REPORT TO

ALLEN GROUP DEVELOPMENTS PTY LTD

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED RESIDENTIAL DEVELOPMENT

AT

33-35 FAIRLIGHT STREET, FAIRLIGHT, NSW

Date: 10 January 2022
Ref: 34479SJrpt

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ATTACHMENTS

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed residential development at 33-35 Fairlight Street, Fairlight, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Mr Oscar Guzman of Lighthouse Project Group on behalf of Allen Group Developments Pty Ltd by email on 28 September 2021 and was completed in general accordance with Option 2 in our proposal, Ref: P55078SJ, dated 22 September 2021.

Based on the architectural drawings prepared by Platform Architects (Ref: Drawings prepared for DA Submission, dated October 2021, Revision A), we understand that following demolition of the existing residences, garage and other structures on site, it is proposed to construct a four-storey apartment building that requires cutting into the hillside that slopes down to the south. The ground floor level is shown at RL39.05m and so we anticipate bulk excavation level to be about RL38.8m. A car lift is proposed at the northern end and three other lifts are proposed, where we anticipate bulk excavation level to be about RL37m. We estimate excavations for the building to a maximum depth of about 10m will be required. Localised deeper excavations may be required for buried services. A landscaped courtyard, bin collection area (shown at RL38.95m) and an OSD overflow structure are shown to the south of the building. Another paved area is shown between the OSD overflow structure and the building (shown at RL38.95m). At this stage details on the OSD tank dimensions have not been provided. We anticipate excavations for the bin collection area (western side) and other paved area (eastern side) to depths of about 2m and 1m, respectively, to be required.

Based on the Technical Brief for Geotechnical Engineers prepared by the structural engineer, M+G Consulting (Ref: Project 5631, dated 14 September 2021) we understand column working loads will be in the range of 500 to 1500kN.

The purpose of the investigation was to develop a geotechnical model for the site based on the subsurface information obtained at the test locations and from our site observations. Based on this we have provided comments and recommendations on dilapidation surveys, excavation and hydrogeology, retention, footings, slabs on grade, engineered fill and aggressivity.

2 INVESTIGATION PROCEDURE

Prior to the commencement of the fieldwork, we carried out the following:

- Review of the proposed development and existing site constraints to determine appropriate investigation locations,
- Liaison with the Client in regards to access, and,
- Completion of a dial before you dig buried services search and an on-site services search using electromagnetic induction measures completed by a buried services subcontractor.

Our geotechnical investigation was carried out on 29 October and 1 November 2021 using portable hand operated equipment and comprised the following:

- Two boreholes, which were initially advanced using hand techniques to refusal depths of 0.82m (BH1) and 0.3m (BH4), and then extended using portable rotary diamond rock coring techniques (TT56) to depths of 6.34m (BH1) and 12.86m (BH4).
- Seven DCP tests were completed adjacent to the boreholes (DCP1 and DCP4) and at five other locations (DCP2, DCP3 and DCP5 to DCP7) to refusal depths ranging between 0.2m and 1.55m.
- Groundwater observations were made in the boreholes, during and on completion of drilling and following drilling as discussed in more detail below. We note that water is introduced into the boreholes during core drilling (BH1 and BH4) and therefore the water levels after coring are likely to be artificially higher than actual levels.

The apparent compaction of the fill was assessed from the DCP test results. The refusal depth of DCP tests can provide an indicative depth to bedrock, though we note that refusal can also occur on obstructions in fill, 'floaters' and other hard layers within the soil profile. Based on the two tests carried out at the borehole locations, outcropping sandstone and test results we anticipate that the refusal depths in DCP2, DCP3 and DCP5 to DCP7 are likely to be on sandstone bedrock.

The strength of the bedrock in the boreholes was assessed by examination of the recovered rock core and, subsequent correlation with Point Load Strength Index ($I_{S\,(50)}$) testing. The results of the Point Load Strength Index tests are presented on the attached Table A and on the cored borehole logs. The Unconfined Compressive Strengths (UCS's) were estimated from the Point Load Strength Index test results and are also summarised in Table A. Photographs of the recovered core are presented at the rear of this report with the borehole logs.

The investigation location plan is included as Figure 2. Due to access constraints, investigation locations were limited to areas outside the buildings and other structures. The locations were set out by taped measurements from apparent surface features, as shown on the survey plan prepared by Bee & Lethbridge Pty Ltd (Drawing No. 21589, Revision No. 00, dated 19 March 2020). The approximate surface levels, as shown on the borehole logs and DCP test results sheet, were estimated by interpolation between spot levels shown on the survey plan and are therefore only approximate. The datum for the levels is Australian Height Datum (AHD), as noted on the survey drawing.

Selected samples were returned to Envirolab Services Pty Ltd (Envirolab), a NATA accredited laboratory, for pH, sulphate content, chloride content, and resistivity testing. The results are presented in the attached Envirolab Certificate of Analysis No. 282373.

Groundwater monitoring wells were installed in BH1 and BH4. Well construction details are shown on the attached borehole logs. On 1 November 2021 we used a bailer to remove water from the wells. We revisited the site on 26 November and 1 December 2021 to remeasure the wells. A summary of groundwater levels is provided in Section 3.2.

Our Associate, Mr Jarett Mones, visited the site on the morning of 29 October 2021 prior to the investigation to carry out an overall walkover and inspection. The investigation was carried out in the full-time presence of our geotechnical Engineer, Mr Quang Minh Vu under the management of Mr Mones, who set out the investigation locations, nominated the sampling and testing, and prepared logs of the strata encountered. The borehole logs (including core photographs) and DCP test results sheet, are attached to the report together with our Report Explanation Notes, which further describe the investigation techniques adopted, and define the logging terms and symbols used.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site description below should be read with reference to Figure 2 which includes the survey plan as the base plan and outcropping sandstone bedrock mapped during the fieldwork.

The site is located on a south facing hillside that falls towards Fairlight Beach.

The site is rectangular in shape with dimensions of 30.5m (north to south) by 40.2m wide (east to west). The site slopes/steps down at about 10° to 15° towards the south, with an elevation difference of about 8m to 11m.

The site encompassed two developed properties at the time of fieldwork.

