REPORT

TO
KOGARAH 048 SERVICE PTY LTD

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED RESEDENTIAL DEVELOPMENT

AT
12 – 24 STANLEY STREET
KOGARAH, NSW

26 April 2018
Ref: 31240S rev1

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

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Date: 26 April 2018
Report No: 31240Srpt rev1
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For and on behalf of
JK GEOTECHNICS
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1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed 11-storey residential development at 12-24 Stanley Street, Kogarah, NSW. The site location is shown on the attached Figure 1. The investigation was commissioned by Poly Australia on behalf of Kogarah 048 Service Pty Ltd and was completed in accordance with our proposal (Ref: P46070S, dated 3 November 2017).

We understand that the proposed development consists of the construction of an 11-storey building over a 4-level basement carpark. The finished floor level for basement B4 is expected to be at RL5.25m. The basement will extend close to the site boundaries except in the north-east and south-west corners and with a small set-back of about 1m along Stanley Lane on the southern boundary.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions at the borehole locations. Based on this information we have provided comments and recommendations on excavation, retention, earth pressures, footing design, slabs on grade, hydrogeology and soil aggression. Due to access limitations it was only possible to drill in the eastern half of the site and we understand it is proposed to drill in the western portion when access is available. We would not expect there to be differences in ground conditions in the western part of the site which would invalidate the comments and recommendations which follow, although there may be differences that require detail changes in the design.

2 INVESTIGATION PROCEDURE

The subsurface investigation comprised the drilling of three boreholes using our track mounted JK305 drill rig. The boreholes were initially advanced through the soils and upper weaker rock using solid flight augers with an attached Tungsten Carbide (TC). At depths ranging from 3.7m (BH3) to 5.9m (BH) auger drilling techniques were ceased, rotary diamond coring techniques commenced and the boreholes extended to termination depths of 17.35m.

The borehole locations are shown on the attached Figure 2 and were set out by taped measurements from features shown on the survey plan prepared by SDG Land Development Solutions (Project No. 7578, dated 13 February 2018). The reduced levels shown at the top of the borehole logs were interpolated from spot levels shown on the above plan and are consequently approximate only.

Prior to commencement of fieldwork a Dial Before you Dig services search was completed and the borehole locations were electromagnetically scanned by a specialist subcontractor so that all boreholes could be located clear of buried services.

The strength of the soils was interpreted from the Standard Penetration Test (SPT) 'N' values. This interpretation was augmented by hand penetrometer tests completed on the samples recovered in the SPT split tube sampler. Where the bedrock was augered, rock strengths were assessed from observation of the penetration resistance and by examination of the recovered rock cuttings. We note that rock strengths estimated in this way are indicative and variations of at least one strength order should not be unexpected.

Where the bedrock was cored, the recovered core was returned to Soil Test Services (STS) for photographing and Point Load Strength Index (I_{s50}) testing. Using established correlations the unconfined compressive strength (UCS) of the bedrock was then calculated from the I_{s50} results. These point load strength test results are presented in the attached Table A and are also shown on the borehole logs.

Selected soil and rock samples were also sent to a NATA registered laboratory for pH, chloride and sulphate content and resistivity testing. These results are presented in the attached Envirolab Certificate of Analysis – 185783.

Groundwater observations were made in the boreholes both during and on completion of auger drilling. PVC standpipes were installed in the boreholes on completion to allow longer term monitoring of groundwater level fluctuations. The water levels were measured on 28 February 2018, approximately two weeks after drilling was completed, which should have given the water introduced during the drilling process sufficient time to reach a state of equilibrium.

Our geotechnical engineer was present on a full-time basis during the fieldwork, to direct the electromagnetic scanning, set out the borehole locations, nominate testing and sampling and prepare the borehole logs. The borehole logs which include field test results and groundwater observations are attached to this report together with a set of explanatory notes, which describe the investigation techniques, their limitations and define the logging terms and symbols used.

3 RESULTS OF INVESTIGATION

3.1 Site Description

For ease of understanding the following description, reference should be made to the attached Figures 1 and 2.

The site is located on a hillside that slopes down to the east at approximately 2°. The site is bound by Stanley Street to the north and Stanley Lane to the south. The site is roughly rectangular in plan, being about 33.5m wide (north to south) by about 73m long (east to west) and has a maximum elevation relief of about 3.0m from the eastern to western site boundaries.

At the time of the fieldwork, the site comprised seven properties all with one and two storey brick and clad frame houses. The houses had front and rear gardens, concrete driveways and single storey structures of either brick, metal or clad construction adjacent to Stanley Lane.

The neighbouring property to the west contained a double storey brick house occupying the central portion of the site, set back approximately 2.5m from the common boundary. A single storey brick and clad frame garage was located at the rear of the site and abutted the common boundary. Concrete surfaces were located at the front and rear of the property with the remainder of the site covered by brick pavers and small garden beds.

A single storey clad frame house and single storey garage were set back approximately 2.5m and 0.4m, respectively, from the eastern boundary of the subject site. A small grass covered yard was located at the front of the site with a brick paved walkway located adjacent to the western boundary of the site.

Vegetation across the site and within the surrounding area contained small to medium sized bushes and trees. The buildings and structures mentioned above generally appeared in a good condition based on a cursory inspection from within the site and street frontages although the brick pavers within the neighbouring property to the west were in a poor condition with localised subsidence up to 50mm.

3.2 Subsurface Conditions

The 1:100,000 Geological Map of the Sydney Region indicates that the site is underlain by the Hawkesbury Sandstone. To the east of the site is the geological boundary with Quaternary sediments comprising sands, silts and clays of low strength.

The boreholes disclosed a profile comprising a thin layer of fill overlying natural silty clay and silty sandy clay that in turn were underlain by sandstone bedrock. A more detailed summary of the subsurface conditions encountered is provided below. Reference should be made to the attached borehole logs for detailed descriptions of the subsurface conditions at particular borehole locations.

Pavements and Fill

Concrete pavements were encountered at Boreholes 1 and 2 with thicknesses of 80mm to 120mm. In Boreholes 1 and 2 a silty sand fill was encountered below the pavements and extended to depths of 0.4m to 0.6m respectively. In Borehole 3 a silty sandy clay topsoil was present.

Residual Silty Clay

Residual silty clays and silty sandy clays of very stiff to hard strength and mainly medium plasticity were encountered either below the topsoil or fill. These residual clays extended to depths ranging from 3.0m to 4.9m.

