

Durbach Block Jagers Architects
Level 2, 9 Roslyn Street
POTTS POINT NSW 2011

Attention: David Jagers
Email: david@durbachblock.com

**GEOTECHNICAL DESKTOP ASSESSMENT
PROPOSED GARAGE AND STUDIO
13A LODGE STREET, BALGOWLAH, NSW**

1 INTRODUCTION

This letter report presents the results of our geotechnical desktop assessment for the proposed garage and studio at 13A Lodge Street, Balgowlah, NSW. The location of the site is shown in Figure 1. The assessment was commissioned by David Jagers and Sarah Kirkham by return of a signed Acceptance of Proposal form dated 22 October 2024. The commission was on the basis of our fee proposal, Ref. P70444PN, dated 22 October 2024.

We have been provided with architectural drawings prepared by Durbach Block Jagers Architects (Project: Garage and Studio, Drawing Nos DA 01 – DA 07, Issue P1, dated October 2024) and a survey plan prepared by Beveridge Williams (Project No. 2402566, Drawing Ref. SUR-001, Version A, dated 16 October 2024).

Based on the supplied information, we understand that following demolition of the existing carport in the north-eastern corner of the site, a new two car garage, driveway and studio are proposed, which will require excavation to a maximum depth of about 1.1m to achieve design levels. The proposed finished floor level of the driveway and garage appear to be at approximately the same level as the adjacent street, i.e. reduced level (RL) 17.8m.

We have previously completed a limited scope geotechnical investigation at the site in August 2010, and the results were presented in our report, Ref: 24234Zlet dated 25 August 2010 [JKG2010], to inform the design and construction of alterations and additions to the existing house. The factual information from our previous investigation is attached as Appendix A. The locations of our previous investigations are shown on the attached Investigation Location Plan (Figure 2) which is based on the provided survey drawing, with also shows the outline of the proposed garage and driveway overlain.



The purpose of the desktop assessment was to review the results of our previous investigation and to use this as a basis for providing comments and recommendation on geotechnical aspects of the proposed garage and studio works. A current site visit was not allowed for.

2 SUMMARY OF PREVIOUS GEOTECHNICAL INVESTIGATION

2.1 Site Description

Review of available Nearmap aerial imagery, it appears that alterations and additions to the house on site were completed in 2011, along with construction of the existing carport in the north-eastern corner of the site. However, the neighbouring properties to the east and south appear to be unchanged since 2010. Relevant information from JKG2010 is reproduced below.

The site is located on a gently sloping north facing hillside which generally grades at about 3° to 4° on the south-east corner of Condamine Street and Lodge Street.

The neighbouring property to the south was occupied by a two storey brick house which was set back approximately 4m to 5m beyond the southern site boundary. A concrete driveway was located just beyond this boundary and was supported between 0.5m and 1m above the subject site by a brick retaining wall.

The neighbouring property to the east was occupied by numerous two storey brick town houses which were set back a minimum of approximately 3m beyond the south-western corner of the subject site.

The neighbouring buildings and structures appeared to be in good condition when viewed from within the subject site. Other than the southern site boundary, ground levels across the remaining site boundaries were essentially similar.

2.2 Subsurface Conditions

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

Our previous investigation disclosed a generalised subsurface profile comprising shallow sandy fill over residual sand of loose and medium dense relative density. Sandstone bedrock was inferred from below about 1.5m to 1.7m depth, however, as the equipment used for our previous investigation does not provide sample return, this has not been confirmed. Groundwater was not encountered within the depth of our previous investigation (1.4m), though no long term groundwater level monitoring has been carried out.

3 COMMENTS AND RECOMMENDATIONS

3.1 Excavation Conditions

Prior to any excavation commencing we recommend that reference be made to the Safe Work Australia 'Excavation Work Code of Practice' dated January 2020.

Excavation for the proposed garage is expected to be to a maximum depth of about 1.1m below existing levels, and based on the results of our previous investigation, will encounter soils only. We expect excavation will be readily achieved using the bucket of a small hydraulic excavator. To reduce the potential for transmission of vibrations which could potentially damage nearby structures or services, the excavator must be limited to no larger than about 5 tonnes size.

