REPORT

TO

MARCHESE PARTNERS

ON

PRELIMINARY GEOTECHNICAL INVESTIGATION

FOR

PROPOSED SENIORS LIVING DEVELOPMENT

AT

3 QUARRY ROAD, DURAL, NSW

18 June 2018

Ref: 31137Srpt_Rev1

dlcs 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 a proposed seniors living development at 3 Quarry Road, Dural, NSW. The investigation was commissioned by Mr Enrique Blanco De Cordova of Marchese Partners International Pty Ltd and was completed in accordance with our proposal (Ref: P46131S) dated 10 November 2017.

We have been provided with preliminary architectural drawings prepared by Marchese Partners International Pty Ltd (Job No: 16033, Drawing Nos. DA2.01 to DA.28 inclusive) all dated 7 June 2018, Revision G. From these drawings, we understand it is proposed to construct eight 3 level buildings (known as Buildings A to G & RAC), a single level Wellness Centre building and a single level facilities building. Buildings A, D, F and G will have single level basements, while Buildings B, C, E and RAC will have split one to two level basements. We understand the basements will have Finished Floor Levels (FFL) between RL199.85m and RL194.35 and will require excavation to depths between about 2.6m and 7.6m, below existing surface levels.

Along the western and southern boundaries, the basements will be setback about 10.0m and 14.1m, respectively; with the basements of Building D, E and F setback at least 16.3m from the eastern boundary with No.5 Quarry Road. The RAC building basement will be setback about 1.2m from the northern boundary with Vineys Road and 3.0m from the eastern boundary with No.6 Vineys Road; with the basement of Building C setback about 0.8m to 2.1m from the northern boundary with Vineys Road. The Building G basement will be setback about 10.0m from the northern boundary, with No 5 Vineys Road.

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

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 14 December 2017, and included the auger drilling of eight boreholes (BH1 to BH8) to refusal depths between 0.8m and 3.0m. The boreholes were drilled using our track mounted JK300 drill rig.

The borehole locations, as shown on Figure 2, were set out using taped measurements from existing surface features and were electromagnetically scanned for buried services prior to drilling commencing. The approximate surface levels, as shown on the borehole logs, were estimated by interpolation between spot levels shown on the supplied survey plan prepared by Marchese Partners International Pty Ltd (Job No. 16033, Drawing No. DA1.03, dated 29/5/2018, revision F) and should be considered approximate. We have assumed the datum of the levels is Australian Height Datum (AHD).

The nature and composition of the subsurface profile was assessed by logging the materials recovered during drilling. The relative compaction/strength of the subsoils were assessed from the Standard Penetration Test (SPT) 'N' number, augmented by hand penetrometer readings on the on cohesive samples recovered in the SPT split tube sampler. The strength of the sandstone bedrock was assessed by observation of the auger penetration resistance using a tungsten carbide 'TC' drill bit, together with examination of the recovered rock cuttings and from correlations with subsequent moisture content test results on recovered rock chips. It should be noted that strengths assessed in this way are approximate and variances of one strength order should not be unexpected.

Groundwater observations were made during and on completion of auger drilling. No longer term groundwater monitoring has been carried out.

Our geotechnical engineer, Mr Michael Serra, set out the borehole locations, nominated sampling and testing locations and prepared logs of the strata encountered. The borehole logs are attached, together with a set of explanatory notes, which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

Selected samples were returned to Soil Test Services Pty Ltd (STS), a NATA accredited laboratories, for testing to determine moisture contents, liquid limit and linear shrinkage values. The results of the laboratory testing are summarised in STS Table A.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located in undulating topography which slopes down to the north-east at between 2° to 3°, towards Tunks Creek. The site is cut by a broad gully, with the southern portion of the site sloping down at between 3° to 4° to the north-east and the northern portion sloping down at between 4° to 6° to the south. Near the centre of the site are the headwaters of a meandering creek, which flows towards the north-east. The site is bound by Vineys Road and Quarry Road to the north and south, respectively.

The site contains a single level fibro-clad house and a single level fibro-clad cabin fronting Vineys Road and Quarry Road, respectively. The remainder of the site is grassed. There is a segmental block retaining wall up to about 1.5m in height located near the eastern corner boundary with No.6 Vineys Road.

It appears the southern portion of the site was heavily vegetated and has been recently cleared of many large trees, with the southern boundary of the site lined with trees up to about 15m in height. Various outcrops of sandstone bedrock are exposed around the site. Based on inspection, the sandstone was assessed to be slightly weathered and of high strength.

Neighbouring the site to the south-west and south-east, along its frontage with Quarry Road, are No 1 and No 5 Quarry Road, respectively. No 1 contains a 2 level brick house with an inground pool to its rear; the house appears in good external condition and is set back 8m from the common boundary. No 5 is currently occupied by The Green Gallery Nursery which contains several green-houses and above-ground water tanks.

Neighbouring the site, to the north-west and north-east, along its frontage with Vineys Road, are No 2 and No 6 Vineys Road, respectively. No 2 contains a 2 level brick house with a dam in its rear (southern) yard. The dam is located near the central portion of the subject site and in discussions with the current occupant of the site, there is sub-surface drainage which traverses the subject site from the dam to the inlet into the meandering creek. No 6 Vineys Road contains a single level brick homestead with an in-ground pool to its rear. The houses at No 2 and No 6 both appear in good external condition and are setback at least 12m and 6m from their common boundaries with the subject site, respectively.