Bedrock

Underlying the residual silty clays sandstone bedrock was encountered. The bedrock comprised sandstone, mostly fine to medium grained but some being fine to coarse grained. When first encountered the sandstone was of extremely low to low strength but within 0.2m to 0.4m became at last low strength and within 1m to 1.5m became medium or medium to high strength. At depths of 8m to 10m the sandstone was mainly high strength. The sandstone units were mainly distinctly cross-bedded with only occasional bands of more massive/homogeneous material. Minor shale bands/lenses were present in each borehole. Defects were almost entirely bedding partings at 0^0 to 20^0 though a few joints at around 25^0 were also noted.

Groundwater

All boreholes were dry on completion of auger drilling. Coring involves the pumping of water down the borehole to aid in the drilling process and the removal of rock cuttings which results in elevated water levels. Slotted PVC standpipes were installed into the boreholes on completion to allow monitoring of groundwater over a longer time frame. Standing water was observed at depths of 8.42m to 8.78m on 28 February 2018 which was about 2 weeks after completion of drilling.

3.3 Laboratory Test Results

The point load strength index tests showed good correlation with the field logging strengths as shown on the borehole logs.

A summary of the soil chemistry test results is provided in the table below:

Borehole	Sample Depth (m)	Description	pH Units	Sulfate (mg/kg)	Chloride (mg/kg)	Resistivity (ohm cm)
BH1	1.5 – 1.95	Silty sandy clay	5.1	75	<10	17,000
BH2	4.5 - 4.85	Silty sandy clay	4.8	30	10	29,000
BH3	0.5 – 0.95	Sandy clay	6.8	10	<10	57,000

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Issues

The main geotechnical issues associated with the excavation and construction would seem to be the design and installation of a shoring system and excavation of the relatively strong sandstone bedrock. Foundations for the building appear to be good and the use of simple pad footings at basement level should be straightforward. There will be seepage into the excavation though it should be readily controlled by normal good design and construction practices.

4.2 Excavation

Excavation for the proposed development will extend to depths of about 11 to 13.5m, being deeper at the western side of the site where ground levels are higher. Excavation to these depths will require the removal of silty and sandy clays and sandstone bedrock. Excavation of the soils and bedrock of up to low strength should be able to be completed using medium sized excavators (say 15 to 20 tonnes) with buckets with “tiger teeth” attached for the weak rock. Where the bedrock is of greater than low strength, “hard rock” excavation techniques will be required.

Where percussive rock excavation techniques are adopted it is recommend that considerable caution be taken as there will be direct transmission of ground vibrations to adjoining buildings and structures. Prior to rock excavation commencing, dilapidation reports should be completed on the adjoining properties to the east and west. A copy of these reports should be provided to the respective property owners and they should be asked to confirm that the reports present a fair record of existing conditions. The dilapidation reports may then be used as a benchmark against which to assess possible future claims for damage resulting from the works. In this manner the reports protect the builder from unfounded claims relating to damage existing prior to the commencement of work.

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

Where percussive excavation methods (ie. rock hammers) are adopted we recommend that hammer size be carefully selected with respect to the adjoining movement sensitive structures and the potential for transmitted vibrations to damage these structures. In this regard quantitative vibration monitoring should be completed in the initial stages of percussive excavation to provide feedback to the contractor on the suitability of the equipment and excavation techniques adopted. Following this initial monitoring, periodic or full time quantitative vibration monitoring should be undertaken throughout the duration of percussive excavation, depending upon the degree of assurance required that tolerable limits are not exceeded on the adjacent structures. The purpose of this monitoring is to provide feedback to the contractor on the suitability of their equipment and to build up a record of the magnitude of transmitted vibrations generated during percussive excavation. Where acceptable transmitted vibration limits are exceeded, hammer sizes should be reduced and consideration should be given to the installation of continuous vibration monitors if not already installed. We have attached our Vibration Emission Design Goals to provide guidance on the upper limits for transmitted vibrations. In order to reduce the magnitude of transmitted vibrations we recommend that the following techniques be adopted:

- That rock hammers be used in short burst such that transmitted vibrations are not magnified,
- That the excavation be progressively opened up by breaking wedges out of the face of the excavation and
- That a sharp moil be maintained.

Alternatively, non-percussive excavation methods may be adopted. These methods may consist of the use of rock saws, rotary grinders, rock splitting or ripping tynes. Where ripping tynes are used we consider that they will need to be attached to heavy excavators to effectively rip sandstone bedrock of better than low strength, but even so it is not likely that ripping will be effective without saw-cutting the high quality bedrock that will be encountered with depth. Given the relatively small area of the excavation it is not expected that ripping with heavy bulldozers will be viable. Where ripping tynes attached to excavators are used, care must be taken to ensure that the tyne is not hammered into the rock in an attempt to break or dislodge the bedrock as this action will result in the creation of possibly damaging transmitted vibrations. Where non-percussive excavation techniques are adopted we consider that vibration monitoring will not be necessary.

4.3 Groundwater Control

Provided excavation does not extend below about 8m depth we expect that some groundwater seepage flows will occur at the soil-rock interface and through joints and bedding planes within the completed cut faces, particularly after periods of heavy rain. Seepage, if any, is expected to be satisfactorily controlled by sump and pump that discharges groundwater inflows to the Council stormwater system both during and after construction. During construction some treatment may be required to reduce silt content and manage pH levels.

Where excavation extends significantly below 8m depth then inflows are likely to be more persistent and of greater volume. Nevertheless the permeability of the high quality sandstone which is present at the site is normally quite low and total inflows would be expected to be within normally tolerable volumes. Further testing and analysis could be undertaken if this issue is to be addressed in more detail and would enable estimates of inflow volumes to be calculated.

4.4 Retention

Given the depth of soil cover at the site we do not consider it practical to form temporary batter slopes as these would need to be at least battered back to 1 Horizontal (H) : 1 Vertical (V), probably with a mid-height bench where soil depths exceed 3m. As the excavation extends up to or close to the site boundaries temporary batters are not physically possible. Therefore we recommend that prior to the commencement of excavation a retention system must be installed to support the soils and poorer quality bedrock. For this site an anchored soldier pile retaining wall with shotcrete and mesh infill panels may be adopted. It is anticipated that bored piers can be used on this site.