On the eastern half is No. 33 Fairlight Street. At this property we observed the following:

- In the north-western corner at the street level a single storey rendered garage which appeared to be in fair condition.
- Along the northern boundary, a partially mortared sandstone block retaining wall, maximum 1.9m high, supports the footpath along Fairlight Street. The retaining wall appeared to be in poor condition based on the wall leaning towards the south at approximately 8° to 10°. The concrete footpath above appeared to be in good condition. Plate 1a is a photograph of this wall.
- Over the central portion, a one, two and three storey rendered clad, brick and stone house which appeared to be in fair to poor external condition. This was based on horizontal cracks up to approximately 2mm to 3mm wide and a detached portion of rendering (over an area of about 0.5m x 0.3m), which were both observed on the northern side of the house. Plate 1b includes two photographs of the northern side of the house.
- There were concrete paved footpaths around the house which appeared to be in poor condition based on longitudinal cracking.
- East and adjacent to the house was a brick retaining wall, maximum 1.3m high, that had failed. The wall had a crack about 50mm wide and was bulging towards the east.
- To the south and east of the house sandstone is outcropping and there are sandstone rock ledges, with heights of about 0.5m and 1.5m. The sandstone had indistinct rock fabric and was assessed to be of medium to high strength. There was an overhang about 0.6m deep in the lower 0.3m to 0.4m of the 1.5m high rock ledge.
- South of the rock ledges, in the rear yard area, there were dry stacked sandstone block walls, about 0.5m to 0.7m high which appeared to be in good condition.

On the western half of the site is No. 35 Fairlight Street. At this property we observed the following:

- Along the northern boundary, a mortared sandstone block retaining wall, maximum 2.4m high, which supports the footpath along Fairlight Street. The retaining wall appeared to be in good condition. The concrete footpath above appeared to be in good condition.
- In the front yard area, there were irregular composite rock retaining walls, maximum 0.9m high, and a mortared irregular sandstone block retaining wall, less than 1m high, which appeared to be in good condition. Near the western boundary there were sandstone rock ledges, maximum 1.7m high. The bedrock had indistinct rock fabric and was assessed to be of medium to high strength. Also, there were some loose blocks of rock in this area. Plate 2 includes two photographs of this area.
- Over the central portion, a one and two storey clad, brick and stone house which appeared to be in fair external condition.
- There were concrete paved footpaths around the house which appeared to be in fair condition.
- To the south of the house sandstone is outcropping and there is a sandstone rock ledge, 1.1m high. We observed one bedding parting dipping below horizontal at about 20°. The sandstone was assessed to be of medium to high strength.

The following observations were made along and across the eastern, southern and western boundaries.

At the eastern boundary of No. 33 we observed the following measured from the northern boundary:

- 0m to 3m – Concrete retaining wall, maximum 0.5m high supporting No. 31, which appeared to be in good condition.
- 3m to 14m – Levels were about the same across the boundaries.
- 14m to 40.2m - Concrete retaining wall, 0.2m to 0.5m high supporting No. 31, which appeared to be in good condition. One layer of sandstone blocks was at the base this wall along a portion of its length.

The neighbouring property to the east, No. 31 Fairlight Street, contained a two and three storey brick building, which was set back about 2m to 3m from the boundary and appeared to be in good external condition when viewed from within the site and Fairlight Street.

At the southern boundary of the combined sites, we observed the following measured from the western boundary:

- 0m to 12m – Retaining wall, maximum 1.4m high supporting No. 35 (subject site). The wall was vegetated but appeared to be a dry stacked sandstone block wall.
- 12m to 15m – Sandstone rock ledge, about 1.4m high with the property to the south being lower. Sandstone bedrock was observed in the lawn area in the property to the south, i.e., No. 12 Clifford Avenue.
- 15m to 30.5m – Dry stacked sandstone block retaining wall, about 0.7m high supporting No. 33 Clifford Avenue (subject site).

The neighbouring properties to the south included Nos. 14, 12 and 10 Clifford Avenue from west to east, respectively. Measured from the western boundary, No. 14 Clifford Avenue extended from 0m to 5m, No. 12 Clifford Avenue extended from 5m to 15m and No. 10 extended from 15m to 30.5m.

- No. 14 contained a storage shed with a corrugated metal roof at the boundary and a multi-storey brick house setback about 2m from the boundary. It was difficult to see these structures from the site but the portions visible appeared to be in good external condition.
- No. 12 contained a multi-storey rendered and stone house that was setback about 18m from the site.
- No. 10 contained a multi-storey rendered and stone building and a swimming pool, which were set back about 6m to 7m and 1m to 2m from the boundary, respectively. These structures appeared to be in good condition when viewed from within the subject site.

At the western boundary of No. 35, we observed the following measured from the northern boundary:

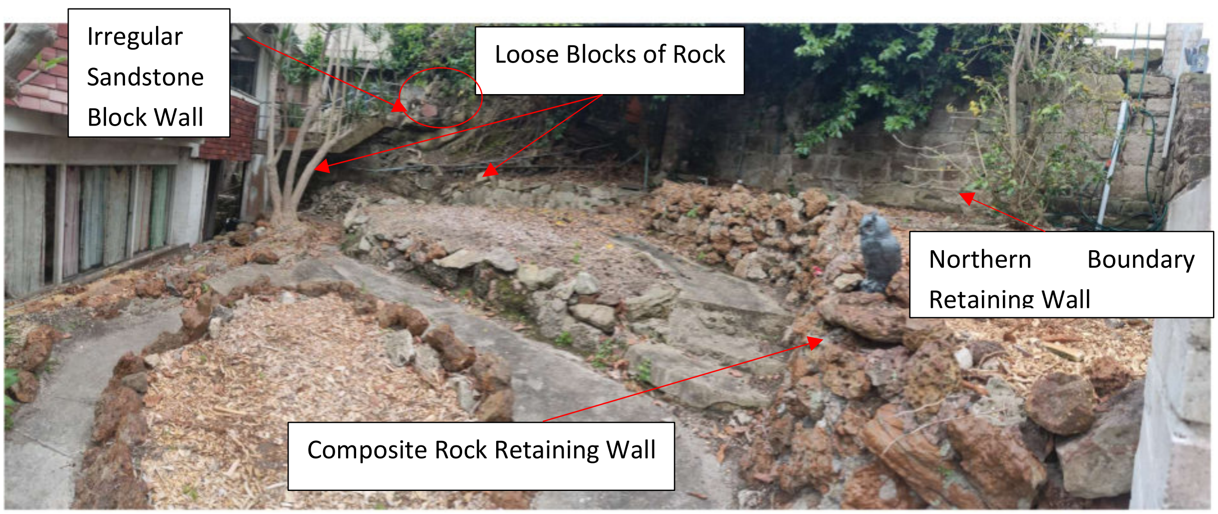
- 0m to 5m - Rendered brick retaining wall, maximum 1.8m high supporting No. 37, which appeared to be in good condition.
- 5m to 14m – Sandstone rock ledge maximum 1.7m high.
- 14m to 26m – Concrete retaining wall, 0.2m to 0.4m high, supporting No. 37.
- 26m to 40.2m - Irregular sandstone block retaining wall, maximum 0.4m high.