3.2 Subsurface Conditions

Reference to the Sydney 1:100,000 Geological Series Sheet indicates that the site is underlain by Ashfield Shale of the Wianamatta Group, but close to the contact with the underlying Hawkesbury Sandstone. The investigation has revealed a generalised subsurface profile comprising fill over residual clays with sandstone bedrock at shallow depths. Some of the characteristic features of the substrata encountered are described below. For further details of the conditions encountered at each location, reference should be made to the attached borehole logs

- Fill comprising silty sand was encountered in all boreholes to depths between 0.3 and 0.5m.
- Residual silty clay was encountered below the fill BH1, BH2 and BH4. In BH1 and BH4 the clay was of hard strength, while in BH2 the clay was of stiff to hard strength..The clay was assessed to be of medium plasticity.
- In BH7, a thin layer of residual clayey sand was encountered beneath the fill.
- Sandstone bedrock was encountered in all boreholes at depths between 0.3m (BH6) and 2m (BH2). On first contact the bedrock ranged from extremely low to high strength, quickly becoming high strength in all boreholes with auger refusal occurring at depths between 0.8m and 3.0m.
- Groundwater was not encountered during or on completion of drilling. No long term ground monitoring was undertaken.

3.3 Laboratory Test Results

Based on the Liquid Limit and Linear Shrinkage test results, the residual silty clay tested is of medium plasticity and is assessed to have a moderate potential for shrink/swell movements with changes in moisture content. The results of moisture content tests on selected samples of the bedrock correlate reasonably well with the field strength assessments.

4 COMMENTS AND RECOMMENDATIONS

4.1 Excavation

All excavation recommendations should be complemented by reference to Safe Work Australia's 'Excavation Work Code of Practice', dated July 2015 and AS3798 'Guidelines on Earthworks for Commercial and Residential Developments'.

4.1.1 Dilapidation Surveys

Prior to the commencement of demolition and excavation, we recommend that dilapidation surveys be completed on the neighbouring buildings to the east and west of the site that lie within 30m of the proposed excavation.

The dilapidation surveys should include detailed internal and external inspections of the neighbouring buildings, where all defects including defect location, type, length and width are rigorously described and photographed.

The respective owners should be asked to confirm that the dilapidation survey reports present a fair record of existing conditions. The dilapidation survey reports may be used as a benchmark against which to assess possible future damage claims. We could prepare a proposal to carry out the dilapidation surveys, if requested.

4.1.2 Excavation Techniques

Prior to the commencement of excavation, demolition of the existing houses, as well as the removal of any vegetation within the development footprint, will be required. Any deleterious or contaminated fill should also be stripped and disposed appropriately off-site.

Based on the investigation results, excavation to a maximum depth of about 7.6m will extend through the soil profile, but mostly into sandstone bedrock, initially of variable, but predominantly high strength. There may of course be variation in rock strength at depth and features such as weathered shale bands may be present; such features can be identified with more certainty following detailed investigation which should include a series of diamond cored boreholes throughout the site.

The soil cover should be readily excavatable using conventional earthworks equipment (eg. hydraulic excavators or small dozers). Some of the underlying weathered bedrock of extremely or very low strength, may also be excavated by a large bucket excavator (possibly with some ripping). However, the rock is predominantly of high strength and presents 'hard' rock excavation conditions and will require the use of hydraulic impact hammers for the majority of the work. Given the size of the site the use of a heavy tractor (D11 or equivalent) for ripping will probably be preferred for the bulk excavation, though productivity will be low in the high strength sandstone.

During the use of hydraulic impact hammers, precautions must be made to reduce the risk of vibrational damage to adjoining structures. At the commencement of the use of hydraulic impact hammers we recommend that some quantitative vibration monitoring be carried out on the adjoining residences by an experienced vibration consultant or geotechnical engineer to check that vibrations are within acceptable limits. The attached vibration emission guidelines provide some advice on acceptable vibrations in this regard.

If during excavation with the hydraulic impact hammers, vibrations are found to be excessive or there is concern, then alternative lower vibration emitting equipment, such as rock saws, rock grinders or smaller hammers may need to be used. The use of a rotary grinder or rock sawing in conjunction with excavator ripping presents an alternative low vibration excavation technique, however, productivity is likely to be slower. When using a rock saw or rotary grinder, the resulting dust must be suppressed by spraying with water.

We recommend that only excavation contractors with appropriate insurances and experience on similar projects be used. Excavation contractors should be provided with a copy of this geotechnical report, including the borehole logs and point load strength test results, so that they can make their own assessment of suitable excavation equipment.

The excavated material will also need to be classified for disposal purposes, which will require environmental testing of the various materials.

4.1.3 Seepage

Groundwater inflows into the excavation may occur as local seepage flows within the fill/residual soil interface, at the soil/rock interface, and through joints and bedding partings within the bedrock profile, particularly after heavy rain.

Seepage volumes into the excavation, if any, are expected to be controllable by gravity or conventional sump and pump methods. Notwithstanding, groundwater seepage monitoring should be carried out by site staff during excavation so that any unexpected conditions can be addressed.

A toe drain should be provided at the base of all rock cuttings to collect groundwater seepage and lead it to a sump for pumping to the stormwater system.

4.2 Excavation Retention

4.2.1 Battered Slopes

Where battered slopes can be accommodated within the site boundaries, we consider that the upper soils and rock will be able to be temporarily battered and the following excavation recommendations are appropriate;

- Temporary batter slopes no steeper than 1 Vertical (V) in 1.5 Horizontal (H) through the fill and natural soils.
- Temporary batter slopes no steeper than 1V in 1H through extremely weathered bedrock or any bedrock of less than low strength.
- Temporary batter slopes through sandstone bedrock of at least low strength may be cut vertically subject to inspections by the geotechnical engineers at not greater than 1.5m

depth intervals. Where adverse defects are encountered then these will need to be stabilised such as with rock bolts. It is likely that some staging of the excavation will be recommended.