Given that the depth of excavation is going to be between about 11m and 13.5m and that good quality sandstone was encountered from depths between about 4.5m and 6.5m in the boreholes, it is not necessary to socket the piles below bulk excavation level provided at least two rows of ground anchors are installed to provide stability. The toe level of the piles will be dictated by the level of the floor slabs used to provide permanent propping. It will be essential to ensure the piles are founded at sufficient depth that they are on high quality sandstone which will form a stable face in the short and long term. As piles will rely on the stability of the rock face for support we recommend that they are not used to support substantial structural elements of the building and that those loads are transferred below bulk excavation level by internal columns. We recommend that pile loads should not exceed 1000kPa applied bearing pressure.

For the design of anchored walls we recommend that a trapezoidal earth pressure distribution (refer to Figure 3) be adopted for the full height of the proposed shoring wall. Where movement sensitive structures (including movement sensitive services present within the road corridors) are located within the zone of influence of the excavation (defined as a distance 2H extending horizontally out from the crest of the excavation where H is the height of retained materials) a pressure of 8H kPa should be adopted over the central 50% of the pressure distribution to help limit behind wall deflections. Where no movement sensitive structures are located within the zone of influence of the excavation, a pressure of 5H kPa may be adopted. Appropriate hydrostatic pressures and surcharge loads such as nearby building footing loads, traffic loads, stock piles etc should be added to above pressure distributions.

Bedrock of medium strength or better is expected to be self-supporting, unless affected by inclined joints. It is essential that the excavation is inspected by a geotechnical engineer at frequent intervals in order that the presence off any adversely inclined joints is identified in good time and stabilisation works implemented to prevent instability developing. The maximum vertical interval for inspections should be 1.5m but the inspections may need to be staggered on different sides of the excavation as progress is unlikely to be uniform. Stabilisation works may include rock bolts and the use of reinforced shotcrete or dental concrete where weak seams occur. In the long term any rock bolts which extend beyond site boundaries will have to be replaced by retaining walls or other structural works. We do recommend that all faces of the excavation be saw cut rather than hammered as this usually results in cost savings with respect to long term stability of the rock faces.

Where temporary anchors are installed below neighbouring properties, permission will need to be obtained from the respective property owners. For the preliminary design of anchors a bond stress of 500kPa may be adopted for anchors formed within medium or better strength bedrock provided the bond length starts below a line drawn upwards from the base of the excavation at 1V:1H. All anchors should be proof loaded in four equal increments to a minimum of 1.3 times the design load. All proof loading should be undertaken in the presence of an independent party engaged directly by the principal. Lift off tests should be carried out on at least 10% of anchors at least 48 hours after stressing; should any anchors fail then all anchors should be tested. Only experienced contractors with the appropriate insurances should be considered for this work. Anchors should be a design and construct sub-contract to avoid contractual disputes should any anchors fail test load.

4.5 <u>Footing Design</u>

Based upon the boreholes completed to date excavation for the proposed development will result in the exposure of bedrock over the proposed building footprint. Consequently, we recommend that all footings be uniformly founded on the underlying bedrock. The table below details the allowable bearing pressures (ABP) that may be adopted at varying depths at each of the borehole locations. It should be noted that where the higher bearing pressures are adopted further site proving will be required to confirm these ABP's. The allowable bearing pressure is based upon settlements not exceeding 1% of the footing width.

ABP (kPa)	***Borehole 1***	***Borehole 2***	***Borehole 3***
1,000	6.0m (RL10.2m)	5.3m (RL11.9m)	4.6m (RL12.6m)
3,500	7.6m (RL8.6m)	5.3m (RL11.9m)	4.6m (RL12.6m)
6,000	8.8m (RL7.4m)	7.3m (RL9.9m)	5.6m (RL11.6m)

Notes:

- *ABP assumes the footing is below bulk excavation level ie does not apply to shoring piles.*
- *In BH1 a weak zone from 14.2m depth to 15.3m depth would affect the bearing capacity of footings above where the seam is within the zone of influence.*
- *In BH3 a weak seam at the base of the borehole from 17.15m to 17.36m could affect bearing capacity of footings above where the seam is within the zone of influence.*
- *The zone of influence for square or rectangular footings is taken to be 1.5x the footing width.*

Prior to pouring concrete all footings should be inspected by a geotechnical engineer to confirm that the design ABP's are achieved. Where ABP's of 3,500kPa or 6,000kPa are adopted further testing may be required. This will either comprise spoon tests or further cored boreholes. Where an ABP of 3,500 is adopted further testing should be reassessed after the planned three additional boreholes have been completed. Where an ABP of 6,000kPa is adopted spoon testing or cored boreholes will be required at 50% of all footings evenly spread across the site in addition to the three additional cored boreholes.

Prior to pouring concrete all footings should be free from all loose and softened materials. Where water ponds in the base of footings, the footings should first be pumped dry and then re-excavated to remove all loose and softened materials.

4.6 Earthquake Design

The site classifies as Class $C_{e,}$ shallow soil site, in accordance with AS1170.4.

4.7 Slabs on Grade

Bulk excavation is anticipated to result in the exposure of sandstone bedrock over all of the site.

All slabs should be provided with underfloor drainage. The underfloor drainage should comprise a strong, durable, single sized washed aggregate, such as 'blue metal' gravel. The subgrade should be shaped to promote runoff. The underfloor drainage should collect groundwater seepage and direct it to the stormwater system. Either a continuous blanket or discrete drains on a reticulated grid may be used. If the latter is preferred then we recommend that trafficable concrete pavements be underlain by 100mm of DGB20 or other similar good quality base-course material. Slabs should be designed with keyed or dowelled joints to transmit shear forces but not bending moments.

4.8 Soil Aggression

Based on the Envirolab Services test results, 'mild' and 'non-aggressive' exposure classifications are applicable in accordance with AS2159-2009 for concrete and steel piles, respectively.

The site is not in an area nor does it have the type of geology usually associated with acid-sulphate soils.

4.9 Further Work

The recommend that the following further works be completed before or during construction:

- Monitoring of groundwater levels in Boreholes 1 to 3.
- Drilling of 3 additional cored boreholes in the western portion of the site.
- Dilapidation reports of adjoining movement sensitive structures prior to the commencement of construction.
- Review of structural drawings to confirm the intentions of this report have been successfully implemented in the design.
- Periodic or continuous vibration monitoring during percussive excavation.
- Proof loading of all anchors to confirm that their design loads are achieved.
- Inspection and/or testing of all footings by a geotechnical engineer to confirm that the design ABP's are achieved.