The neighbouring property to the west, No. 37 Fairlight Street, contained a two and three storey rendered, clad and brick unit building, set back about 1m from the boundary. The building appeared to be in good external condition when viewed from the site and Fairlight Street.

Plate 1a: Northern Boundary Partially Mortared Sandstone Block Retaining Wall

Plate 1b: Northern Side of House, No. 33 (2 Photographs)

Plate 2: Loose Blocks of Rock and a Mortared Irregular Sandstone Block Retaining Wall (2 Photographs)



3.2 Geology and Subsurface Conditions

Reference to the Sydney 1:100,000 Geological Series Sheet indicates that the site is underlain by Hawkesbury Sandstone of the Wianamatta Group. Hawkesbury Sandstone comprises medium to coarse grained quartz sandstone with very minor shale and laminite lenses.

The investigation disclosed subsurface conditions generally comprising sandy and clayey fill that in turn overlay sandstone bedrock at relatively shallow depths. Some of the more pertinent details of the strata encountered are described below. For further details of the conditions encountered at a particular test location, reference should be made to the attached borehole logs and DCP test results sheet.

Fill

Fill encountered in the boreholes comprised silty sand and sandy clay and extended to depths of 0.82m (BH1) and 0.3m (BH4). In BH1, hand auger refusal occurred at a depth of 0.82m within the fill (and DCP refusal occurred at a depth of 0.8m) and then 'No Core' was retrieved between depths of 0.82m and 0.92m; we have inferred that at this location refusal occurred on the extremely weathered sandstone, but it is possible that the fill extends to a depth of 0.92m.

It is likely that fill was present above the refusal depths in the DCP tests carried out at locations 2, 3 and 5 to 7, though this cannot be confirmed. There may be pockets of residual soil where the rock is deeper.
The fill was assessed to be poorly compacted.

Sandstone Bedrock

Sandstone bedrock was encountered (BH4) and inferred (BH1/DCP1) at the hand auger and DCP refusal depths of about 0.3m (BH4) to 0.82m/0.8m (BH1/DCP1). At BH1, the initial 100mm where there was 'no core', i.e. depth range of 0.82m to 0.92m, the sandstone was inferred to be extremely weathered and of soil strength and was likely washed away during the coring process. It is possible that the top of the bedrock at this location is actually at a depth of 0.92m.

The sandstone bedrock in BH4 and in BH1 from a depth of 0.92m was assessed to be moderately weathered, slightly weathered and fresh and generally of medium strength. In BH4, a band of high strength sandstone was encountered between a depth range of about 2.65m and 2.82m, and medium to high strength sandstone was encountered at a depth of 7.8m.

Based on the DCP blow counts, sandstone was inferred at depths of about 0.85m at DCP2, 0.505m at DCP3, 0.2m at DCP5, 1.5m at DCP6 and 1.55m at DCP7. As noted earlier, the refusal depth of DCP tests can also occur on obstructions in fill, 'floaters' and other hard layers within the soil profile.

Sandstone bedrock is outcropping on the site as shown on Figure 2. Where outcropping the sandstone bedrock was generally indistinctly bedded and was assessed to be of medium to high strength.

Groundwater

At BH1 and BH4, the boreholes were dry on completion of hand augering to depths of 0.82m and 0.3m, respectively. We note that water is introduced into the boreholes during core drilling and therefore the water levels after coring are typically artificially high. So, following core drilling we installed monitoring wells so that further measurements could be made. The following is a summary of groundwater notes and measurements:

- BH1 (Surface RL˜41.2m)
    - 1 November 2021 – After three days of drilling and installation of the well groundwater was measured at a depth of 1.7m (˜RL39.5m). We used a bailer to remove water and lowered the water level to a depth of 5.7m (˜RL35.5m).
    - 26 November 2021 – About 3½ weeks after drilling and bailing the water to the depth above groundwater was measured at a depth of 0.28m (˜RL40.9m).
    - 1 December 2021 – About 4 weeks after drilling and bailing the water to the depth above groundwater was measured at a depth of about 1.4m (˜RL39.8m).
    - Based on the measurements and observations above we anticipate that the water levels were influenced by stormwater seepage through the fill.

- BH4 (Surface RL~46.8m)
  - 1 November 2021 - Following drilling and installation of the well groundwater was measured at a depth of 2.34m (~RL44.5m). We used a bailer to remove water continuously for 30 minutes and water was measured at a depth of 2.74m (~RL44.1m).
  - 26 November 2021 – About 3½ weeks after drilling groundwater was measured at a depth of about 4.5m (~RL42.3m).
  - 1 December 2021 – About 4 weeks after drilling groundwater was measured at a depth of about 4.6m (~RL42.2m).

3.3 Laboratory Test Results

The results of the Point Load Strength Index tests carried out on the recovered rock cores from each borehole correlated well with our field assessment of bedrock strength. Point Load Strength Index ($I_{s(50)}$) tests ranged from 0.3MPa to 1.6MPa. These are also plotted on the attached borehole logs. Estimated unconfined compressive strength (UCS), based on the relationship of UCS = 20 x $I_{s(50)}$), ranged from 6MPa to 32MPa.

The results of the pH, sulphate, chloride and resistivity tests are summarised in the table below. Envirolab Certificate of Analysis No. 282373 is attached and presents the details of these tests. Reference should be made to Section 4.8 for potential impact of the aggression on concrete and steel structures in contact with the ground.

Borehole	Depth (m)	Sample Type	pH	Sulphates SO_4 (ppm)	Chlorides Cl (ppm)	Resistivity ohm.cm
BH1	0.6-0.8	Fill: Sandy Clay	6.4	<10	<10	40,000
BH4	0.1-0.2	Fill: Silty Sand	6.0	<10	<10	22,000
BH4	0.3-0.4	Sandstone	7.0	<10	<10	66,000

4 COMMENTS AND RECOMMENDATIONS

4.1 Adjoining Properties and Dilapidation Surveys

Dilapidation surveys of adjoining buildings and structures that fall in the area of influence of the excavation are a necessary part of the process of claim protection, i.e., avoiding spurious claim of damage where, in fact, the damage existed prior to excavation or demolition commencing. Consequently, prior to demolition and excavation commencing, we recommend that detailed dilapidation reports be compiled on buildings and structures that fall within the zone of influence of the excavation. The zone of influence regarding excavation for dilapidation surveys may be defined by a distance back from the excavation perimeter of twice the depth of the excavation. At a minimum this should include Nos. 31 and 37 Fairlight Street, the northern boundary retaining wall, footpath and Fairlight Street. The dilapidation surveys should comprise detailed inspections of the adjoining properties and structures (including retaining walls), both externally and internally, with all defects rigorously described, i.e., defect location, defect type, crack width, crack length, etc. The respective owners should be asked to confirm in writing that these reports represent a fair record of actual conditions. These reports should be carefully reviewed prior to excavation commencing to ensure that appropriate equipment is used. In particular, the size/energy of the rock impact breakers should be considered.

