REPORT TO

ALLAN GROUP DEVELOPMENTS

ON

GEOTECHNICAL DESK STUDY

FOR

PROPOSED RESIDENTIAL DEVELOPMENT

AT

10-12 CLIFFORD STREET & 33-35 FAIRLIGHT STREET, FAIRLIGHT, NSW

Date: 9 December 2024

Ref: 37056Lrpt-rev1

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1 INTRODUCTION

This report presents the results of a geotechnical desktop study for the proposed residential development at 10-12 Clifford Street and 33-35 Fairlight Street, Fairlight, NSW. The location of the site is shown in Figure 1. The geotechnical desktop study was commissioned by Mr Oscar Guzman by email dated 13 September 2024 and was carried out in accordance with our fee proposal, Ref: P60532S, dated 30 April 2024.

Based on the provided relevant architectural drawings prepared by Platform Architects (Project No. FSF2, Drawing No's. DA0000, DA0050, DA0100, DA0400, DA0500, DA1000 to DA1008, DA1950, DA2000 to DA2003, DA3000 to DA3003, DA5100 and DA5101, all Revision A, dated 5 December 2024), we understand the proposed works include the following:

- Demolition of all site structures.
- Construction of 2No. four and five storey structures at the northern and southern ends of the site, respectively, above a common two-storey basement. The northern structured herein referred to as the Fairlight Street structure and the southern structure is herein referred to as the Clifford Avenue structure
- The proposed new building steps up to the north in three levels, having the following floor levels and offsets:
    - The 'Clifford Avenue Ground Floor' extends over the southern third of the site and has a floor level at RL25.75m, resulting in a maximum excavation depth of about 12.6m at the northern end, reducing to about 2.4 at the southern end. The ground floor footprint is variable, with minimum offsets of about 5.7m and 2.4m from the western and eastern boundaries respectively.
    - The 'Clifford Avenue Level 2' comprises of 'carpark 1' and 'carpark 1.5'. This carpark is accessed from the Clifford Street Ground Floor Level and extends into the central and northern thirds of the site, respectively. Carpark 1 has a proposed floor level at RL30.35m, whilst carpark 1.5 has a proposed floor level at RL31.95m, resulting in a maximum excavation depth of about 15.5m. On the western side, the majority of the Clifford Avenue Level 2 footprint is set-back about 6m from the western boundary, although for a small portion at the southern end of carpark 1, it is set-back only about 1.5m. Elsewhere, the Clifford Avenue Level 2 footprint of set-back from the eastern and northern boundaries by about 1.5m and 6.3m, respectively.
- Construction of a courtyard adjacent to the Fairlight Street frontage, which will have an approximate finished level at RL45.75, which will result in excavation of about 4.2m below the Fairlight Street frontage. The courtyard will abut the northern boundary, and will be set-back about 1.2m from the eastern and western boundaries.
- Installation of an OSD tank below the proposed driveway at the southern end of the site, which will require excavation to about 3.3m below existing ground levels. The OSD tank is set-back from the southern, western and eastern boundaries by about 0.6m, 20m and 6.6m, respectively.

- The existing sandstone cliff face within the north-western corner of the site will remain.

The purpose of this desktop report is to assess the geotechnical suitability of the site for the proposed development. This has been undertaken by reviewing subsurface information from previous investigations carried out by JK Geotechnics at No's.33-35 Fairlight Street and at No.10 Clifford Avenue together with a walkover assessment of the site and the immediate surrounds. Based on the information obtained we have provided our comments on the expected subsurface profile and our preliminary geotechnical recommendations for the proposed development to inform the design process. The site geotechnical model, comments and recommendations will be refined following completion of further site-specific subsurface investigations and confirmation of the likely extent and nature of the development.

2 INVESTIGATION PROCEDURE

The geotechnical assessment comprised:

- A review of the previous investigations carried out by JK Geotechnics at No.33-35 Fairlight Street.
- A review of the previous investigations and construction phase inspections carried out by JK Geotechnics at 10 Clifford Avenue.
- A search of the JK Geotechnics project database to identify any other relevant geotechnical investigations nearby to the site.
- A site walkover by our Senior Geotechnical Engineer, Mr Ben Sheppard on 3 October 2024.
- A review of aerial photography (Google Earth and Six Maps).
- A review of the regional geological map (1:100,000 geological map of Sydney).
- A review of the proposed development shown on the preliminary architectural drawings.

Some geotechnical subsurface investigations have been carried out at the site, although they were tailored to the proposed development works at the time of the fieldwork and not to the current proposed development. As such, the boreholes have not been drilled to below the proposed BEL. Further detailed geotechnical and hydrogeological investigations will need to be caried out as part of the detailed design stages of the proposed development works.

The following geotechnical investigations have been completed by JK Geotechnics at, or within the vicinity of the site:

- **33-35 Fairlight Street, Fairlight** (JK Geotechnics (JKG), JKG Ref:34479SJrptRev2 dated 30 August 2023): Geotechnical investigation that comprised two cored boreholes, to depths of 6.34m and 12.86m below existing ground levels. Due to site access constraints, the boreholes were drilled with specialised Melvelle coring equipment. A Dynamic Cone Penetration (DCP) test was carried out adjacent to each borehole location and at five additional locations. The location of the boreholes and DCP tests are shown on the attached Figure 2. Based on the borehole logs and DCP test results, the sub-surface conditions at the site are as follows:

- Silty sand and sandy clay fill were encountered at both borehole locations and extended to the surface of the sandstone bedrock. The fill was assessed to be poorly compacted.
- Sandstone bedrock was encountered at depths of about 0.3m and 0.9m at the borehole locations and was typically moderately to slightly weathered and medium strength from first contact. Defects within the rock mass comprised sub-horizontal bedding partings, crushed and clay seams up to 30mm thick and joints inclined up to 80°.
- Groundwater monitoring wells were installed within the boreholes and the groundwater monitored for a period of about 4 weeks. Groundwater was measured at depths of about 1.4m and 4.6m about 4 weeks after completion of the fieldwork, though it is believed that the water levels have likely been influenced by stormwater seepage within the fill.

- **10 Clifford Avenue, Fairlight** (JK Geotechnics (formerly Jeffery and Katauskas), JKG Ref: 20098VBrpt dated 10 March 2006): Geotechnical investigation that comprised two cored boreholes, to depths of 4.56m and 9m below existing ground levels. Due to site access constraints, the boreholes were drilled with specialised Melvelle coring equipment. A Dynamic Cone Penetration (DCP) test was carried out adjacent to each borehole location. The location of the boreholes are shown on the attached Figure 2. JK Geotechnics were also involved in construction phase inspections which comprised nine site inspections, predominantly for cut face stability. Based on the borehole logs and reference to construction inspections, the sub-surface conditions at the site are as follows:
  - Silty sand and clayey sand fill were encountered at both borehole locations and extended to depths of about 0.4m and 0.7m below existing ground levels. The fill was assessed to be poorly compacted. Residual sandy clay was encountered in one of the boreholes and was assessed to be of low plasticity and stiff strength.
  - Some deeper soil was encountered above the sandstone in some areas, and contained sandstone floaters.
  - No groundwater was encountered within the boreholes during auger drilling, and based on limited photographs, the cut faces appear to be relatively ‘dry’ during excavation besides some minor seepage emanating from clay seams.
  - Sandstone was encountered at depths of 0.4m and 1.2m, below existing ground levels within the boreholes and was generally assessed to be of slightly weathered and medium strength. Some cross-bedded units were also noted near the surface of the sandstone. A localised ‘pocket’ of very low strength sandstone was encountered at the surface of the sandstone within the western cut face above the better-quality sandstone. A 0.7m thick extremely weathered seam was encountered slightly below the surface of the sandstone, and was generally extensive across the cut faces. A 0.9m to 1m thick shale band was encountered slightly above the bulk excavation level within the western and northern cut faces, and appears to slope to below the bulk level within the eastern cut face. The shale band comprised very low to low strength siltstone, with closely spaced defects. Several sub-vertical joints were observed within the boreholes and cut faces and were generally inclined at about 70° to 90° from the horizontal, with some being soil infilled. Based on the limited photographs, two dominant sub-vertical joint sets appear to be

present and appear to have a very approximate orientation (strike) to the north-east /south-west and east/west.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site comprises four lots, including No.10 and No.12 Clifford Avenue and No.33 and No.35 Fairlight Street. The 'site' refers to all four lots and this site description should be read in conjunction to the attached Figures 1 and 2, which includes the survey plan as the base plan and outcropping sandstone bedrock mapped during the fieldwork.

The site is located about mid-slope on a south facing hillside, which slopes and steps down from an approximately east-west striking ridgeline to Fairlight Beach at an overall grade of about 10°. The site is bound to the north and south by Fairlight Street and Clifford Avenue, respectively, and by residential properties on its remaining sides. A total elevation relief of about 23m exists between the Fairlight Street and Clifford Avenue frontages, with ground levels reducing to the south by a series of sandstone clifflines, retaining walls and sloping gardens and lawns.

The south-eastern quadrant of the site, which comprises No.10 Clifford Avenue, contains a five-storey concrete apartment building, which covers the majority of the lot. Concrete driveways and pathways, garden beds, pools and verandas surround the structure. Access along the eastern side of the structure was restricted, although ground levels appear to be similar to slightly higher than the neighbouring property to the east. We understand from our involvement in the construction of this structure that a basement level exists below the structure, and it has been excavated into the hillside. Based on our knowledge, the existing basement has a floor level at about RL26m. The approximate outline of the existing basement is shown on the attached Figure 2.

