

REPORT ON GEOTECHNICAL SITE INVESTIGATION

for

PROPOSED BASEMENT LEVEL EXTENSION

at

65 BANGAROO STREET, NORTH BALGOWLAH

Prepared For

Grant Findlay

Project: 2016-134

July, 2016

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**GEOTECHNICAL REPORT FOR PROPOSED BASEMENT LEVEL EXTENSION
65 BANGAROO STREE, NORTH BALGOWLAH, NSW**

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for the proposed basement level extension at 65 Bangaroo Street, North Balgowlah, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of the architect Matt Elkan on behalf of the client Grant Findlay.

The site is situated on the low eastern side of Bangaroo Street within gentle to moderately south east sloping topography. The site is currently occupied by a single storey timber dwelling at the front of the site with a paved courtyard at the front and a sloping lawn at the rear.

It is understood that the proposed works involve a basement level extension. The design of the proposed works is yet to be finalised, with the depth of excavation and distance to the site boundaries still unknown. Therefore this report is an assessment of the potential for excavation and not for Council submission.

The investigation and reporting were undertaken as per the Tender P16-218, Dated: 10th May 2016.

The investigation comprised:

- a) A detailed geotechnical inspection and mapping of the site and adjacent properties by a Geotechnical Engineer.
- b) Drilling of two boreholes using hand tools along with Dynamic Cone Penetrometer testing (DCP) to investigate the subsurface geology, depth to bedrock and identification of a water table.
- c) Excavation of one test pit to expose footings and founding conditions.

The following plans and drawings were supplied by the client for the work;

- Site survey plan by Burton and Field Pty Ltd, Plan Reference: E1301-55805, Date of Survey: 21st August 2007.

2. SITE FEATURES:

2.1. Description:

The site is an irregular shaped block located on the low east side of Bangaroo Street. It has a front west boundary of 11.89m, north side boundary of 42.73m, south side boundary of 43.78m and rear east boundary of 21.46m as referenced from the provided survey plan.

At the front of the site, there is a concrete paved courtyard and single storey timber dwelling. A gravel and concrete path adjacent to the north boundary provides access to the rear east side of the property where there is a sloping lawn and bedrock outcrop.

2.2. Geology:

Reference to the Sydney 1: 100,000 Geological Series sheet (9130) indicates that the site is underlain by Hawkesbury Sandstone (Rh) which is of Triassic age. The rock unit typically comprises of medium to coarse grained quartz sandstone with minor lenses of shale and laminite. Units of this rock were identified at the rear east side of the existing house.

Morphological features often associated with the weathering of Hawkesbury Sandstone are the formation of near flat ridge tops with steep angular side slopes. These slopes often consist of sandstone terraces and cliffs with steep colluvial slopes below. The terraced areas above these cliffs often contain thin sandy (low plasticity) soil profiles with intervening rock (ledge) outcrops. The outline of the cliff areas are often rectilinear in plan view, controlled by large bed thickness and wide spaced near vertical joint pattern, many cliff areas are undercut by differential weathering. Slopes below these cliffs are often steep 15° to 23° with a moderately thick sandy colluvial soil profile that are randomly covered by sandstone boulders.

3. FIELD WORK:

3.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site and adjacent properties on the 4th July 2016 by a Geotechnical Engineer. It included a photographic record of site conditions as well as geological/geomorphological mapping of the site and adjacent land with examination of outcrops, vegetation and existing structures. It also included the drilling of two boreholes (BH1 □ BH2) using a hand auger due to site access limitations and excavation of one test pit to expose footing and foundation conditions.

Dynamic Cone Penetrometer (DCP) testing was carried out from ground surface adjacent to the boreholes in accordance with AS1289.6.3.2 □ 1997, □Determination of the penetration resistance of a soil □ 9kg Dynamic Cone Penetrometer□to estimate near surface soil conditions and confirm depths to bedrock.

Explanatory notes are included in Appendix: 1. Mapping information and test locations are shown on Figure: 1, along with detailed log sheets in Appendix: 2. A geological model/section is provided as Figure: 2 and 3, Appendix: 2.

3.2. Field Observations:

The site is located on the low eastern side of Bangaroo Street within gentle to moderately south east sloping topography. Bangaroo Street contains a bitumen pavement with a low concrete gutter, which is gently south sloping (-4°) where it passes the site. There were no signs of excessive cracking or deformation within the road pavement to suggest any movement.

The front of the site consists of an even surfaced paved courtyard with garden beds adjacent to the boundaries. Adjacent to the east side of the courtyard, there is a single storey timber dwelling with tile roof which appears to be approximately 40 years of age and is in good condition with no obvious signs of cracking or settlement.

Timber stairs adjacent to the south side boundary provide access to a grassy and pebble surfaced area. Below the stairs and adjacent to the south of the dwelling, there is an access door to the sub-floor cavity of the dwelling, revealing brick and metal columns founded on concrete footings. However the founding conditions of these footings could not be observed as the surface was covered with construction debris. Bedrock outcrop was also not exposed within the sub-floor cavity.

