REPORT TO

ARCH MANLY MANAGEMENT (AUSTRALIA) PTY LTD ATF ARCH MANLY TRUST

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED RESIDENTIAL DEVELOPEMENT

AT

195-197 SYDNEY ROAD, FAIRLIGHT, NSW

Date: 29 January 2021
Ref: 33708PNrpt

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DOCUMENT REVISION RECORD

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1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed residential development at 195–197 Sydney Road, Fairlight, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Arch Manly Management (Australia) Pty Ltd ATF Arch Manly Trust by return of a signed Acceptance of Proposal form dated 2 December 2020. The commission was on the basis of our fee proposal (Ref: P53140PN), dated 26 November 2020.

We have been provided with the following relevant documents/drawings:

- Architectural drawings (Project No.20025, Drawing Nos. A-2000^{P9}, 2001^{P10}, 2002^{P6}, ,2003^{P4}, 2004^{P4}, 2005^{P4}, 4000^{P3}, 4001^{P3}, 4002^{P2}, 4003^{P4}, 4004^{P1}, and, 5000^{P2}), prepared by Mostaghim & Associates.
- Survey Drawings by Bee & Lethbridge – Project No. 19628B-01, Sheet 1 to Sheet 2, dated 22 June 2018 and 30 April 2018.

From the architectural drawings, we understand the proposed development will comprise two separate 4-storey buildings over one or two basement levels. The northern portion of the basement will be at Reduced Level (RL) 47.925m, but the southern portion of the basement will ramp up to about RL48.5m. Excavation to a maximum depth of about 12m is expected to be required for the proposed basement, but this will reduce to near 0m at the northern boundary. The southern side of the basement will be offset about 5m from the southern site boundary, and the eastern and western sides of the basement will be offset about 2m from the respective boundaries over the majority of the site, but will extend to the boundaries near the northern end of the site.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions, and to use this as a basis for comments and recommendations on excavation conditions, hydrogeology, shoring options, retaining wall design, footing options, and on-grade floor slabs.

This report confirms and amplifies our preliminary advice provided by email dated 12 January 2021.

2 INVESTIGATION PROCEDURE

Eight boreholes, BH1 to BH8, were drilled to depths between 1.57m (BH6) and 12.0m (BH4) predominantly using rotary diamond coring techniques with our portable Melvelle coring equipment. BH2, BH3, BH4 and BH6 were core drilled from surface, and for the remaining boreholes, the soil profile was drilled to a maximum depth of 0.72m using a hand auger prior to coring commencing. The compaction of the fill was assessed from the results of Dynamic Cone Penetrometer (DCP) tests completed adjacent to BH1, BH5 and BH8. The strength of the bedrock was assessed from tactile examination of the recovered rock core and the results of laboratory Point Load Strength Index ($I_{S(50)}$) tests. The results of the Point Load Strength Index tests are presented on the attached Table A and are plotted on the cored borehole logs. Groundwater observations were made during, on completion of drilling each borehole, as well as about 3 weeks after completion of drilling in BH1 to BH6 inclusive.

The borehole locations, as shown on the attached Borehole Location Plan (Figure 2) were set out by taped measurements from existing surface features. The approximate surface levels at the borehole locations were estimated by interpolation between spot heights shown on the provided survey plan. The datum of the levels is Australian Height Datum (AHD).

Our Geotechnical Engineer, Mr Ben Sheppard, was on site full time during the fieldwork and set out the borehole locations, nominated the sampling and testing, prepared the borehole logs and recorded the DCP test results. The borehole logs, core photographs, and DCP test results sheet are attached to this report, together with our Report Explanation Notes which describe the investigation techniques adopted and define the logging terms and symbols used.

Testing for possible soil or groundwater contamination was outside the agreed scope of the investigation.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located towards the crest of a north facing hillside which grades down at about 10° overall. The site itself slopes down to the north at about 12°, resulting in an elevation relief of about 14m between the southern and northern ends of the site. Sydney Road bounds the site to the North.

At the time of the fieldwork, two single storey houses of brick, sandstone masonry, and fibro construction were located centrally on the site, and two brick garages were located close to the northern boundary. The structures all appeared to be in good condition, based on a cursory inspection. To the north and south of the houses were landscaped areas which were divided into more level areas by masonry and stacked sandstone walls a maximum of about 2m high. Sandstone bedrock was exposed in a number of locations across the site, to both the north and south of the existing houses.

To the west of the site was a two-storey rendered residential building over the northern portion of the site, and which abutted the north-west corner of the site, but was set back about 3m elsewhere. At the southern end of the neighbouring building was an asphaltic concrete carparking area which was accessed by a driveway adjacent to the subject site. Along the eastern side of the drive and carpark, adjacent to the site, and at the southern end of the carpark was a sandstone cut face a maximum of about 4m high. A concrete wall was located above the cut face along a portion of the eastern side of the driveway and supported the subject site to a maximum height of about 1.5m. Adjacent to the site, several subvertical joints were observed in the cut face, including one which appeared to have been ‘jacked’ open by tree roots, along with sub-horizontal weathered seams. To the south and above the southern portion of the cut face was a landscaped area. Along the eastern side of the landscaped area, set back about 2m from the site boundary, was a sandstone cliff a maximum of about 6m high with the landscaped area below this cliff.

To the east of the site were two residential properties, with a two-storey brick and clad house adjacent to the houses on site, and set back about 1m from the site boundary. At the southern end of the site a four-storey brick unit building was set back about 5m from the site boundary. Surface levels across the eastern boundary were predominantly similar.

To the south of the site were two separate multi-storey residential apartment buildings which were set back at least 7m from the site boundary, but for the western property, it appeared a garage below the building may extend to the site boundary, as a brick wall with sandstone masonry footing and vent holes was located on the boundary, below a landscaped area.

3.2 Subsurface Conditions

The 1:100,000 Geological Map of Sydney indicates the site to be underlain by Hawkesbury Sandstone.

The boreholes disclosed a generalised subsurface profile comprising shallow fill over sandstone bedrock. Natural soils were not encountered in the boreholes. Reference should be made to the attached borehole logs for detailed subsurface descriptions at specific locations. A summary of the encountered subsurface conditions is presented below.

Pavement

Concrete 20mm thick was penetrated from the surface in BH7.

Fill

Fill comprising silty sand was encountered from the surface in BH1, BH5 and BH8 to depths of 0.37m, 0.43m and 0.55m, respectively. The fill contained inclusions of brick and plastic fragments, and based on the DCP test results, was predominantly assessed to be poorly compacted.

Sandstone Bedrock

Sandstone bedrock was encountered from the surface in BH2, BH3, BH4 and BH6, and from beneath the fill in the remaining boreholes. In BH1 and BH8, the initial approximately 0.1m of the sandstone bedrock was extremely weathered, and in BH7, it is possible that the upper bedrock was a sandstone boulder over soil which was washed away during coring.

The sandstone bedrock was predominantly of medium strength, however, some limited areas of low strength sandstone was also encountered, predominantly over the upper 1m to 21m of the bedrock profile. In BH8, siltstone of limited was encountered between about 3.3m and 4.5m depth. High strength bedrock was also encountered in some boreholes.

Relatively few defects were encountered within the majority of the bedrock profile, and included inclined joints, bedding partings and weathered/clay seams. The 'no core' zones most likely represent bands of weaker rock or soil strength material which was washed away during coring. The majority of the 'no core' zones were of limited thickness, however, as discussed above, the thicker zone in BH7 may be soil below a sandstone boulder, and the thicker seam in BH1 may be extremely weathered siltstone from the same band as was encountered in BH8. This agrees with the observations in the rock cutting in the property to the west, apart from the near vertical joint towards the northern end of that cutting. Given the presence of that large defect, it is likely that similarly oriented joints will be encountered in the proposed excavation.

Groundwater

BH1, BH5, BH7 and BH8 were 'dry' during and on completion of auger drilling. Whilst standing water was measured in the boreholes on completion of coring, as flusg water is introduced into the boreholes during coring, these levels are not considered representative of natural water levels.

About three weeks after BH1 to BH6 were drilled, standing water was measured in these boreholes, however, given the surface topography of the site, with surface levels on the northern boundary lower than the measured water levels, these are also not considered representative of natural groundwater levels. Instead, these elevated levels may be due to surface water ingress into the open boreholes, but also indicate the lower portion of the bedrock profile may be of relatively low permeability which maty not have allowed the flush water to dissipate.

3.3 Laboratory Test Results

The results of the Point Load Strength Index tests (Is_{50}) correlated well with our field assessment of the bedrock strength. The Unconfined Compressive Strength (UCS) of the rock core, estimated from the point load strength index test results, ranged from 1MPa to 36MPa, but were predominantly between 10MPa and 26MPa.

4 COMMENTS AND RECOMMENDATIONS

4.1 Dilapidation Surveys

Prior to any demolition and excavation commencing, we recommend that detailed dilapidation reports be prepared for the adjoining properties to the south, east and west of the site. The dilapidation surveys should comprise detailed inspections both externally and internally, with all defects rigorously described, e.g. defect location, defect type, crack width, crack length, etc. The respective property owners should be provided with a copy of the dilapidation reports and be asked to confirm that they present a fair representation of existing conditions. Such reports can be used as a baseline against which to assess possible future claims for damage arising from the works. We note that Council may also require that any damage to their adjoining assets be reported prior to any works commencing on site.

4.2 Excavation and Vibration

4.2.1 Excavation Conditions

Excavation for the proposed development is expected to extend to a maximum depth of about 12m below existing surface levels. Excavation to such depths will extend through the thin soil profile (where present), and be mostly within the sandstone bedrock profile.

Following installation of appropriate shoring, where necessary, excavation of the soils as well as any extremely low and very low strength bedrock, where encountered, is expected to be readily achievable using conventional techniques such as the buckets of hydraulic excavators.