- Inspection of the excavation to review groundwater seepage and drainage.

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. 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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SOIL TEST SERVICES

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client: JK Geotechnics
Project: Proposed Residential Development
Location: 14-24 Stanley Street, Kogarah, NSW

Ref No: 31240S
Report: A
Report Date: 26/02/2018

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	6.01 - 6.05	0.3	6
	6.56 - 6.60	0.7	14
	6.89 - 6.92	0.9	18
	7.31 - 7.35	1.1	22
	7.79 - 7.82	0.7	14
	8.24 - 8.28	1.4	28
	8.77 - 8.81	1.0	20
	9.23 - 9.28	1.0	20
	9.85 - 9.90	0.9	18
	10.27 - 10.31	1.7	34
	10.53 - 10.56	1.5	30
	10.85 - 10.90	2.2	44
	11.31 - 11.36	1.2	24
	11.79 - 11.84	1.7	34
	12.30 - 12.35	1.6	32
	12.81 - 12.85	1.3	26
	13.31 - 13.35	1.7	34
	13.73 - 13.77	1.1	22
	14.14 - 14.18	0.9	18
	14.40 - 14.43	0.1	2
	14.81 - 14.84	2.1	42
	15.28 - 15.31	0.2	4
	15.31 - 15.35	1.4	28
	15.75 - 15.80	1.0	20
	16.25 - 16.30	1.6	32

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BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	16.71 - 16.75	1.1	22
	17.21 - 16.25	2.0	40
2	5.48 - 5.51	0.9	18
	5.84 - 5.88	1.0	20
	6.31 - 6.36	1.0	20
	6.85 - 6.90	0.4	8
	7.30 - 7.36	1.3	26
	7.80 - 7.84	0.7	14
	8.25 - 8.30	1.4	28
	8.77 - 8.82	1.6	32
	9.31 - 9.37	2.1	42
	9.86 - 9.91	1.5	30
	10.26 - 10.30	1.0	20
	10.81 - 10.86	1.7	34
	11.31 - 11.37	1.5	30
	11.85 - 11.91	1.7	34
	12.24 - 12.29	1.2	24
	12.80 - 12.85	1.9	38
	13.28 - 13.34	1.7	34
	13.77 - 13.83	1.9	38
	14.31 - 14.35	1.8	36
	14.75 - 14.81	1.3	26
	15.23 - 15.27	1.9	38
	15.78 - 15.83	1.7	34
	16.24 - 16.28	1.4	28

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BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
2	16.76 - 16.81	1.2	24
	17.04 - 17.08	0.04	1
	17.30 - 17.35	2.2	44
3	3.81 - 3.85	0.06	1
	4.15 - 4.18	0.7	14
	4.83 - 4.87	0.9	18
	5.21 - 5.26	0.8	16
	5.79 - 5.83	1.1	22
	6.27 - 6.31	1.0	20
	6.79 - 6.84	0.8	16
	7.24 - 7.28	0.9	18
	7.74 - 7.80	2.2	44
	8.24 - 8.29	1.2	24
	8.84 - 8.89	0.9	18
	9.24 - 9.30	1.0	20
	9.78 - 9.84	1.4	28
	10.19 - 10.24	1.4	28
	10.83 - 10.87	0.7	14
	11.25 - 11.31	2.0	40
	11.86 - 11.92	1.3	26
	12.18 - 11.24	1.3	26
	12.83 - 12.88	1.7	34
	13.21 - 13.27	1.5	30
	13.79 - 13.84	1.4	28
	14.24 - 14.29	2.4	48

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3	14.83 - 14.87	1.4	28
	15.21 - 15.25	1.4	28
	15.76 - 15.81	2.1	42
	16.22 - 16.27	0.7	14
	16.81 - 16.86	2.0	40
	17.31 - 17.36	0.4	8

NOTES:

1. In the above table testing was completed in the Axial direction.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RMS T223.
4. For reporting purposes, the $I_{S(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa
5. The Estimated Unconfined Compressive Strength was calculated from the point load Strength Index by the following approximate relationship and rounded off to the nearest whole number :
   U.C.S. = 20 $I_{S(50)}$

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CERTIFICATE OF ANALYSIS 185783

Client Details

Client	JK Geotechnics
Attention	Michael Egan, Paul Stubbs
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**31240S, Kogarah**
Number of Samples	3 Soil
Date samples received	22/02/2018
Date completed instructions received	22/02/2018

Analysis Details

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

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

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

Report Details

Date results requested by	01/03/2018
Date of Issue	26/02/2018

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Results Approved By
Nick Sarlamis, Inorganics Supervisor

Authorised By

David Springer, General Manager

Misc Inorg - Soil				
Our Reference		185783-1	185783-2	185783-3
Your Reference	UNITS	BH1	BH2	BH3
Depth		1.5-1.95	4.5-4.85	0.5-0.95
Date Sampled		13/02/2018	14/02/2018	15/02/2018
Type of sample		Soil	Soil	Soil
Date prepared	-	23/02/2018	23/02/2018	23/02/2018
Date analysed	-	23/02/2018	23/02/2018	23/02/2018
pH 1:5 soil:water	pH Units	5.1	4.8	6.8
Sulphate, SO4 1:5 soil:water	mg/kg	75	30	10
Chloride, Cl 1:5 soil:water	mg/kg	<10	10	<10
Resistivity in soil*	ohm m	170	290	570

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Alternatively determined by colourimetry/turbidity using Discrete Analyer.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	185783-2
Date prepared	-			23/02/2018	1	23/02/2018	23/02/2018		23/02/2018	23/02/2018
Date analysed	-			23/02/2018	1	23/02/2018	23/02/2018		23/02/2018	23/02/2018
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	1	5.1	5.1	0	102	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	75	70	7	110	120
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	<10	<10	0	100	90
Resistivity in soil*	ohm m	1	Inorg-002	<1	1	170	180	6	[NT]	[NT]

Result Definitions

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

Quality Control Definitions

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

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

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. 1

Client: KOGARAH 048 SERVICE PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 12-24 STANLEY STREET, KOGARAH

Job No.: 31240S **Method:** SPIRAL AUGER **R.L. Surface:** ~16.2 m
Date: 13/2/18 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** M.E/P.S

