SPECIALIST ADVICE TO

PATHWAYS RESIDENCES CREMORNE

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED SENIORS LIVING DEVELOPMENT

AT

50-88 PARRAWEEN STREET AND
59-67 GERARD STREET, CREMORNE, NSW

Date: 4 July 2023
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1 INTRODUCTION

This specialist advice report presents the results of a geotechnical investigation for the proposed seniors living development at 50-88 Parraween Street and 59-67 Gerard Street, Cremorne, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Graeme Skerritt of Pathways Residences Cremorne and was carried out in accordance with our proposal dated 20 December 2023, Ref: P57914S.

We were provided with the following documents for the preparation of our report:

- Development Application drawings prepared by Morrison Design Partnership (Project: Pathways Residences Cremorne, dated 16 June 2023)
- Survey plan prepared by Geosurv (Drawing No. 220107-DT-01 [A], dated 4 August 2022.

From the above supplied documents, we understand that following the demolition of the existing houses and apartment building, four buildings of between four and eight levels over two basements will be constructed for a seniors living development. The lowest basement level has a proposed finished floor level (FFL) of 76.8m, which will require excavation to a maximum depth of about 9m. Localised deeper excavation may be required for lift pits etc. The basement typically extends to within 3m to 7m of the boundaries except along most of the street frontage with Parraween Street where the basement will extend up to the boundary and in the south-western and north-eastern corners of the Parraween Street portion where the setbacks are between 9m and 12m.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions, and to use this as a basis for providing comments and recommendations on geotechnical aspects of the proposed development, such as excavation, shoring, batter slopes, earth pressures, vibration control, foundations, groundwater and subgrade preparation.

This geotechnical investigation was carried out in conjunction with an environmental site assessment by our environmental division, JK Environments (JKE). Reference should be made to the separate report by JKE, Ref: E35736PTrpt, for the results of the environmental site assessment.

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

2 INVESTIGATION PROCEDURE

The investigation comprised the drilling of four boreholes with our tracked JK205 drilling rig. The boreholes were initially drilled using spiral auger techniques with an attached tungsten carbide (TC) bit. Once bedrock was encountered the boreholes were extended to depths ranging from 10m to 15m using diamond coring techniques with water flush. Once core drilled, the boreholes were reamed out for groundwater well installation. In each borehole a 50mm Class 18 PVC monitoring well was installed; details of each installation are provided on the attached borehole logs. At BH3 the drilling was interrupted at 5.8m depth and so when drilling was restarted about 1 month later a new borehole was open-hole drilled to 5.8m depth immediately adjacent to the first borehole and continued by diamond coring to the planned termination depth of 11.24m; the borehole log is differentiated as BH 3A.

The borehole locations, as shown on Figure 2, were set out by tape measurements from existing surface features. The approximate surface levels of the borehole logs were interpolated from spot heights and contours shown on the survey plan prepared by Geosurv surveyors. The height datum is Australian Height Datum (AHD).

The strength of the soils and degree of compaction of the fill were assessed from the Standard Penetration Test (SPT) 'N' values, which were augmented by hand penetrometer readings completed on cohesive samples recovered from the SPT split tube sampler. The strength of the bedrock in the augured portion of the boreholes was assessed from the observation of drilling resistance and examination of rock cuttings recovered from the augers. It should be noted that rock strengths assessed in this way are approximate and variation of one order of strength should not be unexpected.

The recovered core of the bedrock was photographed and tested to determine point load strength index values (I_{s50}). Using established correlations, the unconfined compressive strength (UCS) of the bedrock was then calculated. Photographs of the core are presented with the borehole logs while the Point Load Strength Index test results are summarised on the borehole logs and presented in Table A.

Groundwater observation were made in the boreholes during and on completion of drilling. The water levels in the wells were all measured on 27 February 2023, about 1 to 6 weeks after drilling was completed, and the relevant details are recoded on the borehole logs.

Selected samples were returned to Envirolab Services Pty Ltd for pH, chloride content, sulphate content and resistivity testing. The results of this testing are presented on the attached Envirolab Certificate of Analysis 314797.

Our engineering geologist, Mr Tom Foster, was present full-time during the fieldwork to set out the borehole locations, direct the buried services scan, log the encountered subsurface profile and nominate in-situ testing and sampling. The borehole logs (which include groundwater observations) are attached, together with a glossary of logging terms and symbols used. Further details of the methods and procedures employed in the investigation are presented in the attached Report Explanation Notes.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located on top of a ridgeline that is relatively level with no major slope changes across the site. The site consists of multiple properties (50-88 Parraween Street and 59-67 Gerard Street) that have a south-eastern frontage onto Parraween Street and north-western frontage onto Gerard Street.

At the time of the investigation, the site was occupied by a four-storey brick apartment building and multiple single storey brick residential houses that appeared to be in poor to fair condition when viewed externally. The properties on Parraween Street contained either concrete or paved driveways (with the exception of 54 Parraween Street) which all appeared to be fair to good condition. The four-storey apartment building located within the site on Gerard Street contained a garage on the ground floor level with a concrete driveway and external carpark.

To the north-west of the site is Gerard Street that slopes down to the north-east at about 2-3°. On the opposite side of Gerard Street was the edge of the ridge crest and the topography changes with moderate falls towards the north. North-east of the site (81a-81b Gerard Street) contained two multi-storey brick apartment buildings that appeared to be in fair/good condition when viewed from within the subject site. A basement carpark was observed underneath 81a Gerard Street with the entrance to the basement located off Parraween Street.

West of the site, were three multi-storey apartment buildings with each one containing basement parking. Two of the apartment buildings (47-51 Gerard Street and 9 Paling Street) appeared older and were in fair condition when viewed from the street. Immediately west of 59-61 Gerard Street was a newer multi-storey apartment building that appeared to be in good condition when viewed from the street.

South of the site was Cremorne's shopping village which consisted of multi-storey adjoined buildings that extend from Parraween Street to Military Road. The ground floor level was occupied by shops with residential apartments on the upper levels of the buildings. The buildings appeared to be in fair to good condition when viewed from the street.

3.2 Subsurface Conditions

Reference to the 1:100,000 Geological Series Sheet 9130 'Sydney' indicates that the site is underlain by Hawkesbury Sandstone. The results of the investigation revealed that the site is underlain by relatively shallow layer of soil overlying the sandstone bedrock. The more pertinent details of the materials encountered are provided below. Reference should be made to the borehole logs for a detailed description of the subsurface conditions encountered.

Fill

Fill comprising of silty sand was encountered to depths of 0.3m to 1.0m. In BH4, a gravelly clayey sand was encountered to a depth of 1.0m. The fill generally contained gravel sized inclusions of ironstone and igneous rock. In BH2 and BH4 the fill was assessed as being poorly to moderately compacted.

Residual Soils

Underlying the fill, a residual clayey sand and sandy clay of low to medium plasticity was encountered to depths ranging from 2.2m to 2.4m. The sandy clay varied in strength from stiff to hard.

Sandstone Bedrock

Sandstone bedrock was encountered in all boreholes ranging from 2.2m to 2.4m. When first encountered it was extremely weathered and of hard clay strength. Sandstone of very low to low strength was encountered at depths of 2.5m to 2.9m. The sandstone has been classified in accordance with Pells et al (Australian Geomechanics 2019) as shown in the flowing table:

Borehole	**Approx. Surface RL (m) AHD**	**Engineering Classification Sandstone Bedrock**				
		Depth (m)/ (RL) Top of Class V	Depth (m)/ (RL) Top of Class IV	Depth (m)/ (RL) Top of Class III	Depth (m)/ (RL) Top of Class II	Depth (m)/ (RL) Top of Class I
1	83.3	2.4* (RL 80.9)	3.4* (RL 79.9)	-	-	-
2	85.2	2.2 (RL 83.0)	4.3 (RL 80.9)	8.3(RL76.9)	9.0(RL76.2)	11.0$^+$(RL74.2)
3	84.4	2.2* (RL 82.2)	2.9$^+$ (RL 81.5)	8.3 (RL 76.1)	9.2 (RL 75.2)	-
4	82.9	2.0 (RL 80.9)	3.0 (RL 79.9)	8.5m$^?$(RL74.4)	-	-

* Although generally Class IV there are bands from 5.0m to 5.8m and 7.3m to 7.5m of Class V material.
\+ From 11.0m the rock is just Class I borderline Class II.
? From 9.0m the rock is borderline Class II.

It can be seen from the table that there is a significant difference in sandstone quality across the site with the rock in Boreholes BH2 and BH3 being of much better quality than that in BH1 and BH4. BH1 in particular showed Class IV sandstone extending to the termination depth of the borehole at 10.1m

Groundwater

Groundwater inflows and levels were recorded during auger drilling as shown on the borehole logs. BH1 to BH3 were dry on completion of augering but in BH4 water flowed from 0.5m depth and rose in the borehole to that depth on completion of augering. This is suggestive of there being a broken service pipe in the vicinity.