- We recommend all surcharge loads (such as construction traffic etc) are kept well clear of the crest of the temporary batter slope (at least twice the height of the batter slope from the crest), unless geotechnical assessment and/or stabilisation of excavation faces is carried out and the geotechnical engineers confirm that surcharge loads can be placed closer than the above recommendations.

Consideration of the excavation size will need to be taken into account regarding the requirements for stabilisation of vertical rock cuts. Stabilisation may include rock bolts, shotcrete and mesh. Stabilisation works may need to be permanent if basement walls are not designed as permanently supporting structures.

Permanent batters through soils or sandstone bedrock of less than medium strength should be no steeper than 1V:2H unless supported by shotcrete, mesh and dowels. Permanent batters through the sandstone bedrock of medium strength or better may be cut vertically but localised stabilisation measures may be necessary and we recommend that the rock face be progressively inspected by a geotechnical engineer at no more than 1.5m depth intervals, to identify adverse defects and to proposed appropriate stabilisation measures.

Along the northern boundary, we recommend that a series of additional boreholes be carried out to determine the feasibility of battered slopes. Where battered slopes cannot be accommodated than the excavation will need to be supported by an engineered shoring system.

4.2.2 Excavation Support

If for any reason battered slopes are not considered feasible, (i.e. potentially along the northern boundary), then the excavation will need to be supported by an engineered shoring system installed prior to excavation commencing. A shoring system comprising anchored soldier pile walls with shotcrete infill panels would be suitable. The soldier piles must have sufficient embedment below bulk excavation level to ensure lateral toe restraint or a second row of rock anchors will be required, the latter option being the more likely.

Where temporary stabilisation of rock faces by means of rock bolts occurs, the rock bolts must be replaced as construction progresses by permanent support such as retaining walls braced off adjacent floor slabs. Provision in the design and construct documents for such support is essential.

We note that relatively high strength sandstone bedrock is anticipated and the potential piling contractors must ensure that pile boring rigs capable of drilling into this material be used, where appropriate.

Where sandstone of low strength has been cut vertically during construction it will require protection in the long term to avoid fretting and erosion which will block drains and ultimately could lead to instability. Where space permits shotcrete secured by rock bolts may be appropriate, but if rock bolts would extend across site boundaries then propping from the permanent structure would be necessary. Such areas mush be identified and treated in the course of excavation.

4.3 Earth Pressures

The major consideration in the selection of earth pressures for the design of the retaining walls is the need to limit deformations occurring outside the excavations. The following characteristic earth pressure coefficients and subsoil parameters may be adopted for the static design of temporary or permanent retaining walls/retention systems:

- Cantilever walls which will be restrained by the proposed floor slabs or which support movement sensitive elements, should be designed using a triangular lateral earth pressure distribution and an 'at rest' earth pressure coefficient, Ko, of 0.55, for the soil profile and any very low strength bedrock, assuming a horizontal retained surface.
- For anchored or internally propped walls, where there are no structures in the zone of influence of the excavation, we recommend the use of a trapezoidal earth pressure distribution of 4H kPa for the soil profile and extremely to very low strength bedrock, where 'H' is the retained height in metres. These pressures should be assumed to be uniform over the central 50% of the support system. Where structures are present in the zone of influence a higher pressure of 6H kPa should be adopted.
- For bedrock having quality better than very low strength, a uniform rectangular lateral load of 5kPa could be provisionally adopted (subject to inspection during construction).
- A bulk unit weight of 20kN/m3 should be adopted for the soil profile and 24kN/m3 for bedrock.
- Any surcharge affecting the walls (eg. traffic loading, construction loads, adjacent high level footings, etc) should be allowed in the design using the at rest earth pressure coefficient from above.
- The retaining walls should be designed as drained and measures taken to provide permanent and effective drainage of the ground behind the walls. The soldier pile walls

should incorporate intermediate strip drains which are wrapped in a non-woven geotextile fabric (eg. Bidim A34) to act as a filter against subsoil erosion. The strip drains should discharge from the base of the walls.

- Where rock anchors extend beyond the site boundaries the permission from the neighbours should be obtained before installation. Rock anchors should have a free length of at least 3m and be bonded into medium strength or better rock with an allowable bond stress of 300kPa applicable. However, anchors should be a design and construct sub-contract to avoid contractual disputes in the event that any anchors fail test loading. All anchors should be proof-tested to 1.3 times the working load under the direction of an experienced engineer, independent of the anchor contractor. Lift-off tests should be carried out on at least 10% of anchors after 24 - 48 hours of initial stressing.
- The lateral resistance of pile toes embedded into the sandstone may be taken as 300kPa for low strength or better rock. The first 0.3m of any socket should be ignored to allow for overbreak. Socket lengths must also be below the zone of influence of local excavations for services, footings etc.
- We assume that permanent lateral support of the top of the retaining walls will be provided by the new structure.

4.6 Footings

The bulk excavation will expose sandstone bedrock and pad or strip footings may be used. Where buildings extend beyond the basement perimeters it may be necessary to use short bored piers to economically transfer loads to the bedrock.

The footings/piers may be designed for an allowable bearing pressure (ABP) of up to 1,000kPa, subject to inspection by a geotechnical engineer. Higher allowable bearing pressures of 3,500kPa or more are probably feasible, however, further boreholes including diamond coring of the bedrock and strength testing the recovered rock core are required in order to confirm the founding conditions.

4.7 Pavements and Subgrade Preparation

For any on-grade pavements, the subgrade should be stripped of all vegetation, root affects soils, deleterious fill or other deleterious materials to expose the residual clay.