Excavation through the sandstone bedrock of low and greater strength will be expected to be slower, and we recommend grid sawing and hammering with smaller excavators and/or ripping using a large excavator (at least 30 tonne in size) in combination with sawing. Extreme care must be taken when excavating along the western side of the basement where a plinth of rock will be left between the excavation and the excavation and cliff face within the neighbouring property. For example, when ripping with a 30 tonne excavator, it would be possible to rip blocks out of the plinth, thereby destabilising it. Further details on protection measures during and following excavation are provided in Section 4.3 below.

4.2.2 Potential Vibration Risks

We recommend that considerable caution be taken during rock excavation on the site as there will likely be direct transmission of ground vibrations to the neighbouring buildings to the south, east and west.

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

Excavation using hydraulic rock hammers should commence away from likely critical areas (i.e. commence within the central portion of the site). We recommend that continuous vibration monitoring be carried out during all demolition and excavation works. Vibrations, measured as Peak Particle Velocity (PPV), must be limited to no higher than 5mm/sec for the nearby residential buildings, subject to confirmation by the project structural engineer and/or a specialist vibration consultant that these vibration levels can be tolerated by those structures. This vibration limit must also be reviewed following completion of the dilapidation reports on the nearby buildings. If higher vibrations are recorded, they should be assessed against the attached Vibration Emission Design Goals as higher vibrations may be feasible depending on the associated vibration frequency. If it is confirmed that transmitted vibrations are excessive, then it would be necessary to use smaller plant or alternative techniques, e.g. grid sawing in conjunction with ripping.

The use of a rotary grinder or grid sawing in conjunction with ripping presents an alternative low vibration excavation technique, however, productivity is likely to be slower. 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.
- Use rock hammers in short bursts only to reduce amplification of vibrations.
- Maintain a sharp moil on the hammer.

We recommend use of excavation contractors with experience in such work with a competent supervisor who is aware of vibration damage risks. The contractor should be provided with a full copy of this report and have all appropriate statutory and public liability insurances.

4.2.3 Drainage

The augered portion of the boreholes were 'dry' during and on completion of hand auger drilling, and the water measured in the boreholes after coring is not considered representative of natural groundwater levels. Groundwater, if present, is expected to occur as localised flows of relatively limited volume at the soil-bedrock interface, with some flows also possible along open defects within the bedrock profile, with possibly higher flows during and following heavy rainfall events.

We consider that construction of a drained structure would be feasible and appropriate. Drainage of the minor seepage inflows would not be expected to cause any adverse effects on any surrounding structures or improvements given the very shallow soil profile and very stiff nature of the bedrock.

Long term groundwater flows would be expected to be of limited volume and would be able to be controlled by draining to a sump, or sumps, for gravity disposal to the stormwater system. The completed excavation should be inspected by the hydraulic consultant to confirm that the drainage allowed is sufficient for the actual seepage flows.

Groundwater seepage into the excavation should be monitored by the site foreman and geotechnical engineer as excavation progresses to confirm that seepage volumes are within the range anticipated.

Overall, it is considered that the construction of the proposed development will not be adversely affected by groundwater provided engineer designed drainage systems are constructed. Similarly, it is not expected that the development will have an adverse effect on the regional groundwater flows.

4.3 Retention

4.3.1 Retention Options

Retention of the soil profile, and any extremely weathered bedrock, is expected to be required in at least some portions of the site as excavation will extend to, or very close to, the site boundaries towards the northern end of the site. However, where the excavation is set back from the boundaries, the soil and extremely weathered bedrock profile can be battered at no steeper than 1 Vertical in 1.5 Horizontal in the temporary case. Rather than battered excavations, consideration could also be given to excavating a trench just outside the excavation perimeter, with a width of half the soil depth (minimum 0.4m wide), and extending down to low strength or stronger bedrock. Drainage should be installed such as with lengths of geotextile wrapped strip drains down the face of the trench, across the base, and up the near face at approximately 1.5m centres. The trench could then be filled with mass concrete to form a gravity retaining wall founded on bedrock.

The majority of the sandstone bedrock profile is expected to be suitable to stand vertically unsupported in the long term subject to geotechnical inspection at no greater than 1.5m excavation increments. The purpose of such inspections is to check for the presence of adversely orientated defects or weathered seams which may require treatment, e.g. rock bolts, shotcrete, etc. Allowance must be made in both the project budget and program for such inspections and installation of any required treatment. In particular, treatment is expected to be required for the following features:

- The thick siltstone seam and no core zone encountered in BH8 and BH1 respectively, if encountered above excavation level. This will likely require bolting and shotcrete;
- The thinner weathered seams and no core zones encountered in all of the boreholes. These will likely require dry packing with mortar; and
- Bolting of potentially unstable blocks formed by joints, similar to features identified within the cut face in the neighbouring property to the west.

In the permanent case, any unstable features should be supported by the building structure to avoid the need for obtaining easements/permission for permanent rock bolts extending across site boundaries (if required), or the need for designing for long term corrosion considerations. If the sandstone cut faces within the basement are to be left unsupported in the permanent case, access must be provided for cleanup of any fretting. Alternatively, walls within 0.1m of the cut faces could be backfilled with a free draining gravel.

With regards to the plinth which will be formed between the proposed excavation and the cut within the neighbouring property to the west, we note that over the northern end of the neighbouring site, several potentially unstable blocks were observed. In addition, as excavation progresses, existing defects may well lead to additional unstable blocks being formed.

From a practical perspective, we consider the most suitable method to address the risk of instability of the existing cut face and plinth would be for the plinth to be removed in conjunction with the excavation for the proposed development. However, this would require negotiating with, and permission from, the neighbouring property owner. Careful staging would also be required so that risks to both the occupants of the neighbouring property and staff on site are appropriately managed. Provisionally, we recommend that if this option is adopted, then the plinth be completely removed back from the northern boundary to the point where the width of the plinth is no greater than its height above driveway level in the neighbouring property.

Alternatively, the existing potentially unstable features in the cut face could be stabilised, and the plinth could be strengthened in conjunction with the excavation works. This will likely require the following:

- Bolting and shotcreting of existing unstable features from within the neighbouring property prior to excavation commencing;
- Possible strengthening or replacement of the existing shoring wall at the top of the cut face;
- Installation of tensioned vertical rock anchors from the top of the plinth to below bulk excavation level. Indicatively, approx. 32mm diameter bars at 2m centres would be expected to be required;

- Progressive installation of horizontal bolts through the plinth from within the excavation to ‘stitch’ the plinth together. Indicatively 24mm bars on a 2m by 2m grid would be expected to be required, however, additional bolts may also be required to address specific features; and
- Shotcreting of the exposed face of the plinth from within the excavation to ‘tie’ the rock bolts together.

This option would also require negotiation with, and approval of, the neighbouring property owner as a portion of the works would extent into their property. All of the bolts/anchors within the plinth would need to be designed as permanent elements with an appropriate allowance for long term corrosion. Indicatively, encapsulated hot dip galvanised bars with a corrosion allowance or stainless steel bars would be required. To finalise the design of the strengthening measures, sections should be prepared at 2m increments along the northern boundary showing the dimensions of the plinth and details of the existing cut face in the neighbouring property. Further, these rock bolts would extend across the boundary and would likely require the registration of an easement for support.

4.3.2 General Shoring Design Parameters

Free standing cantilever walls, where some wall movements are tolerable, should be designed using a triangular lateral earth pressure distribution and with an ‘active’ earth pressure coefficient, K_a, of 0.35 for the soil and any extremely weathered bedrock, as well as for any backfill materials, assuming a horizontal retained surface.

Cantilever walls which will be propped or restrained by structures and subsequently backfilled, or where wall movements are to be limited, should be designed using a triangular lateral earth pressure distribution and with an 'at rest' earth pressure coefficient, K_0, of 0.5 for the soil and extremely weathered bedrock profile, as well as for any backfill materials, assuming a horizontal retained surface.

A bulk unit weight of 20kN/m^3 should be adopted for the retained materials (existing soils and backfill materials).

Any surcharge affecting the walls (e.g. traffic loads, construction loads, etc.) should be allowed for in the design using the appropriate earth pressure coefficient from above. Sloping retained surfaces should be treated as a surcharge or, alternatively, the earth pressure coefficient should be appropriately increased.

Complete and permanent drainage of the ground behind the walls should be provided. Subsurface drains should incorporate a non-woven geotextile fabric (e.g. Bidim A34), to act as a filter against subsoil erosion.

Temporary and permanent rock bolts bonded into medium or greater strength sandstone bedrock may be designed on the basis of a maximum allowable bond stress of 250kPa.

4.4 Footing Design

On completion of excavation, sandstone bedrock of predominantly medium strength is expected to be exposed across the entire footprint of the proposed basement.

We therefore recommend that the new house be uniformly supported on pad and strip footings founded within the sandstone bedrock profile.

For shallow footings founded in sandstone bedrock of at least medium strength, an allowable bearing pressure of 2,500kPa can be adopted, based on serviceability criteria. All footings must be clean of any loose or water softened material and free of standing water prior to pouring concrete.

All footings must be inspected by a geotechnical engineer to confirm that an appropriate foundation material has been achieved. In addition, spoon testing must be completed on at least 1/3 of all footings to check for the presence of open joints or compressible material below the base of the footing. Over the northernmost 15m of the site, spoon tests must be completed in all footings to confirm the lateral extent of the siltstone seam identified in BH8. Depending on what is found from the initial spoon testing, additional tests may be recommended.

Should open joints or compressible material be identified by spoon testing, it may be necessary to reduce the allowable bearing pressure for some footings, or deepen the footing excavation to found below such features.

Special consideration will need to be given to any proposed footings, which are to be located close to the crest of excavation cuts, or natural steps in the bedrock. Any footings within a distance of 5m from the crest of a cut face or step will require special consideration. Thorough inspection of the nearby rock face will be required to check for the presence of adversely orientated defects which may require additional stabilisation.