4.2 Excavation and Hydrogeology

A detailed demolition, excavation and retention methodology should be developed by the builder/excavation contractor and approved 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.

We understand that excavation to a maximum depth of about 10m will be required. There will be deeper localised excavations for buried services, subsurface drainage and possibly an OSD tank. The proposed ground floor level footprint is shown on Figure 2. The perimeter walls are shown to be set-in about 3m from the eastern and western boundaries and about 1.5m to 4.5m and 8m from the northern and southern boundaries, respectively. Typically, excavations are carried out closer (at least 1m) to the boundaries to allow for construction of the building walls.

Since the northern boundary retaining wall will likely be supported on a strip footing the footing details should be investigated prior to any excavation being carried out so that the feasibility of temporary batters can be reviewed, as discussed in more detail in Section 4.3. The investigation should include a minimum of four small test pits that are inspected by the geotechnical and structural engineers. Subsequent to this investigation, confirmation on whether temporary batters are suitable can be provided by the geotechnical engineer. In addition, it is important that excavation does not compromise the stability of the walls. A structural engineer should review the stability of the existing walls 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 (Ref: Plate 1a), which is leaning towards the site, be temporary propped or replaced prior to excavations. There is also an irregular sandstone block wall and loose blocks of rock at the north-western portion of the site (Ref: Plate 2) which should be supported or removed prior to excavation. We note that in the long term the boundary walls will all be replaced.

Excavation will result in the removal of some soil, generally silty sand and sandy clay fill but probably some natural soils including extremely weathered sandstone, but mainly sandstone bedrock of medium to high strength. Where sandstone bedrock of low strength or greater is encountered "hard rock" excavation conditions will be encountered and will require the adoption of either percussive or non-percussive rock excavation techniques. A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. We can provide the appropriate testing if required.

Excavation of the soils and extremely weathered sandstone 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.

"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. Consequently, we recommend that considerable caution be exercised and that the excavation procedures and the dilapidation reports be carefully reviewed prior to excavation commencing so that appropriate equipment is used. We recommend that the following measures be taken:

- During percussive excavation continuous quantitative vibration monitoring should be completed. Vibration monitors should ideally be attached to the adjoining structures closest to the location of the percussive excavation. Where non-percussive excavation techniques are adopted, we consider that only an initial quantitative assessment of vibrations would be necessary at the commencement of rock excavation. Further continuous vibration monitoring is not likely to be required provided non-percussive techniques are employed.
- Percussive excavation should be completed so that the excavation is progressively enlarged by breaking small wedges out of the face.
- Rock hammers should only be operated in short bursts to prevent amplification of vibrations.
- Where transmitted vibrations exceed prescribed limits, excavation techniques must be altered to reduce transmitted vibrations to within acceptable limits. This may mean that the size of percussive equipment used may need to be reduced or non-percussive techniques adopted. Whether reducing the size of the percussive equipment is effective in controlling transmitted vibrations must be confirmed by further quantitative vibration monitoring.

The prescribed vibration limits that should be adopted on this site are set out in the Vibration Emission Design Goals attached to the rear of this report.

Based on the site geometry, levels and groundwater measurements and previous experience in the area we anticipate that the elevated water levels recorded in BH1 were impacted by stormwater seepage through the fill. In BH4 the water level stabilised between depths of 4.5m (~RL42.3m) and 4.6m (~RL42.2m) after about 3½ to 4 weeks. We anticipate that during the excavation initially groundwater water seepage will be higher at about this level, and at the interface between the soils and bedrock, and will likely slow over time. However, higher groundwater seepage flows along the soil/rock interface and through defects within the rock mass typically occur during and immediately following periods of wet weather. As groundwater seepage will be encountered during excavation works submersible pumps should be kept available on site for use as required.

We can carry out longer term groundwater monitoring, infiltration testing and groundwater seepage analysis to assess the impact rainfall has on groundwater levels and to estimate groundwater extraction volumes during construction. Alternatively, we recommend that measurements of the groundwater seepage rates are carried out during construction so that the drainage design can be confirmed. We recommend that an inspection by a hydraulic engineer, during construction and/or once the bulk excavation has been carried out to confirm the drainage design.

4.3 Retention

The proposed walls are shown to be set-in 3m from the eastern and western boundaries, at the closest point 1.5m from the northern boundary and 8m from the southern boundary. Based on the depths of the soils overlying the medium to high strength sandstone bedrock encountered in the boreholes and depths inferred from the DCP tests and 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 low strength are greater, are likely to be feasible. The feasibility of temporary batters along the northern boundary will need to reviewed following the recommended test pit excavations along the northern boundary wall.

As previously discussed, we recommend the northern boundary wall at No. 33 (Ref: Plate 1a), which is leaning towards the site, be temporary propped/anchored or replaced prior to excavations. There is also an irregular sandstone block wall and loose blocks of rock at the north-western portion of the site (Ref: Plate 2) which should be supported or removed prior to excavation. The likely support is to include shotcrete, mesh and dowels/bolts.

Where space allows and provided movement sensitive structures are not located within 1m from the crest, temporary batters formed through sand may be formed at 1 Vertical (1):1.5 Horizontal (H) and through clay and sandstone bedrock of less than low strength at 1V:1H. The geotechnical engineer should inspect the temporary batters during construction to confirm that the temporary batters are not cut steeper. Based on the borehole results and maximum depth of excavation of about 10m, 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.

Sandstone bedrock of low strength or greater should be able to be cut vertically and left unsupported provided it is free from adverse defects. 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 if walls will not be backfilled against the cuts in the permanent case, grubbing out and dry packing weaker seams. Considering the scale of the proposed excavation in sandstone bedrock some provision should be made for stabilisation of the bedrock. Approvals to install rock bolts beyond boundaries will be required. Backfill must be placed as engineered fill (Ref: Section 4.6) or alternatively a free draining single sized aggregate such as a 'blue metal' may be used.

Where temporary batters are not suitable or not preferred, such as along the northern side where the building footprint is close to the northern boundary wall, an alternative shoring system could comprise construction of low-height, mass concrete walls dowelled into the underlying bedrock. This option is considered to be more practical and economical than a contiguous pile wall due to the shallow depth of the underlying bedrock. Locally steeper batters than recommended above may need to be adopted to form the walls and in this regard the construction will need to be carried out incrementally, say in 2m to 3m lengths. We note that there are deeper soils, to a depth of about 1.5m over about a length of 7m, along the southern end of the eastern side. Where an alternative shoring option, such as a contiguous pile wall, is preferred in this area or other areas please contact us for further advice.