The exposed clay subgrade should then be proof rolled with at least 8 passes of a minimum 7 tonne dead weight, smooth drum, vibratory roller. The final pass of the proof rolling should be carried out without vibration and within the presence of a geotechnical engineer to detect any weak subgrade areas. The use of vibration may have to be curtailed or eliminated to avoid damage to nearby structures.

Any weak areas detected during proof rolling or where the clay subgrade is exposed to periods of rainfall and 'softening'; the subgrade should be locally excavated to a sound base and the excavated material replaced with engineered fill, or as directed by the geotechnical engineer during the proof rolling inspection.

Where weathered sandstone is exposed at the subgrade level no subgrade improvement works would be required, though a separation layer of roadbase or drainage gravel would be required.

We recommend that preliminary design of pavements on fill or clay soil be based on a design CBR of 2%, or an estimated modulus of subgrade reaction of 20kPa/mm (750mm plate) for the clay subgrade. This design CBR value should be confirmed by laboratory testing of samples of the subgrade soils if pavements are to support more than light vehicular loads.

Concrete pavements should have a sub-base layer of at least 100mm thickness of crushed rock to RTA QA specification 3051 (1994) unbound base material (or equivalent good quality and durable fine crushed rock), which is compacted to at least 100% of SMDD. Concrete pavements should be designed with an effective shear transmission at all joints by way of either doweled or keyed joints.

4.8 Engineered Fill

Engineered fill must be used where excavated material is to be replaced or where ground surface levels are to be raised.

Engineered fill should preferably comprise well graded granular materials, such as ripped or crushed sandstone, free of deleterious substances and having a maximum particle size not exceeding 75mm. Such fill should be compacted in horizontal layers of not greater than 200mm loose thickness, to a density of at least 98% of Standard Maximum Dry Density (SMDD).

The excavated clay and weathered sandstone may be used as engineered fill provided it is free of deleterious materials and particles greater than 75mm in size. Any clay fill should be compacted in 200mm loose thickness layers to a density strictly between 98% and 102% of SMDD and at moisture contents within 2% of Standard Optimum Moisture Content (SOMC).

Where space permits, we recommend that engineered fill extend a horizontal distance of at least 1m beyond the design fill embankment slope, so that adequate edge compaction can be achieved. On completion of filling any excess fill can be trimmed off.

Backfill behind retaining walls and for service trenches should also comprise engineered fill. Due to limited access for machinery, compaction of backfill immediately behind retaining walls and in service trenches may need to be completed using smaller compaction equipment (e.g. upright rammer compactors, sled compactors or small rollers). Due to the reduced energy output of such equipment, fill in such areas must be placed in maximum 100mm loose thickness layers, and have a maximum particle size not exceeding 40mm.

Density tests should be regularly carried out on the engineered fill to confirm the above specifications are achieved. All density testing must be completed over the full thickness of each compacted fill layer.

The frequency of density testing for engineered fill should be at least one test per layer per $500m^2$ or one test per $100m^3$ distributed reasonably evenly throughout the full depth and area, whichever requires the most tests. The frequency of density testing for engineered backfill behind retaining walls and trenches should be at least one test per two layers per 50 linear meters.

Compaction of engineered fill behind free standing retaining walls can be problematic and the use of a single sized durable gravel, such as “blue metal” or crushed concrete gravel (free of fines), which do not require significant compactive effort could be considered if good performance is a priority. Such material should be nominally compacted using a hand operated vibrating plate (sled). Free draining backfill material must be separated from the in-situ soils or general embankment fill by a non-woven geotextile fabric (e.g. Bidim A34), to act as a filter against subsoil erosion. Provided the gravel backfill is placed as recommended above, density testing of the gravel backfill would not be required. The geotextile should then be wrapped over the surface of the gravel backfill and capped with at least a 0.3m thick compacted layer of engineered fill to reduce infiltration of surface water.

4.9 Basement Floor Slab

Based on the investigation results, the proposed basement floor slabs will directly overlie the sandstone bedrock. We therefore recommend that underfloor drainage be provided. The underfloor drainage should comprise a strong, durable, single-sized washed aggregate such as 'blue metal' gravel. The underfloor drainage should connect with the perimeter drains and lead groundwater seepage to a sump for pumped disposal to the stormwater system.

Joints in the basement concrete on-grade floor slabs should be designed to accommodate shear forces but not bending moments by using dowelled or keyed joints.

4.10 Further Geotechnical Input

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

- Detailed investigation including cored boreholes.
- Dilapidation surveys of neighbouring buildings.
- Quantitative vibration monitoring during rock hammer operation.
- Monitoring of groundwater seepage into bulk excavation.
- Density testing of engineered fill, base and sub-base materials.
- Inspection of the subgrade proof rolling.
- Inspection of the initial pile drilling and footing excavations.
- Proof-testing of anchors.
- Inspection of rock faces at intervals not exceeding 1.5m.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. 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.

The long term successful performance of floor slabs and pavements may be dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance program should not be limited to routine compaction density testing only. Other critical factors associated with the earthworks may include subgrade preparation, selection of fill materials, control of moisture content and drainage, etc. The satisfactory control and assessment of these items may require judgment from an experienced engineer. Such judgment often cannot be made by a technician who may not have formal engineering qualifications and experience. In order to identify potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the earthworks requirements and potential difficulties. This meeting should clearly define the lines of communication and responsibility.

