REPORT

TO
BISHI CONSTRUCTIONS PTY LTD

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED RESIDENTIAL DEVELOPMENT

AT
15-17 DENT STREET, JAMISONTOWN, NSW

2 November 2016
Ref: 29853Zrpt

AS/NZS ISO 9001 certified
AS/NZS 4801 certified

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

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For and on behalf of
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1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed residential development at 15-17 Dent Street, Jamisontown, NSW. The investigation was commissioned by Mr Bishi Tancev of Bishi Constructions, by signed 'Acceptance of Proposal' form dated 10 October 2016. The commission was on the basis of our fee proposal (Ref P43495Z Jamisontown) dated 30 September 2016.

We understand from the provided architectural drawings (Project No 15015, Drawing Nos DA000^{C}, 101^{B}, 102^{B}, 201^{B} to 208^{B}, 301^{B} to 304^{B}, 401^{B} and 402^{B}) prepared by Alan Johnson Architect, that following demolition of existing improvements on site, a new six-storey building with two basement levels will be constructed. The basements will be set back approximately 1.3m, 5.6m, 2.5m and 3.2m from the northern, eastern, southern and western site boundaries, respectively, and will require a maximum excavation depth of approximately 7.5m to achieve the lower basement finished floor at Reduced Level (RL) 21.55m to RL22.56m. We have assumed that typical structural loads for this type of development apply.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions as a basis for comments and recommendations on excavation conditions, shoring, retaining walls, footings and on-grade floor slabs.

Our environmental consulting division, Environmental Investigation Services (EIS), was commissioned to concurrently carry out a preliminary waste classification and salinity assessment. The geotechnical report must therefore be read in conjunction with the environmental report (Ref E29853KJ).

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 20 October 2016 and comprised the auger drilling of two boreholes (BH1 and BH2) to depths of 11.0 and 6.0m, respectively, using our track-mounted JK300 drill rig. In addition, one Dynamic Cone Penetration (DCP) test (DCP3) was completed to a refusal depth of 2.5m, where access to our rig was not available.

The investigation locations, as indicated on attached Figure 2, were set out using taped measurements from existing surface features and were electromagnetically scanned for buried services prior to drilling commencing. The surface RLs shown on the borehole logs and DCP test were estimated by interpolation between spot heights indicated on the provided survey plan (Drawing No DWG-01, Rev. B, dated 10 May 2016) prepared by Spatial Technologies. Figure 2 is based on the survey plan and the survey datum is the Australian Height Datum (AHD).

The nature and composition of the subsoils were assessed by logging the materials recovered during drilling. The strength of the soil profile was assessed from the Standard Penetration Test (SPT) 'N' value, Solid Cone Penetration Test (SCPT) 'Nc' value, and hand penetrometer readings on clayey samples recovered in the SPT split tube sampler. Groundwater observations were made during, on completion and shortly following completion of drilling individual boreholes. A slotted PVC standpipe was installed into BH1 for subsequent groundwater monitoring. Longer term groundwater monitoring was not carried out as part of this investigation. For further details on the investigation procedure adopted, reference should be made to the attached Report Explanation Notes.

Our geotechnical engineer was present full-time on site during the fieldwork and set out the borehole locations, directed the electromagnetic scan, nominated the sampling and testing and logged the subsurface profile. The borehole logs and DCP test results are presented with this report, together with a glossary of the logging terms and symbols used.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located in a region of relatively level terrain, sloping no more than 1°. The site topography conforms to that of the region.

The site comprised two properties (No. 15 and No. 17) which, at the time of the investigation, were occupied by single-storey brick dwellings with tile roofs. Both dwellings appeared to be in good condition, based on a cursory external inspection. The remainder of the site was covered by pavements and landscaped gardens. Trees up to approximately 5m high were located at the eastern and western ends of the site. The pavements were in good condition, except for the concrete driveway along the northern site boundary which exhibited block cracking up to 10mm wide.