A gravelly and paved path adjacent to the northern boundary provides access to the rear east side of the site. At the rear there is a timber deck overlooking sandstone outcrop adjacent to the rear east side of the dwelling and backyard which extends to the rear east boundary. Further east below the timber deck, there is a stone paved area supported by a 0.70m high cemented sandstone block wall on its east side. The sandstone block wall is founded on sandstone bedrock which extends from the timber stairs adjacent to the north side boundary across to the south boundary fence as a low cliff line.

Sandstone bedrock supporting the retaining wall (top of rock RL 75.48m □ 75.76m) is characterised as moderately weathered, medium to coarse grained and of medium to high strength with sub-horizontal bedding defects dipping at 5° to 10° to north west. There is an undercut of up to 0.80m depth with a 0.80m thick overhang at the northern side of the bedrock outcrop. The overhang thins out to the south side where

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another, approximately 1.00m deep, undercut is located below the retaining wall. The minimum thickness of the overhang supporting the retaining wall is 0.20m, however the arching bedrock supporting the retaining wall and the retaining wall itself appear to be in a stable condition with no obvious signs of fractures within the bedrock or cracks in the retaining wall.

Adjacent to the low cliff line there is a moderately south east sloping lawn (-16°) which extends to the rear boundary fence.

The neighbouring property to the north (No. 67) consists of a two storey weatherboard dwelling located within gentle to moderately south east sloping topography. The building structure appears to be less than 20 years of age and is in good condition, with no obvious signs of cracking or settlement on its exterior walls. The property is at a similar ground level along the boundary, with the building located within 1.00m of the common boundary.

The neighbouring properties to the south (No. 38, 40 and 42 Woodbine Street) consist of single storey brick, timber and rendered dwellings respectively. The properties are all located within gentle to moderately south east sloping topography. The dwellings are between 30 to 50 years of age, however their current condition could not be observed from within the site. The properties are at a similar ground level along the boundary with the buildings located within 10.00m of the common boundary. The properties No. 42 and No. 40 contain a concrete block garage and a metal shed respectively, located at the common boundary.

The neighbouring property adjacent to the rear south-east portion of the site (No. 2 Kalai Street) consists of a two storey dwelling with a swimming pool at the rear. A single storey dwelling within the backyard is located within 2.00m of the common boundary. The neighbouring property adjacent to the rear north-east portion of the site (No. 4 Kalai Street) consists of a two storey dwelling with a swimming pool at the rear which is within 10.00m of the common boundary. The existing conditions of both properties could not be observed from within the site area.

The neighbouring buildings and properties were only inspected from within the site or from the road reserve however the visible aspects did not show any significant signs of large scale slope instability or other major geotechnical concerns which would impact the site.

3.3. Boreholes and Test Pits:

The boreholes (BH1 and BH2) were drilled within the garden beds at the front west and south side of the site house, with hand auger refusal encountered at 0.70m and 1.00m depth below the existing ground surface on sandstone bedrock.

Dynamic Cone Penetrometer (DCP) tests were carried out from ground surface adjacent to the boreholes with the DCP1 refusing at 0.70m depth and DCP2 refusing at 0.90m depth.

Based on the field borehole logs and DCP test results the subsurface conditions at the project site can be classified as follows:

- **FILL** □ this layer was encountered below the gravel layer to depths varying from 0.55m (BH1) to 0.70m (BH2) below the existing ground surface. It is classified as very loose, fine grained, moist sand with some sandstone gravels and clay;
- **SAND** – this layer was encountered below the fill to depths varying from 0.70m (BH1) to 1.00m (BH2) below the existing ground surface. It is classified as medium dense, fine to coarse grained, wet sand;
- **SANDSTONE BEDROCK** □ based on the geological mapping of the bedrock outcrops over the project site it is classified as moderately weathered, medium to coarse grained, medium to high strength sandstone. Based on the results of DCP testing in areas of soil cover, the depth to the sandstone bedrock of a minimum of very low strength was interpreted to vary from 0.70m (DCP 1) to 0.90m (DCP2) below the existing ground surface.

A free standing ground water table or significant water seepage were not identified within any of the boreholes. No signs of ground water were observed after the retrieval of the DCP rods. However minor seepage was observed over the bedrock surface.

One test pit (TP1) was excavated adjacent to the north side of the site house to expose the footing and inspect the foundation conditions. The test pit was completed adjacent to a brick column which was observed in the sub-floor cavity. The test pit extended to 0.60m depth below the existing ground surface with refusal encountered on sandstone bedrock, confirming that the column footing was founded on low to medium strength sandstone bedrock.

4. COMMENTS:

4.1. Geotechnical Assessment:

The site investigation identified the presence of a sandy fill layer of shallow thickness ($\approx 0.70\text{m}$) across the project site overlying sandy soils which are underlain by sandstone bedrock. Based on DCP testing in the areas of soil cover and test pit excavation, the depth to the sandstone bedrock of a minimum of very low to low strength is interpreted to be varying from 0.60m (TP1) to 0.90m (DCP2) below the existing ground surface, over the location of the proposed works. This very low to low strength bedrock is expected to grade very quickly to medium to high strength bedrock, which is outcropping at the rear east side of the project site.

The proposed works involve excavation for a basement level extension within the existing building footprint. The final design of the proposed works is yet to be completed and the required excavation depth and distances to the site boundaries are still unknown. It is expected that any excavation will extend through sandstone bedrock of low to medium and potentially high strength bedrock along with intersecting $<1.00\text{m}$ thick layers of sandy fill and sandy soils.