Reference should be made to Section 4 below for guidance on the offsite disposal of soil.

For the maximum excavation depth of 1.1m, temporary batter slopes through the sandy soils of no steeper than 1 Vertical (V) in 1 Horizontal (H) are feasible, provided all surcharge loads (e.g. stockpiles, construction materials, existing footings, etc.) are kept at least 2m clear of the crest of the batters. We note that at such a slope, some instability of the batters could occur, e.g. localised slumping. In most areas, we expect this would not be expected to be of concern, however, on the eastern side of the excavation, such slumping could extend into the neighbouring property, and as such, we recommend seeking approval for minor disturbance to the landscaping in the adjoining property. If such permission is not granted, then further geotechnical advice on providing some temporary shoring must be sought before excavation commences.

3.2 Retaining Wall Design Parameters

The major consideration in the selection of earth pressures for the design of retaining walls is the need to limit deformations occurring outside the excavation. The characteristic earth pressure coefficients and soil parameters provided below may be adopted for the design of the retention systems.

Free-standing cantilevered retaining walls supporting landscaped areas where some movement of the wall can be tolerated should be designed using a triangular lateral earth pressure distribution and an 'active' earth pressure coefficient (K_a) of 0.35 for the soil, assuming a horizontal retained/backfill surface. For the design of walls where movement is to be limited, we recommend the adoption of a triangular lateral earth pressure distribution and an 'at rest' earth pressure coefficient (K_a) of 0.55 for the soil, also assuming a horizontal retained/backfill surface.

A bulk unit weight of 21kN/m³ should be adopted for the soils above any water table.

Any surcharge loads affecting the walls (e.g. nearby footings, inclined backfill, compaction stresses during backfilling, etc.) should be allowed in the design using the relevant earth pressure coefficient from above.

The sandy soil profile is expected to be of relatively high permeability and no groundwater was encountered to at least excavation level. As such, we consider that the retaining walls will not require formal subsoil

drainage. However, if any of the walls are founded on bedrock, subsurface drainage incorporating a non-woven geotextile filter fabric to protect against the migration of fines must be constructed behind such walls, with an invert level at about 0.1m below the garage floor slab level. The subsurface drainage must discharge to the stormwater system. Further, to limit surface water inflows, we recommend good surface drainage be constructed.

Lateral toe restraint may be achieved by passive resistance of the soil profile in-front of the wall using a triangular lateral earth pressure distribution and a 'passive' earth pressure coefficient, K_p , of 3.0 for the soil profile. Due to the large movements required to mobilise full passive pressures, we recommend that a Factor of Safety of at least 2.0 be adopted. We recommend that the upper 0.3m below garage subgrade level be ignored for such analysis to allow for potential disturbance and over-excavation.

3.3 Footing Design

For the proposed garage, conventional high level strip and pad footings founded in the natural sands may be considered. The allowable bearing pressure in sands is dependent on the sand density, the footing geometry and the depth to groundwater. For preliminary design, for a 0.5m wide strip footing or a 0.5m square pad footing founded at a depth of 0.5m in loose sand, an allowable bearing pressure of 100kPa could be adopted. The elastic settlement for such footings would be expected to be about 5mm.

However, if bedrock is expected within the excavation for any footings, for uniformity of support, we recommend the entire garage be founded on the bedrock, for which an allowable bearing pressure of 600kPa could be adopted.

Following excavation, the base of all footings in sand should be well watered, and rigorously compacted using an upright rammer compactor or vibrating sled compactor. Following such treatment, trafficking of the base of the footing excavation must be avoided as this will tend to loosen the foundation material.

All footing excavations must be inspected by a geotechnical engineer to assess an appropriate foundation material has been achieved. For footings in sands, Dynamic Cone Penetration (DCP) testing will be required to confirm the relative density of the foundation material.