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

ABN 43 002 145 173

TABLE A
MOISTURE CONTENT, LIQUID LIMITS AND LINEAR SHRINKAGE TEST REPORT

Client: JK Geotechnics
Project: Proposed Seniors Living Development
Location: 3 Quarry Road, Dural, NSW

Ref No: 31137S
Report: A
Report Date: 20/12/2017

AS 1289	TEST METHOD	2.1.1	3.1.2	3.4.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**LINEAR SHRINKAGE %**
1	2.00-2.50	5.5		
2	0.50-0.95	9.9	39	9.0
2	2.40-2.70	3.5		
3	1.30-1.80	5.0		
3	2.50-2.80	4.0		
4	1.40-1.70	4.5		
4	2.00-2.50	3.8		
5	1.00-1.50	5.8		
6	0.30-0.80	4.0		
7	1.30-1.50	5.2		
8	1.30-1.70	5.4		

Notes:
- The test sample for liquid limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 15/12/2017

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BOREHOLE LOG

Borehole No.
1
1/1

Client: MARCHESE PARTNERS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 3 QUARRY ROAD, DURAL, NSW

Job No. 31137S
Date: 14/12/17
Method: SPIRAL AUGER JK300
Logged/Checked by: M.S./P.S.
R.L. Surface: ≈ 207.0m
Datum: AHD ASSUMED

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
DRY ON COMPL -ETION			0			FILL: Silty sand, fine to medium grained, dark brown, trace of root fibres.				GRASS COVER
		N = 10 3,4,6			CL	SILTY CLAY: medium plasticity, light grey mottled red and yellow brown, with fine to medium grained sand.	MC<PL	H	500 >600 >600	RESIDUAL
		N > 4 10,4/10mm	1		-	SANDSTONE: fine to medium grained, light grey mottled red and yellow brown.	XW	EL		BANDED VERY LOW 'TC' BIT RESISTANCE
							DW	L		LOW RESISTANCE
			2				SW	H		HIGH RESISTANCE
			3			END OF BOREHOLE AT 2.9m				'TC' BIT REFUSAL
			4							
			5							
			6							
			7							

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BOREHOLE LOG

Borehole No. **2**
1/1

Client: MARCHESE PARTNERS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 3 QUARRY ROAD, DURAL, NSW

Job No. 31137S
Date: 14/12/17
Method: SPIRAL AUGER JK300
Logged/Checked by: M.S./P.S.
R.L. Surface: ≈ 199.7m
Datum: AHD ASSUMED

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
DRY ON COMPL -ETION	ES		0			FILL/TOPSOIL: Silty sand, fine to medium grained, dark brown, trace of clay.				GRASS COVER
	ES, DS	N = 6 2.2.4			CL	SILTY CLAY: medium plasticity, yellow brown and light grey mottled red brown.	MC<PL	St-VSt	180 190 210	RESIDUAL
			1							
	DS	N = 14 12,7,7			CL	SANDY CLAY: medium plasticity, light grey and red brown, with bands of XW sandstone.		H	>600 >600	
	DS		2		-	SANDSTONE: fine to medium grained, light grey and red brown.	DW	L H		LOW 'TC' BIT RESISTANCE HIGH RESISTANCE
			3			END OF BOREHOLE AT 2.7m				'TC' BIT RESISTANCE
			4							
			5							
			6							
			7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
3
1/1

Client: MARCHESE PARTNERS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 3 QUARRY ROAD, DURAL, NSW

Job No. 31137S
Date: 14/12/17
Method: SPIRAL AUGER JK300
Logged/Checked by: M.S./P.S.
R.L. Surface: ≈ 205.1m
Datum: AHD ASSUMED

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
DRY ON COMPL -ETION			0			FILL: Silty sand, fine to medium grained, grey brown, trace of fine to medium grained ironstone gravel.	M			GRASS COVER
		N = SPT 4/50mm			-	SANDSTONE: fine to medium grained, light grey mottled red and yellow brown.	DW	VL-L		BANDED LOW 'TC' BIT RESISTANCE
			1					M		MODERATE RESISTANCE
						as above, but light grey and brown.		L		LOW RESISTANCE
			2			as above, but light grey mottled red brown.	SW	H		HIGH RESISTANCE
			3			END OF BOREHOLE AT 3.0m				'TC' BIT REFUSAL
			4							
			5							
			6							
			7							

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
4
1/1

Client: MARCHESE PARTNERS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 3 QUARRY ROAD, DURAL, NSW

Job No. 31137S
Date: 14/12/17
Method: SPIRAL AUGER JK300
Logged/Checked by: M.S./P.S.
R.L. Surface: ≈ 204.7m
Datum: AHD ASSUMED

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
DRY ON COMPL -ETION			0			FILL: Silty sand, fine to medium grained, brown.	M			
		N > 27 15,12/ 100mm			CL	SILTY CLAY: medium plasticity, red brown mottled light grey, with high strength iron indurated bands.	MC<PL	H	>600 >600	RESIDUAL
			1		-	SANDSTONE: fine to medium grained, red brown mottled light grey, with high strength iron indurated bands.	DW	L		LOW 'TC' BIT RESISTANCE
								M		MODERATE RESISTANCE WITH HIGH STRENGTH BAND
			2			as above, but light grey mottled red brown.	SW	H		HIGH RESISTANCE
			3			END OF BOREHOLE AT 2.7m				'TC' BIT REFUSAL
			4							
			5							
			6							
			7							

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
5
1/1

Client: MARCHESE PARTNERS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 3 QUARRY ROAD, DURAL, NSW

Job No. 31137S
Date: 14/12/17
Method: SPIRAL AUGER JK300
Logged/Checked by: M.S./P.S.
R.L. Surface: ≈ 201.5m
Datum: AHD ASSUMED