The south-western quadrant of the site, which comprises No.12 Clifford Avenue, contains a one and two storey split level house, centrally located towards the Clifford Avenue frontage. The front yard is higher than Clifford Avenue, and is supported by a 1.2m to 1.3m high sandstone block wall. Sandstone outcropped near the base of the wall and was assessed to be of medium strength, and cross bedded. The rear of the site contains a lawn which is generally level and contains some minor level changes due to sandstone outcrops and low-height retaining walls. Sandstone was mapped to be outcropping at multiple locations within the rear yard and was generally assessed to be medium to high strength. A 'plenum' extended around the rear (northern) side of the house, and resulted in a 1.5m to 3m high vertical cut formed through sandstone bedrock assessed to be of at least medium strength. Defects within the exposed sandstone outcrops observed within the site comprise sub-horizontal bedding partings, extremely weathered/clay seams up to 300mm thick and two sub-vertical joint sets striking at about 090° some of which were infilled with soil by up to about 100mm thick. Some sandstone beds were cross-bedded at about 15° to 20° and dipping to about 030°.

The north-western quadrant of the site, which comprises No.35 Fairlight Street, contains a one and two storey split level house, which is centrally located. A 2m to 2.4m high sandstone block wall extends along the Fairlight Street frontage and appears to be in good condition. The concrete footpath above also appeared to be in good condition. Several irregular low-height cobble retaining walls, up to about 0.9m high, are present within the front yard and were in fair to poor condition. A stepped sandstone cliffline is present within the north-western corner of the site, and it is up to about 2.4m high. The sandstone was assessed to comprise medium to high strength sandstone. Sandstone also outcrops below the south-eastern corner of the house, and is cross bedded at 25°, dipping to about 050°. A large sandstone floater is present within the rear yard.

The north-eastern quadrant of the site, which comprises No.33 Fairlight Street, contains a one, two and three storey split level house, which is centrally located. A 1.2m to 1.9m high sandstone block wall extends along the Fairlight Street frontage and appears to be in poor condition, with tilting of the wall by about 8°. The concrete footpath above appeared to be in good condition. To the front and rear of the house are garden beds, pathways and lawns, along with several low-height brick masonry and sandstone cobble/block walls which appear to be in poor condition. Sandstone is outcropping below and adjacent to the southern portion of the house and was assessed to be of medium strength. The sandstone outcrops are cross-bedded at about 20°, dipping to about 040° in some areas, along with a 0.9m deep undercut section below a 1.5m high cliffline.

The neighbouring properties to the west, No.37 Fairlight Street and No.14 Clifford Avenue, contain a two and three storey brick apartment building and a one and two storey apartment building, respectively. Both structures generally appear to be in good condition based on a cursory inspection from within the subject site. Ground levels along the length of western boundary, measured from the Fairlight Street frontage are summarised below:

- 0m to 6m – 2.6m high rendered brick retaining wall which supports the neighbouring property and appears to be in good external condition.
- 6m to 28m – Ground levels generally similar.
- 28m to 40m – 0.9m high sandstone block wall which supports the neighbouring property and appears to be in good external condition, with no obvious signs of bulging or displacement.
- 40m to 50m – Ground levels generally similar.
- 50m to 55m – 2.4m high sandstone block wall which supports the neighbouring property and appears to be in fair condition, although is densely vegetated.
- 55m to 59m – 1.5m high sandstone block wall above a sandstone outcrop (lower 0.4m) which supports the neighbouring property.
- 59m to 73m – 2m to approximately 4m high (inferred) sandstone outcrop which is densely vegetated and so observations were heavily restricted. Some sandstone cobble/block walls may be present near the crest. An undercut about 1.5m deep was present at the southern end of the outcrop.
- 73m to 83m – Ground levels generally similar.

The neighbouring properties to the east, No.31 Fairlight Street and No.8 Clifford Avenue, contain a two and three storey brick apartment building and a four-storey rendered apartment building, respectively. Both structures generally appear to be in good condition based on a cursory inspection from within the subject site. Ground levels along the length of eastern boundary generally appeared to be similar, although access to the southern portion of the eastern boundary was restricted and so ground levels along this portion of the common boundary are generally unknown. Nevertheless, the site and the neighbouring property to the east along the southern portion of the eastern boundary generally comprise several ground level changes due to the presence of retaining walls, raised planter beds, etc. and so ground levels are expected to be variable.

3.2 Subsurface Conditions

The 1:100,000 Geological Map of Sydney indicates that the site is underlain by Hawksbury Sandstone comprising *"medium to coarse grained quartz sandstone, very minor shale and laminate lenses".*

From review of the investigations completed at Nos.33-35 Fairlight Street and No.10 Clifford Avenue, geological maps, and the site walkover, we anticipate that following demolition, the site will be underlain by surficial fill and residual soils overlying weathered sandstone bedrock at relatively shallow depth. The sandstone is expected to be generally of good quality on, or close to, first contact, with defects within the rock mass comprising sub-vertical joints, extremely weathered and clay seams and shale bands. Cross bedded units are anticipated, which dip up to about 25° and generally have a dip direction to the north-east (between about 030° and 050°). A shale band is likely to be encountered at the lowest basement level and anticipated to be extensive.

A Groundwater table is not anticipated within the proposed excavation depth, however we expect that there will be some transient seepage flows at the fill/natural soil and soil/rock interface, along with some seepage through bedding and joints within the bedrock.

4 COMMENTS AND RECOMMENDATIONS

4.1 Principal Geotechnical Issues

We consider that the main geotechnical issues relating to this development will be as follows, and are discussed in further detail in the following sections of the report:

- At this stage, boreholes drilled at the site do not extend deep enough to determine the full geotechnical model in respect to the proposed development, with the exception to the two boreholes drilled within No.10 Clifford Avenue which extend to about RL23.7m. Nevertheless, there appears to be sufficient borehole information to guide the structural design until such time that access for a drilling rig is feasible. Further investigation will be required to determine the quality of the rock below known borehole information depths to confirm if unsupported cuts within the rock are feasible and to assist in the structural design of footings. In this regard, considering the size of the proposed development and to optimise footing design, we recommend that at least an additional four cored boreholes be drilled over the site, extending to below the Bulk Excavation Level (BEL). Considering the depth of the excavation, boreholes will need to be drilled to depths of up to about 18m and as such, boreholes will need to be drilled following demolition when access for a drilling rig is feasible. Deeper boreholes will be required to satisfy WaterNSW requirements, as discussed below.

- The upper portion of the excavation, through any soils and poor-quality bedrock, will need to be either temporarily battered (where space exists) or supported by low-height engineered walls. Any good-quality sandstone may be cut vertically and left temporarily unsupported, although this will require close consultation with the geotechnical engineer to ensure the stability of the cut faces are maintained during construction. Stabilisation of cut faces will almost certainly be required, and time required for inspections and costs for stabilisation must be allowed for in the pre-planning phase of the project. At this stage the current joint orientations indicate that substantial rock face stabilisation may be required for all east-west orientated rock cuts, with less substantial stabilisation along the north-south oriented rock cuts.

- Excavation is proposed close to, and in some areas adjacent to existing structures and will require careful staging of demolition and excavation. Test pits excavated prior to or during demolition will be required adjacent to all potentially impacted structures, which appears to primarily comprise boundary retaining walls. Following inspection of the test pits, the need for underpinning and/or shoring can be determined by the structural and geotechnical engineers.

- Even good-quality bedrock will erode and fret with time. If the rock faces are to be left generally unsupported, access must be created to the void so that ongoing maintenance can be carried out to clear any debris caused by erosion and fretting from the dish drains at the base of the cut faces. If such maintenance is not caried out the drains may become blocked, resulting in dampness issues for the basement walls. Alternatively, and our recommended methodology is for the cut faces to be protected from spalling with reinforced shotcrete to prevent the need for such maintenance as our experience is that the maintenance to clear drains is often neglected.

- The majority of the excavation will require 'hard' rock excavation, which will induce ground borne vibrations that will need to managed such that they do not damage nearby structures.

- Several approvals will be required from various regulatory bodies and these processes can take time and should be addressed early in the planning phase.

4.2 Regulatory Considerations and Geotechnical Design Constraints

The following geotechnically relevant regulatory requirements are expected to apply to the proposed development:

- **Sydney Water**: detailed geotechnical Finite Element Analysis (FEA) will be required to confirm that impacts to Sydney Water assets are within limits considered acceptable to Sydney Water. This will need to be completed in accordance with Sydney Water Specialist Engineering Assessment (SEA) procedure requirements, and input from the structural engineer would also be necessary. The proposed excavation appears to impact the sewer mains extending within the centre of the site and within the south-western corner of the site and the water main below the Fairlight Street frontage.

- **WaterNSW**: A groundwater table is not anticipated to be encountered within the depth of excavation, although dewatering of seepage flows will likely be required during construction, and likely in the permanent case as a drained basement is expected to be adopted. To obtain permission for extracting seepage flows, groundwater level monitoring (using installed data loggers), seepage analysis, Groundwater quality assessment and preparation of a Site Hydrology Report (SHR), Dewatering Management Plan (DMP) to accompany an application to WaterNSW will be necessary, if deemed necessary.

4.3 Dilapidation Survey and Adjacent Buildings

Prior to commencement of any site works, we recommend that detailed internal and external dilapidation reports be completed on all neighbouring buildings. Dilapidation surveys of the surrounding roadways and footpaths may be required by Council. Utility owners, such as Sydney Water, may also require dilapidation surveys of their assets.