4.5 On-Grade Floor Slabs

The sandstone bedrock is considered a suitable subgrade for on-grade floor slabs.

Drainage, comprising single size aggregate and subsoil drains should be provided below all on grade slabs. The hydraulic consultant should inspect the completed excavation to assess if the design drainage system is adequate for the actual seepage flows.

4.6 Earthquake Design

In accordance with AS1170.4-2007, the site subsoil class is 'Class B_e – Rock', and the hazard design factor (z) is 0.08.

4.7 Further Geotechnical Input

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

- Detail design of stabilisation/strengthening measures for the plinth on the western boundary;
- Quantitative vibration monitoring during rock excavation;
- Seepage monitoring during excavation;
- Progressive inspection of excavated cut faces to confirm if additional stabilisation measures are required;
- Geotechnical footing inspections including spoon testing.

5 GENERAL COMMENTS

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

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:	Arch Manly Management (Australia) Pty Ltd ATF Arch Many Trust	**Ref No:**	33708PN
Project:	Proposed Residential Development	**Report:**	A
Location:	195-197 Sydney Road, FAIRLIGHT, NSW	**Report Date:**	4/01/20

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BOREHOLE NUMBER	DEPTH (m)	IS (50) (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
1	0.48 - 0.50	0.1	2	A
	0.82 - 0.85	0.2	4	A
	1.11 - 1.13	0.2	4	A
	1.86 - 1.90	0.06	1	A
2	0.12 - 0.16	0.5	10	A
	0.70 - 0.74	0.4	8	A
	1.02 - 1.06	0.4	8	A
	1.71 - 1.75	0.3	6	A
	2.22 - 2.26	0.6	12	A
	2.69 - 2.73	0.3	6	A
	3.26 - 3.30	0.3	6	A
	3.88 - 3.91	0.8	16	A
	4.31 - 4.64	0.3	6	A
	4.77 - 4.81	0.9	18	A
	5.07 - 5.10	0.9	18	A
	5.74 - 5.77	0.8	16	A
	6.24 - 6.27	0.6	12	A
	6.79 - 6.83	0.7	14	A
	7.08 - 7.11	0.4	8	A
	7.76 - 7.78	1.3	26	A
	8.17 - 8.19	1.6	32	A
	8.80 - 8.82	1.1	22	A
	9.23 - 9.27	1.1	22	A
	9.83 - 9.85	1.5	30	A
	10.24 - 10.28	0.9	18	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 IS(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 IS(50).

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	Arch Manly Management (Australia) Pty Ltd ATF Arch Many T	**Ref No:**	33708PN
Project:	Proposed Residential Development	**Report:**	A
Location:	195-197 Sydney Road, FAIRLIGHT, NSW	**Report Date:**	4/01/20
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BOREHOLE NUMBER	DEPTH (m)	IS (50) (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
3	0.29 - 0.31	0.6	12	A
	0.83 - 0.86	0.8	16	A
	1.15 - 1.17	0.7	14	A
	1.57 - 1.60	0.7	14	A
4	0.09 - 0.12	0.6	12	A
	0.76 - 0.79	0.8	16	A
	1.12 - 1.16	1	20	A
	1.75 - 1.78	0.5	10	A
	2.16 - 2.19	0.9	18	A
	2.96 - 3.00	1.6	32	A
	3.17 - 3.20	1.4	28	A
	3.75 - 3.78	1.6	32	A
	4.06 - 4.09	1.5	30	A
	4.83 - 4.85	0.9	18	A
	5.24 - 5.28	0.6	12	A
	5.80 - 5.83	0.7	14	A
	6.12 - 6.15	1.5	30	A
	7.21 - 7.24	0.6	12	A
	7.74 - 7.77	0.7	14	A
	8.35 - 8.38	1.2	24	A
	8.85 - 8.88	1.8	36	A
	9.24 - 9.28	1.3	26	A
	9.24 - 9.28	1.3	26	A
	9.24 - 9.28	1.3	26	A
	9.24 - 9.28	1.3	26	A

NOTE: SEE PAGE 1

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	Arch Manly Management (Australia) Pty Ltd ATF Arch Many T	**Ref No:**	33708PN
Project:	Proposed Residential Development	**Report:**	A
Location:	195-197 Sydney Road, FAIRLIGHT, NSW	**Report Date:**	4/01/20

Page 3 of 1

BOREHOLE NUMBER	DEPTH (m)	IS (50) (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
4	9.24 - 9.28	1.3	26	A
	9.24 - 9.28	1.3	26	A
5	0.45 - 0.48	0.8	16	A
	0.86 - 0.89	1	20	A
	1.26 - 1.29	1	20	A
	1.76 - 1.80	0.4	8	A
	2.16 - 2.19	0.7	14	A
	2.75 - 2.78	0.7	14	A
	3.18 - 3.22	0.6	12	A
	3.77 - 3.80	0.8	16	A
	4.15 - 4.18	0.6	12	A
	4.80 - 4.83	0.8	16	A
	5.37 - 5.41	0.9	18	A
	5.85 - 5.87	0.8	16	A
	6.24 - 6.27	1.2	24	A
	6.79 - 6.83	1.2	24	A
	7.17 - 7.20	0.9	18	A
	7.84 - 7.87	0.9	18	A
	8.24 - 8.27	1.7	34	A
	8.85 - 8.87	1.4	28	A
	9.22 - 9.25	1.8	36	A
	9.69 - 9.73	1.2	24	A
	10.00 - 10.04	1.2	24	A
	10.61 - 10.64	1.1	22	A
6	0.25 - 0.28	0.8	16	A

NOTE: SEE PAGE 1

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	Arch Manly Management (Australia) Pty Ltd ATF Arch Many T	**Ref No:**	33708PN
Project:	Proposed Residential Development	**Report:**	A
Location:	195-197 Sydney Road, FAIRLIGHT, NSW	**Report Date:**	4/01/20
		Page 4 of 1	

BOREHOLE NUMBER	DEPTH (m)	IS (50) (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
6	0.72 - 0.75	0.7	14	A
	1.10 - 1.13	0.9	18	A
	1.53 - 1.57	0.9	18	A
7	0.09 - 0.11	0.4	8	A
	1.15 - 1.18	0.6	12	A
	1.57 - 1.60	0.5	10	A
8	1.16 - 1.19	0.2	4	A
	1.73 - 1.75	0.5	10	A
	1.83 - 1.86	0.6	12	A
	2.10 - 2.13	0.6	12	A
	3.10 - 3.13	0.7	14	A
	4.33 - 4.37	0.04	1	A
	4.90 - 4.94	1.3	26	A
	5.19 - 5.22	1.2	24	A
	5.89 - 5.93	0.8	16	A
	6.02 - 6.06	1	20	A

NOTE: SEE PAGE 1

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BOREHOLE LOG

Borehole No.
1
1 / 2

Client: ARCH MANLY MANAGEMENT (AUSTRALIA) PTY LTD ATF ARCH MANLY TRUST
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 195-197 SYDNEY ROAD, FAIRLIGHT, NSW

Job No.: 33708PN
Method: HAND AUGER
R.L. Surface: ~48.9 m
Date: 16/12/20
Datum: AHD
Plant Type:
Logged/Checked By: B.S./N.E.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										
DRY ON COMPLETION OF AUGERING				■ ■	REFER TO DCP TEST RESULTS			▨		FILL: Silty sand, fine to medium grained, dark brown, trace of fine to medium grained sandstone gravel, brick fragments, root fibres and plastic.	M			APPEARS POORLY COMPACTED
									-	Extremely Weathered sandstone: SAND, fine to medium grained, yellow brown and orange brown, trace of fine grained sandstone gravel and silt fines.	XW	VD		HAWKESBURY SANDSTONE
										REFER TO CORED BOREHOLE LOG				HAND AUGER REFUSAL
						48	1							
						47	2							
						46	3							
						45	4							
						44	5							
						43	6							
						42								

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CORED BOREHOLE LOG

Borehole No.
1

Client: ARCH MANLY MANAGEMENT (AUSTRALIA) PTY LTD ATF ARCH MANLY TRUST
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 195-197 SYDNEY ROAD, FAIRLIGHT, NSW

Job No.: 33708PN
Core Size: NMLC
R.L. Surface: ~48.9 m
Date: 16/12/20
Inclination: VERTICAL
Datum: AHD
Plant Type: MELVELLE
Bearing: N/A
Logged/Checked By: B.S./N.E.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
ON 7/1/21					START CORING AT 0.37m							
80% RETURN		48	1		SANDSTONE: fine to medium grained, orange brown and grey, bedded at 0-10°.	SW	L	0.10 0.20 0.20		(0.41m) J, 30°, P, Fe Sn (0.53m) Be, 0°, R, Po, Fe Sn (1.03m) J, 45°, R, Po, Fe Sn (1.18m) J, 40°, R, Po, Clay Ct		Hawkesbury Sandstone
					NO CORE 0.61m							
		47			SANDSTONE: fine to medium grained, orange brown and grey, bedded at 0-10°.	MW	VL	0.060		(1.80m) XWS, 10°, 20 mm.t (1.90m) J, 25°, R, Po, Cn		
		46 45 44 43 42	2 3 4 5 6		END OF BOREHOLE AT 1.95 m				600 200 60 20 600 200 60 20			

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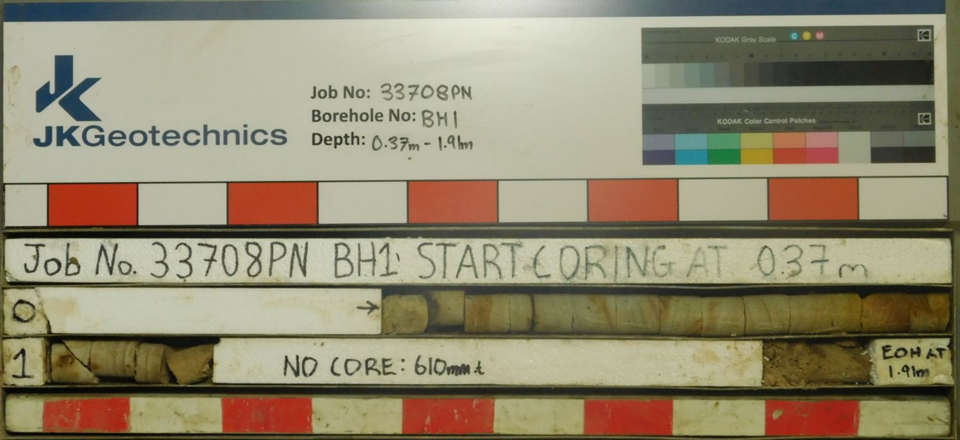
FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



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Job No: 33708PN
Borehole No: BH1
Depth: 0.37m - 1.91m

CORED BOREHOLE LOG

Borehole No.
2
1 / 2

Client: ARCH MANLY MANAGEMENT (AUSTRALIA) PTY LTD ATF ARCH MANLY TRUST
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 195-197 SYDNEY ROAD, FAIRLIGHT, NSW

Job No.: 33708PN
Core Size: NMLC
R.L. Surface: ~55.8 m
Date: 16/12/20
Inclination: VERTICAL
Datum: AHD
Plant Type: MELVELLE
Bearing: N/A
Logged/Checked By: B.S./N.E.S.

