CROZIER
GEOTECHNICAL CONSULTANTS

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Project No: 2015-050

REPORT ON GEOTECHNICAL INSPECTION
INTERNAL & EXTERNAL STRIP OUT
NEWPORT ARMS HOTEL
2 KALINYA STREET, NEWPORT

1. INTRODUCTION:

This report details the results of geotechnical inspection and testing for construction works underway at the Newport Arms Hotel at 2 Kalinya Street, Newport, NSW. The investigation was undertaken by Crozier Geotechnical at the request of SJA Construction Services on behalf of the client Hemmes Trading Pty. Ltd.

The site is located on the western side of Kalinya Street and is bound by Queens Parade along the northern boundary and the Pittwater foreshore along the western and southern side of the site. The site contains a two storey hotel building with open courtyard areas. A basement car park is located below the western side of the hotel which opens to a bitumen carpark to the south.

It is understood that the proposed works involve extensive internal and external strip out of the hotel. As part of the works a new concrete slab and retaining structure is proposed at the southern side of the site.

The site is located within the H1 landslip hazard zone as identified within Pittwater Councils Geotechnical Risk Management Policy Map and is also classified as Acid Sulfate Soils hazard Class ó5ô. The investigation and report were completed as per the tender document P15-485, Dated: 9th November 2015.

The investigation comprised:

- Geotechnical inspection of the site by a Senior Geotechnical Engineer.
- Photographic record of site conditions.
- One Dynamic Penetrometer test to identify sub-surface geology.

Details of the fieldwork are given in the report, together with comments relating to design and construction practice. The following plans were supplied for this work;

- Architectural Plans by Akin Creative, Project No. 0359, Drawing No.ös DA D01 and DA E01, Issue: A, Dated: 04/11/2015.
- Structural Design Sketch by Northern Beaches Consulting Engineers, Job No. 150629, Sketch: SK4.

2. SITE FEATURES:

2.1. Description:

The site is situated on the low, western side of Kalinya Street where the road intersects Beaconsfield Street. It is situated within predominantly gently south-west sloping topography which becomes more moderately sloping immediately adjacent to the Pittwater foreshore which runs along the western and southern side of the site. The property contains a large two storey brick and concrete structure which forms the hotel. A concrete driveway runs along the eastern side of the building to an open bitumen carpark along the southern side of the site. The car park continues to a basement car park to the north. The roof of the carpark extends back to the east to the hotel and consists of open concrete areas. At the time of inspection the property was undergoing renovation works.

2.2. Geology:

Reference to the Sydney 1: 100,000 Geological Series sheet (9130) indicates that the site is underlain by Newport Formation (Rnn) of the Upper Narrabeen Group. Newport Formation (Upper Narrabeen Group) is of middle Triassic Age and typically comprises interbedded laminite, shale and quartz to lithic quartz sandstones and pink clay pellet sandstones.

Narrabeen Group rocks are dominated by shales and thin siltstone/sandstone beds and often form rounded convex ridge tops with moderate angle (<20°) side slopes. These side slopes can be either concave or convex depending on geology, internally they comprise interbedded shale and siltstone beds with close spaced bedding partings that have either close spaced vertical joints or in extreme cases large space convex joints. The shale often forms deeply weathered profiles with silty or medium to high plasticity clays and a thin silty colluvial cover.

3. FIELDWORK

Inspection of the proposed new retaining wall and slab area was carried out by a Senior Geotechnical Engineer from Crozier Geotechnical. The previous structures in the location are understood to be a series of disability ramps that had been demolished with excavation carried out along the northern edge with a vertical face exposed up to approximately 2.50m depth. The exposed face can be classified as follows:

- **CONCRETE** ï steel reinforced concrete footpath slab to 0.15m;
- **FILL** ï this layer was encountered to approximate depths of 1.0m. Fill material is classified as Silty/Sandy CLAY;
- **CLAY** ï this layer was encountered below the fill material to 1.40m and is classified as grey Silty CLAY;
- **CLAY (Residual)** ï encountered below the clay to 2.0m depth and classified as brown and orange-brown moist CLAY.
- **SHALEY CLAY –** encountered from 2.0m depth and classified as grey shaley CLAY with some dark red ironstone and iron cemented areas.

A Dynamic Cone Penetrometer test (DCP1) was carried out in accordance with AS1289.6.3.2 ï 1997, ñDetermination of the penetration resistance of a soil ï 9kg Dynamic Cone Penetrometerò through the base of the excavation at a similar level to the bitumen carpark pavement adjacent. The results indicate very stiff clay to 0.90m depth becoming hard below 0.90m. The test was discontinued at 1.05m depth.