Dilapidation reports provide a record of existing conditions prior to commencement of any site works. The dilapidation reports would therefore be used as a benchmark against which to set vibration limits during rock excavation, and for assessing possible future claims for damage arising from the works. The respective owners of the neighbouring properties should be asked to confirm, in writing, that the dilapidation reports present a fair assessment of existing conditions. As dilapidation reports are relied upon for the assessment of potential damage claims, they must be carried out thoroughly by reputable companies with all defects rigorously described (i.e. defect type, defect location, crack width, crack length etc.) and photographed.

The dilapidation reports should be reviewed by JK Geotechnics and the structural engineer prior to commencement of the works.

4.4 Excavation

A detailed demolition, excavation and retention methodology should be developed by the builder/excavation contractor and approved by the geotechnical and structural engineers prior to commencement of the site works.

Prior to any excavation commencing we recommend that reference be made to the latest version of the WorkCover Authority of NSW's Code of Practice – Excavation Work.

4.4.1 Existing Structures

Where any excavation is proposed close to or adjacent to existing structures, which will comprise all boundary walls, care must be taken during excavation not to undermine such structures. During the initial stages of demolition, test pits must be excavated to confirm the nature of the footings supporting these structures, and the test pits are to be inspected by the geotechnical and structural engineers. Following inspection of the test pits, details for temporary support of the existing footings, where such is required, will need to be developed.

Based on the anticipated subsurface conditions, we expect that some of these structures could be founded on sandstone bedrock, although some could be founded within soils at shallow depths. A structural engineer should review the stability of the walls that will remain and confirm that they satisfy acceptable factors of safety against bearing, sliding and overturning in their current condition for temporary construction works. We recommend the northern boundary wall at No. 33 Fairlight Street, which is leaning towards the site, be temporary propped or replaced prior to excavations. We consider that it is inevitable that some underpinning, temporary propping or stabilisation of boundary walls will be required and allowance will need to be made for such measures.

The existing basement walls within No.10 Clifford Avenue are expected to be removed as part of the proposed development, although if it is decided that some elements are to be retained, then the 'as-built' details of such elements will need to be obtained and the structural engineer must determine the suitability of these to remain.

4.4.2 Excavation Conditions

Excavation for the proposed basement is expected to extend to a maximum depth of about 15m below existing ground levels, with locally deeper excavations required for lift overrun pits and services. Excavation to such depths will extend through relatively shallow fill and natural soils, and then predominantly through good quality sandstone bedrock.

Excavation of the soils and any extremely weathered and very low strength sandstone should be achievable using conventional excavation equipment, such as medium sized excavators (say 15 to 20 tonnes) with buckets and "tiger teeth" attached. Where the sandstone bedrock is of low or higher strength, "harder rock" excavation techniques will be required. We anticipate most of the rock excavation will require "harder rock" excavation techniques.

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal.

4.4.3 Potential Vibration Risks

"Harder rock" excavation techniques may consist of percussive or non-percussive techniques. Percussive techniques comprise the use of rock hammers, while non-percussive techniques comprise rotary grinders, rock saws, ripping, rock splitting, etc. Where percussive excavation techniques are adopted, there is the risk that transmitted vibrations may damage nearby movement sensitive structures such as adjoining buildings, retaining walls and the footpath/road to the north.

We recommend that continuous vibration monitoring be carried out on the adjoining buildings to the east and west during all demolition and excavation works. Additional vibration monitoring will also be required on boundary retaining walls, and the extent of this will be determined when their founding conditions are assessed by the test pitting. Vibrations, measured as Peak Particle Velocity (PPV), can provisionally be limited to no higher than 5mm/sec for the adjoining buildings and boundary retaining walls, although lower vibrations may be more applicable to any sensitive boundary structures and this will need to be further assessed following the test pitting. The vibration limits must be confirmed by the structural engineers and acoustic consultant following their review of the dilapidation reports on the nearby buildings and assessment of the boundary walls. If higher vibrations are recorded, they can be assessed against the attached Vibration Emission Design Goals as higher vibrations may be feasible depending on the associated vibration frequency. If it is confirmed that transmitted vibrations are excessive, then it would be necessary to use smaller plant or alternative lower percussion techniques, e.g. rock grinders, or drilling and splitting. The use of these alternative techniques will have lower productivity. When using a rock saw or rotary grinder, the resulting dust must be suppressed by spraying with water.

The following procedures are recommended to reduce vibrations when rock hammers are used:

- Maintain rock hammer orientated towards the face and enlarge excavation by breaking small wedges off the face.
- When operating more than one hammer at a time, operate hammers in different areas of the site and in short bursts only to reduce amplification of vibrations.
- Use excavation contractors with experience and a competent supervisor who is aware of vibration damage risks, possible rock face instability issues, etc. The contractor should be provided with a full copy of this report and have all appropriate statutory and public liability insurances.

4.5 Hydrogeology

Groundwater was encountered at 4.6m (˜RL42.2m) within one of the boreholes from the investigation completed at Nos.33-35 Fairlight Street. Given the topographical location of the site, we do not expect that this will be a groundwater table as such, but rather due to seepage flowing along the top of the soil/rock interface or through defects within the rock. Such seepage should be expected and will be more prevelent particularly during and following rainfall. We anticipate that initially groundwater seepage will be higher and will likely slow over time.

We expect that the permeability of the sandstone bedrock will be relatively slow and seepage during construction would be able to be controlled using conventional sump and pump techniques. Consequently, from a geotechnical perspective we consider that a drained basement would likely be feasible. Collection of seepage from the basement should not result in 'drawdown' of groundwater in the vicinity of the site, and therefore we consider that from a geotechnical perspective, a drained basement will have a negligible impact on any nearby foundations, services, assets, structures and ecosystems.

However, to adopt a drained basement, approval will be required from WaterNSW and Council, respectively, to allow the extraction and disposal of any collected seepage flows. To obtain approval from WaterNSW for temporary and/or permanent dewatering, further investigation and analysis is required, as discussed below.

Construction of a basement that intersects the groundwater table is considered to be an aquifer interference activity. Such activities are subject to the Water Management Act 2000 and NSW Aquifer Interference Policy and are regulated by the Department of Planning, Industry and Environment (DPIE), WaterNSW and Natural Resource Access Regulator (NRAR). The DPIE's policy on basements is that ongoing or frequent dewatering of basements over their life is inconsistent with the principals of sustainable development and, where such dewatering is required, basements should be tanked. Dewatering during construction is permitted, but is regulated through licencing which must either be obtained from WaterNSW or NRAR.

The DPIE's document, "Minimum Requirements for Building Site Groundwater Investigations and Reporting", dated January 2021 outlines the minimum scope of investigation required where a basement is proposed and may intersect the groundwater table. This scope is quite involved and broadly requires the following:

- Boreholes drilled to a minimum depth, which is defined by the proposed number of basements. In this case of about 15m of excavation, at least one of the boreholes is required to be carried out to a depth of about 15m below the BEL. Two boreholes have previously been drilled to depths of RL23.7m, which correlate to about 2m and 8m below the ground floor and carpark 1.5, respectively.
- The installation of a minimum of three groundwater wells installed in a triangulated fashion. The screen within the wells is required to be located 3m above and below the proposed bulk excavation level. We note that for a site of this size additional monitoring wells will also probably be required.
- Permeability testing to define the coefficient of permeability of the various soil layers.
- Groundwater monitoring for a minimum period of 3 months in the 6 months prior to the submission of documentation to the relevant authority.
- Groundwater modelling to predict the groundwater take, groundwater drawdown behind the retention system and potential impact on nearby structures and other groundwater users.
- Chemical analysis of the groundwater to determine its quality.

Therefore at least three wells (and probably more) will need to be installed to facilitate groundwater monitoring and permeability testing, which will then inform the finite element analysis used to prepare the Site Hydrology Report (SHR). The SHR presents the results of the seepage analysis and predicted water take and the impact on surrounding structures and water users. This report is then incorporated in a Dewatering Management Plan (DMP), which is necessary for submission when applying for the relevant licence(s).

Where dewatering is required, potentially two approvals are required from WaterNSW. These are:

- A Water Access Licence (WAL).
- A Water Supply Works (WSW) approval.

A WAL is a licence that provides an allocation of a certain volume of water in the aquifer to a user. However, it does not provide the right to extract this water. To extract or pump water from an aquifer, such as is required during basement dewatering, a WSW approval is required. The WAL is required where extraction of water from the aquifer exceeds 3ML/annum, where a water year coincides with a financial year. Where extraction volumes are less than this value, a WAL is not required.

Should WaterNSW provide permission for the adoption of a drained basement, permission will also need to be obtained from council should it be proposed to dispose of groundwater to the stormwater system. If

Council do not provide permission for disposal to their system, re-use of all extracted groundwater will need to be accommodated on site, whether by watering of vegetated areas, use in toilets etc.

4.6 Retention

The proposed basement has variable depths and offsets from the site boundaries, although has a minimum offset of about 1.5m from the eastern and western boundaries, respectively and about 6.3m from the northern boundary. Based on the anticipated depths of the soils overlying the sandstone bedrock encountered in the boreholes and depths inferred from the DCP tests, and the presence of outcropping sandstone bedrock, temporary excavation batters in the soils and sandstone up to very low strength, and then vertical cuts through sandstone bedrock of at least medium strength (or stronger) are likely to be feasible.