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING			16			CONCRETE: 90mm.t				
					-	FILL SILTY SAND: fine to medium grained, dark grey, with clay fines, trace fine grained igneous gravel and root fibres.	M			
	DS	N = 12 4,6,6			CI	FILL SAND: fine to medium grained, grey brown.				
			15	1		SILTY SANDY CLAY: low plasticity, orange brown, fine to medium grained, trace fine to medium grained ironstone gravel.	w>PL	VSt	350 290 260	RESIDUAL
	DS	N = 9 4,4,5	14	2					210 310 270	
	DS	N = 16 5,6,10	13	3		SILTY CLAY: medium plasticity, light grey, light brown and red brown, with fine to medium grained sand.		Hd	430 510 530	
			12	4		SILTY SANDY CLAY: medium plasticity, light grey and red brown, sand fine to medium grained.		VSt		
ON COMPLETION OF CORING	DS	N = 19 6,9,10							390 390 310	
			11	5		SANDSTONE: fine to medium grained, light grey, with clay bands.	XW	EL		VERY LOW 'TC' BIT RESISTANCE
						SANDSTONE: fine to medium grained, light grey and red brown.	DW	L		LOW TO MODERATE RESISTANCE
			10	6		REFER TO CORED BOREHOLE LOG				

CORED BOREHOLE LOG

Borehole No. 1

Client: KOGARAH 048 SERVICE PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 12-24 STANLEY STREET, KOGARAH

Job No.: 31240S | **Core Size:** NMLC | **R.L. Surface:** ~16.2 m
Date: 13/2/18 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK305 | **Bearing:** N/A | **Logged/Checked By:** M.E/P.S

Water Loss\Level	Depth (m)	CORE DESCRIPTION (Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components)	Weathering	Strength	DEFECT DETAILS – DESCRIPTION (Type, orientation, defect roughness and shape, defect coatings and seams, openness and thickness) Specific / General
	5.88	START CORING AT 5.88m			
	5.88–5.96	CORE LOSS 0.08M.			
	5.96–6.45	SANDSTONE: fine to medium grained, light grey, with iron induarated bands, bedded at 5-25.	DW	L	(6.05m) J, 15°, P, R, Cn (6.10m) Be, 25°, P, R, Cn (6.17m) J, 35°, P, R, Cn (6.39m) CS, 0°, 100 mm.t
	6.45–7.00	As above, but fine to coarse grained.		M - H	(6.89m) Be, 20°, P, R, Cn
ON 28/2/18	7.00–8.80	SANDSTONE: fine to coarse grained, light grey, light brown and red brown, bedded at 0-20°.	SW		(7.00m) Be, 5°, Un, R, Cn (7.08m) Be, 20°, P, R, Cn (7.20m) Be, 15°, P, R, Cn (7.30m) Be, 20°, P, R, Cn (7.55m) Be, 20°, P, R, Cn (7.65m) J, 5 - 60°, Un, HEALED (7.73m) Be, 15°, P, R, Cn (7.79m) Be, 15°, P, R, Cn (7.85m) Be, 15°, P, R, Cn (8.03m) Be, 20°, P, R, Fe (8.24m) Be, 0°, P, R, Cn (8.50m) Be, 0°, P, R, Cn
	8.80–	SANDSTONE: fine to medium grained, light grey and light brown, bedded at 0-20°.	FR		(8.99m) XWS, 0°, 5 mm.t (9.53m) Be, 10°, P, R, Cn (9.70m) XWS, 0°, 40 mm.t
				H	(10.45m) XWS, 5°, 3 mm.t (10.55m) J, 25°, P, R, Cn (10.65m) XWS, 0°, 2 mm.t

Column scales: RL (m AHD) 11, 10, 9, 8, 7, 6, 5; Depth (m) 6, 7, 8, 9, 10, 11; POINT LOAD STRENGTH INDEX $I_s(50)$: VL -0.1, L -0.3, M -1, H -3, VH -10, EH; SPACING (mm): 600, 200, 60, 20. Graphic Log; Barrel Lift; Formation.

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No. 1

CORED BOREHOLE LOG

Client: KOGARAH 048 SERVICE PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 12-24 STANLEY STREET, KOGARAH

Job No.: 31240S | **Core Size:** NMLC | **R.L. Surface:** ~16.2 m
Date: 13/2/18 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK305 | **Bearing:** N/A | **Logged/Checked By:** M.E/P.S

RL (m AHD)	Depth (m)	Core Description	Weathering	Strength	Defect Details – Specific
4 to 2.3	12.06–14.0	SANDSTONE: fine to medium grained, light grey and light brown, bedded at 0-20°. *(continued)*	FR	H	(12.06m) CS, 20°, 1 mm.t
	13.0–14.3	As above, but with fine to coarse grained quartz gravel and shale inclusions.	FR	H / M	
2	14.3–14.8	SANDSTONE: fine grained, light grey and grey, with dark grey siltstone laminae, bedded at 0 - 70°.	XW	EL	
	14.8–15.3	SANDSTONE: fine to medium grained, light grey and light brown, with dark grey shale inclusions, bedded at 0-20°.	FR	H	
1	15.3	SILTSTONE: dark grey.	XW - DW	EL - VL	(15.31m) XWS, 10°, 30 mm.t
0 to -1	15.3–17.36	SANDSTONE: fine to coarse grained, light grey and light brown, bedded at 0-20°.	FR	H	
-1	17.36	END OF BOREHOLE AT 17.36 m			CLASS 18 PVC STANDPIPE INSTALLED TO 17.36m DEPTH, SLOTTED 17.36m TO 14.36m, CASING FROM 14.36m TO 0.1m, BOREHOLE BACKFILLED WITH 2mm SAND FILTER 17.36m TO 1.50m, BENTONITE SEAL 1.50m TO 0.1m, COMPLETED WITH CONCRETED CAST IRON GATIC COVER

Column headings: Water Loss\Level; Barrel Lift; RL (m AHD); Depth (m); Graphic Log; CORE DESCRIPTION (Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components); Weathering; Strength; POINT LOAD STRENGTH INDEX $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH); DEFECT DETAILS: SPACING (mm) (600, 200, 60, 20); DESCRIPTION (Type, orientation, defect roughness and shape, defect coatings and seams, openness and thickness) Specific / General; Formation

Depth scale: 13, 14, 15, 16, 17, 18. RL scale: 4, 3, 2, 1, 0, -1, -2.


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JOB No. 31240S BH1 START CORING AT 5·88m
5
CORE LOSS 0·08m
6
7
8
9
10
11
12
13
14
15
16
17
END OF BOREHOLE AT 17·36m.