Water levels were recorded in the monitoring wells on 22 February 2023 as follows (and shown on the borehole logs):
BH1: 7.2m depth / RL 76.1m
BH2: 10.8m depth / RL 74.5m
BH3: 4.92m depth / RL 79.5m
BH4: 3.53m depth / RL 79.4m

There is clearly a disparity in the information and it seems that not only may there be a leaking service in the vicinity of BH4 but that seepage is finding its way into the monitoring well. Similarly, seepage must be affecting BH3 in which the standing water level was similar to that in BH4. The levels in BH1 and BH2 differ significantly and are greater than would be expected for a normal water table. BH2 was the deepest borehole, terminating at 15m depth as opposed to 10m to 11m for the other boreholes, and the response zone of BH2 was set deeper from 11.5m to 15m. Given the location of the site on a ridgeline at RL's around 85m and a fairly steep gradient to the harbour on both sides, we consider it most likely that the water table could be at the depth shown in BH2 and that the other water levels are recording seepage flows in the unsaturated zone.

3.3 Laboratory Test Results

The point load strength index test results were consistent with the field logging of rock strengths and confirm the disparity in rock quality between BH1/4 and BH2/3.

The soil aggression test results presented in the Envirolab Report, Certificate of analysis 314797 were as follows:

Borehole No.	Depth (m)	Sample Type	pH	Sulphate (ppm)	Chloride (ppm)	Resistivity (ohm.cm)
1	0.5-0.95	Residual	6.6	<10	<10	6,000
1	2.5-2.7	Sandstone	5.4	22	<10	4,700
3	1.2-1.4	Residual	5.5	23	<10	4,800

The above results indicate that the soils would have an exposure classification for concrete piles of ‘Mild’ when assessed in accordance with Table 6.4.2 (C) of AS2159-2009 “Piling Design and Installation”. The soils would also have an exposure classification for steel piles of ‘non-aggressive’ when assessed in accordance with Table 6.5.2 (C).

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Issues

The primary geotechnical issues with this development will be to maintain the stability of the excavation, avoid vibration damage to neighbouring properties during demolition and excavation of sandstone bedrock and manage groundwater inflows to the excavation. Due to the variable and locally poor rock quality there should be full depth soldier pile shoring throughout.

Dilapidation surveys must also be completed prior to works commencing.

Seepage into the excavation should be expected, as the recorded groundwater levels were above the bulk excavation level (BEL) in two of the monitoring wells; the volumes most probably will not be great though they could be moderate, particularly in the early stages after excavation, but they should be managed without undue difficulty provided the issue is properly addressed by the contractors and hydraulic engineers and any leaking pipes repaired. Some pockets of instability of ground between shoring piles could occur and locally shotcrete may be difficult to apply. In the long term, good drainage provisions should be incorporated into the hydraulic design. It should be noted however, that NSW DPIE/WaterNSW, assert that basements should be water tight (fully tanked) for the life of the building (typically 100 years) and that tanking may be stipulated by the planning consent authority.

As sandstone bedrock will be exposed throughout the excavation the design of footings should be quite straightforward, with moderate bearing pressures being feasible.

These issues are discussed in greater detail below.

4.2 Dilapidation Surveys

Prior to demolition and excavation commencing, detailed dilapidation reports should be compiled on the neighbouring buildings that lie within about 20m of the site boundaries. Only the units closest to the excavation need to be subject to detailed reports. An external appraisal as to the condition of the remainder of the buildings would be prudent. If there are any doubts regarding the condition of the structures, detailed reports should also be completed.

The property owners should be asked to confirm that the reports present a fair record of existing conditions as the reports may assist the developers in defending themselves from unfair damage claims due to vibrations and/or ground surface movements.

4.3 Excavation

4.3.1 Excavation Methods

Prior to the commencement of bulk excavation, the proposed development will require demolition of the existing pavements and houses.

The initial stage of excavation will require stripping of any fill and stockpiling it separately to the natural soils for appropriate disposal off site. A waste classification will be required for disposal.

To achieve the proposed two basement levels, excavation ranging from about 6.5m to 9m below existing grade will be required. Based on the investigation results, the proposed excavations will extend through 2m to 2.4m of fill and residual soil, but the majority of excavation works will be through sandstone bedrock, albeit much of which will be of relatively poor quality. It should be noted that there is not a well-defined contact with rock of medium to high strength from the investigation to date, which should be treated as a preliminary investigation when it comes to detailed design stage; as noted above there are significant variations between the boreholes which are widely spaced at around 70m and a closer grid may reveal more defined patterns in the subsurface conditions. The upper layer of weathered Class IV rock extends from depths of 2.9m to 4.3m to at least 8.3m and over 10.1m at BH1. Reference should be made to the rock classification table in Section 3 above and the borehole logs.

Excavation of the soil profile and any Class 5 bedrock can be carried out using buckets attached to large hydraulic excavators.

Excavation of the Class 4 and possibly Class 3 sandstone bedrock could probably be carried out by ripping using a large (40 tonne) excavators (with assistance using hydraulic rock hammers as required). The area of the excavation is probably sufficient to justify mobilisation and efficient operation of heavy dozers for ripping and contractors may prefer that option.

It is likely therefore, that the rock excavation will be carried out by a combination of ripping, rock sawing and hydraulic hammer techniques. We recommend that all perimeter cuts be sawn where possible to avoid unnecessary overbreak which otherwise occurs when using rock hammers and to minimise transmission of vibration off-site, though we note that sawing between soldier piles may not be practicable or essential. Grid-sawing of the better-quality rock is also likely to make the bulk excavation more effective using rock hammers, especially as the size of rock hammers that can be safely used adjacent to boundaries may be limited to manage the risk of vibration damage to nearby properties.

We recommend that a full copy of this report be provided to the excavation contractor so that the contractor can make their own assessment of excavation conditions.

Groundwater inflows into the excavation should be expected (particularly at depth) and may occur as local seepage flows within the fill (particularly near BH4), at the soil/rock interface and through any joints, faults, shear zones and bedding partings within the bedrock profile, particularly after heavy or prolonged rainfall. Seepage volumes into the excavation are expected to be controllable by conventional sump and pump dewatering systems.

Groundwater seepage monitoring (included seepage locations and inflow volumes) should be carried out during bulk excavation prior to finalising the design of a pump out facility. Outlets into the stormwater system will require Council approval if they are permitted in principle.

4.3.2 Potential Vibration Risks

We recommend that caution be taken during rock excavation as there will likely be direct transmission of ground vibrations to adjoining buildings and structures. The proposed excavations will extend moderately close to the site boundaries.

Excavation procedures and the dilapidation reports should be carefully reviewed prior to the commencement of demolition and excavation, so that appropriate equipment is used.

The excavation with hydraulic rock hammers should commence away from likely critical areas (i.e. commence over the central/southern portion of the site). We recommend that continuous vibration monitoring should be carried out on the buildings to the north, west and east. Details should be provided in a Geotechnical Monitoring Program which should be prepared once detailed design is complete. Subject to review of the dilapidation reports, vibrations, measured as Peak Particle Velocity (PPV), should be limited to no higher than 5mm/sec on the neighbouring buildings. As shown on the attached Vibration Emission Design Goals sheet, higher levels of vibration can be tolerable at higher frequencies but it is necessary to download the data from the monitor in order to determine if this is occurring. If it is found that transmitted vibrations are excessive, then it would be necessary to use a smaller rock hammer or alternative excavation techniques.

When using a rock saw or rotary grinder, the resulting dust must be suppressed by spraying with water.

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

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

4.4 Excavation Support

The proposed excavation will generally extend to between 3m and 7m from the boundaries, locally a little more in some areas and zero along the street frontage with Parraween Street. The use of temporary batters could be considered for at least part of the excavation, but would have many drawbacks including the large additional volume of excavation required, the need to carry out soil nailing in some areas, the lack of space on site for sheds and storage and the need to import and place large volumes of good quality backfill material. In light of these issues, we recommend the use of shoring system around the full perimeter of the excavation.

Given the subsurface conditions encountered and the proximity of adjoining buildings, we consider that the most suitable shoring system will comprise anchored soldier pile walls. Where soldier pile walls are adopted, the excavated face must be inspected by a geotechnical engineer at not greater than 1.5m depth intervals, and prior to spraying shotcrete to confirm that no adverse defects are present within the rock mass and the conditions assumed for design are consistent with the actual ground conditions.

The piles for the shoring system should be founded below the bulk excavation level (including allowance for localised services and footing excavations) with sufficient depth to satisfy stability for all stages of excavation and anchoring. Extending the shoring piles in this way has a number of benefits:

- The piles will be founded on rock and can be designed to carry structural loads if required.
- The socket of the pile toe provides lateral support in the short and long term.

Temporary lateral support of the retaining walls will be required by either internal props or external anchors, and these must be installed progressively as each restraining point is uncovered. Permission will need to be obtained from the owners of adjoining properties before the installation of anchors below those properties. The presence of existing basements on adjoining sites must be determined as this may preclude the use of temporary anchors. In the long term we assume that lateral restraint will be provided by the floor slabs within the structure.

Where some wall movements are tolerable, propped or anchored retaining walls may be designed using a trapezoidal lateral pressure distribution of magnitude 5H kPa within the soil and weathered sandstone to a nominal 6m depth, where H is the depth of the supported depth of excavation in metres. As the boreholes are presently spaced a long way apart it will be necessary to better define the depth to which the shoring should be designed as a soil retaining structure by a grid of closer spaced boreholes at the detailed design stage. Where ground movements are critical and adjoining buildings are located close to the wall, a higher lateral earth pressure distribution of magnitude 8H kPa should be used. These maximum lateral pressures should be kept constant for the central 60% of the distribution. The excavated faces between the solider piles will need to be inspected by a geotechnical engineer at regular intervals to confirm that ground conditions are no worse than the design case. For the weaker, weathered rock below about 6m depth a nominal lateral pressure of 10kPa on the shoring should be assumed to accommodate any minor wedges of joint-bound rock, extremely weathered zones etc.