3.4 Garage Floor Slab

Based on the results of our previous investigation, the subgrade for the garage floor slab will comprise the natural sands. Slab-on-grade construction is therefore feasible however, adequate subgrade preparation must be carried out. The subgrade preparation should include placing a 100mm loose thickness of well graded gravel., e.g. DGB20, with this being rigorously compacted with a, say 2 tonne roller (operating in static mode) or heavy vibrating plate compactor. The purpose of this is to confine and improve the near surface density of the sands following likely disturbance during excavation.



The on-grade floor slab should be separated from all walls, columns, footings, etc, to permit relative movement. Joints in the concrete on-grade floor slab should incorporate keyed or dowelled joints to resist shear forces.

Alternatively, the on-grade slab could be designed as suspended from piles to bedrock, in which case, no particular subgrade preparation would be required.

3.5 Groundwater

Based on the investigation results, groundwater (if present) is below the proposed excavation level and seepage into the excavation is not expected.

In view of the above, we consider the proposed development should not adversely affect the existing transient groundwater seepage flows to the extent that there will be any noticeable impact on the surrounding structures.

4 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, away from, and below the previous boreholes and DCP tests 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.

Should you require any further information regarding the above, please do not hesitate to contact the undersigned.

Yours faithfully
For and on behalf of
JK GEOTECHNICS

A handwritten signature in black ink, appearing to read 'Joanne Lagan'.

Joanne Lagan
Senior Geotechnical Engineer

Reviewed by:

A handwritten signature in black ink, appearing to read 'Nicholas Smith'.

Nicholas Smith
Principal Associate | Geotechnical Engineer

Encl: Figure 1: Site Location Plan
 Figure 2: Investigation Location Plan
 Appendix A: Borehole Log (BH1) & Dynamic Cone Penetration Test Results Sheet



© Document copyright of JK Geotechnics

This report (which includes all attachments and annexures) has been prepared by JK Geotechnics (JKG) for its Client, and is intended for the use only by that Client.

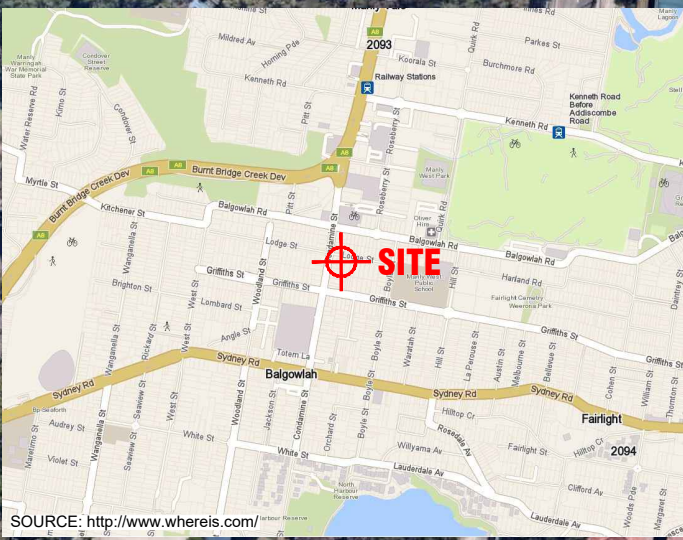
This Report has been prepared pursuant to a contract between JKG and its Client and is therefore subject to:

- a) JKG's proposal in respect of the work covered by the Report;
- b) The limitations defined in the Client's brief to JKG;
- c) The terms of contract between JK and the Client, including terms limiting the liability of JKG.