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
2

BOREHOLE LOG

Client: KOGARAH 048 SERVICE PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 12-24 STANLEY STREET, KOGARAH

Job No.: 31240S **Method:** SPIRAL AUGER **R.L. Surface:** ~17.2 m
Date: 14/2/18 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** M.E/P.S

Groundwater Record	Samples (ES / U50 / DB / DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING			17				CONCRETE 0.12M.				
						-	FILL SILTY SAND: fine to medium grained, grey brown, with bick fragments.	M			
	DS	N = 9 3,4,5	16	1		CL	SILTY SANDY CLAY: low plasticity, orange brown, sand fine to medium grained.	w~PL	VSt	340 260 270	RESIDUAL
	DS	N = 14 4,6,8	15	2		CI	SILTY SANDY CLAY: medium plasticity, light brown mottled light grey and red brown, sand fine to medium grained, with fine to medium grained ironstone gravel.	w>PL		260 240 340	
							SILTY SANDY CLAY: medium plasticity, light grey, sand fine to medium grained.				
	DS	N = 11 4,4,7	14	3						210 260 310	
			13	4							
	DS	N = 20 4,7,13							VSt - Hd	260 420 500	
			12	5			SANDSTONE: fine to medium grained, light grey, with clay bands.	XW	EL	>600 >600 >600	VERY LOW 'TC' BIT RESISTANCE
							SANDSTONE: fine to medium grained, light grey, light brown and red brown.	DW	L - M		MODERATE RESISTANCE
			11	6			REFER TO CORED BOREHOLE LOG				

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
2
2 / 3

CORED BOREHOLE LOG

Client: KOGARAH 048 SERVICE PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 12-24 STANLEY STREET, KOGARAH

Job No.: 31240S | **Core Size:** HQ | **R.L. Surface:** ~17.2 m
Date: 14/2/18 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK305 | **Bearing:** N/A | **Logged/Checked By:** M.E/P.S

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	SPACING (mm)	DEFECT DETAILS DESCRIPTION: Type, orientation, defect roughness and shape, defect coatings and seams, openness and thickness (Specific / General)	Formation
		12			START CORING AT 5.33m						
100% RETURN; ON 28/2/18		11, 10, 9, 8, 7, 6	6, 7, 8, 9, 10, 11		SANDSTONE: fine to medium grained, light brown, light grey and red brown, bedded at 0 - 20.	DW	M - H			(5.51m) Be, 20°, P, R, Fe Ct (5.73m) Be, 20°, P, R, Fe Ct (5.85m) Be, 20°, P, R, Fe Ct (6.16m) Be, 0°, Un, R, Fe Ct (6.82m) CS, 15°, <2 mm.t (6.85m) Be, 10°, P, R, Fe Ct (7.02m) XWS, 0°, 15 mm.t (7.05m) Be, 15°, P, R, Fe Ct	
					SANDSTONE: fine to medium grained, light grey, with grey laminae, bedded at 0 - 20.	FR	H (from ~8.1m)			(7.56m) CS, 5°, 10 mm.t (7.64m) J, 25°, Un, R, Cn (7.97m) CS, 5°, 10 mm.t (8.43m) Be, 15°, St, R, Cn	
					As above but fine to coarse grained.					(10.00m) XWS, 5°, 3 mm.t (10.26m) XWS, 15°, 5 mm.t (11.28m) Be, 20°, St, S, Clay Ct (11.68m) XWS, 15°, 1 mm.t	

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No. 2

Client: KOGARAH 048 SERVICE PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 12-24 STANLEY STREET, KOGARAH

Job No.: 31240S **Core Size:** HQ **R.L. Surface:** ~17.2 m
Date: 14/2/18 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK305 **Bearing:** N/A **Logged/Checked By:** M.E/P.S

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS SPACING (mm)	DEFECT DETAILS DESCRIPTION Type, orientation, defect roughness and shape, defect coatings and seams, openness and thickness — Specific / General	Formation
100% RETURN		5 – 0	12.8 – 17.35		As above but fine to coarse grained. *(continued)*	FR	H			(13.82m) J, 65°, Qz Cn, Healed (14.88m) CS, 5°, Cn, <1 mm.t (17.06m) XWS, 0°, 120 mm.t	
			17.35		END OF BOREHOLE AT 17.35 m					CLASS 18 PVC STANDPIPE INSTALLED TO 17.35m DEPTH, SLOTTED 17.35m TO 14.35m, CASING 14.35m TO 0.1m, BOREHOLE BACKFILLED WITH 2mm SAND FILTER 17.35m TO 13.80m, BENTONITE SEAL 13.80m TO 3.5m, 2mm SAND 3.50m TO 1.0m, BENTONITE SEAL 1.0m TO 0.1m, COMPLETED WITH A CONCRETED CAST IRON GATIC COVER	

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

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JK Geotechnics
JOB No. 31240S BH2 START CORING AT 5·33
5
6
7
8
9
10
11
12
13
14
15
16
17
END OF BOREHOLE AT 17·35m

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
3

BOREHOLE LOG

Client: KOGARAH 048 SERVICE PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 12-24 STANLEY STREET, KOGARAH

Job No.: 31240S **Method:** SPIRAL AUGER **R.L. Surface:** ~17.2 m
Date: 15/2/18 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** M.E/P.S

Groundwater Record	Samples (ES / U50 / DB / DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING			17	0			FILL SILTY SANDY CLAY: low plasticity, dark grey, fine to medium grained sand, trace brick fragments anfd root fibres.	w<PL			GRASS COVER
	DS	N = 10 4,5,5				CL	SANDY CLAY: low plasticity, orange brown and grey, fine to medium grained sand, trace fine to medium grained ironstone gravel.	w>PL	St - VSt	210 180 270	RESIDUAL
			16	1			: as above but with fine to medium grained ironstone gravel				
	DS	N = 8 3,5,3								280 190 210	
			15	2			SANDY CLAY: low plasticity, light grey, brown and red brown, fine to medium grained sand.				
	DS	N=SPT 7/ 80mm REFUSAL	14	3			SANDSTONE: fine to medium grained, light grey, with clay bands.	XW	EL	520 >600 >600	VERY LOW 'TC' BIT RESISTANCE
							SANDSTONE: fine to medium grained, light grey and light brown, with iron indurated bands.	DW	L		LOW TO MODERATE RESISTANCE
			13	4			REFER TO CORED BOREHOLE LOG				
			12	5							
			11	6							