The excavation for the proposed basement extension will most likely be adjacent to and underneath the existing house structure. As such it is recommended that a small scale excavator with small ($<200\text{kg}$) rock hammer and rock saw be utilised. In view of the small scale excavator being employed for the excavation works there is almost no potential for ground vibration damage to occur to the adjacent existing structures. It is recommended that the proposed equipment and methodology be supplied to the geotechnical engineer. Where large scale ($>200\text{kg}$) rock hammer is proposed, there may be a need for additional constraints. The site house footings adjacent to the perimeter of the proposed excavation appear to be founded off low to medium strength sandstone bedrock however this will need confirmation for all footings within a 1.0 (Ver) : 1.0 (Hor) zone of influence drawn up from the base of the excavation.

Whilst there were no stability hazards identified in the investigation, there is a potential for poorly oriented defects within the excavated bedrock to result in localized rock slide/topple failure with potential impact to the work site or the adjacent structures. However through selection of suitable excavation equipment, geotechnical inspection and mapping during the excavation works along with the installation of support measures as determined necessary by the inspections, the risk from the proposed works can be maintained within $\square$ Acceptable $\square$ levels.

The strength of the bedrock with depth is unconfirmed therefore there is a potential for the bedrock to be more deeply weathered and of lesser strength than interpreted. For confirmation of bedrock strength to

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below proposed excavation level will need an investigation utilizing cored boreholes in the actual excavation location however access for such equipment is very limited by site conditions. As such bedrock strength through the excavation and at footing level can be confirmed by geotechnical inspection during initial excavation/construction works.

The recommendations and conclusions in this report are based on an investigation utilising only surface observations and hand drilling tools due to access limitations. This test equipment provides limited data from small isolated test points across the entire site with limited penetration into rock, therefore some minor variation to the interpreted sub-surface conditions is possible, especially between test locations. The results of the investigation provide a reasonable basis for the analysis and subsequent design of the proposed works.



Prepared by:
Shahzada Rizvi
Senior Engineering Geologist



Reviewed by:
Troy Crozier
Principal Engineering Geologist
MAIG. RPGeo; 10197

5. REFERENCES:

1. Australian Geomechanics Society 2007, "Landslide Risk Assessment and Management" Australian Geomechanics Journal Vol. 42, No 1, March 2007.
2. Geological Society Engineering Group Working Party 1972, "The preparation of maps and plans in terms of engineering geology" Quarterly Journal Engineering Geology, Volume 5, Pages 295 - 382.
3. E. Hoek & J.W. Bray 1981, "Rock Slope Engineering" By The Institution of Mining and Metallurgy, London.
4. C. W. Fetter 1995, "Applied Hydrology" by Prentice Hall. V. Gardiner & R. Dackombe 1983, "Geomorphological Field Manual" by George Allen & Unwin
5. V. Gardiner & R. Dackombe 1983, "Geomorphological Field Manual" by George Allen & Unwin.

Appendix 1

NOTES RELATING TO THIS REPORT

Introduction

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

Geotechnical reports are based on information gained from limited subsurface test boring and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Description and classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, Geotechnical Site Investigation Code. In general, descriptions cover the following properties - strength or density, colour, structure, soil or rock type and inclusions.

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (eg. Sandy clay) on the following bases:

Soil Classification	Particle Size
Clay	less than 0.002 mm
Silt	0.002 to 0.06 mm
Sand	0.06 to 2.00 mm
Gravel	2.00 to 60.00mm

Cohesive soils are classified on the basis of strength either by laboratory testing or engineering examination. The strength terms are defined as follows:

Classification	Undrained Shear Strength kPa
Very soft	less than 12
Soft	12 - 25
Firm	25 - 50
Stiff	50 - 100
Very stiff	100 - 200
Hard	Greater than 200

Non-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT) as below:

Relative Density	SPT "N" Value (blows/300mm)	CPT Cone Value (Qc - MPa)
Very loose	less than 5	less than 2
Loose	5 - 10	2 - 5
Medium dense	10 - 30	5 - 15
Dense	30 - 50	15 - 25
Very dense	greater than 50	greater than 25

Rock types are classified by their geological names. Where relevant, further information regarding rock classification is given on the following sheet.

Sampling

Sampling is carried out during drilling to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling to allow information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing a sample of the soil 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 are given in the report.

Drilling Methods

The following is a brief summary of drilling methods currently adopted by the company and some comments on their use and application.

Test Pits – these are excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils if it is safe to descent into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (eg. Pengo) – the hole is advanced by a rotating plate or short spiral auger, generally 300mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling – the hole is advanced by pushing a 100mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling soils, since moisture content is unchanged and soil structure, strength, etc. is only marginally affected.

Continuous Spiral Flight Augers – the hole is advanced using 90 – 115mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or 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, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPT's or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

Non-core Rotary Drilling - the hole is 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.

Rotary Mud Drilling – similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. From SPT).

Continuous Core Drilling – a continuous core sample is obtained using a diamond-tipped core barrel, usually 50mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample. The test procedures is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" – Test 6.3.1.

The test is carried out in a borehole by driving a 50mm diameter split sample tube 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
as 4, 6, 7
N = 13
- In the 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 15, 30/40mm.