To the north of the site was a three-storey brick apartment building with a basement level at approximately 1.5m below street level and set back an estimated 1.5m from the site boundary. A similar building was present to the south of the site, but set back 6.6m from the site boundary and with a basement level approximately 1.8m below street level. A driveway leading down into the basement of this latter building was located along the common boundary, where the subject site was retained up to 1.8m by a rendered wall. Beyond the northern end of the eastern site boundary was a three storey brick apartment building, and beyond the southern end of the boundary was a one-storey brick building. Both buildings were set back an estimated 5m from the site boundary. Surface levels were similar across the northern and western boundaries.

3.2 Subsurface Conditions

The 1:100,000 geological map of Penrith (Geological Survey of NSW, Geological Series Sheet 9030) indicates the site to be underlain by fluvial deposits comprising 'gravel, sand, silt and clay'. The investigation has revealed a generalised subsurface profile comprising surficial silty clay fill over fluvial silty clay and then silty sand with a layer of silty clayey gravel. Groundwater was encountered at moderate depth and bedrock was not encountered. For detailed subsurface conditions at specific locations, reference should be made to the attached borehole logs. A summary of the encountered subsurface conditions is presented below:

- Silty clay topsoil/fill was encountered from the surface of BH1 and BH2 and extended to depths of 0.4m and 0.5m, respectively. Ironstone gravel inclusions were encountered in BH2.

- Fluvial silty clay of low to medium plasticity and stiff to hard strength was encountered beneath the fill/topsoil in both boreholes and extended to depths of 4.0m and 4.6m, respectively.
- The results of DCP3 have been interpreted to indicate silty clay of stiff to very stiff strength to a depth of approximately 2.0m, after which the strength was hard to a depth of at least 2.5m, at which point the test was terminated due to having achieved practical refusal.
- Silty sand was encountered beneath the silty clay in both boreholes and extended to depths of 4.5m and 5.0m, respectively. The sand was loose to medium dense.
- Silty clayey gravel was encountered beneath the silty sand and extended to depths of 8.0m in BH1 and to a refusal depth of 6m in BH2. The gravelly layer was assessed to be dense.
- The gravel layer was only penetrated in BH1 and was underlain by a silty sand which graded to a clayey sand, which extended to the borehole termination depth of 9m. The sands were assessed to be medium dense or denser.
- Groundwater seepage was encountered during drilling BH1 at a depth of approximately 8.0m. On completion of drilling, BH1 collapsed to a depth of 7.8m. Half an hour after completion of drilling BH1 and installation of the groundwater monitoring well, standing groundwater at 8.75m was measured. BH2 was 'dry' during and on completion of drilling.

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Issue

The principal geotechnical issue associated with the proposed development at the subject site relates to the impact of the gravel layer on retention and founding. This issue is discussed in detail in the sections which follow.

However, we recommend that if feasible, the building levels be revised so that the penetration of the bulk excavation into the gravel layer is reduced as much as possible.

4.2 Excavation Conditions

The proposed basement is expected to require excavation to a maximum depth of about 7.5m below existing levels. In addition, localised excavations for the lift overrun pits could extend a further 1.2m locally. The proposed bulk excavation is expected to encounter the fluvial clays and sands and to extend into the gravel layer. The fluvial clays and sands should be readily excavatable using conventional earthworks equipment (eg. hydraulic excavator). However, excavation of the gravel layers, particularly within localised or restricted excavations may require the use of hydraulic impact rock hammers.

Based on the investigation results, the bulk excavation should not encounter the groundwater level. However, localised deeper excavations for the lift overrun pit may extend below the measured groundwater level of 8.75m. Provision for minor dewatering should therefore be made. In this regard, we recommend that regular measurements of the groundwater level in the standpipe which was installed on site be made to confirm any variations due to rainfall or tides.