Considering the variability in ground levels along the site boundaries, the presence of sensitive structures and anticipated variability in depths to self supporting sandstone, it is likely that in some areas temporary batters will not be possible. As such, we recommend that a series of test pits be excavated prior to bulk excavation to assess where sandstone suitable to be cut vertically is present. The test pits should be excavated along the excavation perimeter and in particular where the offsets to site boundaries are at a minimum. The test pits should be inspected by the geotechnical engineer and structural engineer to assess where temporary batters can be adopted and where retention systems are required.

Where space permits and provided movement sensitive structures are founded on sandstone bedrock and are not located within about 2H from the crest, (where H is the depth of soil and sandstone bedrock up to and including very low strength), temporary batters formed through sandy fill may be formed at 1 Vertical (1) in 1.5 Horizontal (H) and through clay and sandstone bedrock of less than low strength at 1V:1H. Such batters should remain stable in the short term, provided all surcharge loads, including construction loads, are kept well clear of the crest of the batters. The geotechnical engineer should inspect the temporary batters during construction to confirm that the temporary batters are not cut steeper. Groundwater seepage is likely to be encountered at the soil/rock interface which may cause localised instability at the toe of the soil batters and as such some toe protection, such as sand bags may be required to maintain temporary stability.

We assume that permanent retaining walls will be constructed and all temporary batters will be backfilled to accommodate the proposed landscaping requirements. Backfill of batters should comprise free draining durable single sized gravel, separated from the soils by non-woven geofabric. A clay capping layer placed over the top will reduce the ingress of water and therefore reduce the water to be drained from the basement sump/drainage system. Backfilling using site won material should only be done with effective compaction of material placed in thin layers to at least 95% standard maximum dry density (SMDD) otherwise excessive settlement may occur at the surface; such effective compaction is very difficult in confined spaces and hence is not recommended.

Where temporary batters are not suitable or not preferred, such as where the offset to the site boundaries do not allow for the formation of temporary batters (such as for the proposed courtyard), an alternative shoring system could comprise construction of low-height, mass concrete walls dowelled into the underlying bedrock. However, considering the observation of cross bedded units, where these units dip into the excavation face then longer dowels may be necessary. This option is considered to be more practical and economical than a contiguous pile wall due to the shallow depth of the underlying bedrock, however are generally only possible where the depth of the soils is less than about 1.5m. Such a system will need to be constructed in small segments as recommended by the geotechnical engineers once specific boundary details are known.

The design of free-standing cantilever retaining walls may be based on a triangular earth pressure distribution and a coefficient of lateral earth pressure, K_a, of 0.35 for areas that are not highly sensitive to lateral movement and that will not be propped in the permanent case. An 'at-rest' earth pressure coefficient K_o, of 0.6 may be adopted for areas that are highly sensitive to lateral movement or for walls that will be propped in the permanent case (i.e., from building slabs). A bulk unit weight of 20kN/m^3 may be adopted for the soils. All surcharge loads such as stockpiles, traffic loads, etc. should be added to the above pressures. Appropriate hydrostatic pressures must also be adopted and are in addition to the above pressures. The design must also consider site geometry, such as sloping ground in front of or behind walls, etc. Complete and permanent drainage of the ground behind the walls should be provided.

Sandstone bedrock of medium strength or greater could be cut vertically and left unsupported provided it is free from adverse defects. This will need to be confirmed following the additional investigation to confirm that the sandstone is of sufficient quality with depth (below the base of currently drilled boreholes). In this regard we recommend that all unsupported vertical excavations through sandstone bedrock be inspected by a geotechnical engineer, at height intervals no greater than 1.5m, so that where adverse defects are present, they may be identified and remedial measures initiated. Remedial measures, should adverse defects be present, are likely to comprise rock bolts, shotcrete and mesh and grubbing out and dry packing weaker seams. Considering the scale of the proposed excavation in sandstone bedrock, and from our experience during construction of No.10 Clifford Avenue, provision should be made for stabilisation of the bedrock. Approvals to install rock bolts beyond boundaries will be required.

Unprotected sandstone rock faces will fret and spall over time as well as experiencing emanation of groundwater seepage which may be laden with chemical precipitates which form a sludgy residue. As a result, toe drains will be become blocked over time and cause damp and other problems. We therefore recommend that the rock faces are covered by protective walls or shotcrete and mesh which is pinned to the face with dowels and includes good drainage by means of strip drains etc.

4.7 Footings

All footings should be founded within the sandstone bedrock to provide uniform support and reduce the risk of differential settlements. Sandstone bedrock is expected to be exposed within the basement excavation and as such, pad and strip footings will likely be suitable.

At this stage, it is not possible to provide detailed recommendations on allowable bearing pressures for the proposed footings as the quality of the bedrock below BEL is not known, besides locally within the south-eastern corner of the site, where two boreholes have been drilled to below the BEL. As a guide and subject to the results of the recommended subsurface investigation, we recommend that pad or strip footings founded within sandstone bedrock of at least low strength may be designed based on an allowable end bearing pressure of 1,000kPa. Higher bearing pressures will likely be feasible following the investigation,. All footings must be dry and clean of any loose material prior to pouring concrete.

If any of the above ground portions of the buildings extend outside the footprint of the proposed basements, they should be supported on piles founded within the rock to provide uniform support. Such piles should be founded below a line drawn up at 45° from the base of the excavation. Alternatively, if founding footings/piles below the zone of influence of the basement is not possible, then the edge of footings near the crest of proposed cut rock faces should be designed for a reduced allowable bearing pressure and should be no closer than 0.3m from the cut face. The cut faces below these footings must be specifically inspected for the presence of adverse defects which could affect stability. Some provision for permanent rock bolts or locally deepening the footing should be allowed for.

All pad and strip footings should be cleaned out and inspected by a geotechnical engineer (prior to the installation of reinforcement cages). If delays in pouring pad and strip footings are envisaged, then we recommend that a concrete blinding layer be provided over the bases to reduce deterioration due to weathering. All piles used for footings should be drilled, cleaned out, inspected and poured on the same day as drilling. It is important to note that the geotechnical engineers can only 'sign off' on footings/piles which they have inspected.

4.8 On-Grade Floor Slabs

The basement floor slab is expected to directly overlie bedrock and no particular subgrade preparation will be required. Slab-on-grade construction is therefore considered appropriate. Underfloor drainage, comprising a washed single size aggregate, must however be provided. Such a layer would also act as a separation between the bedrock and floor slab. The underfloor drainage should connect with the wall drains, where appropriate, and direct groundwater seepage to a sump(s) for pumped disposal to a stormwater system following completion of the analysis and obtaining authority approval. Joints in the on-grade floor slabs should incorporate dowels or keys.

4.9 Sydney Water

There are Sydney Water assets running below Fairlight Street, through the central portion of the site and within the south-western corner of the site. Any diversion works will require liaison with, and approval by, Sydney Water. The presence and diversion of the existing sewer could be a significant design issue to resolve and should be addressed at an early stage.

Prior to the commencement of any demolition and excavation, the structural drawings for the proposed development should be forwarded to Sydney Water for their review and approval.

In our recent experience, Sydney Water will require a Specialist Engineering Assessment (SEA) of the potential impact the excavation and construction of the proposed building and underlying basement will have on their nearby assets. The SEA will need to be prepared by a structural engineer, or a water services co-ordinator (WSC), and will need to include finite element analysis (FEA) of the sewer/water mains and proposed development; we can assist with the FEA. The SEA can take significant time for its preparation and for subsequent approval by Sydney Water, and so the SEA, should be completed at an early stage.

4.10 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:

- Additional geotechnical investigation comprising at least four cored boreholes extending to at least 3m below the BEL to provide advice on the excavatability of the bedrock, retention and footing design. One borehole will need to extend to at least about 15m below the basement level.
- Installation of groundwater monitoring wells within at least three boreholes to facilitate long term groundwater monitoring, which are to be screened above and below the basement.
- A minimum of three months of groundwater monitoring and subsequent analysis and reporting (if required).
- Review of the dilapidation survey reports.
- Approval of a detailed demolition, excavation and retention methodology prior to commencement of the site works.
- Inspection of test pits around the perimeter of the excavation to determine retention requirements, as well as adjacent to any boundary or other structures that will remain near excavations.
- Vibration monitoring.
- Regular inspection of vertical rock cuts at depth intervals of no more than 1.5m to check for adverse defects that require additional support.
- Inspection of all footing excavations to confirm that bedrock of adequate quality for the design allowable bearing pressures has been encountered.
- Inspection by a hydraulic engineer, during construction and/or once the bulk excavation has been carried out to confirm that drainage provisions are appropriate.

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.