Anchors should be bonded within the weathered sandstone and may be provisionally designed based on a maximum allowable bond stress of:

- 80kPa within Class V sandstone,
- 200kPa within Class IV sandstone

The anchor design and construction should also be subject to the following conditions:

- Anchors to have a free length of at least 4m and a bond length of at least 3m with the bond length fully embedded beyond a line drawn up at 45˚ from the bulk excavation level.
- Overall stability, including anchor group interaction, is satisfied.
- All anchors are respectively proof loaded to at least 1.3 times their design working load before locking off at the 80% of the design working load. After lock-off, lift-off tests should be completed to ensure that the anchors are holding their load. Additional lift-off tests should be completed on at least 10% of the anchors 48 hours following initial stressing to ensure that the anchors are holding their working load.

Higher or lower bond stresses may be justified if the results from a testing program on prototype anchors indicate this to be appropriate. It is normal practice to nominate anchor loads and to allow contractors to design and construct anchors to avoid contractual disputes if anchor loads cannot be achieved.

For the design of piled walls socketed into the sandstone bedrock, a maximum allowable toe resistance of 200kPa in Class IV sandstone and 300kPa within Class III sandstone may be used below the base of the excavations, including footing and service excavations. These lateral toe resistances should only be applied for pile sockets which are greater than 0.3m below the bulk excavation level, including any allowances for over-excavation, such as for services or footings.

The above lateral pressures and coefficients assume horizontal backfill surfaces. Where inclined backfill is proposed the lateral pressures should be increased or the inclined backfill taken as a surcharge load. All surcharge loads, particularly those of the adjacent buildings, should be allowed for in the design. A coefficient of earth pressure at rest, K_0, of 0.5 should be used to calculate the effect of surcharge loads. Full hydrostatic pressures should be considered unless measures are undertaken to provide complete and permanent drainage of the ground behind the wall.

Retaining walls may be designed using software such as Plaxis, which often results in more economical designs. We would be pleased to assist in such wall analysis.

4.5 Footings

Based on the investigation results, sandstone bedrock will be present at bulk excavation level. Therefore, pad and strip footings founded within the sandstone will be appropriate. Such footings may be provisionally designed based on the following:

- An allowable bearing pressure (ABP) of 1,500kPa where Class IV sandstone is present
- An ABP of 3500kPa where Class III or better sandstone is present.

Based upon the borehole information summarised in the classification table in Section 3 it can be seen that below the approximate founding depth of RL76.0m a mixture of Class IV and Class III sandstone is expected to be present. Additional boreholes which should be drilled prior to detailed design will probably enable the areas of different rock quality to be better defined. Proving the Class III sandstone quality may require spoon testing in up to one third of the footings as differentiating rock quality from surface inspection may not be easy and all footing excavations must be inspected by a geotechnical engineer following excavation to confirm that the appropriate quality sandstone bedrock has been encountered.

It should be noted that groundwater seepage levels are likely to be above the base of the piers and hence inflows into bored piers may occur. It is likely that piers will have to be poured immediately after drilling, cleaning and inspection and that concrete must be tremied to the base of the piers.

4.6 Groundwater

As noted above, groundwater levels range from about 3.5m to 10.75m in the boreholes, though it is likely that the higher levels are perched groundwater, rather than a true groundwater table. Inflows, particularly at the start of construction could be significant, though they are likely to diminish with time. For a drained basement full drainage must be provided behind all retaining structures and below the floor slabs a drainage blanket is recommended which should comprise at least 100mm thickness of blue metal gravel. Council authority will be needed to permit discharge to the stormwater system.

4.7 Slabs on Grade

On-grade floor slabs should be separated from all walls, columns, footings, etc., to permit relative movements (i.e. designed as ‘floating’ slabs). As the slabs on grade will directly overlie sandstone bedrock the subgrade should not require any special preparation., apart from a drainage blanket being advisable.

4.8 Further Geotechnical Input

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

- Drilling of further boreholes so that the grid spacing is no more than 25m.
- Geotechnical review of structural design to check consistency with the geotechnical reports.
- Dilapidation survey reports on the neighbouring buildings.
- Geotechnical analysis of shoring.
- Geotechnical inspections of shoring piles if these are load bearing.
- Vibration monitoring.
- Geotechnical inspections of cut faces during excavation.
- Witnessing the proof loading of temporary anchors.
- Geotechnical inspection and testing of footing excavations.
- Observation of groundwater seepage into the bulk excavation.

We also recommend a meeting at the commencement of construction to discuss the primary geotechnical issues and inspection requirements.

5 GENERAL COMMENTS

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

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

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

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

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

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client: Pathways **Ref No:** 35736S

Project: Proposed Seniors Living Development **Report:** A

Location: 50-88 Parraween Street and 59-67 Gerard Street, CREMORNE, NSW **Report Date:** 23/02/23

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BOREHOLE NUMBER	DEPTH (m)	$I_{S (50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
1	3.90 - 3.94	0.2	4	A
	4.35 - 4.38	0.1	2	A
	4.83 - 4.87	0.1	2	A
	5.52 - 5.56	0.09	2	A
	5.85 - 5.89	0.1	2	A
	6.24 - 6.28	0.1	2	A
	6.61 - 6.65	0.2	4	A
	7.20 - 7.24	0.4	8	A
	7.74 - 7.78	0.3	6	A
	8.34 - 8.37	0.3	6	A
	8.79 - 8.83	0.3	6	A
	9.24 - 9.28	0.3	6	A
	9.83 - 9.86	0.4	8	A
	10.06 - 10.09	0.9	18	A
2	4.45 - 4.50	0.3	6	A
	4.89 - 4.92	0.1	2	A
	5.18 - 5.21	0.3	6	A
	5.92 - 5.95	0.2	4	A
	6.38 - 6.40	0.05	1	A
	6.89 - 6.91	0.1	2	A
	7.11 - 7.14	0.3	6	A
	7.77 - 7.81	0.3	6	A
	8.19 - 8.23	0.3	6	A
	8.70 - 8.73	0.5	10	A
	9.20 - 9.23	1.1	22	A

NOTE: SEE PAGE 3

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client: Pathways

Ref No: 35736S

Project: Proposed Seniors Living Development

Report: A

Location: 50-88 Parraween Street and 59-67 Gerard Street,
CREMORNE, NSW

Report Date: 23/02/23

BOREHOLE NUMBER	DEPTH (m)	$I_{S (50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
2	9.68 - 9.71	1.2	24	A
	10.23 - 10.27	1.1	22	A
	10.73 - 10.76	0.6	12	A
	11.26 - 11.29	1.8	36	A
	11.70 - 11.73	1.4	28	A
	12.13 - 12.17	1.5	30	A
	12.68 - 12.72	1.4	28	A
	13.24 - 13.28	1.2	24	A
	13.75 - 13.78	1.2	24	A
	14.34 - 14.38	1.6	32	A
	14.76 - 14.79	1.7	34	A
3	3.20 - 3.24	0.3	6	A
	3.74 - 3.77	0.2	4	A
	4.40 - 4.44	0.2	4	A
	4.92 - 4.96	0.2	4	A
	5.34 - 5.38	0.2	4	A
	5.77 - 5.81	0.2	4	A
4	3.15 - 3.17	0.2	4	A
	3.61 - 3.63	0.3	6	A
	4.05 - 4.08	0.1	2	A
	4.71 - 4.74	0.1	2	A
	5.27 - 5.29	0.2	4	A
	5.47 - 5.50	0.3	6	A
	5.72 - 5.74	0.4	8	A
	6.50 - 6.53	0.1	2	A

NOTE: SEE PAGE 3

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client: Pathways **Ref No:** 35736S

Project: Proposed Seniors Living Development **Report:** A

Location: 50-88 Parraween Street and 59-67 Gerard Street, CREMORNE, NSW **Report Date:** 23/02/23

Page 3 of 3

BOREHOLE NUMBER	DEPTH (m)	$I_{S\,(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
4	6.79 - 6.82	0.3	6	A
	7.14 - 7.17	0.3	6	A
	7.47 - 7.50	0.1	2	A
	7.65 - 7.68	0.1	2	A
	8.16 - 8.19	0.4	8	A
	8.68 - 8.71	0.6	12	A
	9.23 - 9.26	0.7	14	A
	9.89 - 9.92	0.6	12	A
3A	6.16 - 6.19	0.4	8	A
	6.84 - 6.87	0.2	4	A
	7.28 - 7.31	0.1	2	A
	7.31 - 7.35	0.1	2	A
	7.52 - 7.55	0.1	2	A
	7.81 - 7.85	0.2	4	A
	8.13 - 8.15	0.4	8	A
	8.55 - 8.58	0.4	8	A
	8.90 - 8.93	0.3	6	A
	9.32 - 9.36	0.7	14	A
	9.75 - 9.79	1.1	22	A
	10.27 - 10.31	0.8	16	A
	10.78 - 10.81	1.1	22	A
	11.11 - 11.15	1.1	22	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 314797

Client Details

Client	JK Geotechnics
Attention	Tom Foster
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**35736S, Cremorne**
Number of Samples	3 Soil
Date samples received	19/01/2023
Date completed instructions received	19/01/2023