4.3 Excavation Support

As the proposed excavation will extend close to the site boundaries, battering of the side slopes is not feasible and a full depth engineered retention system will be required to support the vertical cuts. We are aware that sheet piling has been successfully used as a retention system on a number of excavations with similar subsurface conditions some 2kms to the north. However, due to the subject site being located in a built-up area, the use of sheet piling is not recommended as the installation may result in vibration damage to nearby buildings and structures.

Given the sandy and gravelly profile which will be encountered, we recommend that the retention system comprise a contiguous pile wall using cfa piles.

The piles will need to be installed to sufficient depth below bulk excavation level to satisfy stability considerations and will extend into the gravel layer. Difficult installation conditions must therefore be anticipated. The retention system will need to be temporarily anchored as excavation proceeds so as to reduce lateral deflections. Where the toe depth of the pile wall needs to be raised to reduce penetration into the gravels, a second row of ground anchors may be installed. We note, however, that anchor installation is also likely to be difficult as it will extend into the gravel layer below the groundwater level.

4.4 Retaining Walls

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. In this respect, the location of the neighbouring basements and of buried services around and beyond the site must be accurately determined as part of the shoring design. The following characteristic earth pressure coefficients and subsoil parameters may be adopted for a static design of temporary or permanent retention systems:

- For anchored or propped walls where minor movements can be tolerated (which we anticipated will be case provided there are no buried movement sensitive services present within the road reserve), we recommend the use of a trapezoidal lateral earth pressure distribution of 6H kPa, where 'H' is the retained height in metres. Given, however, the sandy/gravelly nature of the lower profile, the lateral earth pressures should be assumed to be uniform over the lower three quarters of the retained height (ie. lower 0.75H).
- For anchored or propped walls which are relatively sensitive to movement (eg. if there are movement sensitive buildings or buried services present) a lateral earth pressure distribution of 8H kPa should be adopted for the soil profile as above.
- Any surcharge affecting the walls (eg. traffic loads, construction loads, etc) should be allowed in the design using an 'at rest' earth pressure coefficient, K_o, of 0.6.
- The retaining walls should be designed as drained and measures taken to provide permanent and effective drainage of the ground behind the walls.
- Lateral toe restraint may be achieved by embedding the piles to sufficient depth below bulk excavation level. A triangular lateral earth pressure distribution should be adopted for embedment depth design, with a 'passive' earth pressure coefficient, K_p, of 3, assuming horizontal ground in front of the wall. We note that significant deflection is required in order to mobilise the full passive resistance of the soil and, therefore, a factor of safety of at least 2 should be adopted. The upper 0.3m below bulk excavation level should be ignored in the analysis to take excavation tolerances into account. Any localised excavations in front of the wall (eg. for lift overrun pits, buried services, footings, etc) must be taken into account in the design.
- Anchors bonded into the gravels and sands can be designed for an effective friction angle of 35° on the grout/gravel interface using a bulk unit weight of $19kN/m^3$ for the soil profile above the groundwater level, subject to the following conditions:
  - Anchor length of at least 3m behind the active zone of the excavation (taken as a 45° zone above the base of the excavation).

– Overall stability including anchor group interaction is satisfied.

– All anchors are proof-loaded to at least 1.3 times the design working load before being locked off at working load. We recommend that such proof-rolling be inspected by an experienced geotechnical engineer or that full field test records be made available to a geotechnical engineer for review.

We note that as anchors may extend below the groundwater level, this must be addressed by the anchor installation contractor in the method statement. Indicatively temporary casing of the anchor holes will probably be required. The anchors will extend below surrounding properties and the permission of the owners must be obtained before installation. The presence of the neighbouring basements to the north and south must be given due consideration.

Alternatively, the shoring can be designed using computer based methods, and the following parameters may be used:

Profile	Bulk Unit Weight (kN/m^3) *	Effective Cohesion c′ (kPa)	Effective Angle of Friction ϕ' (deg)	Poisson's Ratio υ
Predominantly Clay	18	2	28	0.25
Gravel	20	–	35	0.3

* Above the groundwater level.