For the design of temporary shoring walls, new landscaping walls or to re-build the leaning northern boundary wall at No. 33, the parameters provided in the table below may be used where computer based design using packages such as Wallap, Plaxis, etc. are used. Alternatively, a triangular earth pressure distribution and a coefficient of lateral earth pressure, k_a, of 0.35 may be adopted for areas that are not highly sensitive to lateral movement and that will not be propped in the permanent case. The coefficient of at-rest lateral pressure, 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. Minimum surcharge loads of 5kPa and 10kPa should be adopted for the footpath and road, respectively, above the northern boundary wall. 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.

For the design of the new walls the parameters in the following table may be adopted for the existing fill and natural soil including extremely weathered sandstone.

Preliminary Design Parameters			
Material Unit	**Unit Weight (kN/m^3)**	**Drained Cohesion (c') (kPa)**	**Drained Angle of Friction (Ø') (degrees)**
Existing Fill/ Natural Soils	20	0	28

4.4 Footings

The unaltered site as seen is classified as 'Class P' in accordance with AS2870-2011 due to the presence of uncontrolled fill. The fill is considered unsuitable as a bearing stratum or supporting subgrade for footings, slabs and pavements. Reference should also be made to AS2870-2011 for design, construction, performance criteria and maintenance precautions on 'Class P' sites.

We recommend that all footings be uniformly founded on sandstone bedrock. As the basement excavation is anticipated to uniformly expose sandstone bedrock, shallow pad/strip footings would likely be adequate for the building loads. Pile footings will be required for support of the OSD overflow structure at the south-eastern corner of the site, and possibly at portions of the paved structures south of the building (i.e., bin collection area and area between the OSD overflow structure and the building) if sandstone bedrock is deeper than about 1m below bulk excavation levels. Footings founded on the underlying sandstone bedrock of at least medium strength may be designed for an allowable bearing pressure (ABP) of 1500kPa. Where pile footings are adopted, we recommend they be socketed a minimum of 0.3m into the sandstone bedrock.

A higher allowable bearing pressure of say 3000kPa may be justifiable provided an additional minimum of two cored boreholes are carried out to confirm the bedrock quality and strength.

Where footings are to be located within H of the crest of vertical cuts in sandstone bedrock (where H is the height of the cut), close inspection of the cut faces by a geotechnical engineer for the presence of adverse defects will be required. Redesign of footings or further excavation may be required so that the footings are founded below adverse defects within the sandstone. Where possible footings should be designed to be founded below a line drawn upwards at 1H:1V from the base of any vertical cut, which may be required for footings located just outside of the car lift, lift pit, OSD tank, services, etc.

4.5 Slabs on Grade

We anticipate sandstone bedrock will be exposed at bulk excavation level for the building. All on-grade slabs for the building that are poured directly over sandstone should be provided with an underfloor drainage blanket. The underfloor drainage should comprise a strong, durable, single sized washed aggregate, such as 'blue metal' gravel. The underfloor drainage should collect groundwater seepage and direct it by gravity flow to the stormwater system.

As there is poorly compacted fill to the south of the building, we do not recommend the new pavements in these areas be supported on these materials. Where not designed as fully suspended structures, where existing fill is present, we recommend that these materials are removed and replaced as engineered fill. A geotechnical engineer should inspect the subgrade at bulk excavation to confirm that the fill is removed and where natural soils are present carry out DCP tests to review the soil subgrade strength or alternatively these soils should be proof rolled.

4.6 Engineered Fill

Where a free draining single size aggregate, such as a 'blue metal', is not placed behind retaining walls or uncontrolled fill forms the subgrade to driveways and paths, engineered fill should be placed. Materials preferred for use as engineered fill are well-graded granular materials, such as crushed sandstone, free of deleterious substances and having a maximum particle size not exceeding 75mm. The excavated existing fill may be reused as engineered fill, provided it is free of deleterious materials and particles greater than 75mm in size. Fill containing vegetation and organic matter would not be suitable for reuse as engineered fill.

Engineered fill should be compacted in horizontal layers, to a density of at least 98% of Standard Maximum Dry Density (SMDD). Lift thickness will be controlled by the equipment used and can be trialled during the initial compaction tests. As a guide layer thickness should be limited to 100mm loose thickness. We note that compaction of fill to these standards is difficult on small sites such as this and care must be taken that engineered fill is placed to a high standard.

Density tests should be regularly carried out on the engineered fill to confirm the above specifications are achieved. The frequency of density testing behind the retaining walls should be at least one test per two layers per 40 linear metres or a minimum of 3 tests per visit, and within pavement areas south of the building should be at least one test per each layer or a minimum of 3 tests per visit.

4.7 Aggressivity

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

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

4.8 Further Geotechnical Input

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

- Prepare dilapidation reports on the residences within the zone of influence of the proposed excavation. We recommend, at a minimum, that this includes Nos. 31 and 37 Fairlight Street and the retaining wall, footpath and road to the north.
- Approval of a detailed demolition, excavation and retention methodology prior to commencement of the site works.
- Carry out a minimum of four test pits adjacent to the existing northern boundary wall to assess existing footing details, so that the suitability of temporary batters in this area can be reviewed.
- Carry out continuous vibration monitoring where percussive excavation techniques are adopted and at least some initial vibration monitoring where non-percussive excavation techniques are adopted. If the builder carries out the monitoring the test results should be forwarded to the geotechnical engineer on a regular basis.
- Inspection of temporary batters.
- 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.
- Inspection and testing of subgrade for pavements proposed to the south of the building, where these structures are not designed to be fully suspended.
- Testing of engineered fill by a geotechnician.

5 GENERAL COMMENTS

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

The subsurface conditions between the completed investigation locations 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.