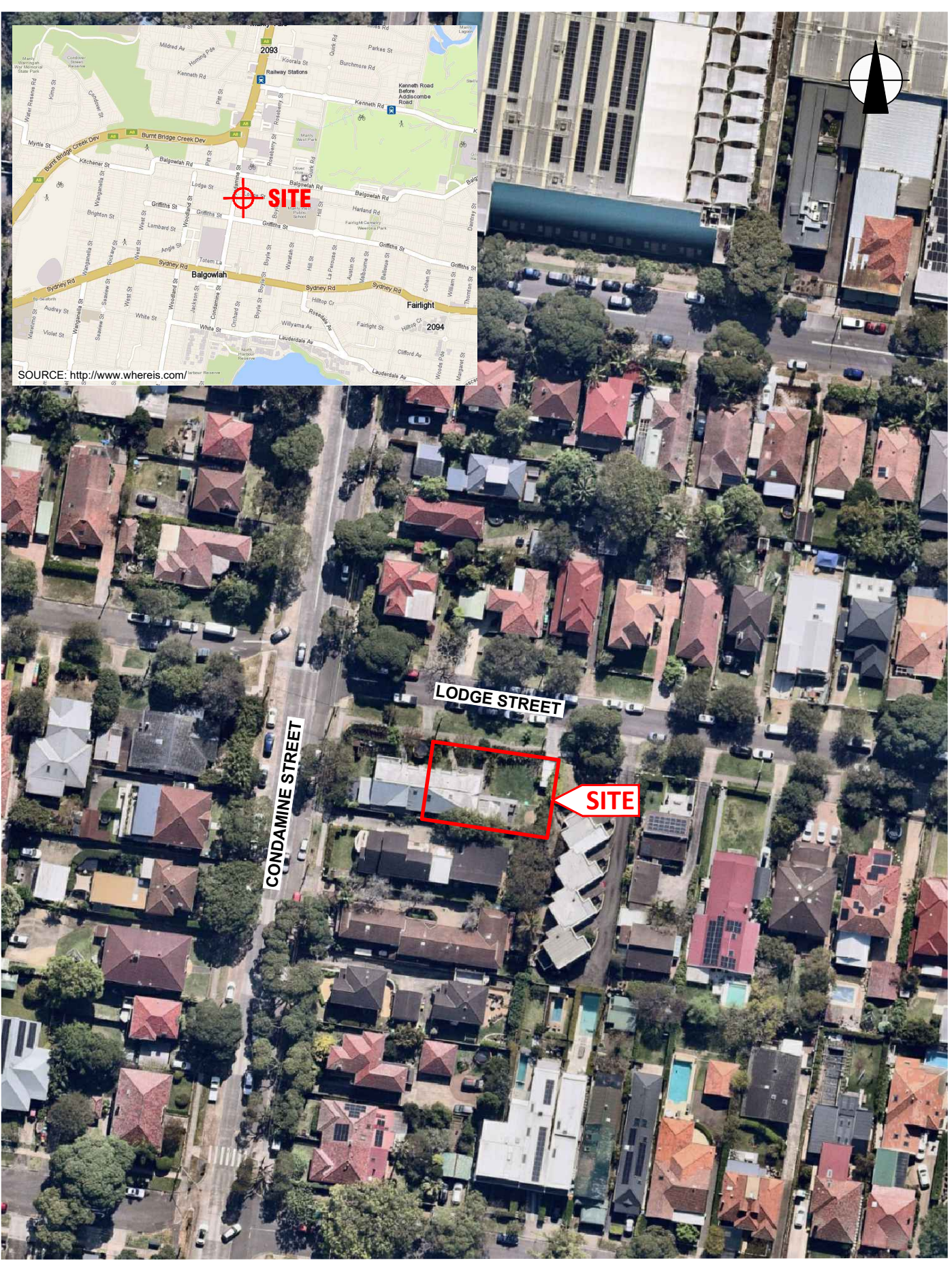
If the Client, or any person, provides a copy of this Report to any third party, such third party must not rely on this Report, except with the express written consent of JKG which, if given, will be deemed to be upon the same terms, conditions, restrictions and limitations as apply by virtue of (a), (b), and (c) above.

Any third party who seeks to rely on this Report without the express written consent of JKG does so entirely at their own risk and to the fullest extent permitted by law, JKG accepts no liability whatsoever, in respect of any loss or damage suffered by any such third party.

At the Company's discretion, JKG may send a paper copy of this report for confirmation. In the event of any discrepancy between paper and electronic versions, the paper version is to take precedence. The USER shall ascertain the accuracy and the suitability of this information for the purpose intended; reasonable effort is made at the time of assembling this information to ensure its integrity. The recipient is not authorised to modify the content of the information supplied without the prior written consent of JKG.