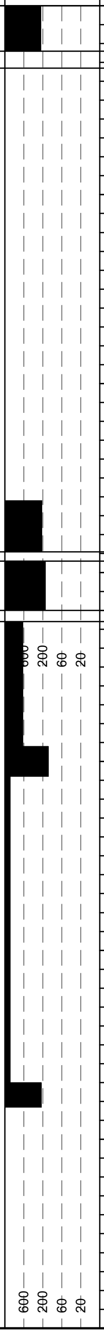
Water Loss\Level: 50% RETURN; ON COMPLETION OF CORING; ON 7/1/21

RL (m AHD): 55, 54, 53, 52, 51, 50, 49
Depth (m): 1, 2, 3, 4, 5, 6

CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components START CORING AT 0.00m	Weathering	Strength	Formation
SANDSTONE: fine to medium grained, grey and orange brown.	SW	M	Hawkesbury Sandstone
NO CORE 0.09m			
SANDSTONE: fine to medium grained, grey and orange brown.	SW	M	
SANDSTONE: fine to medium grained, grey, bedded at 0-10°, with coarse grained bands.			
NO CORE 0.05m			
SANDSTONE: fine to coarse grained, orange brown and grey, with fine grained quartz gravel.	SW	M	
NO CORE 0.06m			
SANDSTONE: fine to coarse grained, orange brown and grey, with fine grained quartz gravel.	SW	M	Hawkesbury Sandstone
SANDSTONE: fine to medium grained, grey.			
SANDSTONE: fine to medium grained, red brown, with iron indurated bands, bedded at 0-15°.	MW		
SANDSTONE: fine to medium grained, grey.	FR		
SANDSTONE: fine to medium grained, grey, with dark grey laminae and carbonaceous lenses, bedded at 0-10°.			

POINT LOAD STRENGTH INDEX $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH): 0.50, 0.40, 0.40, 0.30, 0.60, 0.30, 0.30, 0.80, 0.30, 0.90, 0.90, 0.80, 0.60, 0.70

DEFECT DETAILS



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

Specific	General
(2.62m) CS, 0°, 10 mm.t	
(2.94m) XWS, 0°, 2 mm.t	
(3.92m) Be, 5°, R, Po, Fe Sn	
(4.08m) Be, 0°, Ir, Po, Cn	
(5.70m) Be, 5°, Un, R, Fe Sn	
(5.93m) Be, 5°, Un, R, Cn	

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

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CORED BOREHOLE LOG

Borehole No.
2
2 / 2

Client: ARCH MANLY MANAGEMENT (AUSTRALIA) PTY LTD ATF ARCH MANLY TRUST
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 195-197 SYDNEY ROAD, FAIRLIGHT, NSW

Job No.: 33708PN
Core Size: NMLC
R.L. Surface: ~55.8 m
Date: 16/12/20
Inclination: VERTICAL
Datum: AHD
Plant Type: MELVELLE
Bearing: N/A
Logged/Checked By: B.S./N.E.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
50% RETURN		48 47 46	8 9 10		SANDSTONE: fine to medium grained, grey, with dark grey laminae and carbonaceous lenses, bedded at 0-10°. *(continued)*	FR	M H	0.40 1.3 1.6 1.1 1.1			Hawkesbury Sandstone
					SANDSTONE: fine to medium grained, grey.			1.5 0.90			
		45 44 43 42	11 12 13		END OF BOREHOLE AT 10.30 m				600 200 60 20		

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

Job No: 33708PN
Borehole No: BH2
Depth: 0m - 10.3m

CORED BOREHOLE LOG

Borehole No.
3
1 / 1

Client: ARCH MANLY MANAGEMENT (AUSTRALIA) PTY LTD ATF ARCH MANLY TRUST
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 195-197 SYDNEY ROAD, FAIRLIGHT, NSW

Job No.: 33708PN
Core Size: NMLC
R.L. Surface: ~58.5 m
Date: 16/12/20
Inclination: VERTICAL
Datum: AHD
Plant Type: MELVELLE
Bearing: N/A
Logged/Checked By: B.S./N.E.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 START CORING AT 0.00m	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
60% RETURN					NO CORE 0.10m							
ON COMPLETION OF CORING ON 7/1/21		58 57	1		SANDSTONE: fine to medium grained, light grey mottled orange brown.	SW	M	0.60 0.80 0.70 0.70				Hawkesbury Sandstone
		56 55 54 53 52	2 3 4 5 6		END OF BOREHOLE AT 1.60 m				600 200 60 20			

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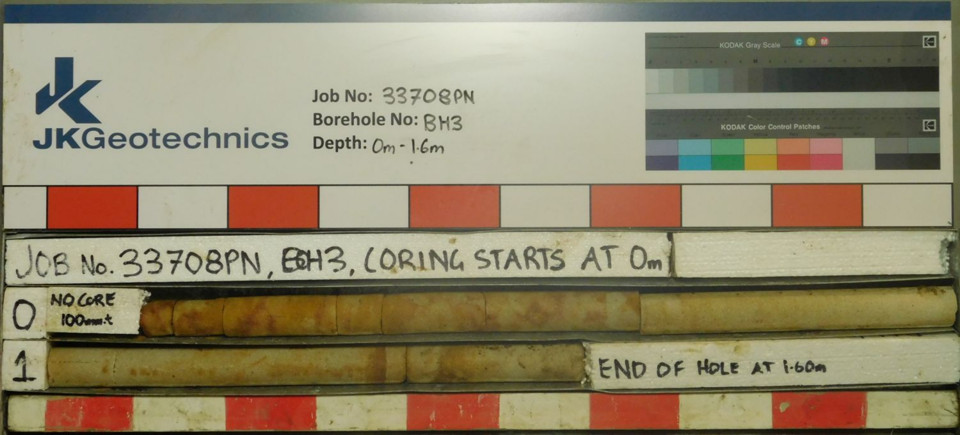
FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



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Job No: 33708PN
Borehole No: BH3
Depth: 0m - 1.6m

JOB No. 33708PN, ~~E~~BH3, CORING STARTS AT 0m

0 NO CORE 100mm±

1 END OF HOLE AT 1.60m

CORED BOREHOLE LOG

Borehole No.
4
1 / 2

Client: ARCH MANLY MANAGEMENT (AUSTRALIA) PTY LTD ATF ARCH MANLY TRUST
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 195-197 SYDNEY ROAD, FAIRLIGHT, NSW

Job No.: 33708PN
Core Size: NMLC
R.L. Surface: ~59.6 m
Date: 16/12/20 TO 17/12/20
Inclination: VERTICAL
Datum: AHD
Plant Type: MELVELLE
Bearing: N/A
Logged/Checked By: B.S./N.E.S.

Water Loss\Level	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components START CORING AT 0.00m	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	Formation
	SANDSTONE: fine to medium grained, light grey.	SW	M - H	0.60 0.80 1.0		Hawkesbury Sandstone
	NO CORE 0.11m					
50% RETURN	SANDSTONE: fine to medium grained, light grey.	SW	M - H H	0.50 0.90 1.6 1.4		Hawkesbury Sandstone
	SANDSTONE: fine to medium grained, light grey, with orange brown laminae, bedded at 0-25°, with occasional coarse grained bands.		H	1.6 1.5	(4.29m) Be, 5°, Un, R, Cn	
ON COMPLETION OF CORING ON 7/1/21	SANDSTONE: fine to coarse grained, red brown and orange brown, bedded at 0-10°, with cross bedding up to 40°.	MW	M - H	0.90 0.60 0.70 1.5	(5.78m) Be, 0°, P, R, Clay Ct	
	NO CORE 0.76m					