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

Borehole No.
1

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, 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 26/11/2021	DS	REFER TO DCP TEST RESULTS	41	0			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
	DS						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							

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 34479SJ FAIRLIGHT.GPJ <<DrawingFile>> 06/01/2022 13:24 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

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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						
0% RETURN 1/12/2021		40	1		NO CORE 0.10m						Hawkesbury Sandstone
1/11/2021				▒	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	
100% RETURN		39, 38	2, 3	▒	as above, but trace of carbonaceous lenses and quartz clasts.			0.50, 0.70, 0.40		(1.99m) Cr, 0°, 10 mm.t; (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						

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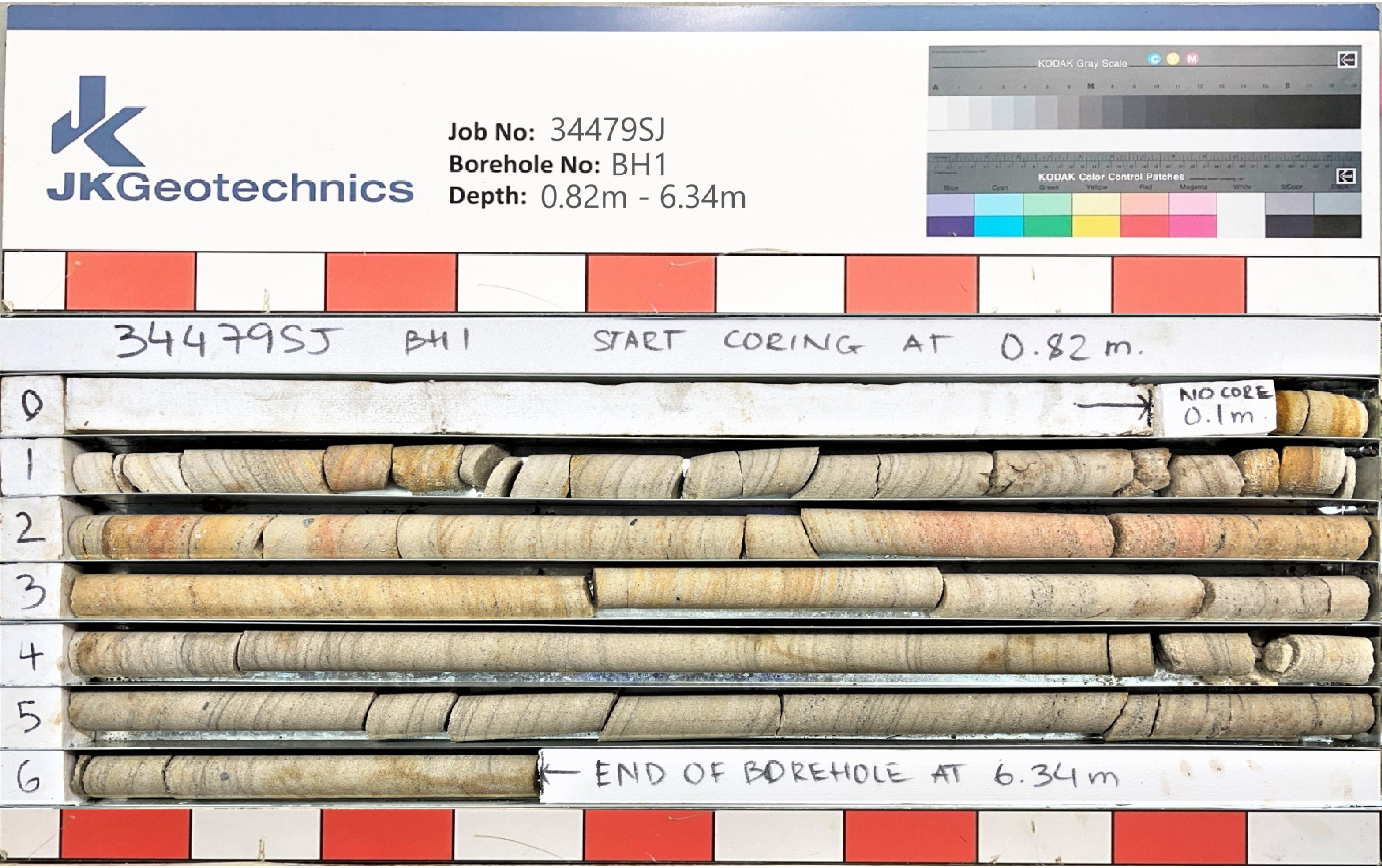
FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


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

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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
										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
						46	1							
						45	2							
						44	3							
						43	4							
						42	5							
						41	6							
						40								

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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 $I_s(50)$	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness (Specific / General)	Formation
					START CORING AT 0.30m					
90% RETURN		46	1		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	0.70, 0.60, 0.50, 0.60	(0.65m) Be, 0°, P, R, Cn; (0.97m) Bex2, 0°, P, R, Fe Sn; (1.10m) Be, 0°, P, R, Fe Sn	Hawkesbury Sandstone
ON COMPLETION OF CORING 70% RETURN		45, 44	2		as above, but with occasional ironstone bands.		M, H (approx. 2.65–2.80m), M	0.80, 1.6	(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	
26/11/2021, 1/12/2021		43, 42, 41	3, 4, 5, 6		SANDSTONE: fine to medium grained, light grey with yellow brown and red brown lamination, distinctly bedded at 0-20°.		M	0.60, 0.70, 0.70, 0.90, 0.50, 0.80, 0.80	(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	
80% RETURN		40			SANDSTONE: fine to medium grained, light grey and red brown with yellow brown laminae, bedded at 0-20°.	MW		0.70		

Point load scale: VL -0.1, L -0.3, M -1, H -3, VH -10, EH. Spacing (mm): 600, 200, 60, 20.

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JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 34479SJ FAIRLIGHT.GPJ <<DrawingFile>> 06/01/2022 13:25 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

CORED BOREHOLE LOG

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)	Core Description (Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components)	Weathering	Strength	Point Load Strength Index $I_s(50)$	Defect Details – Specific (Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness)	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
				SANDSTONE: fine to medium grained, light grey with grey and yellow brown laminae, trace of carbonaceous lenses, distinctly bedded at 0-20°.	FR		1.2; 1.4; 1.3	(9.30m) Be, 10°, P, R, Cb Vn; (9.87m) Be, 5°, P, R, Cn	
80% RETURN		37, 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°.			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	
		33	13	END OF BOREHOLE AT 12.86 m					