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
DRY ON COMPL -ETION			0			FILL: Sandy silt, low to medium plasticity, dark brown.	MC>PL			GRASS COVER AGRICULTURAL ODOUR
		N = SPT 5/20mm			-	SANDSTONE: fine to medium grained, light grey and red brown, with XW bands.	DW	VL-L		BANDED VERY LOW TO LOW 'TC' BIT RESISTANCE
			1			As above, but light grey mottled red brown.		L		LOW RESISTANCE
			2			as above, but light grey.	SW	M-H		MODERATE TO HIGH RESISTANCE
								H		HIGH RESISTANCE
			3			END OF BOREHOLE AT 2.4m				'TC' BIT REFUSAL
			4							
			5							
			6							
			7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
6
1/1

Client: MARCHESE PARTNERS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 3 QUARRY ROAD, DURAL, NSW

Job No. 31137S
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 199.8m
Date: 14/12/17
Datum: AHD ASSUMED
Logged/Checked by: M.S./P.S.

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
DRY ON COMPL -ETION	■					0			FILL: Silty sand, fine to medium grained, brown.				GRASS COVER
				■				-	SANDSTONE: fine to medium grained, light grey and red brown.	DW	M-H		MODERATE TO HIGH 'TC' BIT RESISTANCE
						1			END OF BOREHOLE AT 0.8m				'TC' BIT REFUSAL
						2							
						3							
						4							
						5							
						6							
						7							

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
7
1/1

Client: MARCHESE PARTNERS

Project: PROPOSED SENIORS LIVING DEVELOPMENT

Location: 3 QUARRY ROAD, DURAL, NSW

Job No. 31137S

Date: 14/12/17

Method: SPIRAL AUGER JK300

Logged/Checked by: M.S./P.S.

R.L. Surface: ≈ 197.4m

Datum: AHD ASSUMED

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
DRY ON COMPL -ETION	ES		0			FILL: Silty sand, fine to medium grained, brown.				GRASS COVER
					SC	CLAYEY SAND: fine to medium grained, yellow and red brown.	M			RESIDUAL
	DS	N = SPT 9/100mm			-	SANDSTONE: fine to medium grained, light grey and yellow brown.	XW	EL		
			1			as above, but light grey mottled red and yellow brown.	DW	L		LOW 'TC' BIT RESISTANCE
	DS							M-H		MODERATE TO HIGH RESISTANCE
								H		HIGH RESISTANCE
			2			END OF BOREHOLE AT 2.0m				'TC' BIT REFUSAL
			3							
			4							
			5							
			6							
			7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

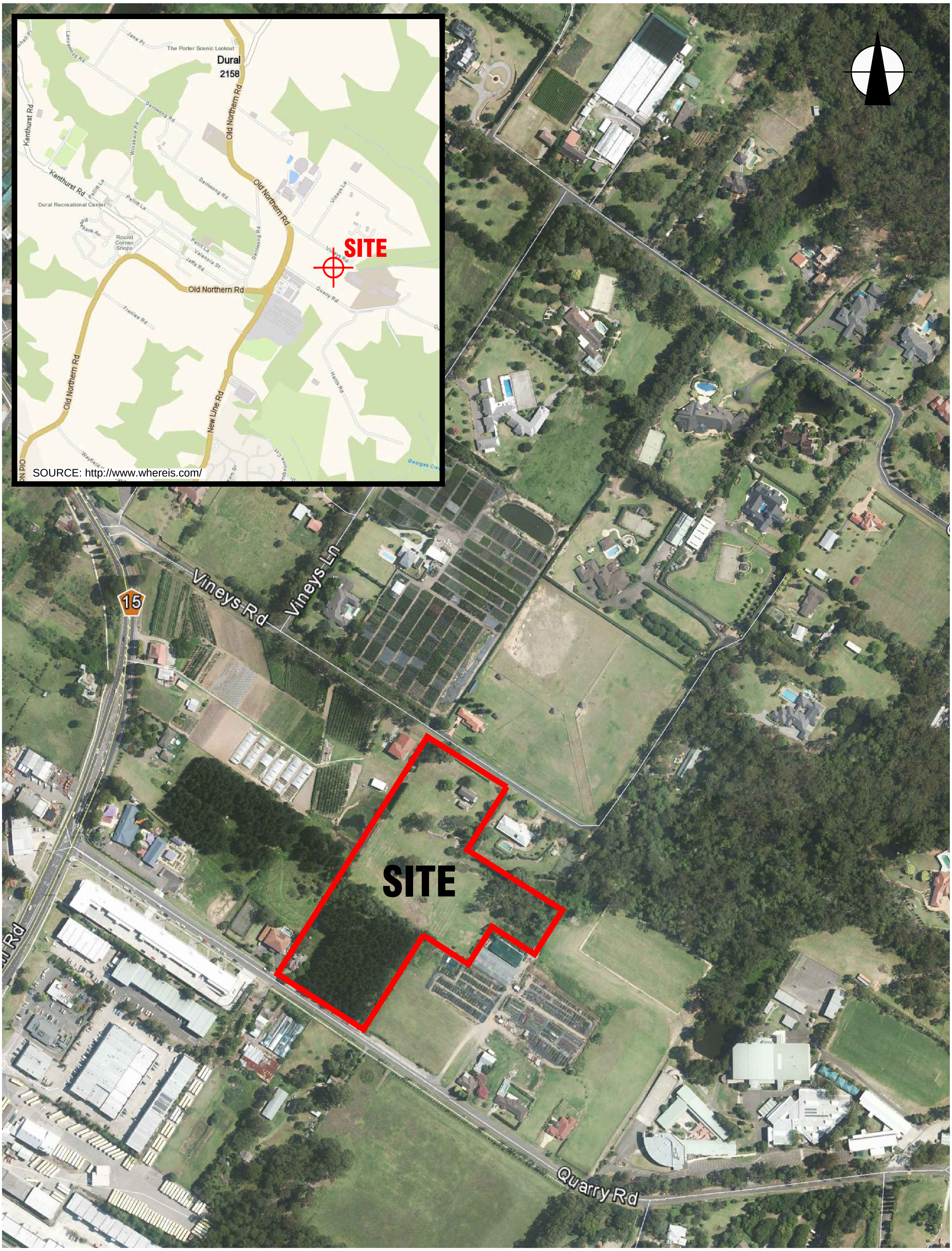
BOREHOLE LOG

Borehole No.
8
1/1

Client: MARCHESE PARTNERS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 3 QUARRY ROAD, DURAL, NSW