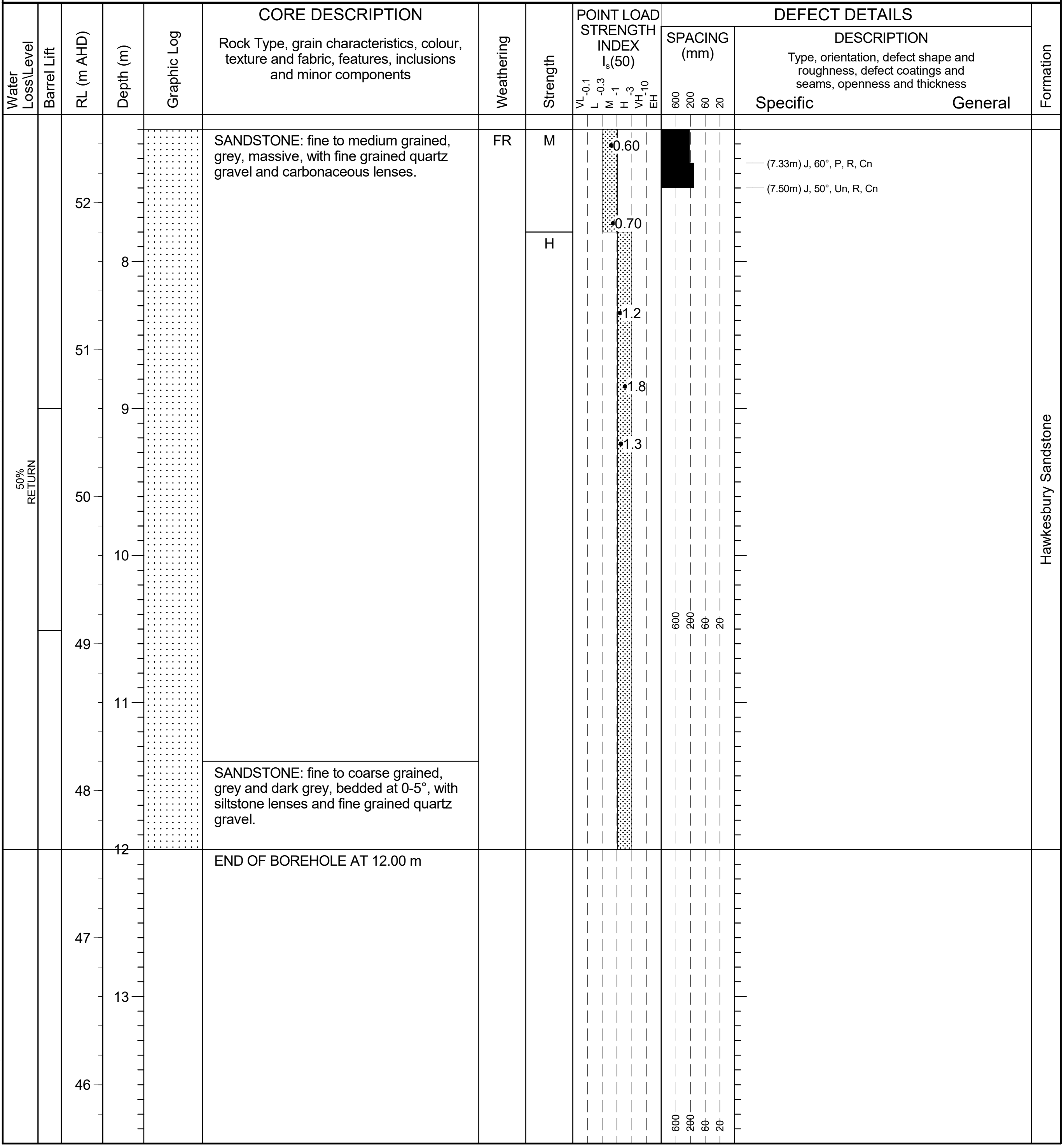
RL (m AHD): 59, 58, 57, 56, 55, 54, 53
Depth (m): 1, 2, 3, 4, 5, 6
Strength scale: VL -0.1, L -0.3, M -1, H -3, VH -10, EH
SPACING (mm): 600, 200, 60, 20

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

CORED BOREHOLE LOG

Client: ARCH MANLY MANAGEMENT (AUSTRALIA) PTY LTD ATF ARCH MANLY TRUST
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 195-197 SYDNEY ROAD, FAIRLIGHT, NSW

Job No.: 33708PN **Core Size:** NMLC **R.L. Surface:** ~59.6 m
Date: 16/12/20 TO 17/12/20 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: MELVELLE **Bearing:** N/A **Logged/Checked By:** B.S./N.E.S.



SANDSTONE: fine to medium grained, grey, massive, with fine grained quartz gravel and carbonaceous lenses.

(7.33m) J, 60°, P, R, Cn

(7.50m) J, 50°, Un, R, Cn

SANDSTONE: fine to coarse grained, grey and dark grey, bedded at 0-5°, with siltstone lenses and fine grained quartz gravel.

END OF BOREHOLE AT 12.00 m

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

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Job No: 33708PN
Borehole No: BH4
Depth: 0m - 12m

Job No. 33708PN BH4 START CORING AT 0m

NO CORE 110mm.t

← NO CORE: 0.75mts.

END OF HOLE AT 12.0m

JKGeotechnics

BOREHOLE LOG

Borehole No.
5
1 / 3

Client: ARCH MANLY MANAGEMENT (AUSTRALIA) PTY LTD ATF ARCH MANLY TRUST
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 195-197 SYDNEY ROAD, FAIRLIGHT, NSW

Job No.: 33708PN
Method: HAND AUGER
R.L. Surface: ~57.3 m
Date: 17/12/20
Datum: AHD
Plant Type:
Logged/Checked By: B.S./N.E.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										
DRY ON COMPLETION OF AUGERING					REFER TO DCP TEST RESULTS			▨		FILL: Silty sand, fine to medium grained, dark brown, trace of root fibres.	M			GRASS COVER
				■		57		▨		FILL: Silty sand, fine grained, trace of fine grained sandstone gravel.	D			APPEARS POORLY COMPACTED
										REFER TO CORED BOREHOLE LOG				APPEARS MODERATELY COMPACTED HAND AUGER REFUSAL
							1							
						56								
							2							
						55								
							3							
						54								
							4							
						53								
							5							
						52								
							6							
						51								

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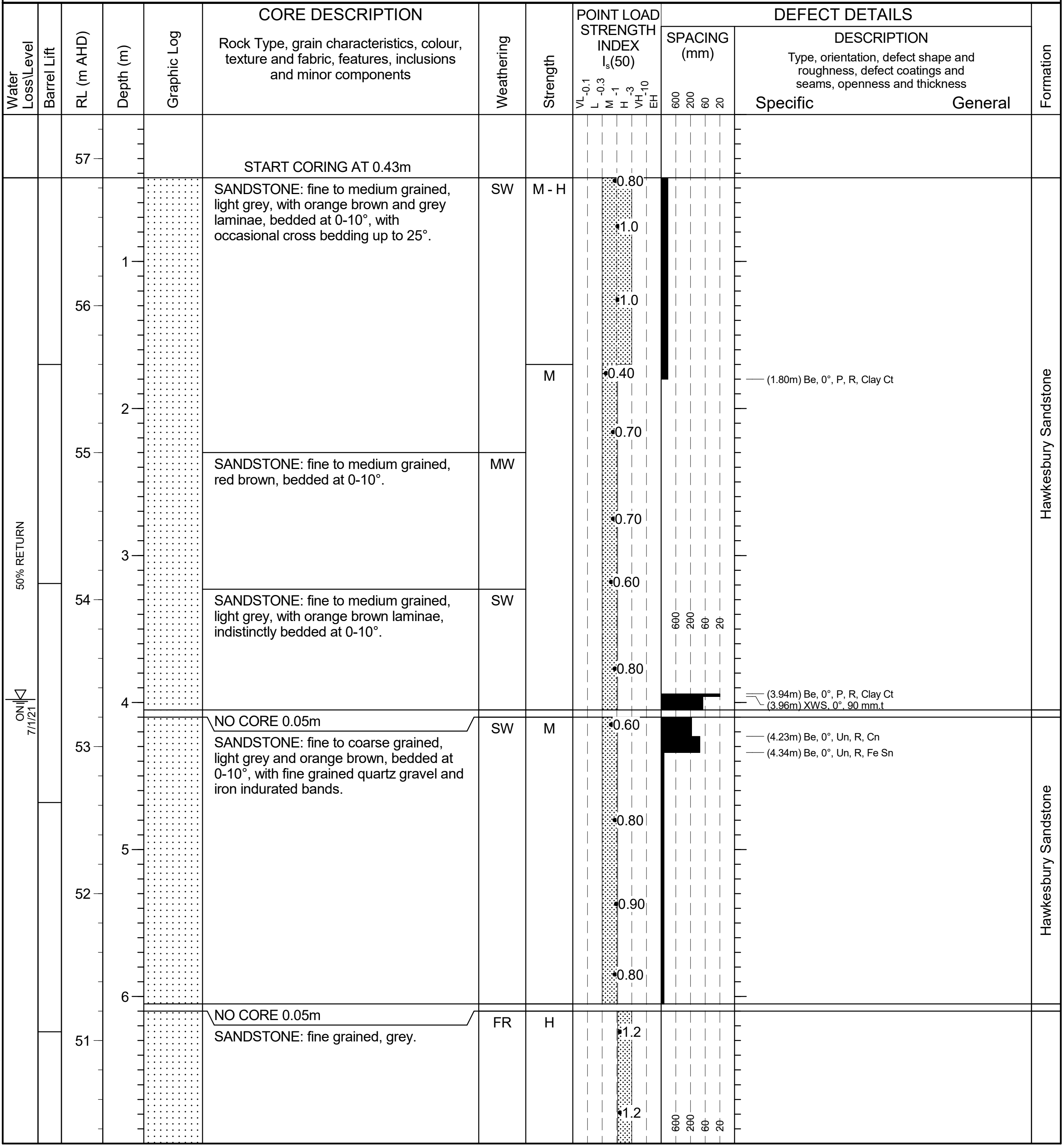
JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 33708PN FAIRLIGHT.GPJ <<DrawingFile>> 28/01/2021 13:45 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

CORED BOREHOLE LOG

Borehole No.
5
2 / 3

Client: ARCH MANLY MANAGEMENT (AUSTRALIA) PTY LTD ATF ARCH MANLY TRUST
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 195-197 SYDNEY ROAD, FAIRLIGHT, NSW

Job No.: 33708PN
Core Size: NMLC
R.L. Surface: ~57.3 m
Date: 17/12/20
Inclination: VERTICAL
Datum: AHD
Plant Type: MELVELLE
Bearing: N/A
Logged/Checked By: B.S./N.E.S.