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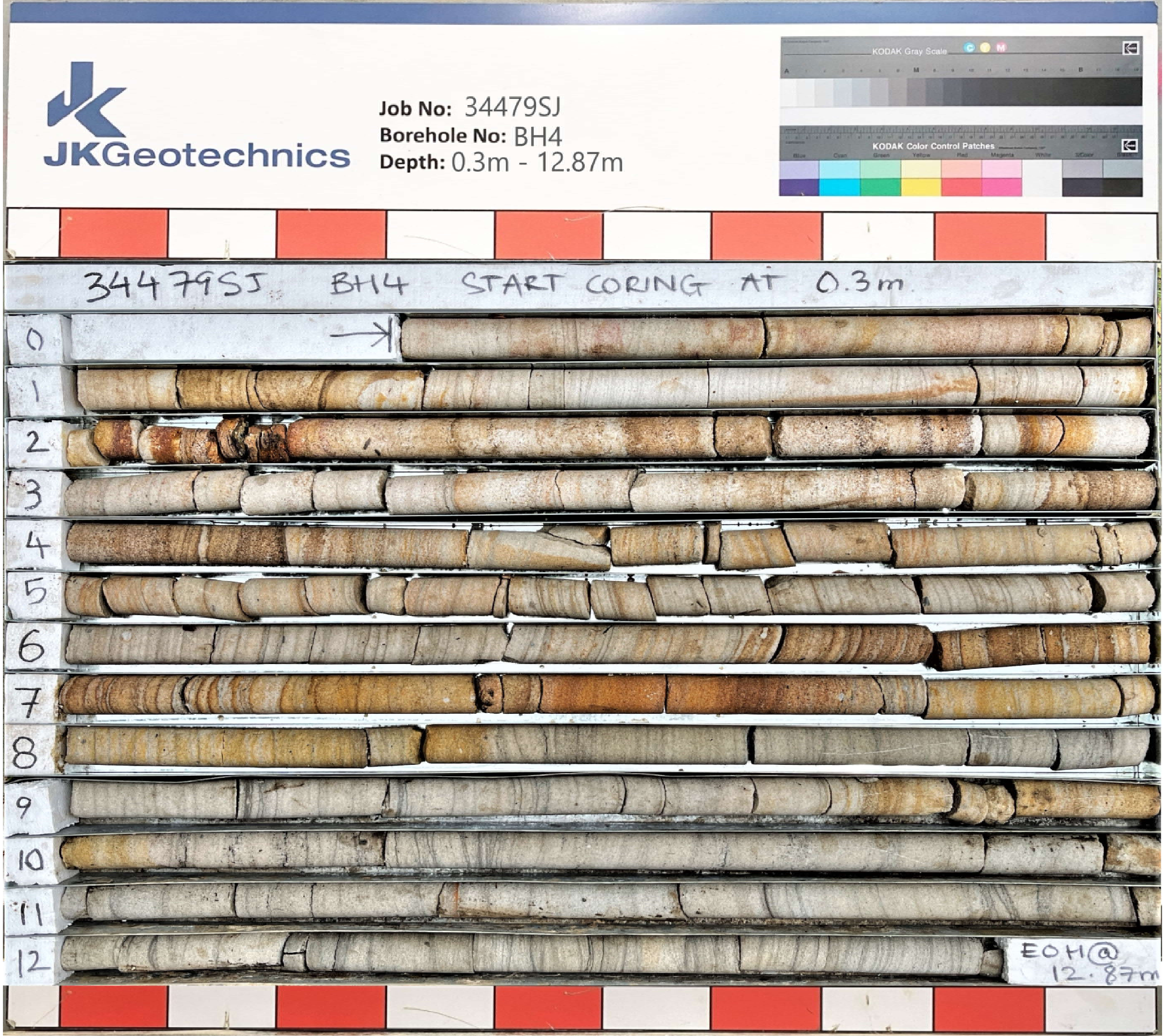
JKGeotechnics
Job No: 34479SJ
Borehole No: BH4
Depth: 0.3m - 12.87m
KODAK Gray Scale
KODAK Color Control Patches
34479SJ BH4 START CORING AT 0.3m
0
1
2
3
4
5
6
7
8
9
10
11
12
EOH@ 12.87m

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

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:	**SITE LOCATION PLAN**
Location:	33-35 FAIRLIGHT STREET, FAIRLIGHT, NSW
Report No:	34479SJ
Figure No:	1

JKGeotechnics

PLOT DATE: 2/11/2021 4:08:06 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\34000'S\34479SJ FAIRLIGHT\CAD\34479SJ.DWG

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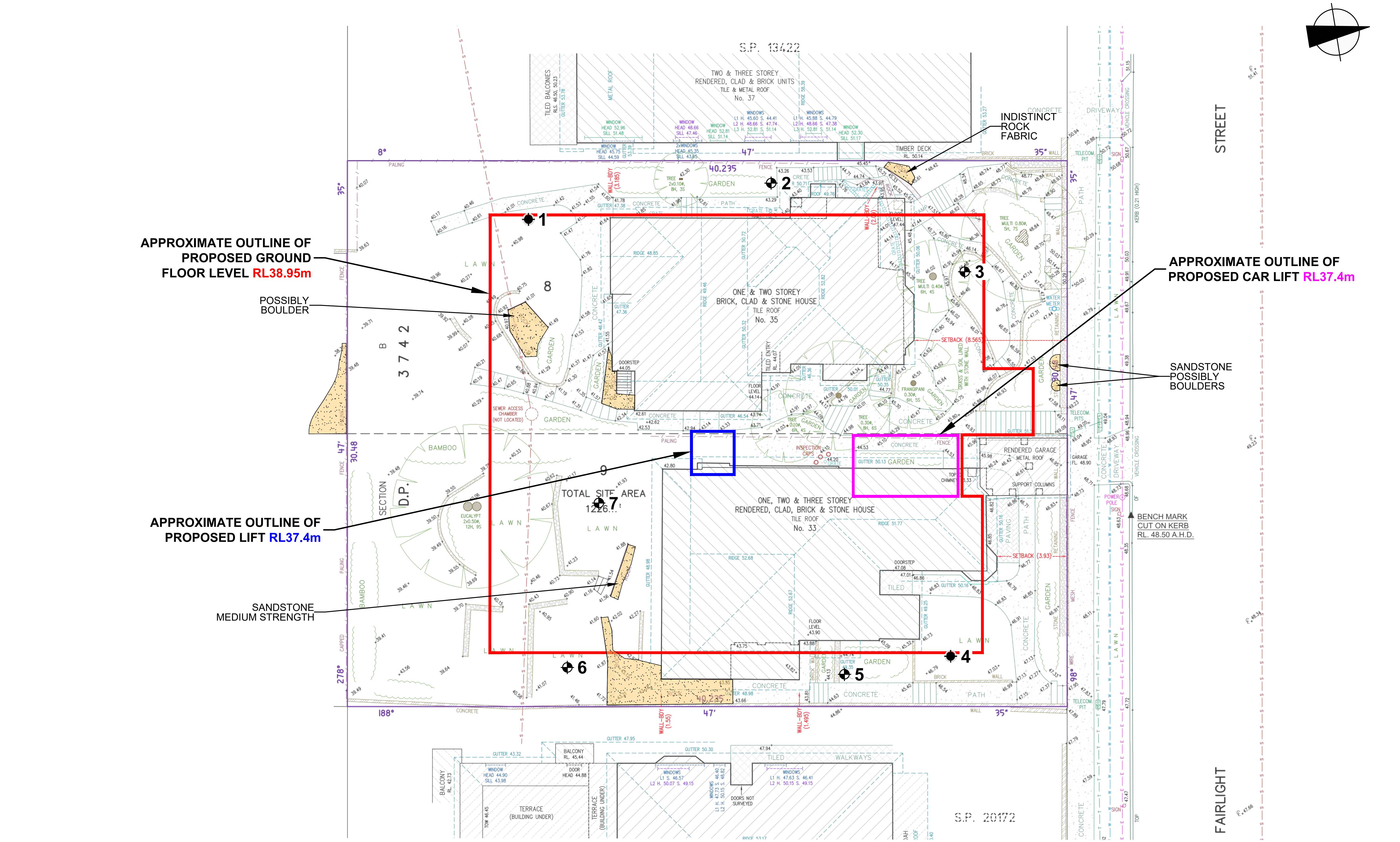