SOURCE: <http://www.wheris.com/>



AERIAL IMAGE SOURCE: [MAPS.AU.NEARMAP.COM](https://maps.au.nearmap.com)

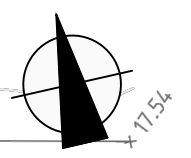
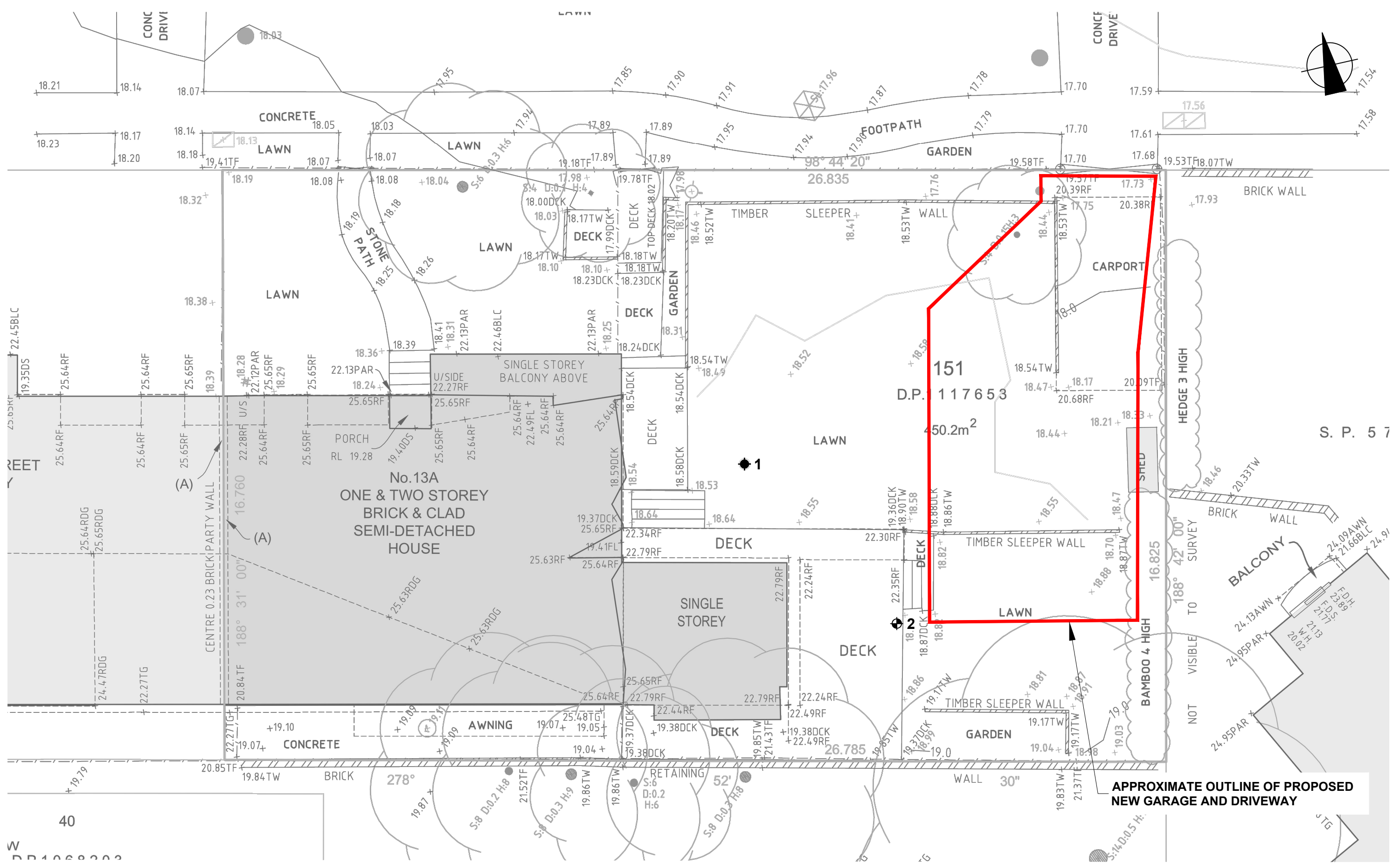
Title:		SITE LOCATION PLAN	
Location:		13A LODGE STREET, BALGOWLAH, NSW	
Report No:	37131PN	Figure No:	1
JKGeotechnics			