4.5 Footings

Based on the results of the investigation, the following footing options are considered feasible for the proposed building:

4.5.1 High Level Footings

A high level footing option consisting of pad footings founded in the gravels which have been inferred to be dense, may be used to support the proposed building. The footings may be designed for an allowable bearing pressure of 300kPa. Associated settlements for the anticipated relatively large footings will be up to 50mm, but are expected to occur rapidly (ie. as the loads are applied, during the construction period).

4.5.2 Piles

If high level footings are not feasible or the predicted settlements are too high, pile footings must be considered. Further geotechnical investigations will be required to determine the depth, strength and uniformity of the rock profile below the site. Specialised drilling techniques will be required to ensure that the gravel layer can be penetrated and the borehole extends into the underlying bedrock which is anticipated at depths roughly of the order of 15m. Generally, we have found that the bedrock which underlies the alluvial plain in this area is competent and of relatively high strength.

For estimation purposes, therefore, assume that piles (cfa) installed from bulk excavation level into the underlying bedrock may be designed for an allowable end bearing pressure of 3.5MPa. In addition, an allowable shaft adhesion may be applied based on an effective friction angle of 35° in the gravels and sands and 350kPa in rock sockets (in compression). However, difficult installation techniques must be anticipated as the gravels are being penetrated. Where the perimeter shoring piles are founded in the sands below the gravels, an allowable end bearing pressure of 600kPa may be adopted. The above parameters are based on serviceability criteria of maximum settlement of 1% of pile diameters.

Piles (cfa) may also be designed using limit state design principles. Ultimate bearing pressures of 1.5MPa and 30MPa may be adopted for the sands and bedrock, respectively. An ultimate shaft adhesion value for rock sockets of 600kPa may be assumed. Settlement limitations to the structure will still need to be satisfied and can be estimated using an elastic modulus value of 800MPa for the bedrock and 80kPa for the gravels and sands.

It should be noted that the ultimate pressures must be used in conjunction with an appropriate geotechnical strength factor as defined in AS2159. The geotechnical strength reduction factor will need to be estimated for the site specific conditions, including the structural design and the pile installation and testing which has been nominated. However, provided there is good workmanship and quality control during construction, we expect that the geotechnical strength reduction factor would be approximately 0.5.

We note that cfa piles would need to be certified by the piling contractor.

4.6 Basement On-Grade Floor Slab

The proposed basement floor slab will directly overlie the gravel profile and slab-on-grade construction is feasible, provided adequate subgrade preparation is carried out.

We recommend that the subgrade preparation include proof-rolling the gravels at design level. The purpose of proof-rolling is to densify the upper profile and to help identify any soft or unstable areas which may be present. If soft or unstable areas are detected, they should be excavated down to a sound base and replaced with well compacted sandy/gravelly fill. Alternatively, further advice on subgrade improvement may be obtained from the geotechnical engineer during proof-rolling inspections.

The proposed 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 dowelled or keyed joints so as to avoid stepping.

4.7 Earthquake Design

Based on the investigation results, the site classifies as Class C_c – Shallow soil site, in accordance with AS1170.4. The hazard factor (Z) for Sydney is 0.08.

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 will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. If the natural soil has been stockpiled, classification of this soil as Excavated Natural Material (ENM) can also be undertaken, if requested. However, the criteria for ENM are more stringent and the cost associated with attempting to meet these criteria may be significant. Analysis takes seven to 10 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) should be expected. We strongly recommend that this issue 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.