The results of the test can be related empirically to the engineering properties of the soil.

Occasionally, the test method is used to obtain samples in 50mm diameter thin wall sample tubes in clay. In such circumstances, the test results are shown on the borelogs in brackets.

Cone Penetrometer Testing and Interpretation

Cone penetrometer testing (sometimes referred to as Dutch Cone – abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in Australia Standard 1289, Test 6.4.1.

In tests, a 35mm diameter rod with a cone-tipped end is pushed continually into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the friction resistance on a separate 130mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected by electrical 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) their information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises: -

- 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 in percent.

There are two scales available for measurement of cone resistance. The lower scale (0 – 5 MPa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main scale (0 – 50 MPa) is less sensitive and is shown as a full line. The ratios of the sleeve friction to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios 1% - 2% are commonly encountered in sands and very soft clays rising to 4% - 10% in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range: -

$$Q_c \text{ (MPa)} = (0.4 \text{ to } 0.6) N \text{ blows (blows per 300mm)}$$

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range: -

$$Q_c = (12 \text{ to } 18) C_u$$

Interpretation of CPT values can also be made to allow estimation of modulus or compressibility values to allow calculations of foundation settlements.

Inferred stratification as shown on the attached reports is assessed from the cone and friction traces and from experience and information from nearby boreholes, etc. This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties, and where precise information on soil classification is required, direct drilling and sampling may be preferable.

Hand Penetrometers

Hand penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150mm increments of penetration. Normally, there is a depth limitation of 1.2m but this may be extended in certain conditions by the use of extension rods.

Two relatively similar tests are used.

- Perth sand penetrometer – a 16mm diameter flattened rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test 6.3.2). The test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.
- Cone penetrometer (sometimes known as Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS 1289, Test 6.3.2). The test was developed initially for pavement sub-grade investigations, and published correlations of the test results with California bearing ratio have been published by various Road Authorities.

Laboratory Testing

Laboratory testing is 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.

Bore Logs

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

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes, the frequency of sampling and the possibility of other than ‘straight line’ variations between the boreholes.

Ground Water

Where ground water levels are measured in boreholes there are several potential problems:

- In low permeability soils, ground water although present, 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. They may not be the same at the time of construction as are indicated in the report.
- The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water observations are to be made. More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be interference from a perched water table.

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 condition, 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 depend partly on bore spacing and sampling frequency,
- changes in policy or interpretation of policy by statutory authorities,
- the actions of contractors responding to commercial pressures,

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

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 than at some later stage, well after the event.

Reproduction of Information for Contractual Purposes

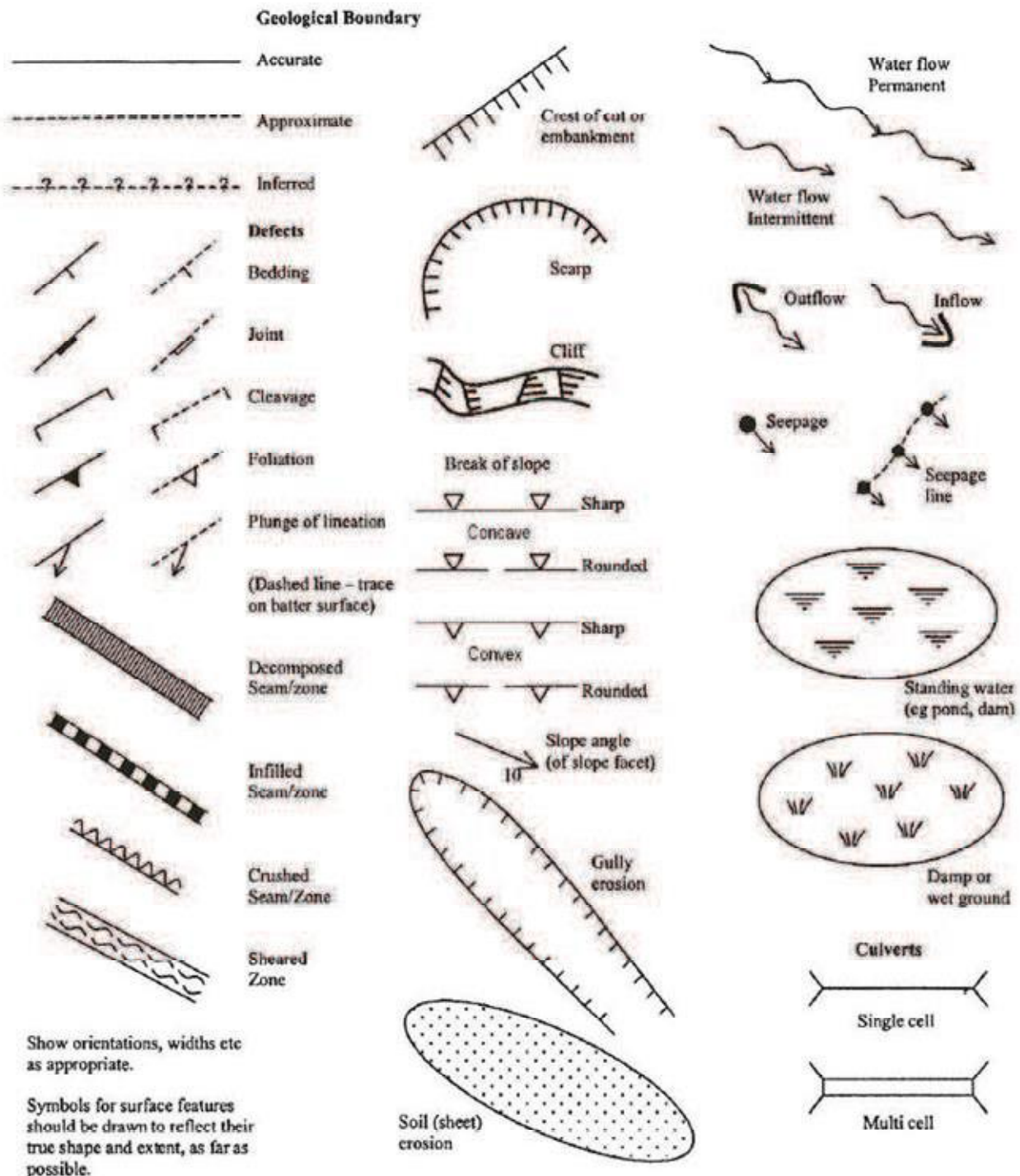
Attention is drawn to the document "Guidelines for the Provision of Geotechnical Information in Tender Documents", published by the Institution of Engineers Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including 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.