Analysis Details

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

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

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

Report Details

Date results requested by	27/01/2023
Date of Issue	27/01/2023

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

Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By
Nancy Zhang, Laboratory Manager

Misc Inorg - Soil				
Our Reference		314797-1	314797-2	314797-3
Your Reference	UNITS	BH1	BH1	BH3
Depth		0.5-0.95	2.5-2.7	1.2-1.4
Type of sample		Soil	Soil	Soil
Date prepared	-	23/01/2023	23/01/2023	23/01/2023
Date analysed	-	23/01/2023	23/01/2023	23/01/2023
pH 1:5 soil:water	pH Units	6.6	5.4	5.5
Chloride, Cl 1:5 soil:water	mg/kg	<10	<10	<10
Sulphate, SO4 1:5 soil:water	mg/kg	<10	22	23
Resistivity in soil*	ohm m	600	470	480

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

Client Reference: 35736S, Cremorne

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			23/01/2023	[NT]	[NT]	[NT]	[NT]	23/01/2023	[NT]
Date analysed	-			23/01/2023	[NT]	[NT]	[NT]	[NT]	23/01/2023	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	100	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	85	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	92	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]

Result Definitions

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

Quality Control Definitions

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

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

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

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

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

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

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

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

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

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

JKGeotechnics

BOREHOLE LOG

Borehole No.
1
1 / 3

Client: PATHWAYS RESIDENCES CREMORNE
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 50-88 PARRAWEEN STREET AND 59-67 GERARD STREET, CREMORNE, NSW

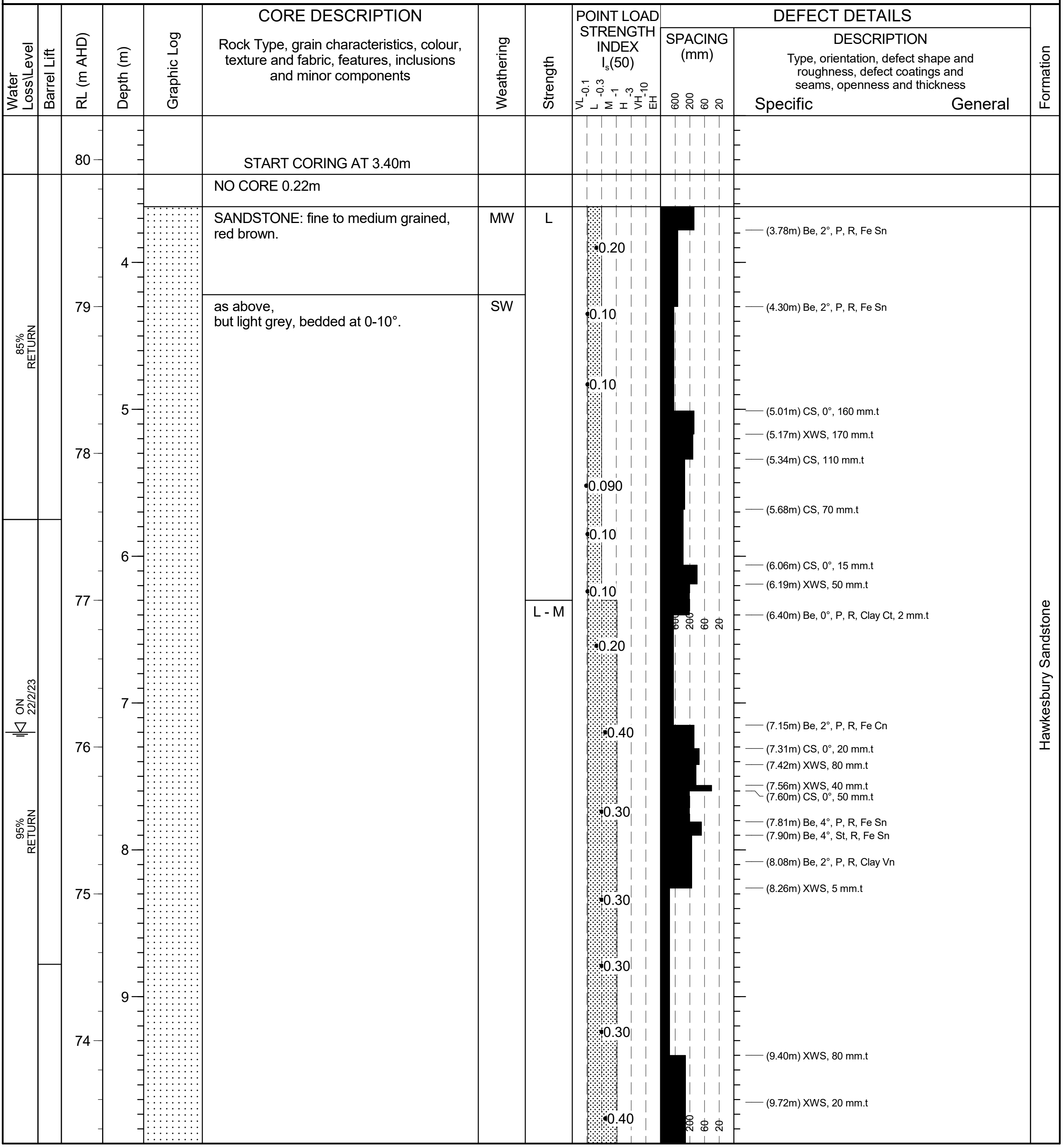
Job No.: 35736S **Method:** SPIRAL AUGER **R.L. Surface:** ~83.3 m
Date: 18/1/23 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** T.F./P.S.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	■					83				FILL: Silty sand, fine to medium grained, grey and brown, with fine to coarse grained igneous gravel, trace of root fibres.	D			GRASS COVER
ON COMPLETION OF CORING	■			■	N = 9 4,4,5		1		SM	Clayey Silty SAND: fine to medium grained, orange brown, with fine to medium grained ironstone gravel.	M			RESIDUAL
DRY ON COMPLETION OF AUGERING	■			■ ■	N = 3 2,1,2	82 81	2		CL-CI	Silty CLAY: low to medium plasticity, orange brown, red brown and light grey, with fine to medium grained sand, trace of fine to medium grained ironstone gravel and root fibres.	w>PL	St	190 110 100	
				■					-	Extremely Weathered sandstone: silty CLAY, low to medium plasticity, light grey, with fine to medium grained sand and iron indurated bands.	XW	Hd		HAWKESBURY SANDSTONE LOW TO MODERATE 'TC' BIT BANDED RESISTANCE
					N=SPT 2/ 0mm REFUSAL	80	3			SANDSTONE: fine to medium grained, orange brown and light grey, with iron indurated bands.	DW	VL		LOW RESISTANCE WITH MODERATE BANDS
						79 78 77	4 5 6			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 10.1m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 10.1m TO 7.1m. CASING 7.1m TO 0m. 2mm SAND FILTER PACK 10m TO 6.6m. BENTONITE SEAL 6.6m TO 0.2m. COMPLETED WITH A CONCRETED GATIC COVER.

CORED BOREHOLE LOG

Client: PATHWAYS RESIDENCES CREMORNE
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 50-88 PARRAWEEN STREET AND 59-67 GERARD STREET, CREMORNE, NSW

Job No.: 35736S
Core Size: NMLC
R.L. Surface: ~83.3 m
Date: 18/1/23
Inclination: VERTICAL
Datum: AHD
Plant Type: JK205
Bearing: N/A
Logged/Checked By: T.F./P.S.