LEGEND

- BOREHOLE AND DCP TEST
- DCP TEST
- SANDSTONE OUTCROP

SCALE 0 2 4 6 8 10 METRES 1:200 @A3

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

Title:	INVESTIGATION LOCATION PLAN
Location:	33-35 FAIRLIGHT STREET, FAIRLIGHT, NSW
Report No:	34479SJ
Figure No:	2

JKGeotechnics

PLOT DATE: 25/11/2021 10:44:51 AM DWG FILE: J:\6F GEOTECHNICAL JOBS\34000'S\34479SJ FAIRLIGHT\CAD\34479SJ.DWG

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VIBRATION EMISSION DESIGN GOALS

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

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

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

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

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

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

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

REPORT EXPLANATION NOTES

INTRODUCTION

These notes have been provided to amplify the geotechnical report in regard to classification methods, field procedures and certain matters relating to the Comments and Recommendations section. Not all notes are necessarily relevant to all reports.

The ground is a product of continuing natural and man-made processes and therefore exhibits a variety of characteristics and properties which vary from place to place and can change with time. Geotechnical engineering involves gathering and assimilating limited facts about these characteristics and properties in order to understand or predict the behaviour of the ground on a particular site under certain conditions. This report may contain such facts obtained by inspection, excavation, probing, sampling, testing or other means of investigation. If so, they are directly relevant only to the ground at the place where and time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726:2017 *'Geotechnical Site Investigations'*. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached soil classification table qualified by the grading of other particles present (eg. sandy clay) as set out below:

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

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

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

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	Strength not attainable – soil crumbles	

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'shale' is used to describe fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

SAMPLING

Sampling is carried out during drilling or from other excavations to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and withdrawing it with a sample of the soil contained in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shrink-swell behaviour, strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling used are given on the attached logs.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application. All methods except test pits, hand auger drilling and portable Dynamic Cone Penetrometers require the use of a mechanical rig which is commonly mounted on a truck chassis or track base.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils and 'weaker' bedrock if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for a large excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of limited reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table.

Rock Augering: Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock cuttings. This method of investigation is quick and relatively inexpensive but provides only an indication of the likely rock strength and predicted values may be in error by a strength order. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be assessed from the cuttings, together with some information from "feel" and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term 'mud' encompasses a range of products ranging from bentonite to polymers. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg. from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils, as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63.5kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

- In the case where full penetration is obtained with successive blow counts for each 150mm of, say, 4, 6 and 7 blows, as

  N = 13
  4, 6, 7

- In a case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm, as

  N > 30
  15, 30/40mm

The results of the test can be related empirically to the engineering properties of the soil.

A modification to the SPT is where the same driving system is used with a solid 60° tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid Cone Penetration Test (SCPT) are shown as 'N_c' on the borehole logs, together with the number of blows per 150mm penetration.

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

In the tests, a 35mm or 44mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck. The CPT does not provide soil sample recovery.

As penetration occurs (at a rate of approximately 20mm per second), the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

  The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Correlations between CPT and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of CPT values can be made to empirically derive modulus or compressibility values to allow calculation of foundation settlements.

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive. The test method provides a continuous profile of engineering properties but, where precise information on soil classification is required, direct drilling and sampling may be preferable.

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than 'straight line' variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

Where groundwater levels are measured in boreholes, there are several potential problems:

- Although groundwater may be present, in low permeability soils it may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes and may not be the same at the time of construction.
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must be washed out of the hole or 'reverted' chemically if reliable water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after the groundwater level has stabilised at intervals ranging from several days to perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from perched water tables or surface water.

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg. bricks, steel, etc) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably assess the extent of the fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is of importance to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building) the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

Reasonable care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.
- Details of the development that the Company could not reasonably be expected to anticipate.

If these occur, the Company will be pleased to assist with investigation or advice to resolve any problems occurring.

SITE ANOMALIES

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. Licence to use the documents may be revoked without notice if the Client is in breach of any obligation to make a payment to us.

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

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

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

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL	ROCK
FILL	CONGLOMERATE
TOPSOIL	SANDSTONE
CLAY (CL, CI, CH)	SHALE/MUDSTONE
SILT (ML, MH)	SILTSTONE
SAND (SP, SW)	CLAYSTONE
GRAVEL (GP, GW)	COAL
SANDY CLAY (CL, CI, CH)	LAMINITE
SILTY CLAY (CL, CI, CH)	LIMESTONE
CLAYEY SAND (SC)	PHYLLITE, SCHIST
SILTY SAND (SM)	TUFF
GRAVELLY CLAY (CL, CI, CH)	GRANITE, GABBRO
CLAYEY GRAVEL (GC)	DOLERITE, DIORITE
SANDY SILT (ML, MH)	BASALT, ANDESITE
PEAT AND HIGHLY ORGANIC SOILS (Pt)	QUARTZITE

OTHER MATERIALS

BRICKS OR PAVERS

CONCRETE

ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory Classification Criteria

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

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

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

NOTES:

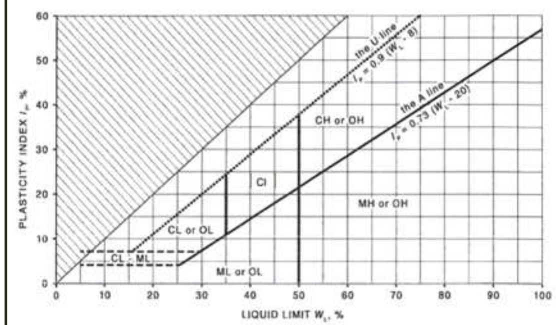
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

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

Log Column	Symbol	Definition	Density Index (I_D) Range (%)	SPT 'N' Value Range (Blows/300mm)
Density Index/ Relative Density (Cohesionless Soils)	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

			Guide to Strength	
Term	**Abbreviation**	**Uniaxial Compressive Strength (MPa)**	**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