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

PLOT DATE: 23/10/2024 12:52:30 PM DWG FILE: JKF GEOTECHNICAL_JOBS\97000\97131\37131PN_BALGOWLAH\CAD\37131PN.DWG

PLOT DATE: 5/11/2024 5:10:26 PM DWG FILE: S:\6 GEOTECHNICAL\GEO\TECHNICAL JOBS\37000\37131PN BALGOWLAH\CAD\37131PN.DWG



- LEGEND**
- BOREHOLE AND DCP TEST
 - DCP TEST

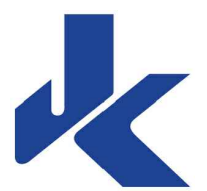
NOTES:
 1. TEST LOCATIONS 1 AND 2 ARE FROM OUR PREVIOUS INVESTIGATION. (Ref 24234Zlet DATED 25 AUGUST 2010)

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

SCALE 1:100 @A3 METRES

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

Title: BOREHOLE LOCATION PLAN	
Location: 13A LODGE STREET, BALGOWLAH, NSW	
Report No: 37131PN	Figure No: 2
JK Geotechnics	



APPENDIX A



Borehole No.

1

1/1

BOREHOLE LOG

Client: DAVID JAGGERS
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 13A LODGE STREET, BALGOWLAH, NSW

Job No. 24234Z **Method:** HAND AUGER **R.L. Surface:** ≈ 18.5m
Date: 18-8-10 **Datum:** AHD

Logged/Checked by: A.P.C. / *[Signature]*

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	USO	DB	DS									
DRY ON COMPLETION					REFER TO DCP TEST RESULTS	0			FILL: Silty sand, fine to medium grained, dark brown, with root fibres and a trace of fine grained sandstone gravel.	M			GRASS COVER
						0.5		SM	SILTY SAND: fine to medium grained, grey.	M	L		RESIDUAL
						0.5			SILTY SAND: fine to medium grained, orange brown and yellow brown, with fine to coarse grained sub rounded sandstone gravel, and a trace of root fibres.				
						1			SILTY SAND: fine to medium grained, yellow brown, with a trace of clay fines.				
						1		SC	CLAYEY SAND: fine to medium grained, yellow brown.				
						1.5			CLAYEY SAND: fine to medium grained, grey brown, with medium to coarse grained angular ironstone gravel.		MD		HAND AUGER REFUSAL ON IRONSTONE GRAVEL.
						1.5			END OF BOREHOLE AT 1.4m				
						2							
						2.5							
						3							
						3.5							

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS



DYNAMIC CONE PENETRATION TEST RESULTS

Client:	DAVID JAGGERS						
Project:	PROPOSED ALTERATIONS AND ADDITIONS						
Location:	13A LODGE STREET, BALGOWLAH, NSW						
Job No.	24234Z						Hammer Weight & Drop: 9kg/510mm
Date:	18-8-10						Rod Diameter: 16mm
Tested By:	A.P.C						Point Diameter: 20mm
Number of Blows per 100mm Penetration							
Test Location	RL ~18.5m	RL ~18.7m					
Depth (mm)	1	2					
0 - 100	2	1					
100 - 200	3	2					
200 - 300	4	1					
300 - 400	5	13					
400 - 500	21	5					
500 - 600	10	2					
600 - 700	5	↓					
700 - 800	4	1					
800 - 900	2	2					
900 - 1000	3	3					
1000 - 1100	2	4					
1100 - 1200	2	3					
1200 - 1300	14	5					
1300 - 1400	16	6					
1400 - 1500	5	10/80mm					
1500 - 1600	8	REFUSAL					
1600 - 1700	22						
1700 - 1800	REFUSAL						
1800 - 1900							
1900 - 2000							
2000 - 2100							
2100 - 2200							
2200 - 2300							
2300 - 2400							
2400 - 2500							
2500 - 2600							
2600 - 2700							
2700 - 2800							
2800 - 2900							
2900 - 3000							
Remarks:	1. The procedure used for this test is similar to that described in AS1289.6.3.2-1997, Method 6.3.2. 2. Usually 8 blows per 20mm is taken as refusal 3. Survey datum is AHD.						