4. COMMENTS

4.1. Geotechnical Assessment:

The inspection identified that the existing footpath slab is underlain by fill to 1.0m depth with clay soils below. Shaley clay was visible from 2.0m depth and is exposed across the base of the excavated footing. The excavation has been carried out vertically up to 2.50m depth and these surficial fill and residual soils will not stand unsupported therefore to ensure safe working conditions they will require battering and then installation of permanent excavation support.

A risk assessment carried out in line with Pittwater Council requirements and the Australian Geomechanics Society Guidleines (2007) identified that the only credible landslip hazard related to the proposed works relates to the excavation. This hazard will be removed by the construction of a new engineer designed retaining structure as proposed.

The proposed works are considered suitable for the site and may be completed with negligible impact to existing structures within the site and adjacent properties provided the recommendations of this report are implemented in the design and construction phases.

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 away from test locations.

The site is situated within Class ó5ôAcid Sulfate Soils hazard zone which is a classification based on proximity to a zone of higher likelihood of Acid Sulfate Soils. The soils encountered during the investigation are not considered to be of the right geological setting to result in actual or potential acid sulfate characteristics and the works will not lower any water table that may be present below the site. Therefore Acid Sulfate Soils will not be impacted by the proposed works and no further investigation into these soils is required.

4.2. Site Specific Risk Assessment:

Based on our site investigation we have identified the following geological/geotechnical hazard which needs to be considered in relation to the existing site and the proposed works. These hazards are:

A. Landslip (Earth slide <5m³) from excavation failure

A qualitative assessment of risk to life and property related to these hazards is presented in Table 1 and 1a, Appendix: 1, and is based on methods outlined in Appendix C of the Australian Geomechanics Society Guidelines for Landslide Risk Management 2007.

The Risk to Life from Hazard A was estimated at Ò6.25 x 10^{-5}, whilst the Risk to Property was considered to be óLowö. The hazard was therefore considered to be óTolerableô when assessed against the criteria of the AGS 2007 and Pittwater Councils Risk Management Policy. However this hazard was assessed as for unsuitable excavation support design and implementation. Through implementation of engineered support as proposed by the new retaining structure the hazard will be reduced to óAcceptableô levels provided the recommendations of this report are adhered to. The Risk to Life following construction of engineered retaining support reduced to 1.25 x 10^{-9} and Risk to Property of óVery Lowô which is óAcceptableö.

4.3. Design & Construction Recommendations:

4.3.1. New Footings:

The results of the investigation suggest that the location of the new retaining structure is underlain by shallow fill and residual clayey soils with shaley clay at the base of the excavated footing. The provided structural sketch indicates that a reinforced concrete block wall and pier footings will be used. The new structure footings should be founded within similar bearing material to limit the potential for differential settlement

The site is considered a Class óPô site as per the Australian Standard for Residential Slabs and Footings AS2870 ï 2011 due to being in an area identified as being prone to landslip risk. However where footings are located at the base of an excavation into shaley clay they may be designed as per a Class óSô site.

Under the Australian Standard *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia* the site Sub-soil classification would be C_e ï shallow soil site.

Footings founded on very stiff shaley clay at the base of the excavation should be designed for a maximum allowable bearing capacity of 300kPa. If higher footing pressures are required then additional testing of the ground below footing level will be required.

All new footings must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity and the insitu nature of the founding strata. This is mandatory to allow them to be ócertifiedô at the end of the project.

4.3.2 Excavation Support

Recommended maximum batter slopes for excavation through fill and natural soils/rock on this site are presented below in Table: 1. Where these batters cannot be implemented then the excavation will require temporary support until permanent retaining walls can be completed. If suitable measures are not implemented then the stability of this excavation until permanent retaining walls are completed cannot be guaranteed. This should be considered in regard to Work Cover/safety requirements.

Table 1 - Safe Batter Slopes

Material	**Safe Batter Slope (H:V)**	
	Short Term/ Temporary	**Long Term/ Permanent**
Fill and natural soils	1:1	2:1
Extremely Low to Very Low strength bedrock	1:1	1.25:1

Water ingress into exposed excavations can result in erosion and stability concerns in both soil and rock portions. Drainage measures will need to be in place during excavation works to divert any surface flow away from the excavation crest and any batter slope.

5.0 CONCLUSION

The inspection identified the existing slab is underlain by fill, clayey soils, and shaley clay. Excavation has been carried out without battering or temporary support measures which presents a stability hazard to construction workers in the area. It is recommended that batters are implemented or temporary support as soon as possible to reduce the risk of instability.