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JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
3

CORED BOREHOLE LOG

Client: KOGARAH 048 SERVICE PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 12-24 STANLEY STREET, KOGARAH

Job No.: 31240S | **Core Size:** HQ | **R.L. Surface:** ~17.2 m
Date: 15/2/18 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK305 | **Bearing:** N/A | **Logged/Checked By:** M.E/P.S

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS: SPACING (mm)	DEFECT DETAILS: DESCRIPTION (Type, orientation, defect roughness and shape, defect coatings and seams, openness and thickness) Specific	General	Formation
		14			START CORING AT 3.73m							
100% RETURN		13	4		SANDSTONE: fine to medium grained, light grey, with iron indurated bands, bedded at 0-25°.	DW	VL			(3.81m) CS, 5°, 5 mm.t (3.88m) J, 15°, P, R, Cn (3.97m) CS, 5°, 20 mm.t		
					NO CORE 0.25m							
		12	5		SANDSTONE: fine to medium grained, light grey and light brown, with iron indurated bands, bedded at 0-25°.	DW	M			(4.60m) CS, 0°, 40 mm.t (5.28m) Be, 15°, P, R, Clay, <1 mm.t (5.40m) XWS, 5 - 15°, 10 mm.t (5.53m) XWS, 10°, 5 mm.t (5.58m) XWS, 0°, 20 mm.t		
		11, 10	6, 7		SANDSTONE: fine to medium grained, light grey and light brown, bedded at 10-15°.	SW	M - H			(5.77m) Be, 20°, P, R, IS (5.94m) Be, 15°, P, R, Cn (5.98m) Be, 15°, P, R, Cn (6.30m) Be, 15°, P, S, CLAY, <1 mm.t (6.82m) Be, 10°, P, R, Cn (7.34m) Be, 5°, Un, R, IS (7.44m) Be, 5°, Un, R, IS, 15 mm.t (7.73m) Be, 5°, Un, R, IS		
		9	8		SANDSTONE: fine to coarse grained, light grey and grey, bedded at 0-25°.	FR	H			(8.19m) XWS, 0°, 20 mm.t		
ON 28/2/18		8	9		As above, but light grey and light brown.							

Point load strength index scale: VL -0.1, L -0.3, M -1, H -3, VH -10, EH

Spacing (mm) scale: 600, 200, 60, 20

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JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No. 3

Client: KOGARAH 048 SERVICE PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 12-24 STANLEY STREET, KOGARAH

Job No.: 31240S **Core Size:** HQ **R.L. Surface:** ~17.2 m
Date: 15/2/18 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK305 **Bearing:** N/A **Logged/Checked By:** M.E/P.S

Water Loss\Level	Depth (m)	Core Description (Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components)	Weathering	Strength	Defect Details – Specific
100% RETURN	10.0–10.25	SANDSTONE: fine to coarse grained, light grey and light brown, bedded at 0-25°.	FR	H	(10.19m) CS, 0°, 1 mm.t
	10.25–10.88	As above, but fine to medium grained.			(10.33m) CS, 5°, 1 mm.t; (10.88m) CS, 5°, 3 mm.t
	10.88–15.54	SANDSTONE: fine to coarse grained, light grey and light brown, bedded at 0-25°.			(11.90m) Be, 20°, P, S, CLAY, 2 mm.t; (13.22m) Be, 20°, P, S, CLAY, 1 mm.t; (14.40m) Be, 25°, P, S, CLAY, 1 mm.t; (14.78m) XWS, 15°, 10 mm.t
	15.54–	As above, but fine to coarse grained.			(15.94-16.00m) DRILLING INDUCED FRACTURES

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
3

CORED BOREHOLE LOG

Client: KOGARAH 048 SERVICE PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 12-24 STANLEY STREET, KOGARAH

Job No.: 31240S | **Core Size:** HQ | **R.L. Surface:** ~17.2 m
Date: 15/2/18 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK305 | **Bearing:** N/A | **Logged/Checked By:** M.E/P.S

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH)	SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS DESCRIPTION: Type, orientation, defect roughness and shape, defect coatings and seams, openness and thickness — Specific	General	Formation
100% RETURN		0				FR	H					
					SANDSTONE: fine to coarse grained, dark grey, with dark grey shale inclusions, bedded at 0-10°.		L			(17.22m) XWS, 20 - 25°, 80 mm.t (17.30m) XWS, 15°, 30 mm.t		
					END OF BOREHOLE AT 17.36 m							
		-1	18									
		-2	19									
		-3	20									
		-4	21									
		-5	22									
		-6	23									

JK 9.00.5 LIB.GLB Log JK CORED BOREHOLE - MASTER 31240S KOGARAH.GPJ <<DrawingFile>> 02/03/2018 10:11 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.00.5 2018-01-11 Prj: JK 9.00.5 2018-01-11

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JK Geotechnics
JOB No. 31240S BH3 START CORING AT 3·73m
3
4
CORE LOSS 0·25m
5
6
7
8
9
10
11
12
13
14
15
16
17
END OF BOREHOLE AT 17·36m