Site Inspection

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

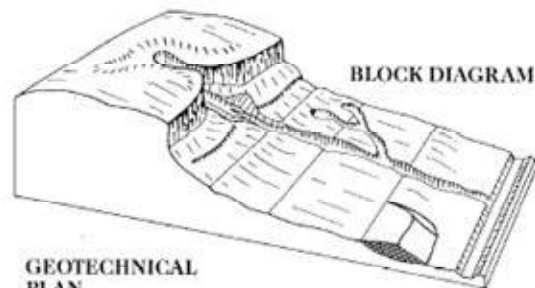
PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT 2007

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY

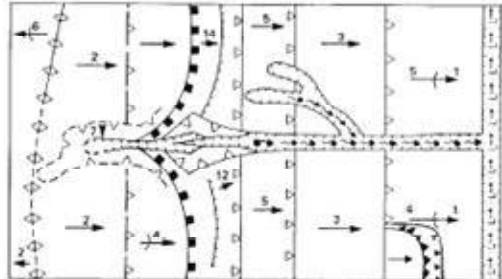


Examples of Mapping Symbols (after Guide to Slope Risk Analysis Version 3.1 November 2001, Roads and Traffic Authority of New South Wales).

PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT 2007



GEOTECHNICAL
PLAN

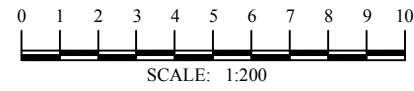
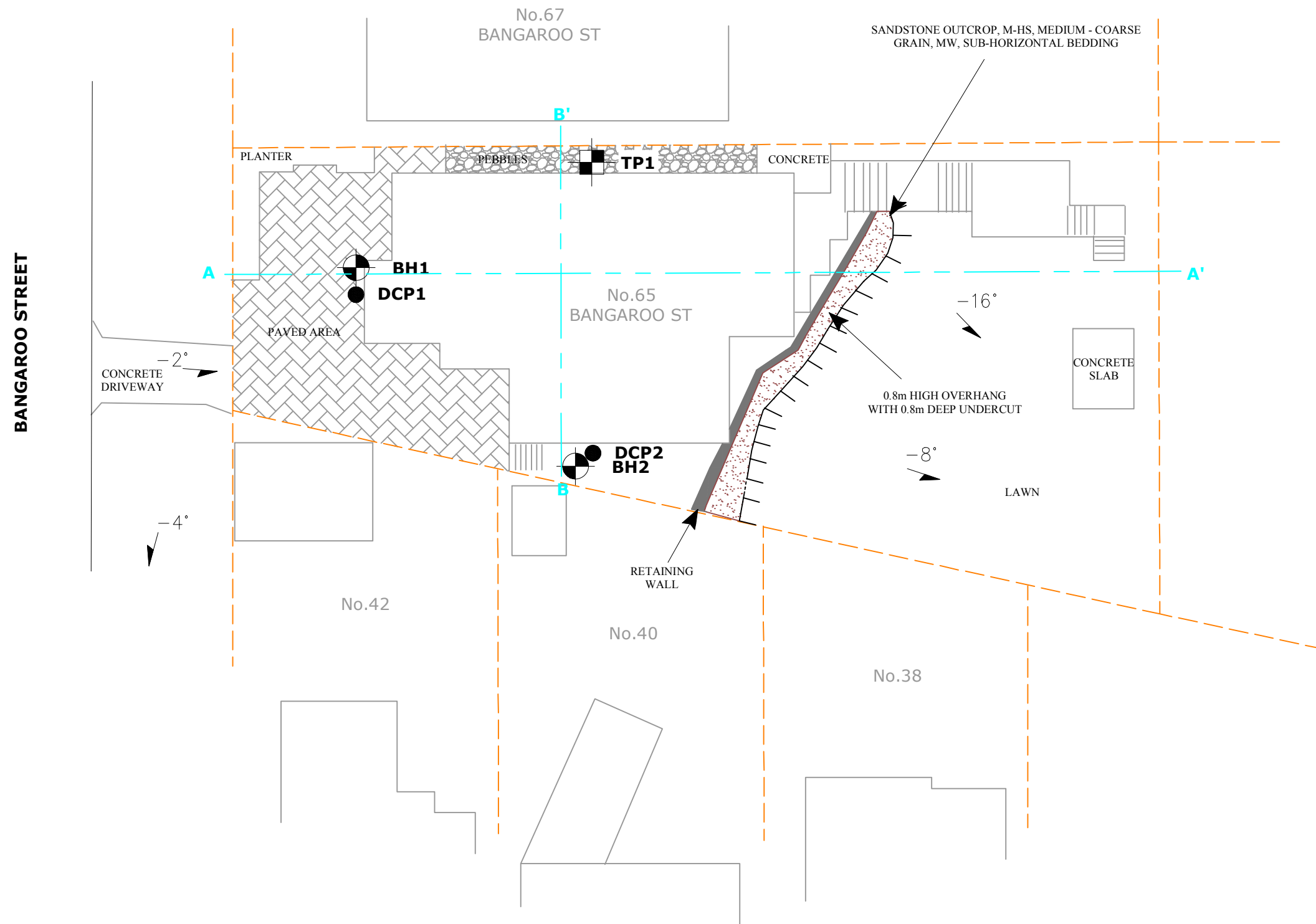