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
1
1/2

Client: BISHI CONSTRUCTIONS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 15-17 DENT STREET, JAMISONTOWN, NSW

Job No. 29853Z
Date: 20-10-16
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 28.6m
Datum: AHD
Logged/Checked by: A.C.K./A.Z.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		TOPSOIL: Silty clay, low plasticity, dark brown.	MC<PL	(St)		GRASS COVER TO 50mm DEPTH
	N = 20 5,7,13		CL	SILTY CLAY: low to medium plasticity, light grey and light brown, trace of root fibres.	MC<PL	H	>600 >600 >600	FLUVIAL
	N = 16 4,6,10			SILTY CLAY: medium plasticity, light grey brown and orange brown.	MC≈PL		475 300 >600	
	N = 32 14,16,16	3		SILTY CLAY: low plasticity, brown, with fine grained sand.	MC<PL		>600 >600 >600	
		4	SM	SILTY SAND: fine to medium grained, brown, trace of clay nodules.	M	(L-MD)		
	N > 15 9,15/ 150mm REFUSAL		GC	SILTY CLAYEY GRAVEL: fine to coarse grained, sub angular and sub rounded igneous, light grey and grey, low plasticity clay, trace of fine grained sand.		(D)		VERY HIGH 'TC' BIT RESISTANCE
		7						

SAMPLES: ES, U50, DB, DS

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. 1

Client: BISHI CONSTRUCTIONS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 15-17 DENT STREET, JAMISONTOWN, NSW

Job No. 29853Z **Method:** SPIRAL AUGER JK300 **R.L. Surface:** ≈ 28.6m
Date: 20-10-16 **Datum:** AHD

Logged/Checked by: A.C.K./A.Z.

Groundwater Record	Samples (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
ON COMPLETION						SILTY CLAYEY GRAVEL: medium to coarse grained, sub angular and sub rounded igneous, light grey and grey, trace of fine grained sand.	M	(D)		VERY HIGH RESISTANCE
			8		SM	SILTY SAND: fine to medium grained, light grey and brown.	W	(MD)		
AFTER 0.5 HRS		Nc= 20 REF.	9		SC	CLAYEY SAND: fine grained, light grey brown, medium to coarse grained gravel.				
			10							
			11			END OF BOREHOLE AT 11.0m				TERMINAL DUR TO COLLAPSE MONITORING WELL INSTALLED TO 9.2m, CLASS 18 MACHINE SLOTTED 50mm PVC 3.2m TO 9.2m, CASING 0m TO 3.2m, 2mm SAND FILTER 2.7m TO 9.2m, BENTONITE SEAL 0.15m TO 2.7m, COMPLETED WITH GATIC COVER AT SURFACE
			12							
			13							
			14							

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BOREHOLE LOG

Borehole No. **2**
1/1

Client: BISHI CONSTRUCTIONS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 15-17 DENT STREET, JAMISONTOWN, NSW

Job No. 29853Z
Date: 20-10-16
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 28.9m
Datum: AHD
Logged/Checked by: A.C.K./A.Z.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		FILL: Silty clay, low plasticity, trace of fine grained ironstone gravel.	MC<PL			GRASS COVER TO 50mm DEPTH
	N = 10 2,4,6		CL	SILTY CLAY: medium plasticity, light grey and light brown, trace of fine grained sand.	MC>PL	VSt	320 230 240	FUUVIAL
	N = 12 3,5,7	1–2					215 180 190	
				SILTY CLAY: low plasticity, grey and orange brown, trace of fine grained sand.				
	N = 15 5,6,9	3				St-VSt	250 140 130	
	N = 25 7,12,13	4	SM	SILTY SAND: fine grained, brown, trace of clayey nodules.	M	MD		
		5	GC	SILTY CLAYEY GRAVEL: coarse grained, light grey igneous, medium to high plasticity clay, trace of fine grained sand.		(D)		VERY HIGH 'TC' BIT RESISTANCE
		6		END OF BOREHOLE AT 6.0m				'TC' BIT REFUSAL
		7						

SAMPLES: ES, U50, DB, DS

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	BISHI CONSTRUCTIONS PTY LTD		
Project:	PROPOSED RESIDENTIAL DEVELOPMENT		
Location:	15-17 DENT STREET, JAMISONTOWN, NSW		
Job No.	29853Z	Hammer Weight & Drop: 9kg/510mm	
Date:	20-10-16	Rod Diameter: 16mm	
Tested By:	A.C.K.	Point Diameter: 20mm	