START CORING AT 0.43m

SANDSTONE: fine to medium grained, light grey, with orange brown and grey laminae, bedded at 0-10°, with occasional cross bedding up to 25°.

SANDSTONE: fine to medium grained, red brown, bedded at 0-10°.

SANDSTONE: fine to medium grained, light grey, with orange brown laminae, indistinctly bedded at 0-10°.

NO CORE 0.05m

SANDSTONE: fine to coarse grained, light grey and orange brown, bedded at 0-10°, with fine grained quartz gravel and iron indurated bands.

NO CORE 0.05m

SANDSTONE: fine grained, grey.

(1.80m) Be, 0°, P, R, Clay Ct
(3.94m) Be, 0°, P, R, Clay Ct
(3.96m) XWS, 0°, 90 mm.t
(4.23m) Be, 0°, Un, R, Cn
(4.34m) Be, 0°, Un, R, Fe Sn

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 33708PN FAIRLIGHT.GPJ <<DrawingFile>> 28/01/2021 13:45 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

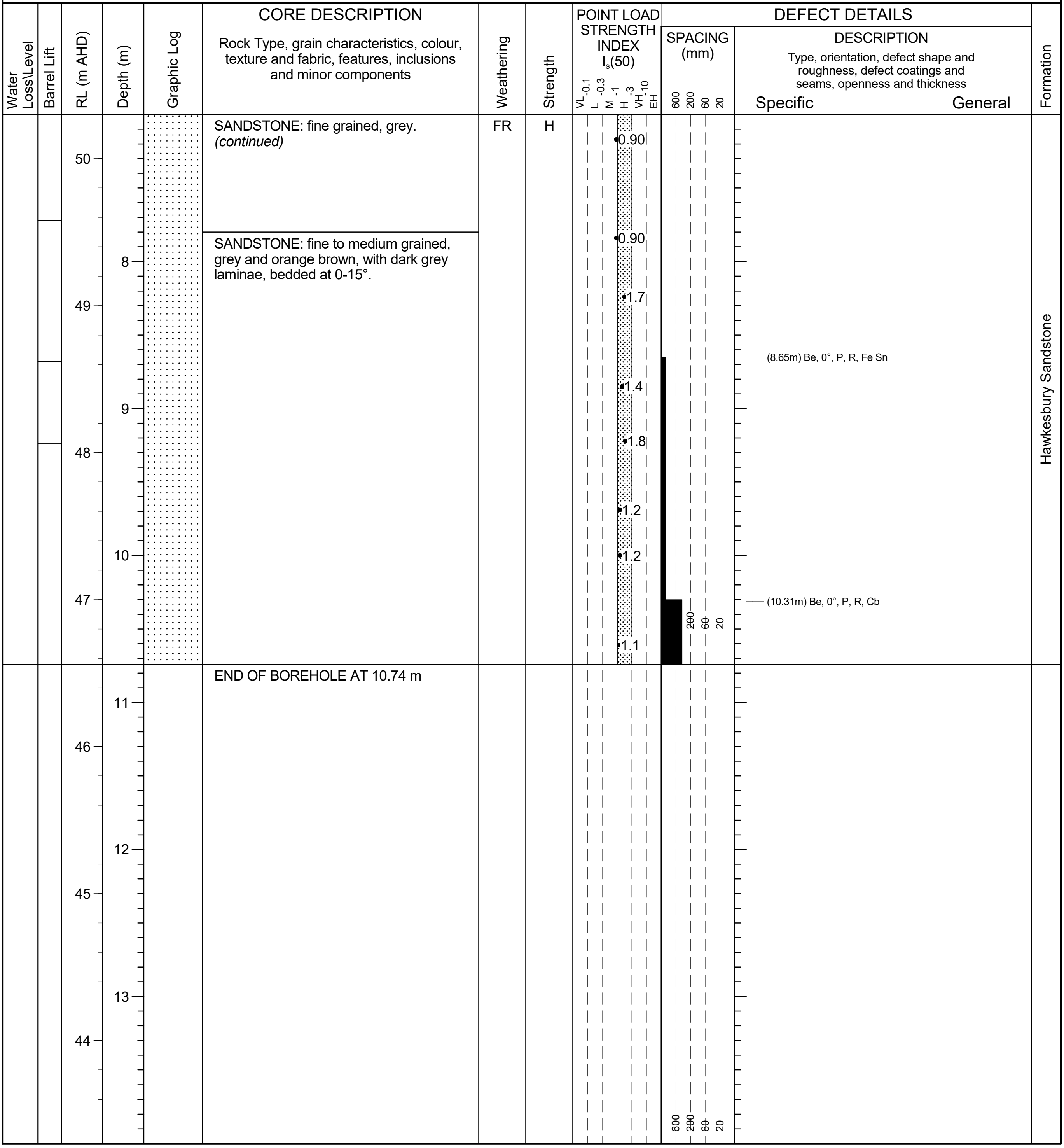
FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

CORED BOREHOLE LOG

Borehole No.
5
3 / 3

Client: ARCH MANLY MANAGEMENT (AUSTRALIA) PTY LTD ATF ARCH MANLY TRUST
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 195-197 SYDNEY ROAD, FAIRLIGHT, NSW

Job No.: 33708PN
Core Size: NMLC
R.L. Surface: ~57.3 m
Date: 17/12/20
Inclination: VERTICAL
Datum: AHD
Plant Type: MELVELLE
Bearing: N/A
Logged/Checked By: B.S./N.E.S.



CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness — Specific	General	Formation
SANDSTONE: fine grained, grey. *(continued)*	FR	H	0.90			Hawkesbury Sandstone
SANDSTONE: fine to medium grained, grey and orange brown, with dark grey laminae, bedded at 0-15°.			0.90 1.7 1.4 1.8 1.2 1.2 1.1	(8.65m) Be, 0°, P, R, Fe Sn (10.31m) Be, 0°, P, R, Cb		
END OF BOREHOLE AT 10.74 m						

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FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


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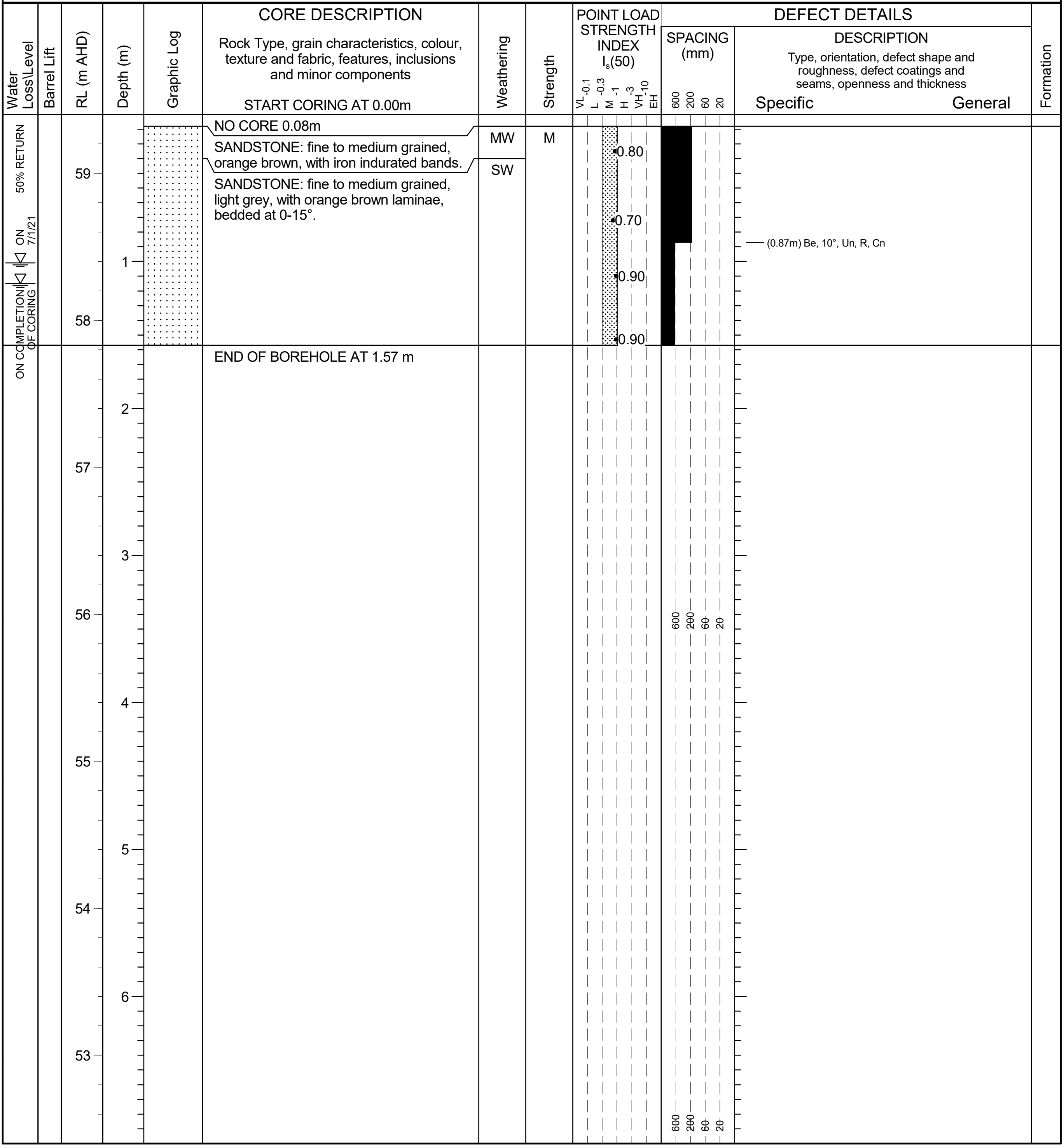
Job No: 33708PN
Borehole No: BH5
Depth: 0.45m - 10.74m
KODAK Gray Scale
KODAK Color Control Patches
JOB NO. 33708PN. BH5. CORING STARTS AT 0.45m
0
1
2
3
4
NO CORE 40mm
5
6
NO CORE 53mm
7
8
9
10
END OF HOLE AT 10.74m