SYMBOL	GROUND PROFILE	
		Convex
		Concave
		Convex
		Concave
		Breaks of slope
		Changes of slope
		Sharp
		Rounded
		Cliff or escarpment or sharp break 40° or more (estimated height in metres)
		15° Uniform slope
		10° Concave slope
		8° Convex slope
		Top
		Bottom
		Hummocky or irregular ground
		Open drain, unlined
		Open drain, lined
		Fence line
		Property boundary
		Dry stone wall
		Major joint in rock face (opening in millimetres)
		Tension crack (opening in millimetres)

Example of Mapping Symbols

(after V Gardiner & R V Dackombe (1983). Geomorphological Field Manual. George Allen & Unwin).

Appendix 2



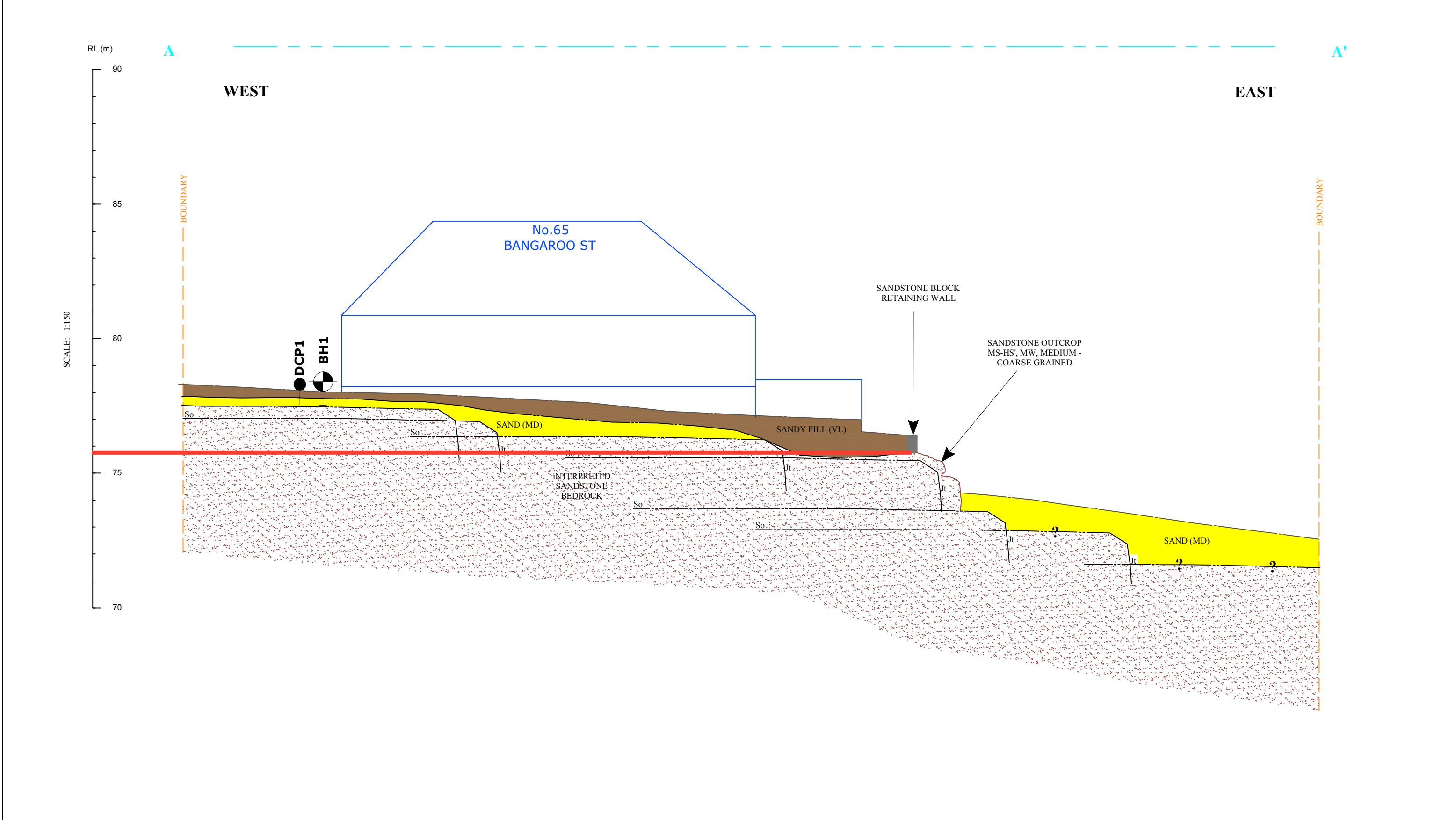
SITE PLAN & TEST LOCATIONS FIGURE 1.