	Number of Blows per 100mm Penetration						
Test Location	RL ~28.5m						
Depth (mm)	**3**						
0 - 100	15						
100 - 200	15						
200 - 300	13						
300 - 400	8						
400 - 500	6						
500 - 600	4						
600 - 700	7						
700 - 800	8						
800 - 900	8						
900 - 1000	8						
1000 - 1100	10						
1100 - 1200	10						
1200 - 1300	8						
1300 - 1400	9						
1400 - 1500	10						
1500 - 1600	13						
1600 - 1700	11						
1700 - 1800	12						
1800 - 1900	14						
1900 - 2000	14						
2000 - 2100	17						
2100 - 2200	15						
2200 - 2300	18						
2300 - 2400	20						
2400 - 2500	25						
2500 - 2600	END						
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.

Ref: JK Geotechnics DCP 0-3m July 2012

AERIAL IMAGE SOURCE: GOOGLE EARTH PRO 7.1.5.1557
AERIAL IMAGE ©: 2015 GOOGLE INC.

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

Title:	SITE LOCATION PLAN
Location:	15-17 DENT STREET JAMISONTOWN, NSW
Report No: 29853Z	Figure No: 1
	JK Geotechnics

PLOT DATE: 31/10/2016 3:07:54 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\29000's\29853Z JAMISONTOWN\CAD\29853Z.DWG

LEGEND

- ● BOREHOLE
- ⊕ DCP TEST

SCALE 0 2 4 6 8 10 METRES, 1:200 @A3	Title: **INVESTIGATION LOCATION PLAN**
	Location: 15-17 DENT STREET JAMISONTOWN, NSW
	Report No: 29853Z — Figure No: 2
This plan should be read in conjunction with the JK Geotechnics report.	**JK Geotechnics**

PLOT DATE: 31/10/2016 3:07:11 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\29000'S\29853Z JAMISONTOWN\CAD\29853Z.DWG

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, the SAA Site Investigation Code. 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 Unified Soil Classification Table qualified by the grading of other particles present (e.g. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2mm
Gravel	2 to 60mm

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	less than 4
Loose	4 – 10
Medium dense	10 – 30
Dense	30 – 50
Very Dense	greater than 50

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

Classification	Unconfined Compressive Strength kPa
Very Soft	less than 25
Soft	25 – 50
Firm	50 – 100
Stiff	100 – 200
Very Stiff	200 – 400
Hard	Greater than 400
Friable	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 thinly bedded to laminated siltstone.

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 shear 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 except test pits, hand auger drilling and portable dynamic cone penetrometers require the use of a mechanical drilling rig which is commonly mounted on a truck chassis.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils 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 an 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. Premature refusal of the hand augers can occur on a variety of materials such as hard clay, gravel or ironstone, 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 relatively lower 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 fragments. 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 determined 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 such as Revert or Biogel. 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, an NMLC triple tube core barrel, which gives a core of about 50mm diameter, 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 CORE LOSS. The location of losses are determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the top end 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, "Methods of Testing Soils for Engineering Purposes" – Test F3.1.

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 63kg 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.

Occasionally, the drop hammer is used to drive 50mm diameter thin walled sample tubes (U50) in clays. In such circumstances, the test results are shown on the borehole logs in brackets.

A modification to the SPT test 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.

Static Cone Penetrometer Testing and Interpretation: Cone penetrometer testing (sometimes referred to as a Dutch Cone) described in this report has been carried out using an Electronic Friction Cone Penetrometer (EFCP). The test is described in Australian Standard 1289, Test F5.1.

In the tests, a 35mm 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 an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm 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.

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.
- 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 EFCP and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of EFCP 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.