The proposed works and surrounding area have been assessed as per the Pittwater Council Geotechnical Risk Management Policy 2009. The assessment identified that the potential landslip hazards created by the proposed works can produce óTolerable risk levels where unsuitable excavation support design and construction measures are undertaken. However it is considered that the site and proposed works can maintain the óAcceptableô risk criteria for the design life of the development, taken as 100 years, provided proper engineering design and construction methods are implemented as proposed, including but not limited to the recommendations of this report.

Prepared by:	Reviewed by:
James Butcher	Troy Crozier
Senior Geotechnical Engineer	Principal Engineering Geologist

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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 separte 130mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected buy 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: -

Qc (MPa) = (0.4 to 0.6) N blows (blows per 300mm)

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

Qc = (12 to 18) Cu

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.

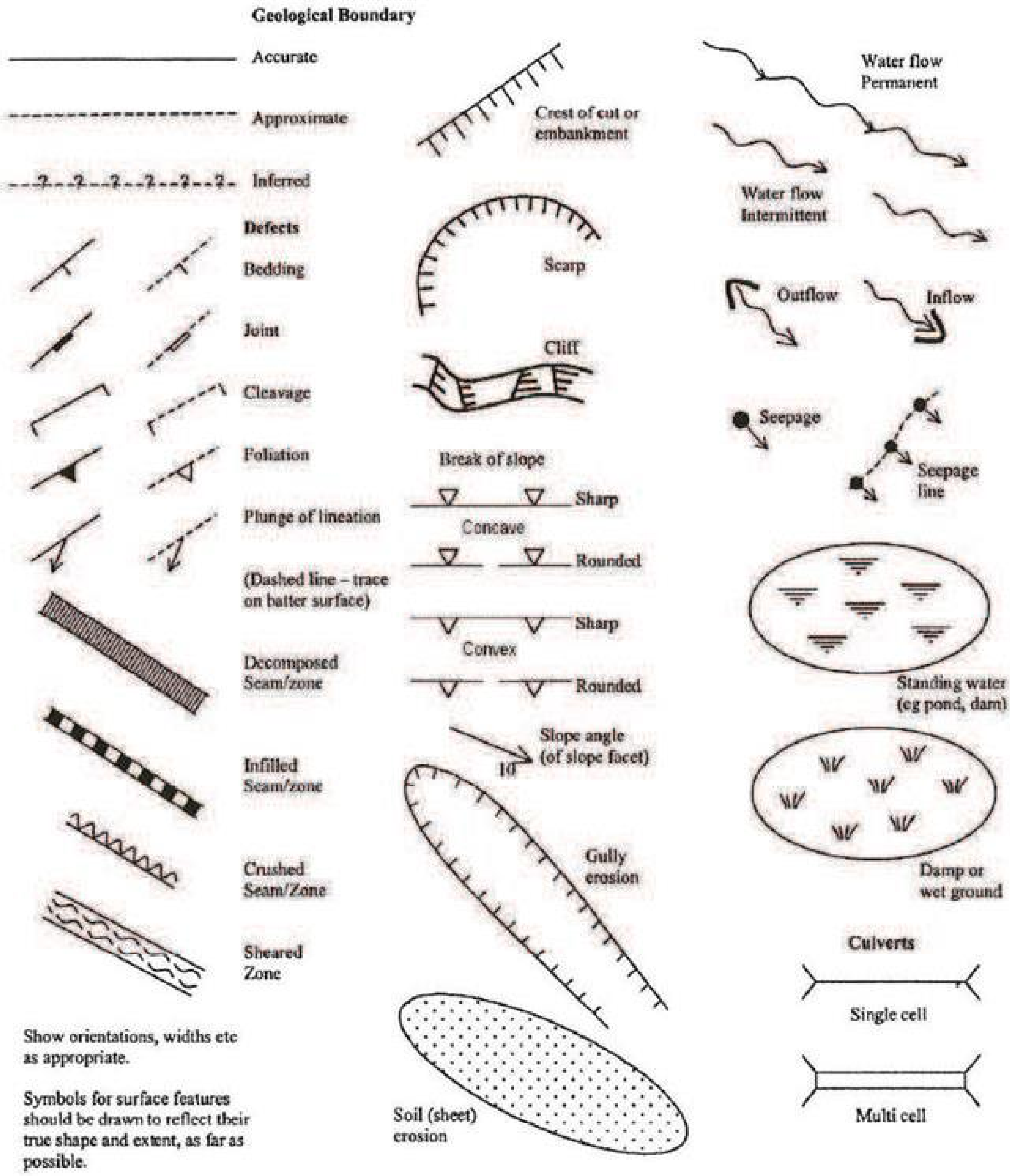
Reproduction of Information for Contractual Purposes

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 special ally 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.