START CORING AT 3.40m

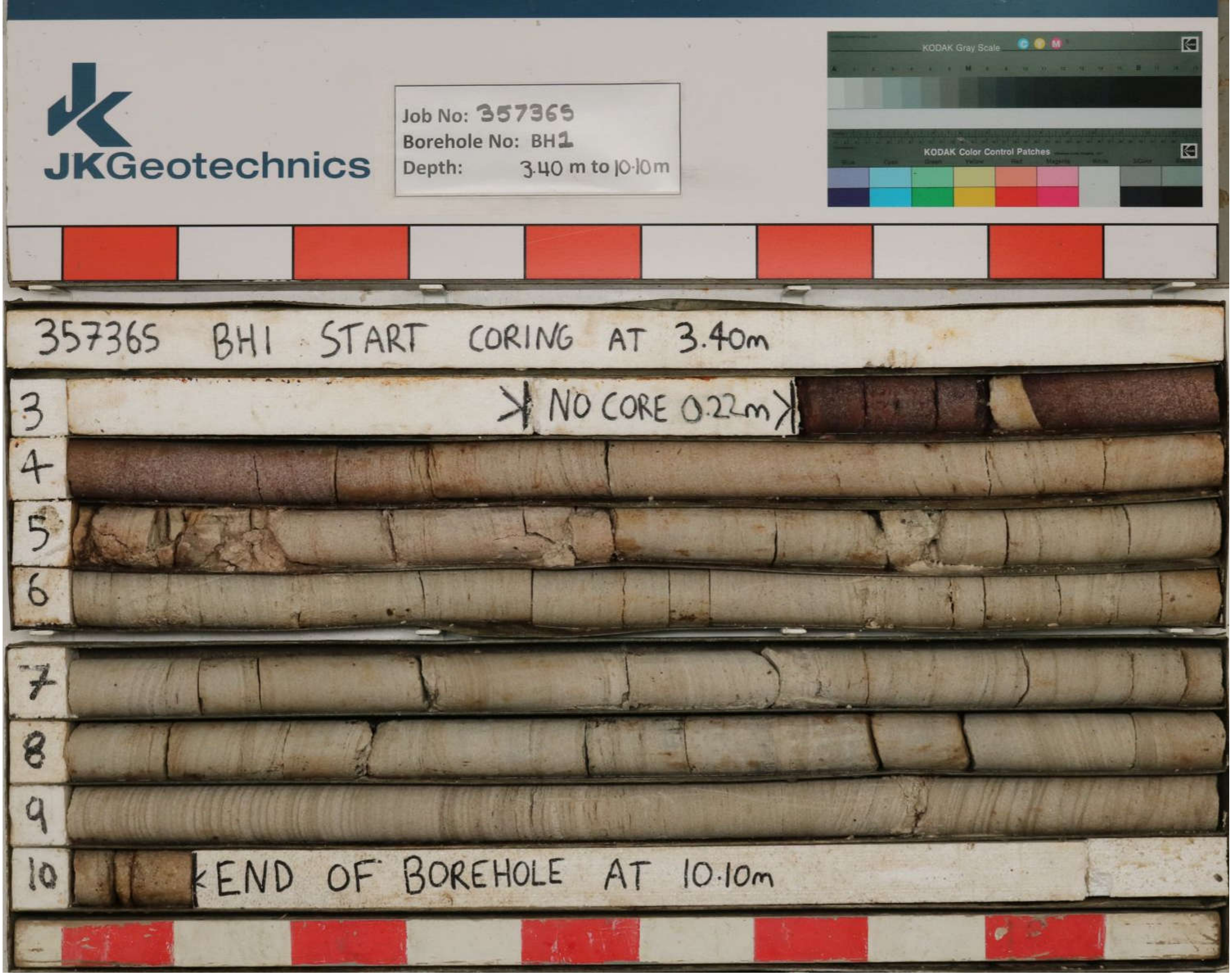
NO CORE 0.22m

SANDSTONE: fine to medium grained, red brown. — MW — L

as above, but light grey, bedded at 0-10°. — SW — L; L - M

(3.78m) Be, 2°, P, R, Fe Sn
(4.30m) Be, 2°, P, R, Fe Sn
(5.01m) CS, 0°, 160 mm.t
(5.17m) XWS, 170 mm.t
(5.34m) CS, 110 mm.t
(5.68m) CS, 70 mm.t
(6.06m) CS, 0°, 15 mm.t
(6.19m) XWS, 50 mm.t
(6.40m) Be, 0°, P, R, Clay Ct, 2 mm.t
(7.15m) Be, 2°, P, R, Fe Cn
(7.31m) CS, 0°, 20 mm.t
(7.42m) XWS, 80 mm.t
(7.56m) XWS, 40 mm.t
(7.60m) CS, 0°, 50 mm.t
(7.81m) Be, 4°, P, R, Fe Sn
(7.90m) Be, 4°, St, R, Fe Sn
(8.08m) Be, 2°, P, R, Clay Vn
(8.26m) XWS, 5 mm.t
(9.40m) XWS, 80 mm.t
(9.72m) XWS, 20 mm.t

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



JKGeotechnics

Job No: 357365

Borehole No: BH1

Depth: 3.40 m to 10.10m

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
1
3 / 3

Client: PATHWAYS RESIDENCES CREMORNE
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 50-88 PARRAWEEN STREET AND 59-67 GERARD STREET, CREMORNE, NSW

Job No.: 35736S
Core Size: NMLC
R.L. Surface: ~83.3 m
Date: 18/1/23
Inclination: VERTICAL
Datum: AHD
Plant Type: JK205
Bearing: N/A
Logged/Checked By: T.F./P.S.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1 L -0.3 M -1 H -3 VH -10 EH	DEFECT DETAILS SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	General	Formation
					SANDSTONE: fine to medium grained, light grey, bedded at 0-10°.	SW	L - M	0.90		(10.04m) Be, 2°, P, R, Fe Sn (10.06m) Be, 2°, P, R, Fe Sn		
					END OF BOREHOLE AT 10.10 m							
		73										
			11									
		72										
			12									
		71										
			13									
		70							600 200 60 20			
			14									
		69										
			15									
		68										
			16									
		67							600 200 60 20			

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JKGeotechnics

BOREHOLE LOG

Borehole No.
2
1 / 3

Client: PATHWAYS RESIDENCES CREMORNE
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 50-88 PARRAWEEN STREET AND 59-67 GERARD STREET, CREMORNE, NSW

Job No.: 35736S **Method:** SPIRAL AUGER **R.L. Surface:** ~85.2 m
Date: 20/2/23 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** T.F./P.S.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING		85			FILL: Silty sand, fine to medium grained, dark grey and brown, trace of fine to medium grained igneous and sandstone gravel, root fibres and ash.	M			GRASS COVER APPEARS MODERATELY COMPACTED
	N = 11 5,8,3				FILL: Silty sand, fine to medium grained, brown, with fine to medium grained sandstone and ironstone gravel, and ash.				
		84	1	CL-CI	Silty CLAY: low to medium plasticity, orange brown, with fine to medium grained sand, trace of fine to coarse grained ironstone gravel.	w~PL	(St)		RESIDUAL ORGANIC ODOUR
	N = 15 3,3,12		2	CL	Silty Sandy CLAY: low plasticity, light grey, with fine to coarse grained ironstone gravel.	w<PL	St	150 180 170	
	N=SPT 16/ 150mm REFUSAL	83 82	3	-	Extremely Weathered sandstone: silty CLAY, low to medium plasticity, light grey and red brown, with fine to coarse grained iron indurated bands.	XW	Hd		HAWKESBURY SANDSTONE VERY LOW 'TC' BIT RESISTANCE
		81	4		SANDSTONE: fine to medium grained, light grey and red brown, with extremely weathered bands and iron indurated bands.	DW	VL		VERY LOW TO LOW STRENGTH BANDS
		80 79	5 6		REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 14.99m. HAND SLOTTED 50mm DIA. PVC STANDPIPE 14.99m TO 11.99m. CASING 11.99m TO 0m. 2mm SAND FILTER PACK 14.99m TO 11.5m. BENTONITE SEAL 11.5m TO 6m. BACKFILLED WITH SAND AND CUTTINGS TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

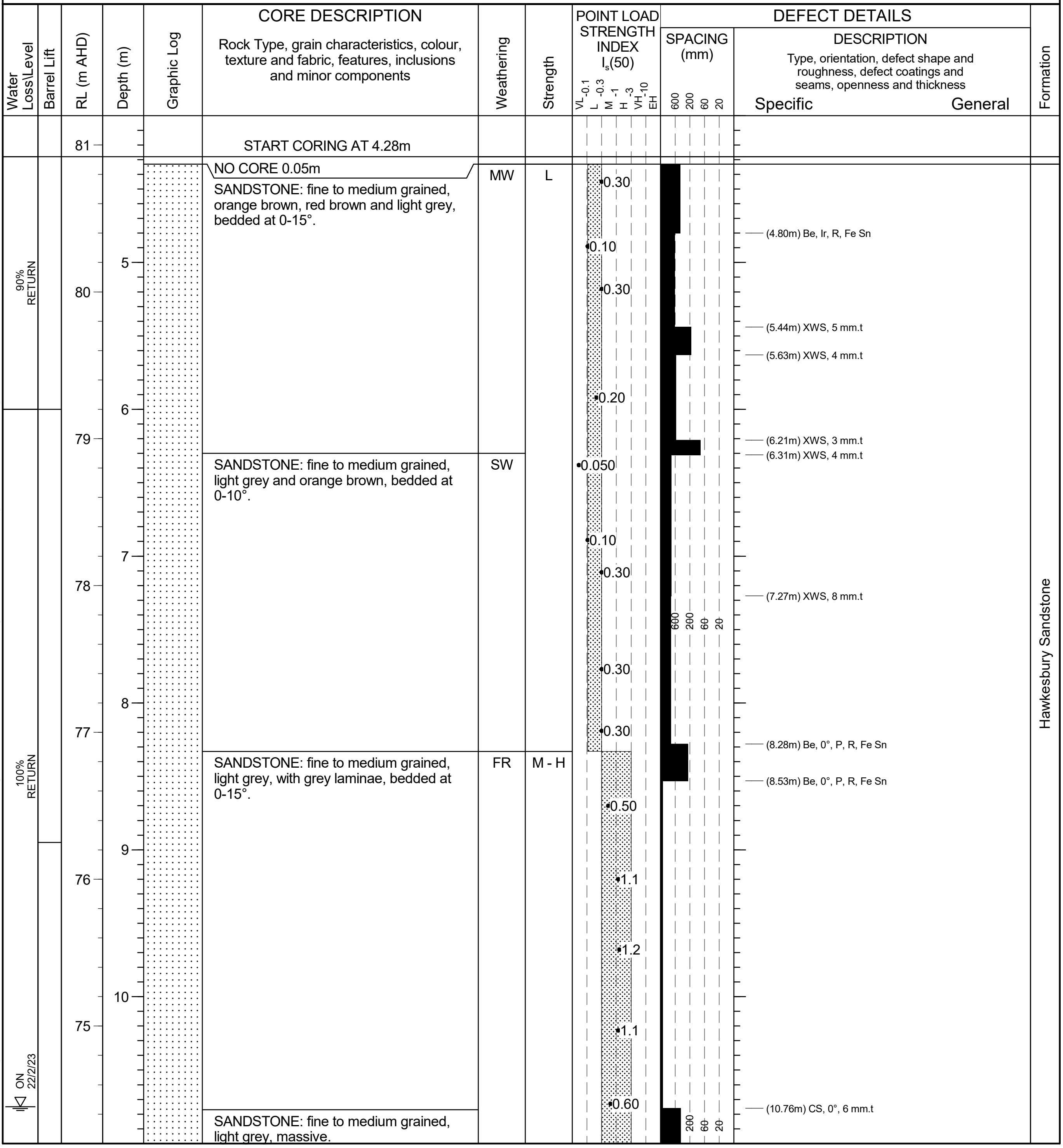
SAMPLES: ES U50 DB DS | Graphic Log

CORED BOREHOLE LOG

Borehole No.
2

Client: PATHWAYS RESIDENCES CREMORNE
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 50-88 PARRAWEEN STREET AND 59-67 GERARD STREET, CREMORNE, NSW

Job No.: 35736S	**Core Size:** NMLC	**R.L. Surface:** ~85.2 m
Date: 20/2/23	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK205	**Bearing:** N/A	**Logged/Checked By:** T.F./P.S.