Crozier Geotechnical ABN: 96 113 453 624
Unit 12, 42-46 Wattle Road Phone: (02) 9939 1882
Brookvale NSW 2100 Fax: (02) 9939 1883
Crozier Geotechnical is a division of PJC Geo-Engineering Pty Ltd

LEGEND					
BH	AUGER LOCATIONS	TP1	TEST PIT		CLIFF LINE
DCP	DYNAMIC CONE PENETROMETER	X°	SLOPE ANGLE		SANDSTONE BEDROCK
			RETAINING WALL		STEPS
					PROPERTY BOUNDARY
				A—A'	CROSS-SECTION REFERENCE LINE

SCALE: 1:200 @ A3	PREPARED FOR:
DRAWING: FIGURE 1	GRANT FINDLAY
DATE: 28/7/2016	
APPROVED BY: TMC	ADDRESS:
DRAWN BY: KB	65 BANGAROO ST
PROJECT: 2016-134	NORTH BALGOWLAH



NB. FOR LOCATION OF SECTION A-A', PLEASE REFER TO FIGURE 1. SITE PLAN AND TEST LOCATIONS

GEOLOGICAL MODEL FIGURE 2.



Crozier Geotechnical
Unit 12, 42-46 Wattle Road
Brookvale NSW 2100
Crozier Geotechnical is a division of PJG Geo-Engineering Pty Ltd

ABN: 96 113 453 624
Phone: (02) 9939 1882
Fax: (02) 9939 1883

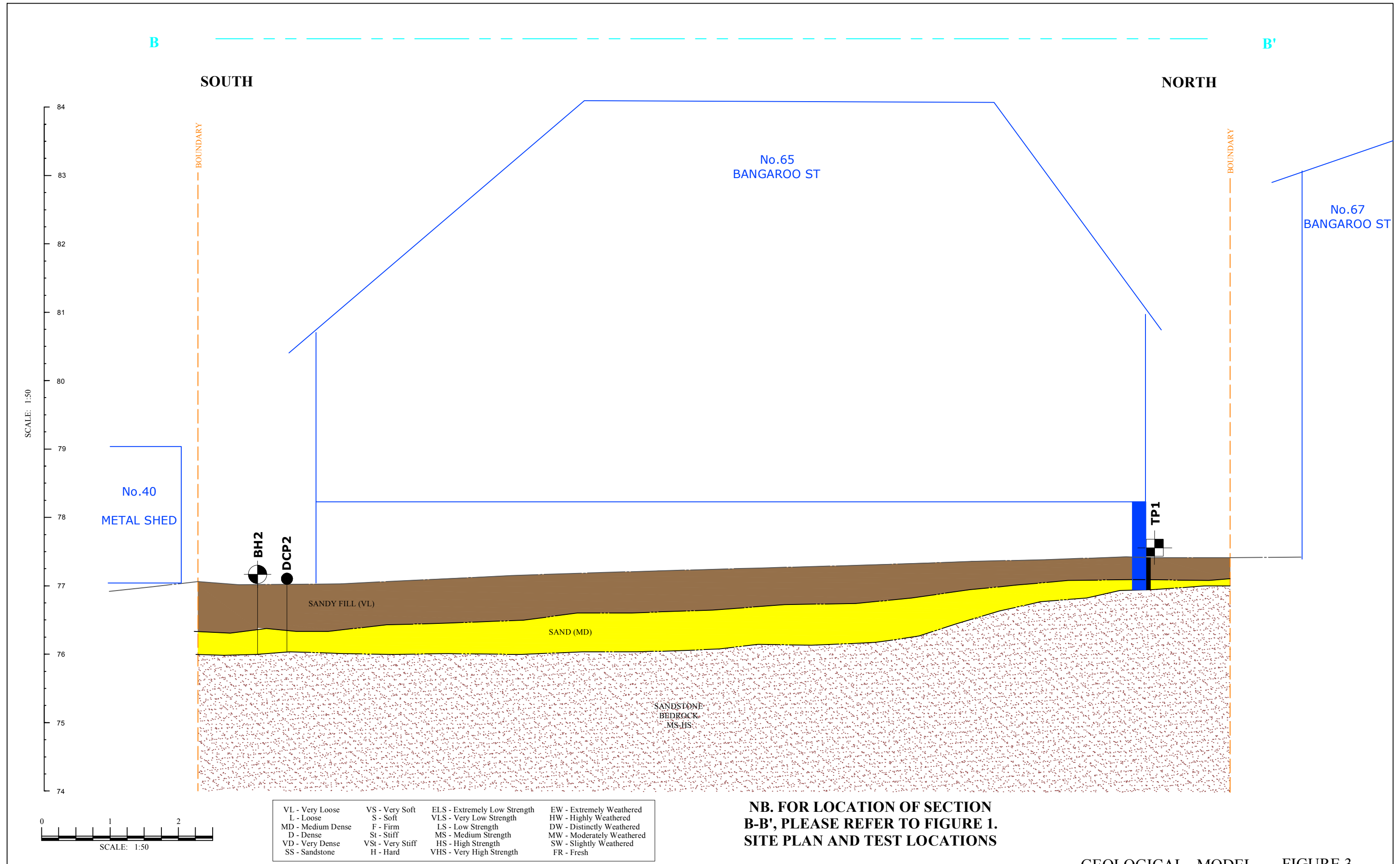
LEGEND			
BH AUGER LOCATIONS	A—A' CROSS-SECTION REFERENCE LINE	So GEOLOGICAL BEDDING	SANDY FILL (VL)
DCP DYNAMIC CONE PENETROMETER	RETAINING WALL	Jt JOINT	SAND (MD)
TP1 TEST PIT	PROPERTY BOUNDARY		SANDSTONE BEDROCK