CORED BOREHOLE LOG

Borehole No.
6
1 / 1

Client: ARCH MANLY MANAGEMENT (AUSTRALIA) PTY LTD ATF ARCH MANLY TRUST
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 195-197 SYDNEY ROAD, FAIRLIGHT, NSW

Job No.: 33708PN
Core Size: NMLC
R.L. Surface: ~59.4 m
Date: 17/12/20
Inclination: VERTICAL
Datum: AHD
Plant Type: MELVELLE
Bearing: N/A
Logged/Checked By: B.S./N.E.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 START CORING AT 0.00m	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
50% RETURN					NO CORE 0.08m							
		59			SANDSTONE: fine to medium grained, orange brown, with iron indurated bands.	MW	M	0.80				
ON 7/1/21 ON COMPLETION OF CORING		58	1		SANDSTONE: fine to medium grained, light grey, with orange brown laminae, bedded at 0-15°.	SW		0.70 0.90 0.90		(0.87m) Be, 10°, Un, R, Cn		
		57, 56, 55, 54, 53	2, 3, 4, 5, 6		END OF BOREHOLE AT 1.57 m							

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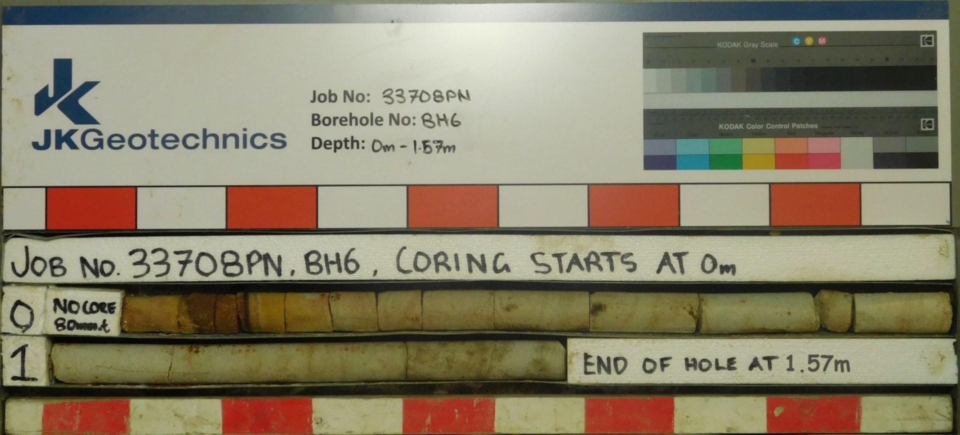
FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



JKGeotechnics

Job No: 33708PN
Borehole No: BH6
Depth: 0m - 1.57m

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BOREHOLE LOG

Borehole No.
7
1 / 2

Client: ARCH MANLY MANAGEMENT (AUSTRALIA) PTY LTD ATF ARCH MANLY TRUST
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 195-197 SYDNEY ROAD, FAIRLIGHT, NSW

Job No.: 33708PN **Method:** DIATUBE **R.L. Surface:** ~54.8 m
Date: 7/1/21 **Datum:** AHD
Plant Type: **Logged/Checked By:** B.S./N.E.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
DRY ON COMPLETION OF AUGERING									-	CONCRETE: 20mm.t				NO OBSERVED REINFORCEMENT
										SANDSTONE: fine to medium grained, orange brown and grey.	MW	M		POSSIBLE BOULDER
										REFER TO CORED BOREHOLE LOG				
						54								
							1							
						53								
							2							
						52								
							3							
						51								
							4							
						50								
							5							
						49								
							6							
						48								

CORED BOREHOLE LOG

Borehole No.
7
2 / 2

Client: ARCH MANLY MANAGEMENT (AUSTRALIA) PTY LTD ATF ARCH MANLY TRUST
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 195-197 SYDNEY ROAD, FAIRLIGHT, NSW

Job No.: 33708PN
Core Size: NMLC
R.L. Surface: ~54.8 m
Date: 7/1/21
Inclination: VERTICAL
Datum: AHD
Plant Type: MELVELLE
Bearing: N/A
Logged/Checked By: B.S./N.E.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 START CORING AT 0.05m	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
ON COMPLETION OF CORING 50% RETURN					SANDSTONE: fine to medium grained, orange brown.	MW	M	0.40		(0.05-0.14m) POSSIBLE BOULDER		Hawkesbury Sandstone
		54	1		NO CORE 0.96m							
					SANDSTONE: fine to medium grained, orange brown and grey, bedded at 0-10°, with dark grey laminae.	SW	M	0.60 0.50		(1.40m) Be, 0°, P, R, Cn		
		53	2		END OF BOREHOLE AT 1.69 m							
		52	3						600 200 60 20			
		51	4									
		50	5									
		49	6									
		48							600 200 60 20			

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 33708PN
Borehole No: BH7
Depth: 0m - 1·69m
KODAK Gray Scale
KODAK Color Control Patches
JOB No. 33708PN, BH7. CORING STARTS AT 0.05m
0
→
← NO CORE 0.96m
1
→
END OF HOLE AT 1.69m

JKGeotechnics

BOREHOLE LOG

Borehole No.
8
1 / 2

Client: ARCH MANLY MANAGEMENT (AUSTRALIA) PTY LTD ATF ARCH MANLY TRUST
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 195-197 SYDNEY ROAD, FAIRLIGHT, NSW

Job No.: 33708PN
Method: HAND AUGER
R.L. Surface: ~50.7 m
Date: 7/1/21
Datum: AHD
Plant Type:
Logged/Checked By: B.S./N.E.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
DRY ON COMPLETION OF AUGERING				■	REFER TO DCP TEST RESULTS			▨		FILL: Silty sand, fine grained, dark brown, with roots and root fibres.	M			GRASS COVER
				■				▨		FILL: Clayey silty sand, fine to medium grained, dark brown, trace of fine to medium grained sandstone gravel, and root fibres.				APPEARS POORLY COMPACTED
						50		⁙	-	Extremely Weathered sandstone: SAND, fine to medium grained, orange brown and grey, trace of silt fines.	XW	D		HAWKESBURY SANDSTONE
							1			REFER TO CORED BOREHOLE LOG				
						49								
							2							
						48								
							3							
						47								
							4							
						46								
							5							
						45								
							6							
						44								

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
8
2 / 2

Client: ARCH MANLY MANAGEMENT (AUSTRALIA) PTY LTD ATF ARCH MANLY TRUST
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 195-197 SYDNEY ROAD, FAIRLIGHT, NSW

Job No.: 33708PN	**Core Size:** NMLC	**R.L. Surface:** ~50.7 m
Date: 7/1/21	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: MELVELLE	**Bearing:** N/A	**Logged/Checked By:** B.S./N.E.S.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS Specific	Formation
		50		START CORING AT 0.72m					
			1	SANDSTONE: fine to medium grained, light grey mottled orange brown, bedded at 0-10°, with carbonaceous lenses.	SW	L - M	0.20		Hawkesbury Sandstone
ON COMPLETION OF CORING		49	2			M	0.50 0.60 0.60		
90% RETURN		48	3	SANDSTONE: fine to medium grained, light grey, bedded at 0-10°.			0.70		
				SILTSTONE: grey, bedded at 0-5°, with fine grained sandstone bands.	HW	VL		(3.30m) Be, 0°, P, R, Clay Ct (3.46m) Be, 0°, P, S, Clay Ct	
		47		Extremely Weathered siltstone, silty CLAY, medium plasticity, grey.	XW	Hd			
			4	NO CORE 0.42m					
				Extremely Weathered siltstone, silty CLAY, medium plasticity, grey.	XW	Hd	0.040		
		46		SANDSTONE: fine to medium grained, grey, bedded at 0-5°, with siltstone lenses.	SW	M			
				NO CORE 0.14m					
			5	SANDSTONE: fine to medium grained, grey, bedded at 0-5°, with siltstone lenses.	FR	M - H	1.3 1.2	(4.90m) Be, 0°, P, R, Clay Vn	Hawkesbury Sandstone
		45		SANDSTONE: fine to medium grained, grey.		VL - L			
			6			M - H	0.80 1.0		
		44		END OF BOREHOLE AT 6.09 m					

POINT LOAD STRENGTH INDEX scale: 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

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JKGeotechnics

Job No: 33708PN
Borehole No: BH8
Depth: 0m - 6.09m

JOB No. 33708PN, BH8, CORING STARTS AT 0.72m

JKGeotechnics

DYNAMIC CONE PENETRATION TEST RESULTS

Client: ARCH MANLY MANAGEMENT (AUSTRALIA) PTY LTD ATF ARCH MANLY TRUST
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 195-197 SYDNEY ROAD, FAIRLIGHT, NSW