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a rod into the ground with a sliding hammer and counting the blows for successive 100mm increments of penetration.

Two relatively similar tests are used:

- Cone penetrometer (commonly known as the Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS1289, Test F3.2). The test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various Road Authorities.
- Perth sand penetrometer – a 16mm diameter flat ended rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test F3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.

LOGS

The borehole or test pit logs presented herein are an engineering and/or geological interpretation of the sub-surface 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 attached explanatory notes define the terms and symbols used in preparation of the logs.

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 water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after stabilising 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 determine 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 Soil for Engineering Purposes'*. 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.

Every 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.

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 that at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Attention is drawn to the document *'Guidelines for the Provision of Geotechnical Information in Tender Documents'*, published by the Institution of Engineers, Australia. 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. License to use the documents may be revoked without notice if the Client is in breach of any objection 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 a senior geotechnical engineer.

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 such as appropriate footing or pier founding depths, or
iii) full time engineering presence on site.

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS

SOIL

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

ROCK

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

DEFECTS AND INCLUSIONS

- CLAY SEAM
- SHEARED OR CRUSHED SEAM
- BRECCIATED OR SHATTERED SEAM/ZONE
- IRONSTONE GRAVEL
- ORGANIC MATERIAL

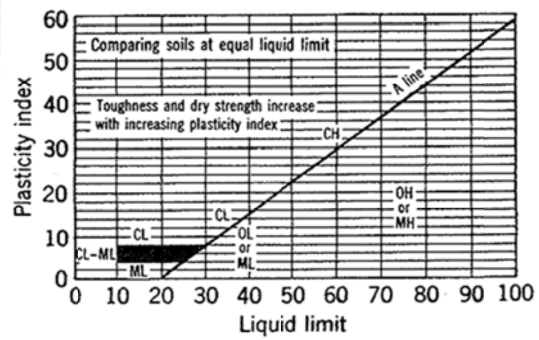
OTHER MATERIALS

- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

UNIFIED SOIL CLASSIFICATION TABLE

Field Identification Procedures (Excluding particles larger than 75 μm and basing fractions on estimated weights)						Group Symbols [a]	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria
Coarse-grained soils More than half of material is *larger* than 75 μm sieve size[b] (The 75 μm sieve size is about the smallest particle visible to naked eye)	Gravels More than half of coarse fraction is larger than 4 mm sieve size	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes			*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing			*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for *GW*
		Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)			*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics	Atterberg limits below "A" line, or *PI* less than 4 — Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)			*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures		Atterberg limits above "A" line, with *PI* greater than 7
	Sands More than half of coarse fraction is smaller than 4 mm sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes			*SW*	Well graded sands, gravelly sands, little or no fines	Example: *Silty sand*, gravelly; about 20% hard, angular gravel particles 12 mm maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (*SM*)	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing			*SP*	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for *SW*
		Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)			*SM*	Silty sands, poorly graded sand-silt mixtures		Atterberg limits below "A" line or *PI* less than 5 — Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)			*SC*	Clayey sands, poorly graded sand-clay mixtures		Atterberg limits below "A" line with *PI* greater than 7
	Identification Procedures on Fraction Smaller than 380 μm Sieve Size		Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)				
Fine-grained soils More than half of material is *smaller* than 75 μm sieve size	Silts and clays liquid limit less than 50		None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses	
			Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays		
			Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions	
	Silts and clays liquid limit greater than 50		Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts		
			High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays	Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)	
			Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity		
Highly Organic Soils			Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils		

Laboratory Classification Criteria (coarse-grained soils): Use grain size curve in identifying the fractions as given under field identification. Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than 75 μm sieve size) coarse grained soils are classified as follows: Less than 5% *GW, GP, SW, SP*; More than 12% *GM, GC, SM, SC*; 5% to 12% *Borderline* cases requiring use of dual symbols.