SCALE: 1:150 @ A3
DRAWING: FIGURE 1
DATE: 1/8/2016

APPROVED BY: TMC
DRAWN BY: KB
PROJECT: 2016-134

PREPARED FOR:
GRANT FINDLAY

ADDRESS:
65 BANGAROO ST
NORTH BALGOWLAH



GEOLOGICAL MODEL FIGURE 3.

TEST BORE REPORT

CLIENT: Grant Findlay

DATE: 4/07/2016

BORE No.: 1

PROJECT: New Basement Level

PROJECT No.: 2016-134

SHEET: 1 of 1

LOCATION: 65 Bangaroo Street, North Balgowlah

SURFACE LEVEL: RL □ 77.96m

Depth (m)	Description of Strata	Sampling		In Situ Testing		
		Type	Depth (m)	Type	Results	
0.00	FILL - GRAVEL					
0.10	FILL - Very loose, dark brown, fine grained, moist sand fill with some gravels * 0.35m a cobble * 0.50m grey-brown and medium grained	D	0.50			
0.55	SAND - Medium dense, light grey, coarse grained, wet sand	D	0.65			
0.70	HAND AUGER REFUSAL at 0.70m depth on sandstone bedrock of at least very low strength					
1.00						
2.00						

RIG: None

DRILLER: KB LOGGED: BL

METHOD: Hand Auger

GROUND WATER OBSERVATIONS: No free standing ground water observed

REMARKS:

CHECKED:

TEST BORE REPORT

CLIENT: Grant Findlay

DATE: 4/07/2016

BORE No.: 2

PROJECT: New Basement Level

PROJECT No.: 2016-134

SHEET: 1 of 1

LOCATION: 65 Bangaroo Street, North Balgowlah

SURFACE LEVEL: RL □ 77.00m

Depth (m)	Description of Strata	Sampling		In Situ Testing		
		Type	Depth (m)	Type	Results	
0.00	PRIMARY SOIL - strength/density, colour, grainsize/plasticity, moisture, soil type incl. secondary constituents, other remarks					
0.05	FILL - GRAVEL					
	FILL - Very loose, dark brown, fine grained moist sand fill					
	* 0.30m some sandstone gravels and clay					
0.70	* 0.55m a cobble					
	SAND - Medium dense, grey-brown, fine grained, wet sand * 0.80m brown					
1.00	HAND AUGER REFUSAL at 1.00m depth on sandstone bedrock of at least very low strength					
2.00						

RIG: None

DRILLER: KB LOGGED: BL

METHOD: Hand Auger

GROUND WATER OBSERVATIONS: No free standing ground water observed

REMARKS:

CHECKED:

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Grant Findlay **DATE:** 4/07/2016
PROJECT: New Basement Level **PROJECT No.:** 2016-134
LOCATION: 65 Bangaroo Street, North Balgowlah **SHEET:** 1 of 1

Depth (m)	Test Location							
	DCP1	DCP2						
0.00 - 0.15	1	1						
0.15 - 0.30	3	4						
0.30 - 0.45	18	11						
0.45 - 0.60	7	5						
0.60 - 0.75	8 (B)	2						
0.75 - 0.90	Refusal at 0.70m	6						
0.90 - 1.05		6 (B)						
1.05 - 1.20		Refusal at 0.90m						
1.20 - 1.35								
1.35 - 1.50								
1.50 - 1.65								
1.65 - 1.80								
1.80 - 1.95								
1.95 - 2.10								
2.10 - 2.25								
2.25 - 2.40								
2.40 - 2.55								
2.55 - 2.70								
2.70 - 2.85								
2.85 - 3.00								

TEST METHOD: AS 1289. F3.2, CONE PENETROMETER

REMARKS: (B) Test hammer bouncing upon refusal on solid object
 -- No test undertaken at this level due to prior excavation of soils