Depth	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength
4.28	START CORING AT 4.28m		
	NO CORE 0.05m		
	SANDSTONE: fine to medium grained, orange brown, red brown and light grey, bedded at 0-15°.	MW	L
	SANDSTONE: fine to medium grained, light grey and orange brown, bedded at 0-10°.	SW	
	SANDSTONE: fine to medium grained, light grey, with grey laminae, bedded at 0-15°.	FR	M - H
	SANDSTONE: fine to medium grained, light grey, massive.		

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

(4.80m) Be, Ir, R, Fe Sn
(5.44m) XWS, 5 mm.t
(5.63m) XWS, 4 mm.t
(6.21m) XWS, 3 mm.t
(6.31m) XWS, 4 mm.t
(7.27m) XWS, 8 mm.t
(8.28m) Be, 0°, P, R, Fe Sn
(8.53m) Be, 0°, P, R, Fe Sn
(10.76m) CS, 0°, 6 mm.t

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JKGeotechnics

Job No: 35736S

Borehole No: BH2

Depth: 4.28 m to 13.00m

CORED BOREHOLE LOG

Borehole No.
2
3 / 3

Client: PATHWAYS RESIDENCES CREMORNE
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 50-88 PARRAWEEN STREET AND 59-67 GERARD STREET, CREMORNE, NSW

Job No.: 35736S
Core Size: NMLC
R.L. Surface: ~85.2 m
Date: 20/2/23
Inclination: VERTICAL
Datum: AHD
Plant Type: JK205
Bearing: N/A
Logged/Checked By: T.F./P.S.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT DETAILS SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Specific Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness	General	Formation
100% RETURN		74 73	12		SANDSTONE: fine to medium grained, light grey, massive. *(continued)*	FR	H	1.8 1.4 1.5		(11.21m) J, 20°, P, R, Fe Sn		Hawkesbury Sandstone
		72 71	13 14		SANDSTONE: fine to medium grained, light grey, with grey laminae, bedded at 0-15°.			1.4 1.2 1.2 1.6 1.7	600 200 60 20			
		70 69 68	15 16 17		END OF BOREHOLE AT 14.99 m				600 200 60 20			

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FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 35736S CREMORNE.GPJ <<DrawingFile>> 24/03/2023 10:18 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

Job No: 35736S

Borehole No: BH2

Depth: 13.00m to 14.99m

13

14

15 END OF BOREHOLE AT 14.99m

JKGeotechnics

BOREHOLE LOG

Borehole No.
3
1 / 2

Client: PATHWAYS RESIDENCES CREMORNE
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 50-88 PARRAWEEN STREET AND 59-67 GERARD STREET, CREMORNE, NSW

Job No.: 35736S **Method:** SPIRAL AUGER **R.L. Surface:** ~84.4 m
Date: 17/1/23 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** T.F./P.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING										CONCRETE: 70mm.t				
									-	FILL: Gravel, fine to medium grained, orange brown and brown, igneous.	M			
					N > 12 4,3,9/ 130mm REFUSAL	84			CL-CI	Sandy Silty CLAY: low to medium plasticity, orange brown, fine to medium grained sand, trace of fine to medium grained ironstone gravel.	w>PL	Hd		RESIDUAL TOO FRIABLE FOR HP TESTING AND IRONSTONE
							1		CL	Sandy CLAY: low plasticity, red brown, fine to medium grained sand, with fine to coarse grained ironstone gravel.	w<PL			HIGH RESISTANCE ON IRONSTONE BAND
					N=SPT 13/ 0mm REFUSAL	83	2			as above, but light grey mottled orange brown.		VSt - Hd	350 300 320 450 420 400	HIGH RESISTANCE BANDS
						82			-	Extremely Weathered sandstone: sandy CLAY, low plasticity, fine to medium grained, with iron indurated bands.	XW	Hd		HAWKESBURY SANDSTONE
										SANDSTONE: fine to medium grained, orange brown and red brown, with iron indurated bands.	DW	VL - L		LOW 'TC'BIT RESISTANCE WITH HIGH RESISTANCE BANDS
						81 80 79 78	3 4 5 6			REFER TO CORED BOREHOLE LOG				LOW TO MODERATE RESISTANCE WITH HIGH RESISTANCE BANDS GROUNDWATER MONITORING WELL INSTALLED TO 11.1m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 11.1m TO 8.1m. CASING 8.1m TO 0m. 2mm SAND FILTER PACK 11.1m TO 7.5m. BENTONITE SEAL 7.5m TO 0.8m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

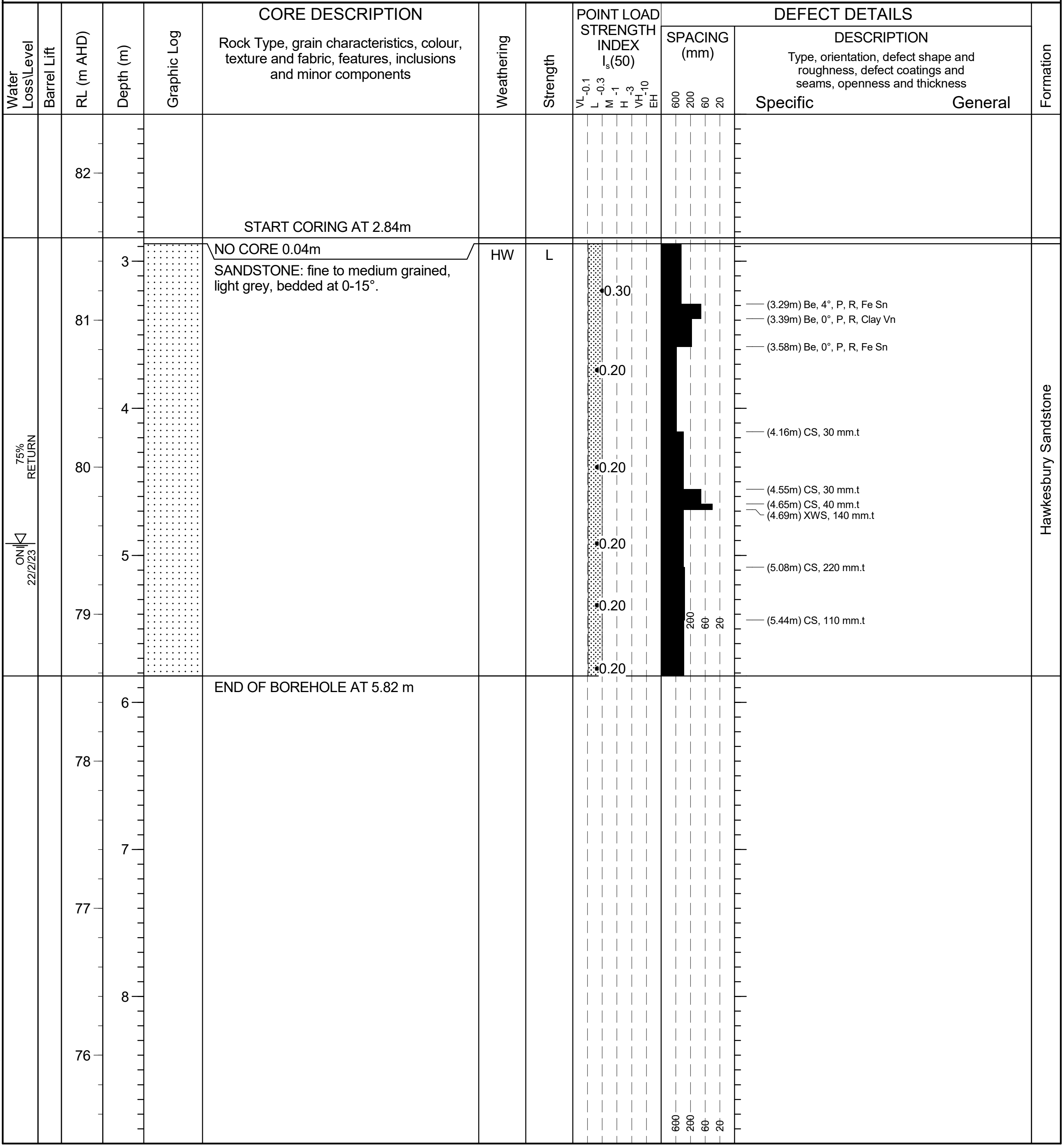
JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 35736S CREMORNE.GPJ <<DrawingFile>> 24/03/2023 10:18 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

CORED BOREHOLE LOG

Borehole No.
3
2 / 2

Client: PATHWAYS RESIDENCES CREMORNE
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 50-88 PARRAWEEN STREET AND 59-67 GERARD STREET, CREMORNE, NSW

Job No.: 35736S
Core Size: NMLC
R.L. Surface: ~84.4 m
Date: 17/1/23
Inclination: VERTICAL
Datum: AHD
Plant Type: JK205
Bearing: N/A
Logged/Checked By: T.F./P.S.



Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS SPACING (mm)	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific General	Formation
		82			START CORING AT 2.84m						
75% RETURN ON 22/2/23		81 80 79	3 4 5		NO CORE 0.04m SANDSTONE: fine to medium grained, light grey, bedded at 0-15°.	HW	L	0.30 0.20 0.20 0.20 0.20 0.20		(3.29m) Be, 4°, P, R, Fe Sn (3.39m) Be, 0°, P, R, Clay Vn (3.58m) Be, 0°, P, R, Fe Sn (4.16m) CS, 30 mm.t (4.55m) CS, 30 mm.t (4.65m) CS, 40 mm.t (4.69m) XWS, 140 mm.t (5.08m) CS, 220 mm.t (5.44m) CS, 110 mm.t	Hawkesbury Sandstone
		78 77 76	6 7 8		END OF BOREHOLE AT 5.82 m						

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JKGeotechnics

Job No: 35736S
Borehole No: BH 3
Depth: 2.84m to 5.82m

BOREHOLE LOG

Borehole No.
3A
1 / 2

Client: PATHWAYS RESIDENCES CREMORNE
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 50-88 PARRAWEEN STREET AND 59-67 GERARD STREET, CREMORNE, NSW

Job No.: 35736S **Method:** SPIRAL AUGER **R.L. Surface:** ~84.4 m
Date: 21/2/23 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** T.F./P.S.

Groundwater Record	SAMPLES				Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
						84								
							1							
						83								
							2							
						82								
							3							
						81								
							4							
						80								
							5							
						79								
							6							
						78								

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 35736S CREMORNE.GPJ <<DrawingFile>> 24/03/2023 10:18 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

CORED BOREHOLE LOG

Borehole No.
3A

Client: PATHWAYS RESIDENCES CREMORNE
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 50-88 PARRAWEEN STREET AND 59-67 GERARD STREET, CREMORNE, NSW

Job No.: 35736S
Core Size: NMLC
R.L. Surface: ~84.4 m
Date: 21/2/23
Inclination: VERTICAL
Datum: AHD
Plant Type: JK205
Bearing: N/A
Logged/Checked By: T.F./P.S.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS SPACING (mm)	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness (Specific / General)	Formation
		79			START CORING AT 5.80m						
100% RETURN					NO CORE 0.08m						Hawkesbury Sandstone
		78 77 76	6 7 8		SANDSTONE: fine to medium grained, light grey, with orange brown laminae, bedded at 0-15°.	MW	L L - M	0.40 0.20 0.10 0.10 0.10 0.20 0.40		(6.90m) Be, 0°, P, R, Fe Sn (8.33m) XWS, 2 mm.t	
		75 74	9 10 11		SANDSTONE: fine to medium grained, light grey, with grey laminae, bedded at 0-15°.	SW	M M - H	0.40 0.30 0.70 1.1 0.80 1.1 1.1			
		73			END OF BOREHOLE AT 11.24 m						

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

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

Job No: 35736S
Borehole No: BH3A
Depth: 5.80m to 11.24m

BOREHOLE LOG

Borehole No.
4
1 / 3

Client: PATHWAYS RESIDENCES CREMORNE
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 50-88 PARRAWEEN STREET AND 59-67 GERARD STREET, CREMORNE, NSW

Job No.: 35736S **Method:** SPIRAL AUGER **R.L. Surface:** 82.9 m
Date: 22/2/23 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** T.F./P.S.

Groundwater Record	SAMPLES				Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
	■									FILL: Silty sand, fine to medium grained, brown and grey, trace of fine to coarse grained ironstone gravel, and root fibres.	M			GRASS COVER APPEARS POORLY COMPACTED
ON COMPLETION OF AUGERING	■			■	N = 5 4,3,2	82				FILL: Gravelly clayey sand, fine to medium grained, grey and brown, fine to coarse grained ironstone gravel.	W			
	■			■	N=SPT 7/ 20mm REFUSAL	81	1		CL	Sandy Gravelly CLAY: low plasticity, light brown, fine to coarse grained ironstone gravel, fine to medium grained sand.	w>PL	Hd		RESIDUAL TOO GRAVELLY FOR HP READING
							2		-	Extremely Weathered sandstone: silty CLAY, low plasticity, red brown, with iron indurated bands.	XW	Hd		HAWKESBURY SANDSTONE VERY LOW 'TC' BIT RESISTANCE WITH HIGH BANDS (IRONSTONE?)
				■						SANDSTONE: fine to medium grained, red brown.	DW	VL - L		
						80	3			REFER TO CORED BOREHOLE LOG				VERY LOW TO LOW RESISTANCE WITH HIGH BANDS GROUNDWATER MONITORING WELL INSTALLED TO 10.0m. CLASS 18 MACHINE SLOTTED/HAND SLOTTED 50mm DIA. PVC STANDPIPE 10.0m TO 7.0m. CASING 7.0m TO 0m. 2mm SAND FILTER PACK 10.0m TO 6.5m. BENTONITE SEAL 6.5m TO 0.5m. COMPLETED WITH A CONCRETED GATIC COVER.
						79	4							
						78	5							
						77	6							
						76								

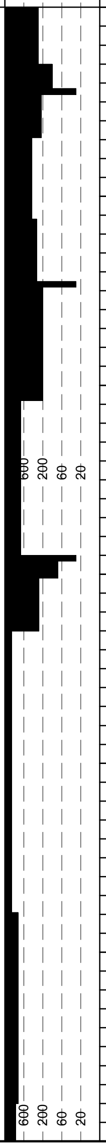
CORED BOREHOLE LOG

Borehole No.
4

Client: PATHWAYS RESIDENCES CREMORNE
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 50-88 PARRAWEEN STREET AND 59-67 GERARD STREET, CREMORNE, NSW

Job No.: 35736S **Core Size:** NMLC **R.L. Surface:** 82.9 m
Date: 22/2/23 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK205 **Bearing:** N/A **Logged/Checked By:** T.F./P.S.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	General	Formation
					START CORING AT 2.80m							
		80			NO CORE 0.20m							
			3		SANDSTONE: fine to medium grained, red brown and light grey, bedded at 0-15°.	MW	L	0.20				Hawkesbury Sandstone
										(3.30m) CS, 0°, 72 mm.t		
										(3.43m) Be, 0°, Ir, R, Fe Sn		
										(3.46m) CS, 0°, 85 mm.t		
ON 22/2/23								0.30				
										(3.69m) Be, 0°, P, R, Fe Sn		
		79										
			4		SANDSTONE: fine to medium grained, light grey, bedded at 0-15°.		VL - L	0.10				
										(4.12m) XWS, 120 mm.t		
75% RETURN							L					
										(4.45m) Be, 0°, P, R, Fe Sn		
										(4.48m) XWS, 90 mm.t		
								0.10				
		78										
			5									
										(5.08m) CS, 0°, 20 mm.t		
								0.20				
								0.30	600 200 60 20			
								0.40				
		77								(5.90m) XWS, 15 mm.t		
										(5.93m) XWS, 90 mm.t		
			6				VL			(6.02m) CS, 280 mm.t		
										(6.30m) XWS, 40 mm.t		
							L - M	0.10				
					SANDSTONE: fine to medium grained, light grey, orange brown and red brown, bedded at 0-10°.							
								0.30				
90% RETURN		76										
			7					0.30				
								0.10				
								0.10				
		75								(7.79m) Be, 0°, P, R, Clay Vn		
			8									
								0.40				
								0.60				
										(8.80m) CS, 10°, 10 mm.t		
		74							600 200 60 20			



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Job No: 35736S
Borehole No: BH4
Depth: 2.80m to 10.00m

CORED BOREHOLE LOG

Borehole No.
4
3 / 3

Client: PATHWAYS RESIDENCES CREMORNE
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 50-88 PARRAWEEN STREET AND 59-67 GERARD STREET, CREMORNE, NSW

Job No.: 35736S
Date: 22/2/23
Plant Type: JK205
Core Size: NMLC
Inclination: VERTICAL
Bearing: N/A
R.L. Surface: 82.9 m
Datum: AHD
Logged/Checked By: T.F./P.S.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1 L -0.3 M -1 H -3 VH -10 EH	DEFECT DETAILS SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	General	Formation
90% RETURN		73	10		SANDSTONE: fine to medium grained, light grey, with grey laminae, bedded at 0-20°. *(continued)*	FR	M	0.70 0.60				
		72 71 70 69 68 67	11 12 13 14 15		END OF BOREHOLE AT 10.00 m							