Plasticity chart
for laboratory classification of fine grained soils

Note: 1 Soils possessing characteristics of two groups are designated by combinations of group symbols (eg. GW-GC, well graded gravel-sand mixture with clay fines).
2 Soils with liquid limits of the order of 35 to 50 may be visually classified as being of medium plasticity.

LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION
Groundwater Record	▼	Standing water level. Time delay following completion of drilling may be shown.
	—c—	Extent of borehole collapse shortly after drilling.
	▶——	Groundwater seepage into borehole or excavation noted during drilling or excavation.
Samples	ES	Soil 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 screening.
	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. 'R' as noted below.
	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 degree 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 (Cohesive Soils)	MC>PL	Moisture content estimated to be greater than plastic limit.
	MC≈PL	Moisture content estimated to be approximately equal to plastic limit.
	MC<PL	Moisture content estimated to be less than plastic limit.
(Cohesionless 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 less than 25kPa
	S	SOFT – Unconfined compressive strength 25-50kPa
	F	FIRM – Unconfined compressive strength 50-100kPa
	St	STIFF – Unconfined compressive strength 100-200kPa
	VSt	VERY STIFF – Unconfined compressive strength 200-400kPa
	H	HARD – Unconfined compressive strength greater than 400kPa
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other tests.
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 15-35 — 4-10
	MD	Medium Dense 35-65 — 10-30
	D	Dense 65-85 — 30-50
	VD	Very Dense >85 — >50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other tests.
Hand Penetrometer Readings	300 250	Numbers indicate individual test results in kPa on representative undisturbed material unless noted otherwise.
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Tungsten carbide wing bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.

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LOG SYMBOLS continued

ROCK MATERIAL WEATHERING CLASSIFICATION

TERM	SYMBOL	DEFINITION
Residual Soil	RS	Soil developed on extremely weathered rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.
Extremely weathered rock	XW	Rock is weathered to such an extent that it has "soil" properties, ie it either disintegrates or can be remoulded, in water.
Distinctly weathered rock	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by ironstaining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Slightly weathered rock	SW	Rock is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh rock	FR	Rock shows no sign of decomposition or staining.

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index (Is 50) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by the International Journal of Rock Mechanics, Mining, Science and Geomechanics. Abstract Volume 22, No 2, 1985.

TERM	SYMBOL	Is (50) MPa	FIELD GUIDE
Extremely Low:	EL		Easily remoulded by hand to a material with soil properties.
-----------------------	--------------	0.03	
Very Low:	VL		May be crumbled in the hand. Sandstone is "sugary" and friable.
-----------------------	--------------	0.1	
Low:	L		A piece of core 150mm long x 50mm dia. may be broken by hand and easily scored with a knife. Sharp edges of core may be friable and break during handling.
-----------------------	--------------	0.3	
Medium Strength:	M		A piece of core 150mm long x 50mm dia. can be broken by hand with difficulty. Readily scored with knife.
-----------------------	--------------	1	
High:	H		A piece of core 150mm long x 50mm dia. core cannot be broken by hand, can be slightly scratched or scored with knife; rock rings under hammer.
-----------------------	---------------	3	
Very High:	VH		A piece of core 150mm long x 50mm dia. may be broken with hand-held pick after more than one blow. Cannot be scratched with pen knife; rock rings under hammer.
-----------------------	---------------	10	
Extremely High:	EH		A piece of core 150mm long x 50mm dia. is very difficult to break with hand-held hammer. Rings when struck with a hammer.

ABBREVIATIONS USED IN DEFECT DESCRIPTION

ABBREVIATION	DESCRIPTION	NOTES
Be	Bedding Plane Parting	Defect orientations measured relative to the normal to the long core axis (ie relative to horizontal for vertical holes)
CS	Clay Seam	
J	Joint	
P	Planar	
Un	Undulating	
S	Smooth	
R	Rough	
IS	Ironstained	
XWS	Extremely Weathered Seam	
Cr	Crushed Seam	
60t	Thickness of defect in millimetres	