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

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

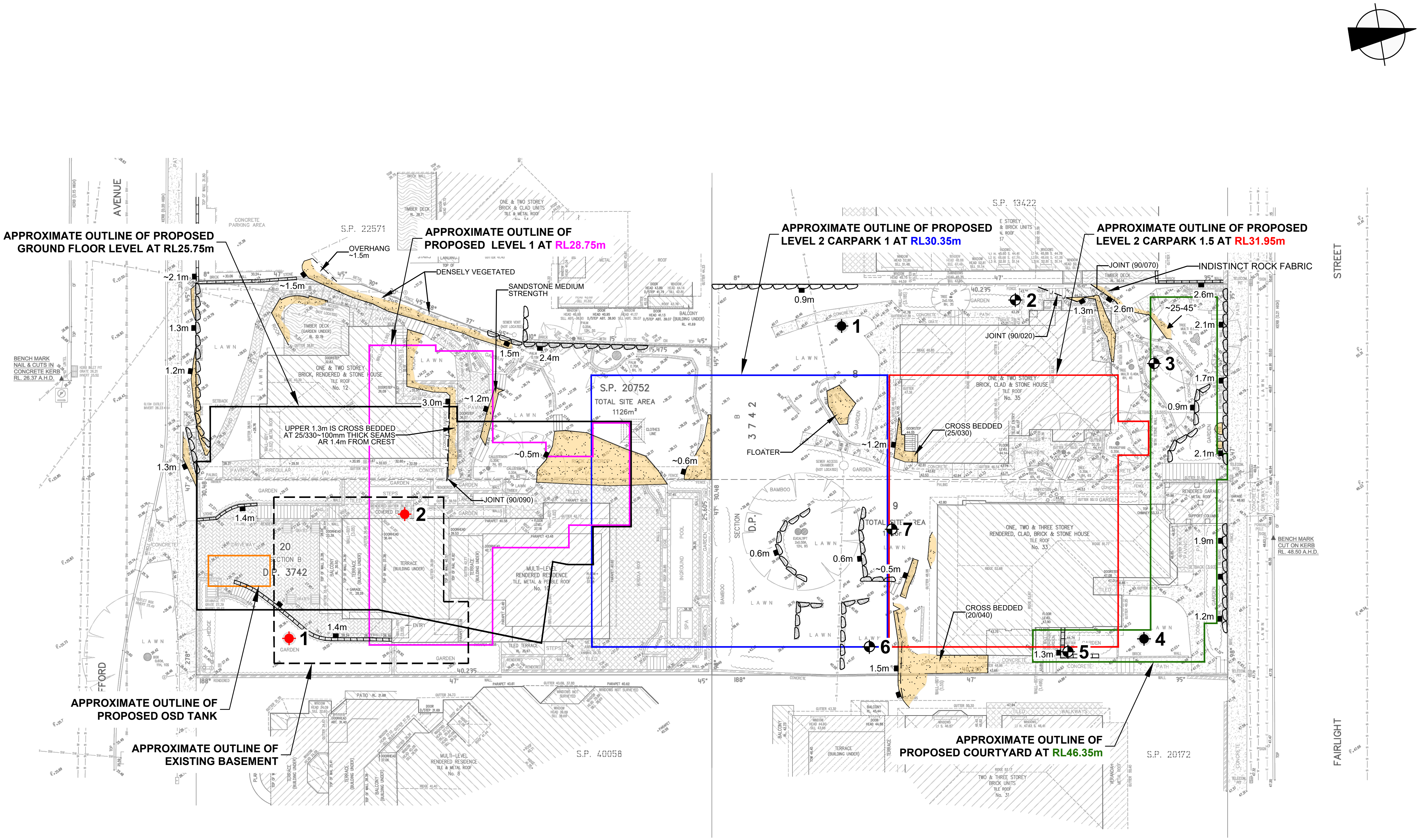
AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:	**SITE LOCATION PLAN**
Location:	10-12 CLIFFORD STREET AND 33-35 FAIRLIGHT STREET, FAIRLIGHT, NSW
Report No: 37056L	Figure No: 1

JKGeotechnics

PLOT DATE: 11/10/2024 9:29:23 AM DWG FILE: J:\6F GEOTECHNICAL JOBS\37000'S\37056L FAIRLIGHT\CAD\37056L.DWG



LEGEND

- BOREHOLE AND DCP TEST (2023)
- DCP TEST (2023)
- BOREHOLE AND DCP TEST (2007)
- SANDSTONE OUTCROP

0 3 6 9 12 15
SCALE 1:300 @A3 METRES

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

Title:	GEOTECHNICAL SKETCH PLAN SHOWING OUTLINE OF PROPOSED WALLS
Location:	10-12 CLIFFORD STREET AND 33-35 FAIRLIGHT STREET, FAIRLIGHT, NSW
Report No:	37056L
Figure No:	2

JKGeotechnics

PLOT DATE: 18/10/2024 11:08:20 AM DWG FILE: J:\6F GEOTECHNICAL JOBS\37000'S\37056L FAIRLIGHT\CAD\37056L.DWG

APPENDIX A

JK Geotechnics 2023 Investigation Results for No.33-35 Fairlight Street

JKG Ref: 34479SJrpt, dated 10 January 2022

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client: ALLEN GROUP DEVELOPMENTS PTY LTD **Ref No:** 34479SJ

Project: PROPOSED RESIDENTIAL DEVELOPMENT **Report:** A

Location: 33-35 FAIRLIGHT STREET, FAIRLIGHT, NSW **Report Date:** 4/11/21

BOREHOLE NUMBER	DEPTH (m)	$I_{S\ (50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
1	0.96 - 0.99	0.3	6	A
	1.12 - 1.16	0.6	12	A
	1.75 - 1.79	1	20	A
	2.21 - 2.25	0.5	10	A
	2.71 - 2.75	0.7	14	A
	3.15 - 3.18	0.4	8	A
	3.76 - 3.80	0.6	12	A
	4.12 - 4.16	0.8	16	A
	4.70 - 4.74	0.8	16	A
	5.11 - 5.15	1	20	A
	5.72 - 5.75	0.9	18	A
	6.13 - 6.17	0.9	18	A
4	0.43 - 0.46	0.7	14	A
	0.85 - 0.89	0.6	12	A
	1.20 - 1.23	0.5	10	A
	1.71 - 1.75	0.6	12	A
	2.31 - 2.35	0.8	16	A
	2.79 - 2.83	1.6	32	A
	3.35 - 3.38	0.6	12	A
	3.88 - 3.91	0.7	14	A
	4.14 - 4.17	0.7	14	A
	4.70 - 4.74	0.9	18	A
	5.35 - 5.38	0.5	10	A
	5.83 - 5.86	0.8	16	A
	6.26 - 6.29	0.8	16	A

NOTE: SEE PAGE 2

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client: ALLEN GROUP DEVELOPMENTS PTY LTD **Ref No:** 34479SJ

Project: PROPOSED RESIDENTIAL DEVELOPMENT **Report:** A

Location: 33-35 FAIRLIGHT STREET, FAIRLIGHT, NSW **Report Date:** 4/11/21

BOREHOLE NUMBER	DEPTH (m)	$I_{S\,(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
4	6.73 - 6.76	0.7	14	A
	7.24 - 7.27	0.9	18	A
	7.81 - 7.85	1.4	28	A
	8.20 - 8.23	1	20	A
	8.69 - 8.73	1.2	24	A
	9.21 - 9.24	1.4	28	A
	9.91 - 9.93	1.3	26	A
	10.20 - 10.24	0.9	18	A
	10.76 - 10.80	1.3	26	A
	11.20 - 11.24	0.9	18	A
	11.75 - 11.79	1.1	22	A
	12.16 - 12.20	1	20	A
	12.69 - 12.72	1.2	24	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 282373

Client Details

Client	JK Geotechnics
Attention	Quang Minh Vu
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**34479SJ, Fairlight**
Number of Samples	3 Soil
Date samples received	09/11/2021
Date completed instructions received	09/11/2021

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Please refer to the last page of this report for any comments relating to the results.

Report Details

Date results requested by	16/11/2021
Date of Issue	16/11/2021

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Accredited for compliance with ISO/IEC 17025 - Testing. **Tests not covered by NATA are denoted with ***

Results Approved By
Diego Bigolin, Inorganics Supervisor

Authorised By
Nancy Zhang, Laboratory Manager

Misc Inorg - Soil				
Our Reference		282373-1	282373-2	282373-3
Your Reference	UNITS	BH1	BH4	BH4
Depth		0.6-0.8	0.1-0.2	0.3-0.4
Date Sampled		29/10/2021	01/11/2021	01/11/2021
Type of sample		Soil	Soil	Soil
Date prepared	-	15/11/2021	15/11/2021	15/11/2021
Date analysed	-	15/11/2021	15/11/2021	15/11/2021
pH 1:5 soil:water	pH Units	6.4	6.0	7.0
Chloride, Cl 1:5 soil:water	mg/kg	<10	<10	<10
Sulphate, SO4 1:5 soil:water	mg/kg	<10	<10	<10
Resistivity in soil*	ohm m	400	220	660

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity (non NATA). Resistivity (calculated) may not correlate with results otherwise obtained using Resistivity-Current method, depending on the nature of the soil being analysed.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			15/11/2021	[NT]	[NT]	[NT]	[NT]	15/11/2021	[NT]
Date analysed	-			15/11/2021	[NT]	[NT]	[NT]	[NT]	15/11/2021	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	102	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	88	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	85	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]

Result Definitions

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

Quality Control Definitions

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

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

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

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

Laboratory Acceptance Criteria

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

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

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

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals (not SPOCAS); 60-140% for organics/SPOCAS (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

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

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

Report Comments
Samples received in good order: Holding time exceedance

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BOREHOLE LOG

Client: ALLEN GROUP DEVELOPMENTS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 33-35 FAIRLIGHT STREET, FAIRLIGHT, NSW

Job No.: 34479SJ | **Method:** HAND AUGER | **R.L. Surface:** ~41.2 m
Date: 29/10/21 | **Datum:** AHD
Plant Type: | **Logged/Checked By:** Q.V./J.M.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING 26/11/2021					REFER TO DCP TEST RESULTS	41				FILL: Silty sand, fine to medium grained, dark grey, with fine to medium grained igneous gravel, trace of clay nodules and root fibres.	M			GRASS COVER APPEARS POORLY COMPACTED
										as above, but with fine to coarse grained sandstone gravel, brown and yellow brown.	w>PL		30 40 30	
						40	1			FILL: Sandy clay, low plasticity, light brown, fine to medium grained sand, trace of fine to medium grained sandstone gravel.				HAND AUGER REFUSAL Groundwater monitoring well installed to 6.34m. Hand slotted 40mm dia. PVC standpipe 0.82m to 6.34m. Casing 0.05m to 0.82m. 2mm sand filter pack 0.2m to 6.34m. Bentonite seal 0.0m to 0.2m. Completed with a concreted gatic cover.
										REFER TO CORED BOREHOLE LOG				
						39	2							
						38	3							
						37	4							
						36	5							
						35	6							