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_b), horizontal stress index (K_0), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_0), 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_0).

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would

be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

- i) a site visit to confirm that conditions exposed are no worse than those interpreted, to
- ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types and appropriate footing or pile founding depths, or
- iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL



FILL



TOPSOIL



CLAY (CL, CI, CH)



SILT (ML, MH)



SAND (SP, SW)



GRAVEL (GP, GW)



SANDY CLAY (CL, CI, CH)



SILTY CLAY (CL, CI, CH)



CLAYEY SAND (SC)



SILTY SAND (SM)



GRAVELLY CLAY (CL, CI, CH)



CLAYEY GRAVEL (GC)



SANDY SILT (ML, MH)



PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK



CONGLOMERATE



SANDSTONE



SHALE/MUDSTONE



SILTSTONE



CLAYSTONE



COAL



LAMINITE



LIMESTONE



PHYLLITE, SCHIST



TUFF



GRANITE, GABBRO



DOLERITE, DIORITE



BASALT, ANDESITE



QUARTZITE

OTHER MATERIALS



BRICKS OR PAVERS



CONCRETE



ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 68% 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	

Laboratory Classification Criteria

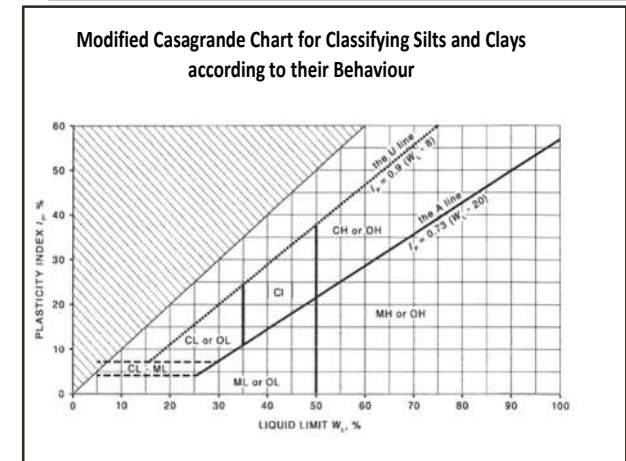
A well graded coarse grained soil is one for which the coefficient of uniformity $C_u > 4$ and the coefficient of curvature $1 < C_c < 3$. Otherwise, the soil is poorly graded. These coefficients are given by:

$$C_u = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_c = \frac{(D_{30})^2}{D_{10} D_{60}}$$




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

- NOTES:**
- 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.
 - 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.
 - Clay soils with liquid limits $> 35\%$ and $\leq 50\%$ may be classified as being of medium plasticity.
 - The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	
fine 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	–	–	–	–



LOG SYMBOLS

Log Column	Symbol	Definition		
Groundwater Record		Standing water level. Time delay following completion of drilling/excavation may be shown.		
		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	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.	
		7		
		3R		
VNS = 25 PID = 100	Vane shear reading in kPa of undrained shear strength. Photoionisation detector reading in ppm (soil sample headspace test).			
Moisture Condition (Fine Grained Soils) (Coarse 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.		
	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 (%)		
	VL	VERY LOOSE	≤ 15	SPT 'N' Value Range (Blows/300mm)
	L	LOOSE	> 15 and ≤ 35	0 – 4
	MD	MEDIUM DENSE	> 35 and ≤ 65	4 – 10
	D	DENSE	> 65 and ≤ 85	10 – 30
	VD	VERY DENSE	> 85	30 – 50
	()	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.	> 50	
Hand Penetrometer Readings	300	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.		
	250			

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	HW	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	
Distinctly Weathered (Note 1)		
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