Job No. 31137S
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 197.5m
Date: 14/12/17
Datum: AHD ASSUMED
Logged/Checked by: M.S./P.S.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPL -ETION	■					0			FILL: Silty sand, fine to medium grained, brown.	M			GRASS COVER
								-	SANDSTONE: yellow brown mottled light grey, with XW bands.	DW	VL		BANDED VERY LOW 'TC' BIT RESISTANCE
						1					L		LOW RESISTANCE
				■					as above, but light grey mottled red and yellow brown.	SW	H		HIGH RESISTANCE
						2							
									END OF BOREHOLE AT 2.1m				'TC' BIT REFUSAL
						3							
						4							
						5							
						6							
						7							

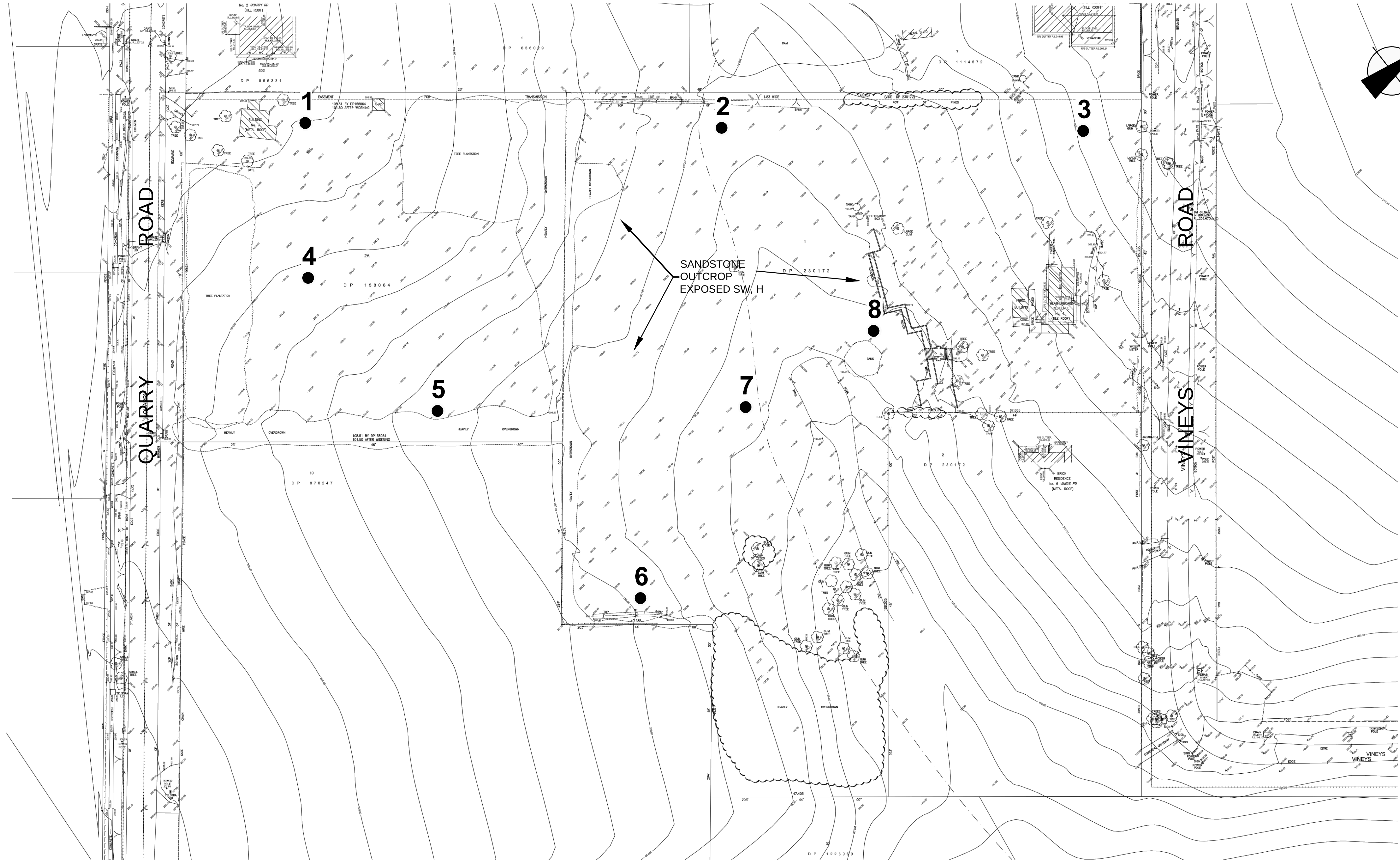


AERIAL IMAGE SOURCE: GOOGLE EARTH PRO 7.1.5.1557

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

Title:	SITE LOCATION PLAN
Location:	3 QUARRY ROAD DURAL, NSW
Report No: 31137S	Figure No: 1
	JK Geotechnics