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CORED BOREHOLE LOG

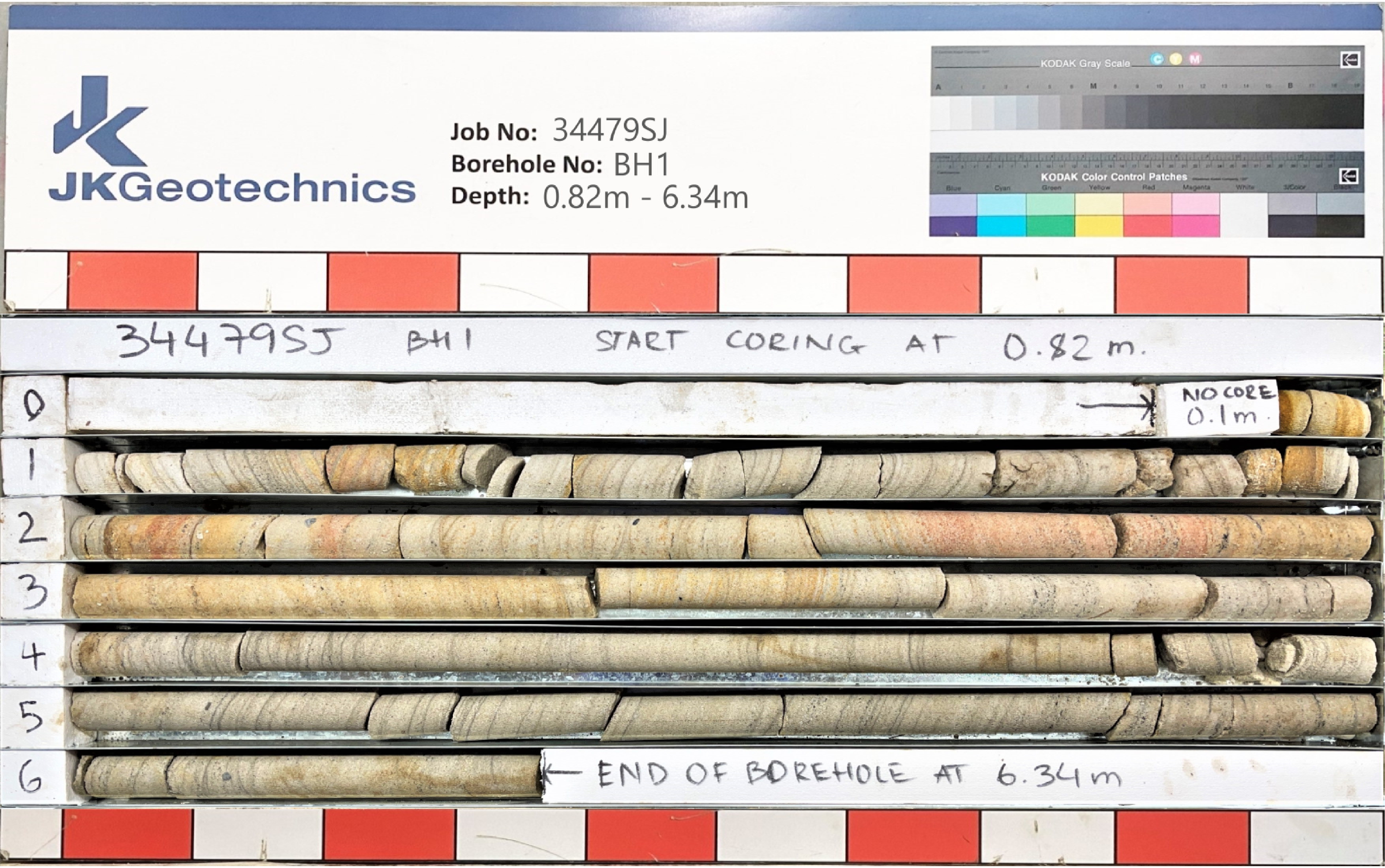
Borehole No.
1

Client: ALLEN GROUP DEVELOPMENTS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 33-35 FAIRLIGHT STREET, FAIRLIGHT, NSW

Job No.: 34479SJ	**Core Size:** TT56	**R.L. Surface:** ~41.2 m
Date: 29/10/21	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: MELVELLE	**Bearing:** N/A	**Logged/Checked By:** Q.V./J.M.

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
		41			START CORING AT 0.82m						
1/12/2021 100% RETURN 1/11/2021		40	1		NO CORE 0.10m						
					SANDSTONE: fine to medium grained, light grey with yellow brown and red brown lamination, distinctly bedded at 0-25°.	SW	M	0.30 0.60 1.0		(0.96m) Be, 5°, P, R, Fe Sn (1.03m) Be, 5°, P, R, Fe Ct (1.30m) Be, 10°, P, R, Cn (1.65m) Ji, 20°, P, R, Cn (1.85m) Bex2, 0°, P, R, Cn (1.99m) Cr, 0°, 10 mm.t	Hawkesbury Sandstone
100% RETURN		39 38	2 3		as above, but trace of carbonaceous lenses and quartz clasts.			0.50 0.70 0.40	600 200 60 20	(2.81m) Be, 5°, P, R, Fe Sn	
		37 36 35	4 5 6		SANDSTONE: fine to medium grained, light grey with grey lamination, trace of carbonaceous lenses, distinctly bedded at 0-20°.	FR		0.60 0.80 0.80 1.0 0.90 0.90		(4.91m) Cr, 0°, 30 mm.t (5.80m) Be, 5°, P, R, Cn (6.07m) CS, 5°, 2 mm.t	
					END OF BOREHOLE AT 6.34 m				600 200 60 20		

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 34479SJ
Borehole No: BH1
Depth: 0.82m - 6.34m
KODAK Gray Scale
KODAK Color Control Patches
34479SJ BH1 START CORING AT 0.82m.
0
NO CORE 0.1m
1
2
3
4
5
6
END OF BOREHOLE AT 6.34m

JKGeotechnics

BOREHOLE LOG

Borehole No.
4

Client: ALLEN GROUP DEVELOPMENTS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 33-35 FAIRLIGHT STREET, FAIRLIGHT, NSW

Job No.: 34479SJ **Method:** HAND AUGER **R.L. Surface:** ~46.8 m
Date: 1/11/21 **Datum:** AHD
Plant Type: **Logged/Checked By:** Q.V./J.M.

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				■	REFER TO DCP TEST RESULTS					FILL: Silty sand, fine to medium grained, dark brown, with fine to medium grained sandstone and igneous gravel, trace of root fibres.	M			GRASS COVER APPEARS POORLY COMPACTED
						46	1			REFER TO CORED BOREHOLE LOG				HAND AUGER REFUSAL Groundwater monitoring well installed to 12.86m. Hand slotted 40mm dia. PVC standpipe 0.86m to 12.86m. Casing 0.05m to 0.86m. 2mm sand filter pack 0.3m to 12.86m. Bentonite seal 0.0m to 0.3m. Completed with a concreted gatic cover
						45	2							
						44	3							
						43	4							
						42	5							
						41	6							
						40								

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
4

Client: ALLEN GROUP DEVELOPMENTS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 33-35 FAIRLIGHT STREET, FAIRLIGHT, NSW

Job No.: 34479SJ	**Core Size:** TT56	**R.L. Surface:** ~46.8 m
Date: 1/11/21	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: MELVELLE	**Bearing:** N/A	**Logged/Checked By:** Q.V./J.M.

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

Depth (m)	Core Description	Weathering	Strength
0.30	START CORING AT 0.30m		
0.30–2.00	SANDSTONE: fine to medium grained, light grey and yellow brown with red brown lamination, trace of quartz clasts, distinctly bedded at 0-23°.	SW	M
2.00–2.96	as above, but with occasional ironstone bands.		M (H from ~2.65 to ~2.80, then M)
2.96–6.65	SANDSTONE: fine to medium grained, light grey with yellow brown and red brown lamination, distinctly bedded at 0-20°.		M
6.65–	SANDSTONE: fine to medium grained, light grey and red brown with yellow brown laminae, bedded at 0-20°.	MW	

Water Loss\Level: 90% RETURN (0.30–1.30m); ON COMPLETION OF CORING ▽; 70% RETURN; 26/11/2021 ▽; 1/12/2021 ▽; 80% RETURN

RL (m AHD): 46, 45, 44, 43, 42, 41, 40

Point Load Strength Index $I_s(50)$: 0.70, 0.60, 0.50, 0.60, 0.80, 1.6, 0.60, 0.70, 0.70, 0.90, 0.50, 0.80, 0.80, 0.70

Defect Details – Specific:

- (0.65m) Be, 0°, P, R, Cn
- (0.97m) Bex2, 0°, P, R, Fe Sn
- (1.10m) Be, 0°, P, R, Fe Sn
- (2.03m) Bex2, 0°, P, R, Fe Vn
- (2.15m) Bex2, 0°, P, R, Fe Ct
- (2.20m) Be, 0°, P, R, Fe Ct
- (2.21m) J, 40°, P, R, Fe Sn
- (2.60m) Be, 0°, P, R, Cn
- (2.65m) Be, 0°, P, R, Fe Sn
- (2.82m) Be, 0°, P, R, Cn
- (3.10m) Be, 0°, P, R, Cn
- (3.15m) Be, 0°, P, R, Cn
- (3.20m) Be, 0°, P, R, Cn
- (3.30m) Be, 0°, P, R, Cn
- (4.38m) Be, 0°, P, R, Fe Sn
- (4.48m) J, 75°, P, R, Cn
- (4.61m) Bex2, 5°, P, R, Cn
- (4.96m) Bex2, 10°, P, R, Clay Ct
- (5.11m) CS, 10°, 2 mm.t
- (5.40m) Bex2, 10°, P, R, Cn
- (5.45m) Be, 10°, P, R, Cn
- (5.65m) J, 80°, C, R, Cn
- (6.40m) Jx2, 70°, P, R, Cn

Formation: Hawkesbury Sandstone

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
4

Client: ALLEN GROUP DEVELOPMENTS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 33-35 FAIRLIGHT STREET, FAIRLIGHT, NSW

Job No.: 34479SJ	**Core Size:** TT56	**R.L. Surface:** ~46.8 m
Date: 1/11/21	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: MELVELLE	**Bearing:** N/A	**Logged/Checked By:** Q.V./J.M.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT DETAILS SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific / General	Formation
70% RETURN		39, 38	8, 9		SANDSTONE: fine to medium grained, light grey and red brown with yellow brown laminae, bedded at 0-20°. *(continued)*	MW	M M - H	0.90 1.4 1.0		(7.40m) Bex2, 5°, P, R, Fe Ct	Hawkesbury Sandstone
70% RETURN / 80% RETURN		37	9, 10		SANDSTONE: fine to medium grained, light grey with grey and yellow brown laminae, trace of carbonaceous lenses, distinctly bedded at 0-20°.	FR	M - H	1.2 1.4 1.3		(9.30m) Be, 10°, P, R, Cb Vn (9.87m) Be, 5°, P, R, Cn	Hawkesbury Sandstone
80% RETURN		36, 35, 34	10, 11, 12		SANDSTONE: fine to medium grained, light grey with grey lamination, trace of carbonaceous lenses, distinctly bedded at 0-20°.	FR	M - H	0.90 1.3 0.90 1.1 1.0 1.2		(11.98m) Be, 10°, P, R, Cb Ct (12.21m) Be, 5°, P, R, Cb Sn	Hawkesbury Sandstone
		33	13		END OF BOREHOLE AT 12.86 m						

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JKGeotechnics

Job No: 34479SJ
Borehole No: BH4
Depth: 0.3m - 12.87m

JKGeotechnics

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	ALLEN GROUP DEVELOPMENTS PTY LTD		
Project:	PROPOSED RESIDENTIAL DEVELOPMENT		
Location:	33-35 FAIRLIGHT STREET, FAIRLIGHT, NSW		
Job No.	34479SJ	Hammer Weight & Drop: 9kg/510mm	
Date:	29-10-21	Rod Diameter: 16mm	
Tested By:	QV	Point Diameter: 20mm	

Test Location	1	2	3	4	5	6	7
Surface RL	≈ 41.2m	≈ 43.2m	≈ 46.5m	≈ 46.8m	≈ 44.5m	≈ 41.3m	≈ 41.7m
Depth (mm)	Number of Blows per 100mm Penetration						
0 - 100	SUNK	SUNK	SUNK	SUNK	1	1	1
100 - 200	↓	↓	↓	1	5	2	1
200 - 300	1	1	1	10/50mm	REFUSAL	2	1
300 - 400	1	1	↓	REFUSAL		2	2
400 - 500	2		1			5	3
500 - 600	2		5/5mm			4	3
600 - 700	3	↓	REFUSAL			3	5
700 - 800	8	2				2	2
800 - 900	REFUSAL	8/50mm				4	2
900 - 1000		REFUSAL				2	3
1000 - 1100						1	3
1100 - 1200						1	3
1200 - 1300						↓	3
1300 - 1400						4	4
1400 - 1500						17	4
1500 - 1600						REFUSAL	10/50mm
1600 - 1700							REFUSAL
1700 - 1800							
1800 - 1900							
1900 - 2000							
2000 - 2100							
2100 - 2200							
2200 - 2300							
2300 - 2400							
2400 - 2500							
2500 - 2600							
2600 - 2700							
2700 - 2800							
2800 - 2900							
2900 - 3000							

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

APPENDIX B

JK Geotechnics (formerly Jeffery & Katauskas) 2006 Investigation Results for No.10 Clifford Avenue

JKG Ref: 20098VBrpt, dated 10 March 2006

Unit 3, 39 Buffalo Road
Gladesville, NSW 2111
Telephone: 02 9809 7322
Facsimile: 02 9809 7626
Email: dtreweek@jkgroup.net.au

SOIL TEST SERVICES
ABN 43 002 145 173

Ref No: 20098VB

TABLE A
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	1.65-1.67	0.6	12
	2.86-2.89	0.5	10
	3.91-3.94	0.8	16
	4.08-4.10	0.6	12
	4.38-4.42	0.9	18
2	0.54-0.57	0.1	2
	1.42-1.45	0.4	8
	2.51-2.54	0.5	10
	3.86-3.89	0.07	2
	4.25-4.27	1.1	22
	5.00-5.02	1.2	24
	5.96-5.99	0.8	16
	6.25-6.28	1.2	24
	7.63-7.66	1.5	30
	8.00-8.03	1.2	24
	8.81-8.84	0.6	12
	8.87-8.90	0.6	12

NOTES:

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

   U.C.S. = 20 $I_{S(50)}$

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BOREHOLE LOG

Borehole No. 1

Client: JA & MT JOSEPH
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 10 CLIFFORD AVENUE, FAIRLIGHT, NSW

Job No. 20098VB **Method:** HAND AUGER **R.L. Surface:** ≈ 28.4m
Date: 23-2-06 **Datum:** AHD
Logged/Checked by: M.T./[illegible]

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION OF AUGERING					REFER TO DCP TEST RESULTS	0			FILL: Silty sand, fine to medium grained, dark brown, with roots and root fibres.	D			GRASS COVER
									FILL: Clayey sand, fine to medium grained, brown and orange brown, with roots, root fibres and sandstone gravel.	M			APPEARS POORLY COMPACTED
						1		CL	SANDY CLAY: low plasticity, orange brown and light grey.	MC<PL	(St)	-	
									REFER TO CORED BOREHOLE LOG				HAND AUGER REFUSAL
						2							
						3							
						4							
						5							
						6							
						7							


Jeffery and Katauskas Pty Ltd
JOB NO. 20098VB BH1 START CORING AT 1.30m
1
1.30
2
CORE LOSS 0.1m
3
4
END AT 4.56m

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CORED BOREHOLE LOG

Borehole No. 1

Client: JA & MT JOSEPH
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 10 CLIFFORD AVENUE, FAIRLIGHT, NSW

Job No. 20098VB | **Core Size:** TT56 | **R.L. Surface:** ≈ 28.4m
Date: 23-2-06 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: MELVELLE | **Bearing:** - | **Logged/Checked by:** M.T./

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION (Rock Type, grain characteristics, colour, structure, minor components.)	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (EL VL L M H VH EH)	DEFECT SPACING (mm) (500 300 100 50 30 10)	DEFECT DETAILS DESCRIPTION (Type, inclination, thickness, planarity, roughness, coating.) Specific / General
		1		START CORING AT 1.30m					
FULL RETURN				SANDSTONE: fine to medium grained, light grey, orange brown and red brown.	SW	M	×		- CS, 3mm.t
		2		CORE LOSS 0.10m					- CS, 40mm.t
		3, 4		SANDSTONE: fine to medium grained, light grey and orange brown, cross bedded at 10-20°.	SW	M	× × × ×		- XWS, 70mm.t
		5, 6, 7, 8		END OF BOREHOLE AT 4.56m					

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BOREHOLE LOG

Borehole No. **2**

Client: JA & MT JOSEPH
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 10 CLIFFORD AVENUE, FAIRLIGHT, NSW

Job No. 20098VB **Method:** HAND AUGER **R.L. Surface:** ≈ 32.7m
Date: 23-2-06 **Datum:** AHD
Logged/Checked by: M.T./

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION OF AUGERING					REFER TO DCP TEST RESULTS	0			FILL: Silty sand,fine to medium grained, dark brown, with roots, root fibres and sandstone gravel.	D			APPEARS POORLY COMPACTED
									REFER TO CORED BOREHOLE LOG				HAND AUGER REFUSAL
						1							
						2							
						3							
						4							
						5							
						6							
						7							

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CORED BOREHOLE LOG

Borehole No. **2**

Client: JA & MT JOSEPH
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 10 CLIFFORD AVENUE, FAIRLIGHT, NSW

Job No. 20098VB | **Core Size:** TT56 | **R.L. Surface:** ≈ 32.7m
Date: 23-2-06 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: MELVELLE | **Bearing:** - | **Logged/Checked by:** M.T./

Water Loss/Level	Depth (m)	Core Description (Rock Type, grain characteristics, colour, structure, minor components)	Weathering	Strength	Defect Description: Specific
	0	START CORING AT 0.40m			
FULL RETURN		SANDSTONE: fine to medium grained, light grey and orange brown, cross bedded at 10-20°.	SW	L	- Be, 0°
	1			M	- CS, 5mm.t
	2				- Be, 10°, CLAY COATING
	3	CORE LOSS 0.17m			
		SANDSTONE: fine grained, light grey.	SW	M	- J, 90°, P, R
		SHALE: dark grey, with thin sandstone laminae.	DW	VL	- XWS, 190mm.t - XWS, 140mm.t - XWS, 15mm.t
	4	SANDSTONE: fine to medium grained, light grey, with dark grey laminae, cross bedded at 10-20°.	SW	M-H	- Be, 10°, CLAY COATING
	5				
	6				- Be, 0°, CLAY INFILL, 5mm.t
	7				- J, 70°, P, R - CS, 5mm.t