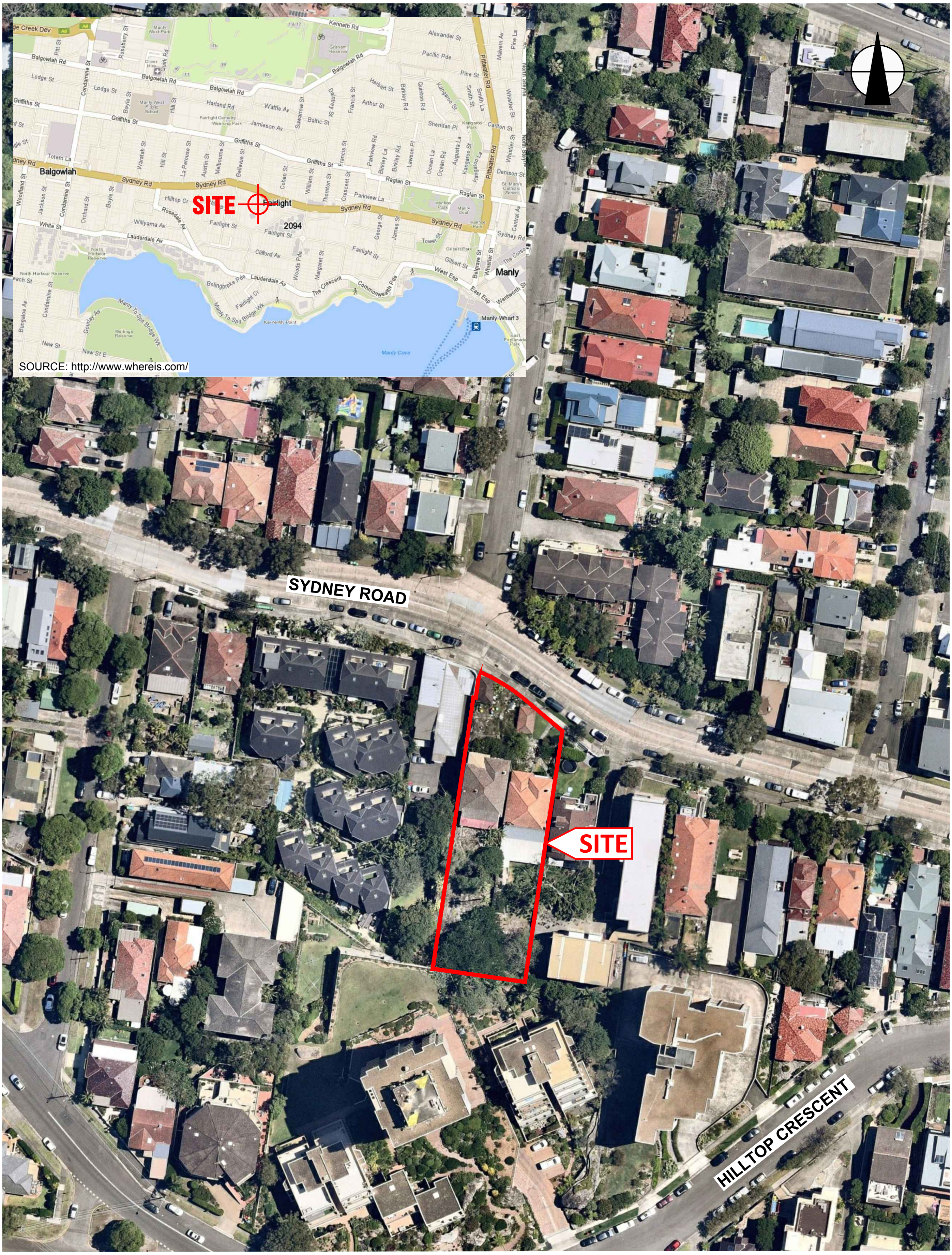
Job No. 33708PN
Date: 16-12-20
Tested By: B.S.

Hammer Weight & Drop: 9kg/510mm
Rod Diameter: 16mm
Point Diameter: 20mm

Test Location	**1**	**5**	**8**				
Surface RL	≈48.9m	≈57.3m	≈50.7m				
Depth (mm)	Number of Blows per 100mm Penetration						
0 - 100	1	SUNK	SUNK				
100 - 200	2	1	↓				
200 - 300	9	3					
300 - 400	8/5mm	10	2				
400 - 500	REFUSAL	9/20mm	4				
500 - 600		REFUSAL	7				
600 - 700			11/95mm				
700 - 800			REFUSAL				
800 - 900							
900 - 1000							
1000 - 1100							
1100 - 1200							
1200 - 1300							
1300 - 1400							
1400 - 1500							
1500 - 1600							
1600 - 1700							
1700 - 1800							
1800 - 1900							
1900 - 2000							
2000 - 2100							
2100 - 2200							
2200 - 2300							
2300 - 2400							
2400 - 2500							
2500 - 2600							
2600 - 2700							
2700 - 2800							
2800 - 2900							
2900 - 3000							

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

Ref: JK Geotechnics DCP 0-3m Rev5 Feb19



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**

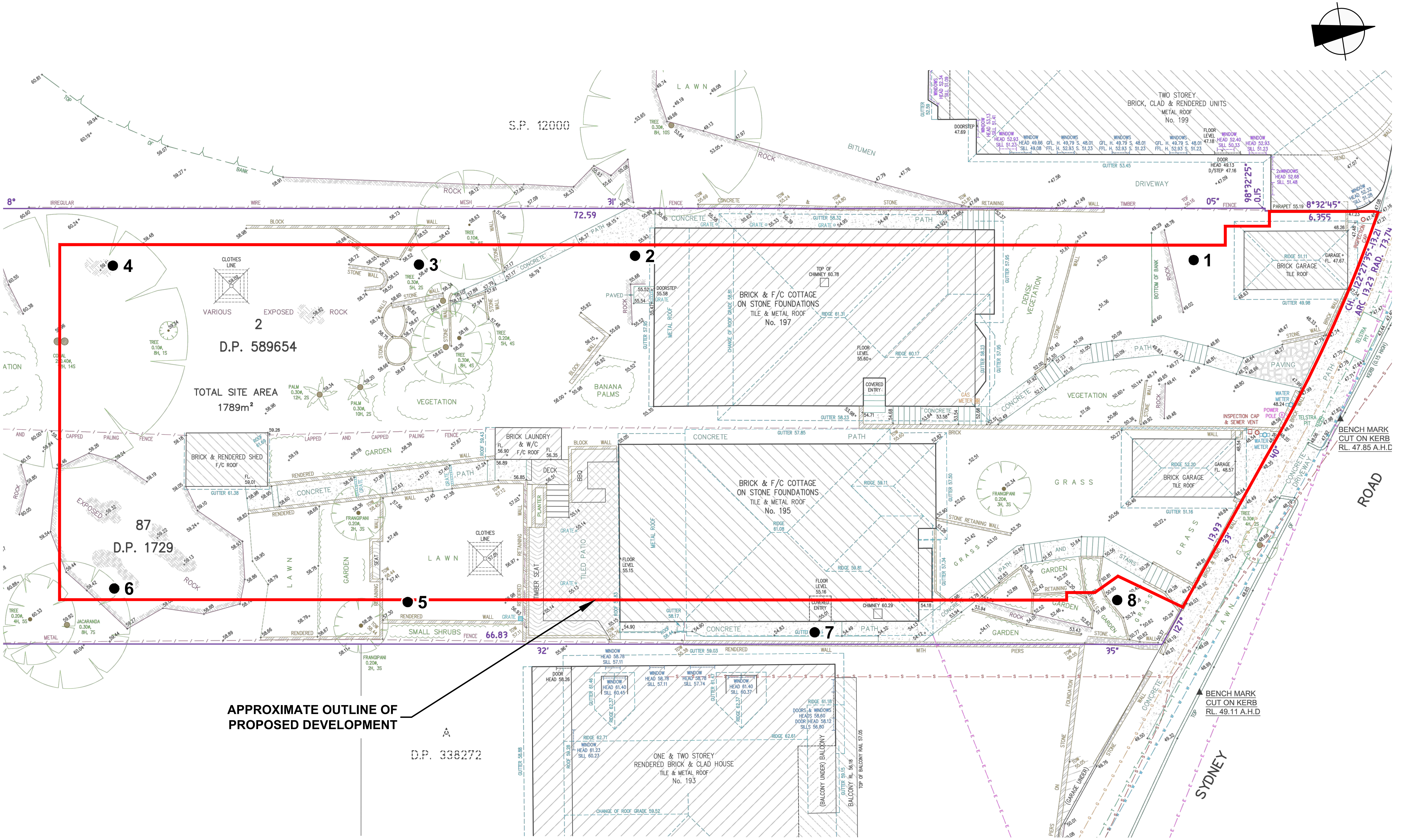
Location: 195-197 SYDNEY ROAD, FAIRLIGHT, NSW

Report No: 33708PN

Figure No: 1

JKGeotechnics

PLOT DATE: 12/01/2021 10:18:34 AM DWG FILE: Y:\33000'S\33708PN FAIRLIGHT\CAD\33708PN.DWG



0 2 4 6 8 10

SCALE 1:200 @A3 METRES

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

Title:

BOREHOLE LOCATION PLAN

Location: 195-197 SYDNEY ROAD, FAIRLIGHT, NSW

Report No: 33708PN

Figure No: 2

JKGeotechnics

PLOT DATE: 12/01/2021 10:19:04 AM DWG FILE: Y:\33000'S\33708PN FAIRLIGHT\CAD\33708PN.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 <u>or</u> where only a limited investigation has been completed <u>or</u> where the geotechnical conditions/constraints are quite complex, it is prudent to have a joint design review which involves an experienced geotechnical engineer/engineering geologist.

SITE INSPECTION

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

Requirements could range from:

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

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

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

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

ROCK

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

OTHER MATERIALS

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

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory **Classification** Criteria

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

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

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

NOTES:

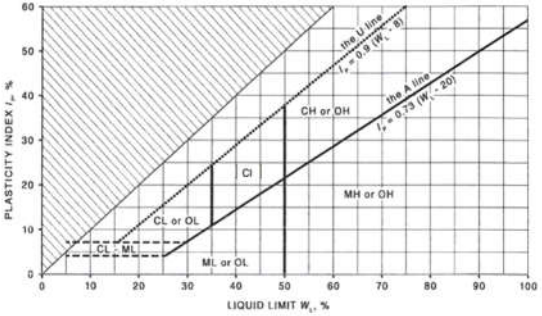
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

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

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Guide to Strength	
			Point Load Strength Index $Is_{(50)}$ (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