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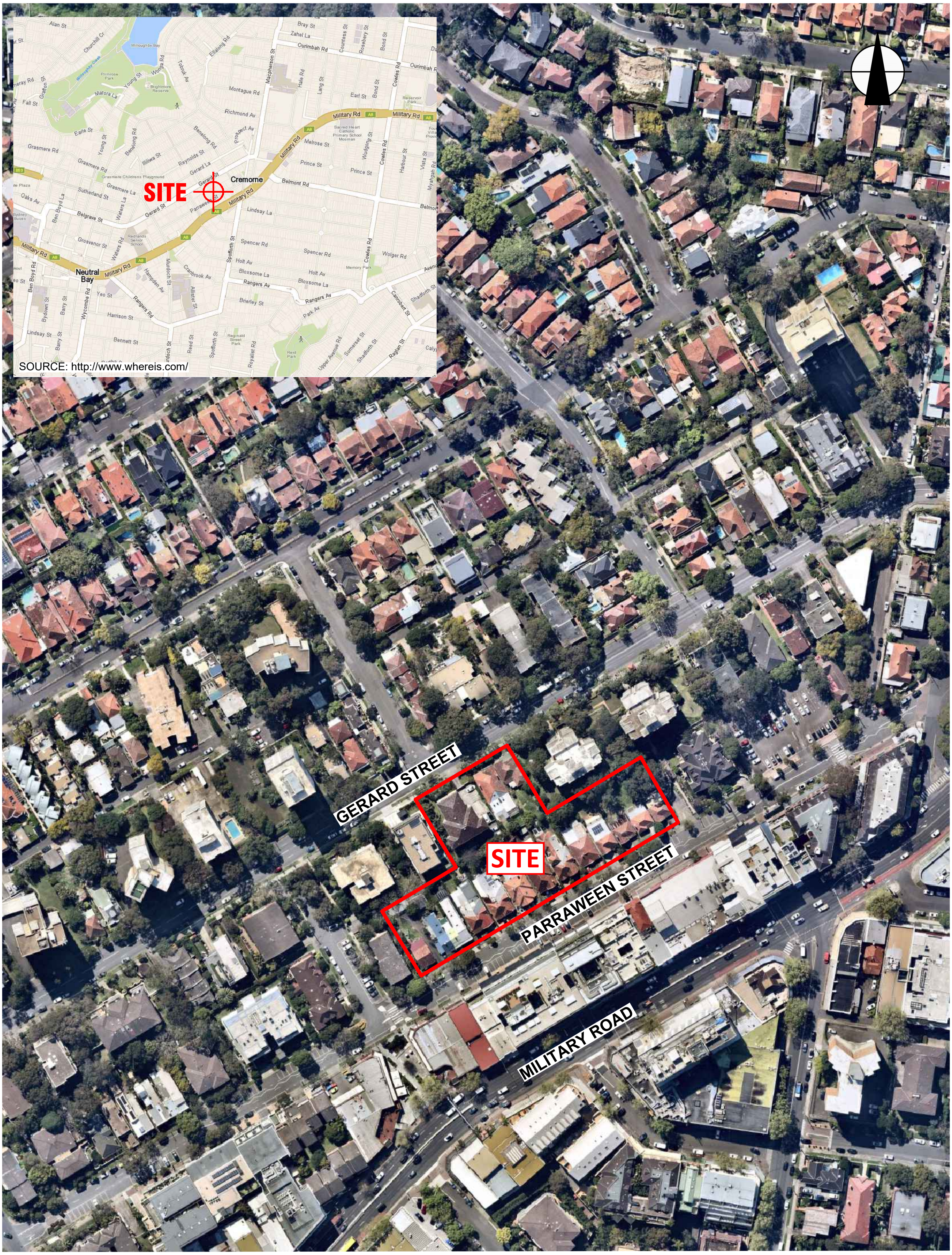
FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



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

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:
SITE LOCATION PLAN

Location:
50-88 PARRAWEEN STREET AND
59-67 GARARD STREET, CREMORNE, NSW

Report No: 35736S

Figure No: 1

JKGeotechnics

PLOT DATE: 27/02/2023 10:35:01 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\35000'S\35736S CREMORNE\CAD\35736S.DWG

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

0 6 12 18 24 30
SCALE 1:600 @A3 METRES

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

Title:
BOREHOLE LOCATION PLAN

Location:
50-88 PARRAWEEN STREET AND
59-67 GARARD STREET, CREMORNE, NSW

Report No: 35736S
Figure No: 2

JKGeotechnics

PLOT DATE: 21/03/2023 10:58:27 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\35000'S\35736S CREMORNE\CAD\35736S.DWG

VIBRATION EMISSION DESIGN GOALS

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

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

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

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

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

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

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

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

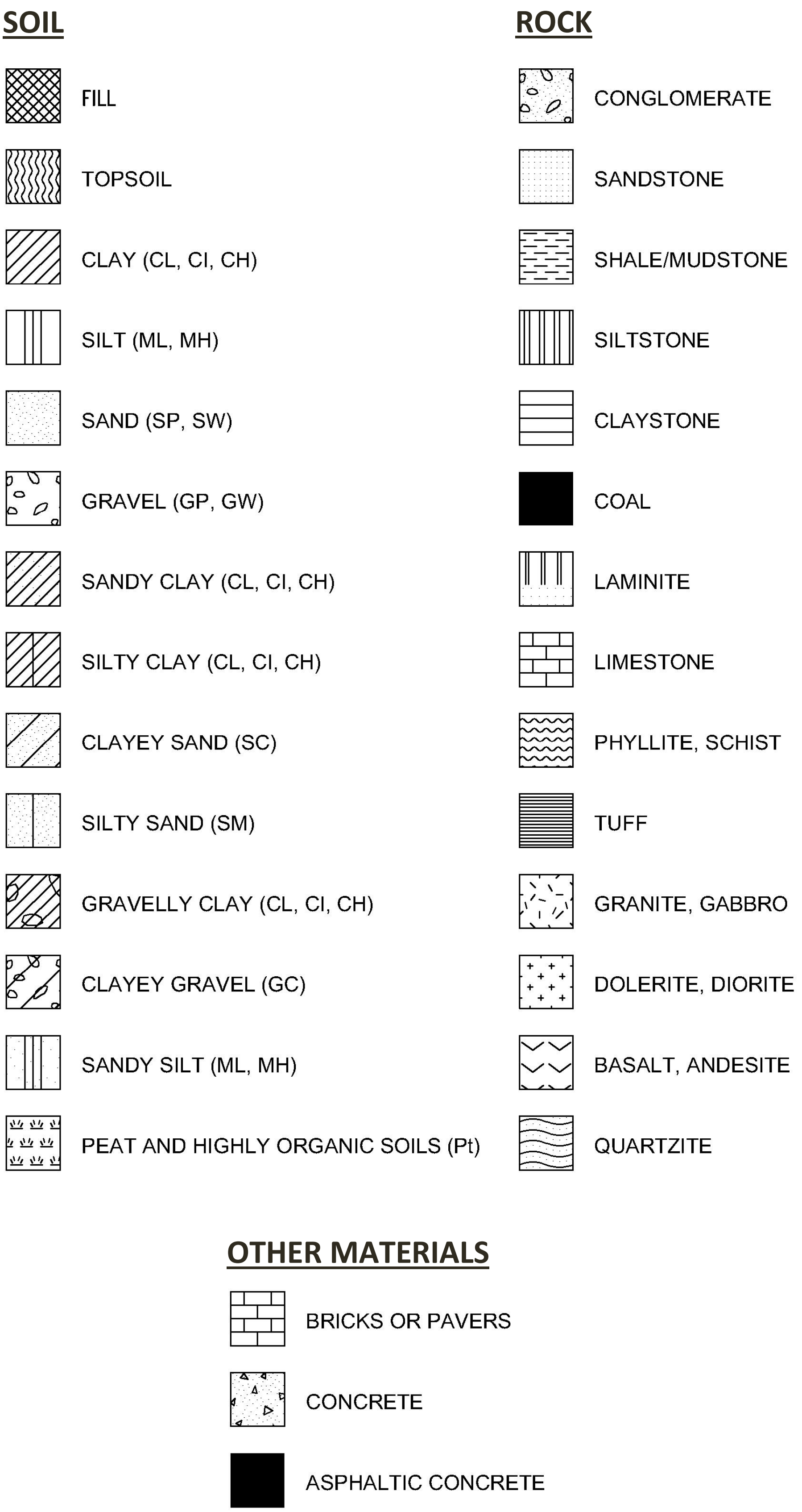
Requirements could range from:

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

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

iii) full time engineering presence on site.

SYMBOL LEGENDS



SOIL

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

ROCK

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

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory **Classification** Criteria

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

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

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

NOTES:

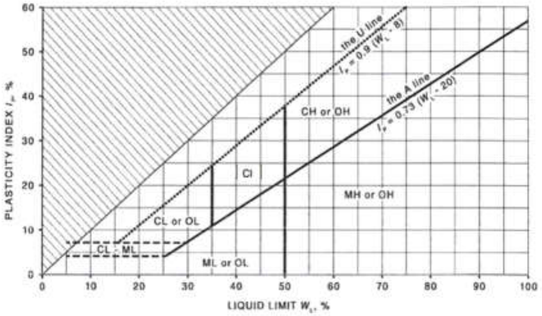
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

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

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

Classification of Material Weathering

<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