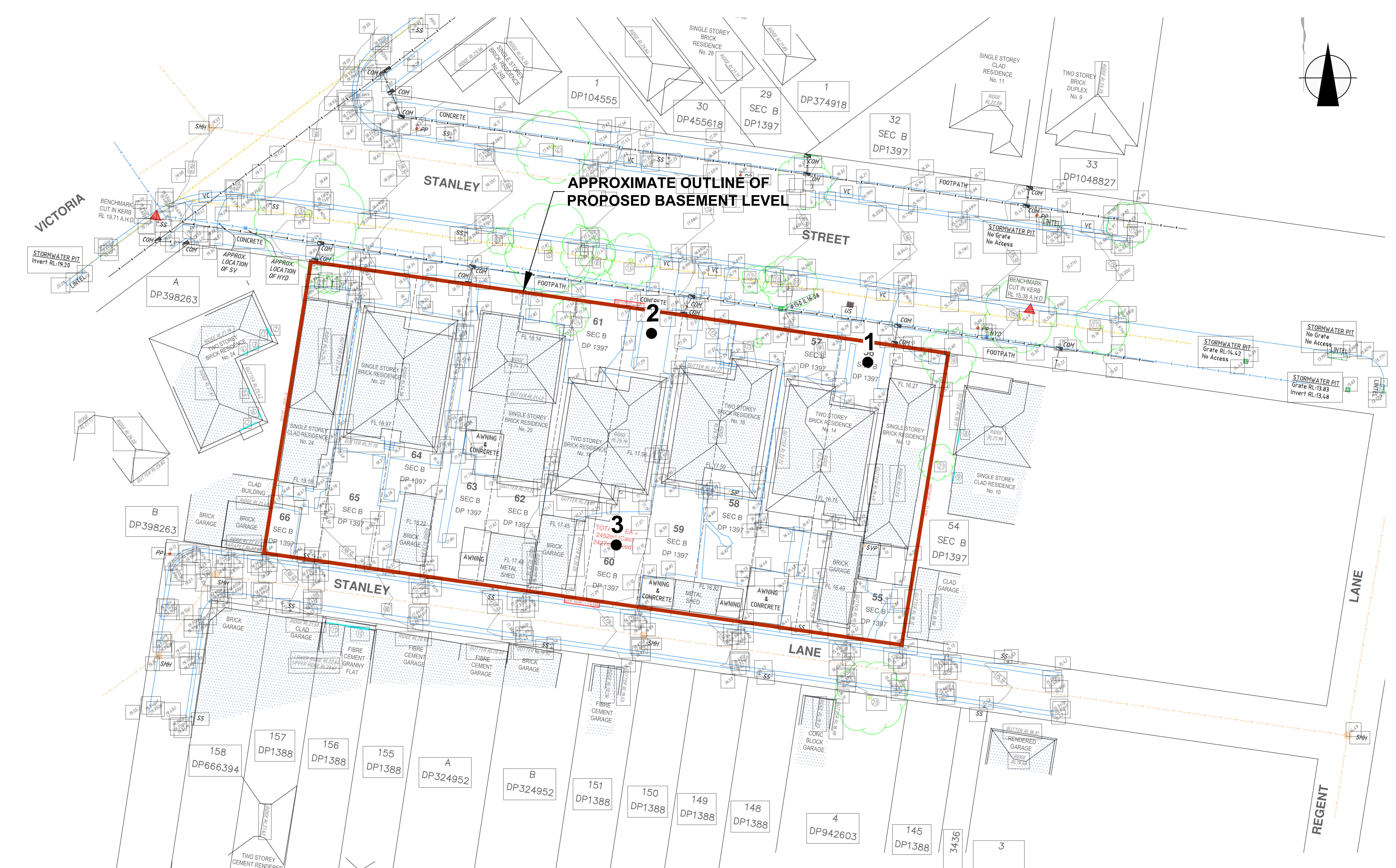
AERIAL IMAGE SOURCE: GOOGLE EARTH PRO 7.1.5.1557
AERIAL IMAGE ©: 2015 GOOGLE INC.

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

Title:	**SITE LOCATION PLAN**
Location:	14-24 STANLEY STREET KOGARAH, NSW
Report No: 31240S	Figure No: 1

JK **Geotechnics**

PLOT DATE: 28/02/2018 11:32:20 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31240S KOGARAH\CAD\31240S.DWG



SCALE 0 4 8 12 16 20 METRES 1:400 @A3	Title: **BOREHOLE LOCATION PLAN**	
	Location: 14-24 STANLEY STREET KOGARAH, NSW	
	Report No: 31240S	Figure No: 2
This plan should be read in conjunction with the JK Geotechnics report.	JK **Geotechnics**	

PLOT DATE: 28/02/2018 11:17:17 AM DWG FILE: S:\6 GEOTECHNICAL JOBS\31000'S\31240S KOGARAH\CAD\31240S.DWG

VIBRATION EMISSION DESIGN GOALS

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

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

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

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

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

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

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

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, 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 (eg. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.06mm
Sand	0.06 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 a Cone Penetrometer Test (CPT). The test is described in Australian Standard 1289, Test F5.1.

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.

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 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.

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

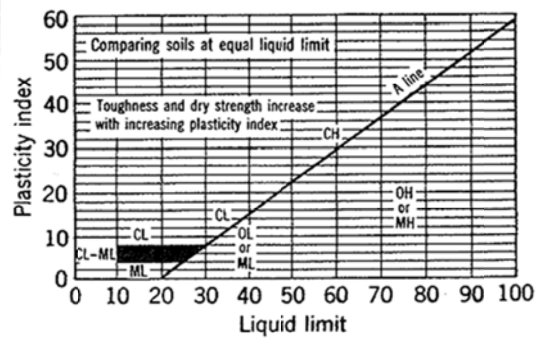
OTHER MATERIALS

- 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 For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics 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*)	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	$C_U = \frac{D_{60}}{D_{10}}$ 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			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			$C_U = \frac{D_{60}}{D_{10}}$ 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)	Group Symbols	Typical Names	Information Required for Describing Soils
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 For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions 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	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	
	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	
		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	



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 U50 DB DS ASB ASS SAL	Soil sample taken over depth indicated, for environmental analysis. Undisturbed 50mm diameter tube sample taken over depth indicated. Bulk disturbed sample taken over depth indicated. Small disturbed bag sample taken over depth indicated. Soil sample taken over depth indicated, for asbestos screening. Soil sample taken over depth indicated, for acid sulfate soil analysis. 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 PID = 100	Vane shear reading in kPa of Undrained Shear Strength. Photoionisation detector reading in ppm (Soil sample headspace test).
Moisture Condition (Cohesive Soils) (Cohesionless Soils)	MC>PL MC≈PL MC<PL D M W	Moisture content estimated to be greater than plastic limit. Moisture content estimated to be approximately equal to plastic limit. Moisture content estimated to be less than plastic limit. DRY – Runs freely through fingers. MOIST – Does not run freely but no free water visible on soil surface. WET – Free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS S F St VSt H ( )	VERY SOFT – Unconfined compressive strength less than 25kPa SOFT – Unconfined compressive strength 25-50kPa FIRM – Unconfined compressive strength 50-100kPa STIFF – Unconfined compressive strength 100-200kPa VERY STIFF – Unconfined compressive strength 200-400kPa 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)	 VL L MD D VD ( )	**Density Index (I_D) Range (%)** **SPT 'N' Value Range (Blows/300mm)** Very Loose <15 0-4 Loose 15-35 4-10 Medium Dense 35-65 10-30 Dense 65-85 30-50 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 'TC' bit T_{60}	Hardened steel 'V' shaped bit. Tungsten carbide wing bit. Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.

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	