Point Load Strength Index $I_s(50)$: EL VL L M H VH EH
Defect Spacing (mm): 500 300 100 50 30 10


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JOB NO 20098VB BH2 START CORING AT 0.4m
0
0.4m
1
2
CORE LOSS
3
0.17m
4
5
6
7
8

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No. **2**
3/3

CORED BOREHOLE LOG

Client: JA & MT JOSEPH
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 10 CLIFFORD AVENUE, FAIRLIGHT, NSW

Job No. 20098VB | **Core Size:** TT56 | **R.L. Surface:** ≈ 32.7m
Date: 23-2-06 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: MELVELLE | **Bearing:** - | **Logged/Checked by:** M.T./

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
				SANDSTONE: fine to medium grainad, light grey, with dark grey laminae, cross bedded at 10-20°.	SW	M-H				
				as above, but light brown and light grey.			×			
		8					×			
				as above, but light grey.			×			
		9		END OF BOREHOLE AT 9.0m						
		10								
		11								
		12								
		13								
		14								

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

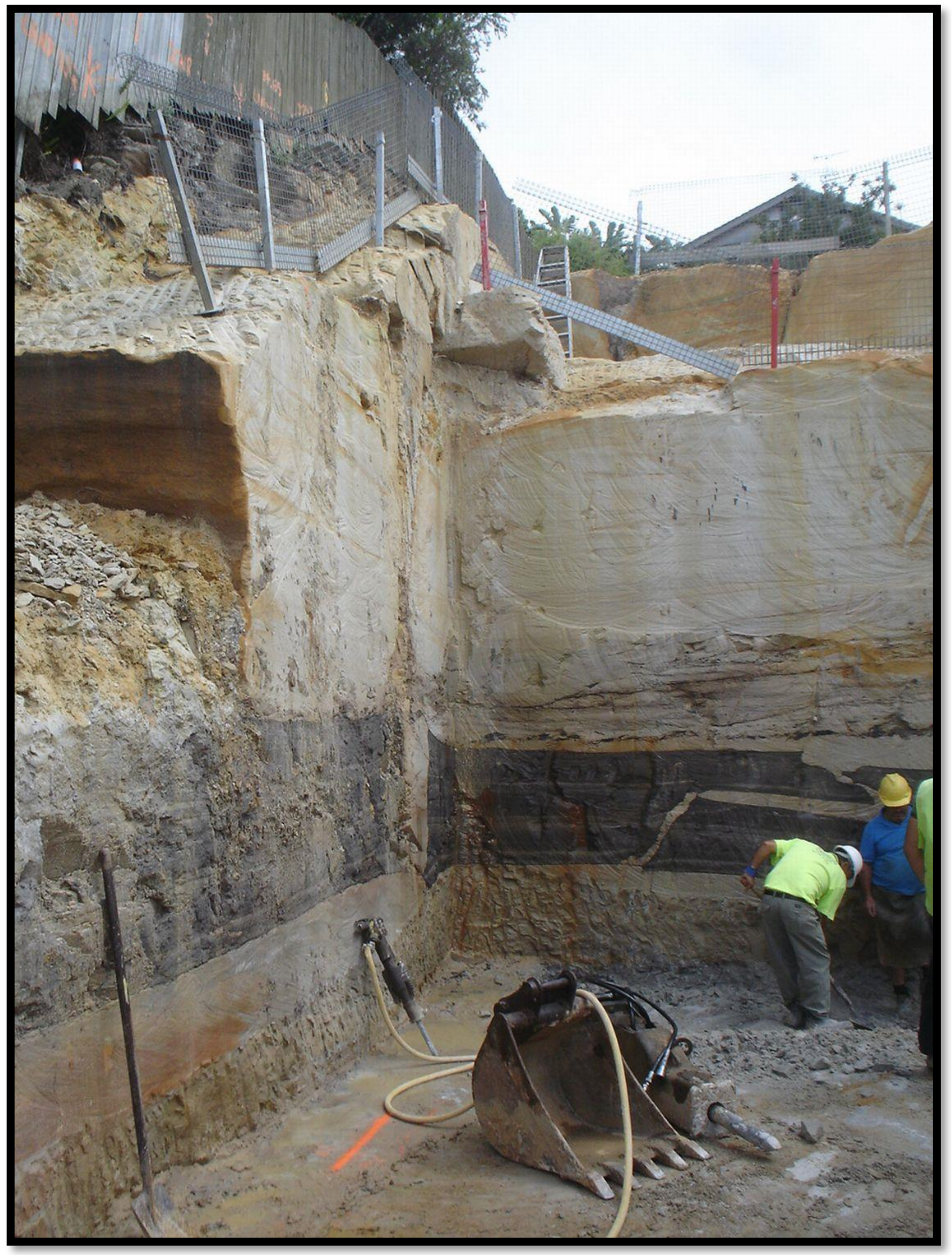
DYNAMIC CONE PENETRATION TEST RESULTS

Client:	JA & MT JOSEPH		
Project:	PROPOSED RESIDENTIAL DEVELOPMENT		
Location:	10 CLIFFORD AVENUE, FAIRLIGHT, NSW		
Job No.	20098VB	Hammer Weight & Drop: 9kg/510mm	
Date:	23-2-06	Rod Diameter: 16mm	
Tested By:	M.T.	Point Diameter: 20mm	

	Number of Blows per 100mm Penetration						
Test Location	RL ~28.4m	RL ~32.7m					
Depth (mm)	**1**	**2**					
0 - 100	1	1					
100 - 200	4	2					
200 - 300	3	2					
300 - 400	3	9/95mm					
400 - 500	2	REFUSAL					
500 - 600	3						
600 - 700	2						
700 - 800	2						
800 - 900	2						
900 - 1000	2						
1000 - 1100	3						
1100 - 1200	3						
1200 - 1300	13						
1300 - 1400	REFUSAL						
1400 - 1500							
1500 - 1600							
1600 - 1700							
1700 - 1800							
1800 - 1900							
1900 - 2000							
2000 - 2100							
2100 - 2200							
2200 - 2300							
2300 - 2400							
2400 - 2500							
2500 - 2600							
2600 - 2700							
2700 - 2800							
2800 - 2900							
2900 - 3000							

Remarks:
1. The procedure used for this test is similar to that described in AS1289.6.3.2-1997, Method 6.3.2.
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

Photographs taken during construction stage inspections at No.10 Clifford Avenue



Rear north-western corner of basement excavation, looking north

Rear, north-eastern corner of basement excavation, looking north

Rear north-western corner of basement excavation, looking west

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 <u>or</u> where only a limited investigation has been completed <u>or</u> where the geotechnical conditions/constraints are quite complex, it is prudent to have a joint design review which involves an experienced geotechnical engineer/engineering geologist.

SITE INSPECTION

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

Requirements could range from:

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

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

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

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

ROCK

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

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory Classification Criteria

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

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

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

NOTES:

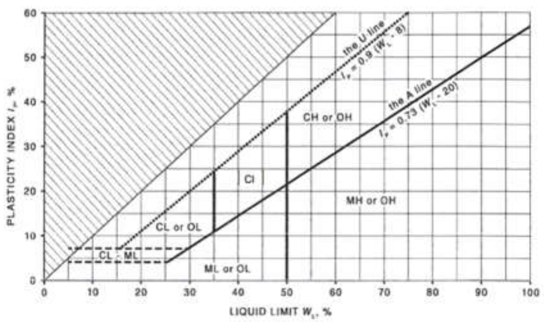
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record		Standing water level. Time delay following completion of drilling/excavation may be shown.
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.
		Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES U50 DB DS ASB ASS SAL	Sample taken over depth indicated, for environmental analysis. Undisturbed 50mm diameter tube sample taken over depth indicated. Bulk disturbed sample taken over depth indicated. Small disturbed bag sample taken over depth indicated. Soil sample taken over depth indicated, for asbestos analysis. Soil sample taken over depth indicated, for acid sulfate soil analysis. Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25 PID = 100	Vane shear reading in kPa of undrained shear strength. Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	w > PL $w \approx$ PL w < PL $w \approx$ LL w > LL	Moisture content estimated to be greater than plastic limit. Moisture content estimated to be approximately equal to plastic limit. Moisture content estimated to be less than plastic limit. Moisture content estimated to be near liquid limit. Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D M W	DRY – runs freely through fingers. MOIST – does not run freely but no free water visible on soil surface. WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS S F St VSt Hd Fr ( )	VERY SOFT – unconfined compressive strength ≤ 25kPa. SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa. FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa. STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa. VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa. HARD – unconfined compressive strength > 400kPa. FRIABLE – strength not attainable, soil crumbles. Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** / **SPT 'N' Value Range (Blows/300mm)**
	VL	VERY LOOSE ≤ 15 / 0 – 4
	L	LOOSE > 15 and ≤ 35 / 4 – 10
	MD	MEDIUM DENSE > 35 and ≤ 65 / 10 – 30
	D	DENSE > 65 and ≤ 85 / 30 – 50
	VD	VERY DENSE > 85 / > 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

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

Classification of Material Weathering

<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