PLOT DATE: 21/12/2017 3:11:18 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31137S DURAL\CAD\31137S.DWG



0 10 20 30 40 50

SCALE 1:1000 @A3 METRES

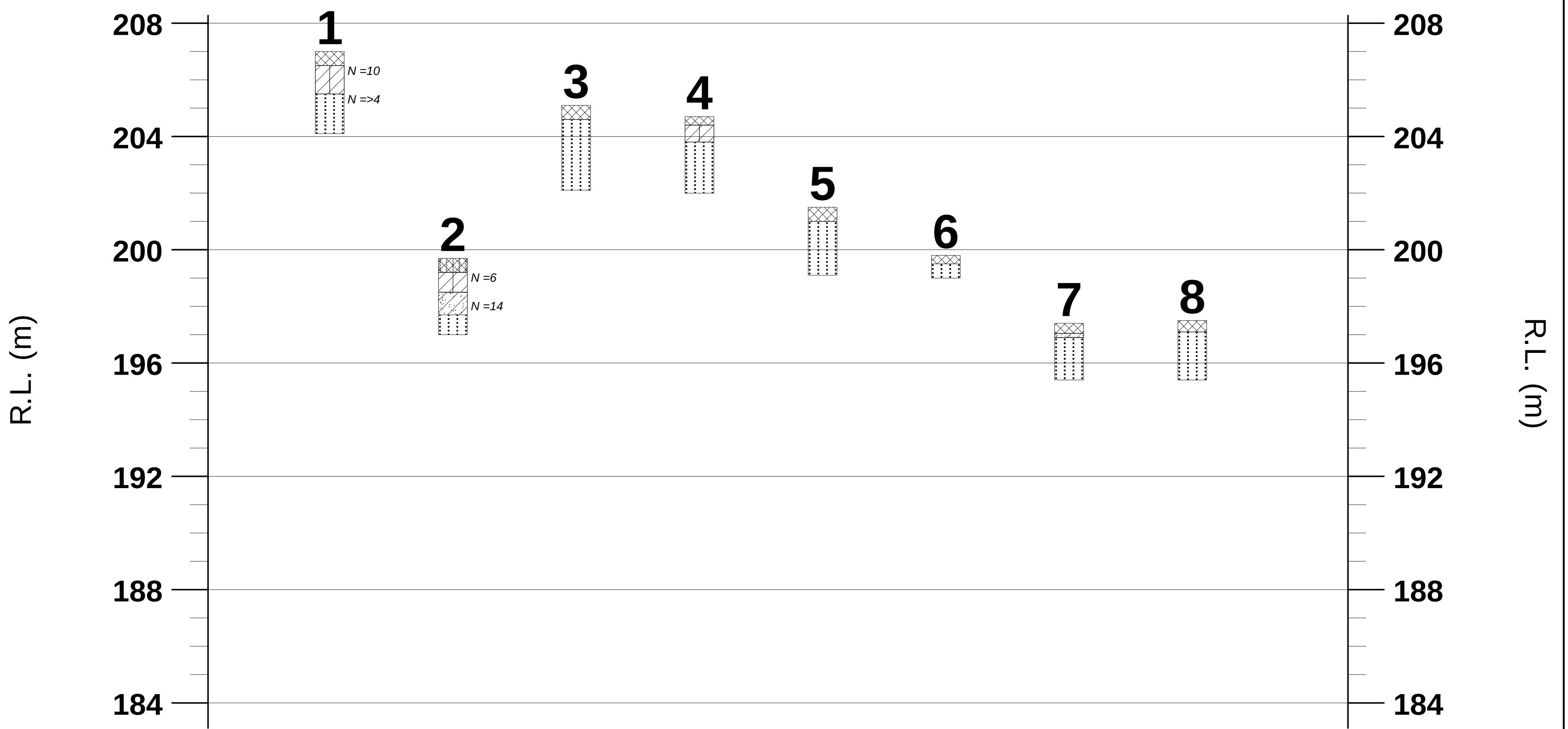
This plan should be read in conjunction with the JK Geotechnics report.

Title:	**BOREHOLE LOCATION PLAN**
Location:	3 QUARRY ROAD DURAL, NSW
Report No: 31137S	Figure No: 2

JK **Geotechnics**

PLOT DATE: 18/06/2018 3:59:28 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31137S DURAL\CAD\31137S.DWG

GRAPHICAL BOREHOLE SUMMARY



Fill

Silty Clay

Sandstone/ Greywacke

Description not given for: "F{"

Sandy Clay

Clayey Sand

N SPT "N" VALUE

Nc SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 200 (vert) ; NTS (horiz)

JK Geotechnics

Job No.: 31137S

Figure No.: 3

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

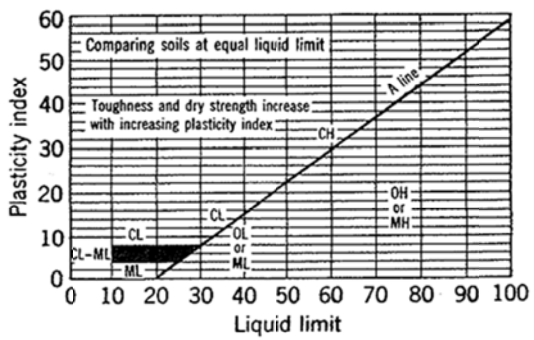
DEFECTS AND INCLUSIONS

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

OTHER MATERIALS

- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

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
Fine-grained soils More than half of material is *smaller* than 75 μm sieve size	Identification Procedures on Fraction Smaller than 380 μm Sieve Size							
		Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)				
	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	—▼— —C— ▶—	Standing water level. Time delay following completion of drilling may be shown. 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	