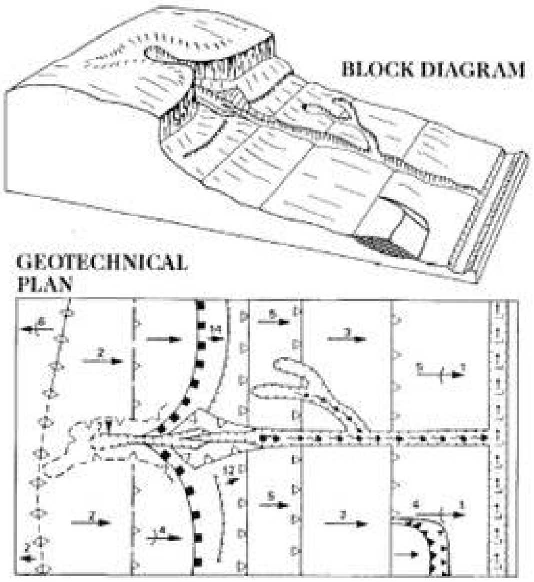
Site Inspection

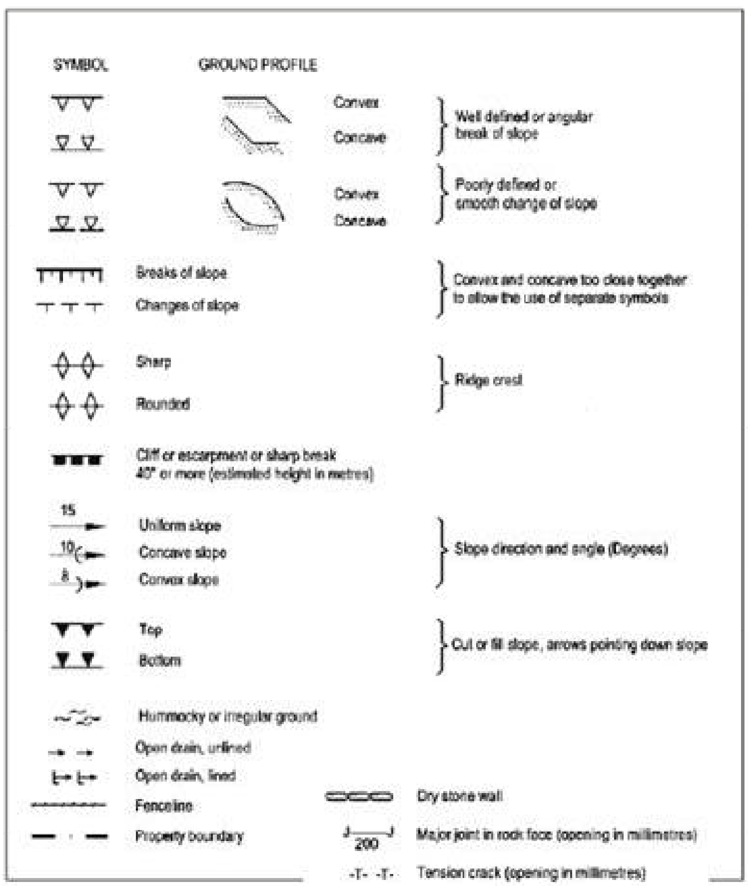
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

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).





Example of Mapping Symbols

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

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Hemmes Trading Pty Ltd **DATE:** 9/11/2015

PROJECT: Strip Out **PROJECT No.:** 2015-240

LOCATION: 2 Kalinya Street, Newport **SHEET:** 1 of 1

	Test Location							
Depth (m)	DCP1							
0.00 - 0.15	10							
0.15 - 0.30	13							
0.30 - 0.45	7							
0.45 - 0.60	7							
0.60 - 0.75	11							
0.75 - 0.90	11							
0.90 - 1.05	15							
1.05 - 1.20	END							
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

TABLE : A

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood	Spatial Impact	Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip (earth/debris slide <10m³) from proposed excavation		Fill over residual soils and deeply weathered bedrock, vertical excavation to 2.50m depth	a) excavation will extend below footpath, may impact 50% b) slide may impact 30% of excavation base c) New retaining structure proposed, failure would be small	a) workers using footpath 6hrs/day b) excavation 8hrs/day c) patrons using footpath 12 hrs/day	a) Possible to not evacuate b) Possible to not evacuate c) Unlikely to not evacuate	a) Person in open space and not buried b) Person in open space and not buried c) Person in open space and not buried	
		a) undermining footpath above	Likely 0.01	0.50	0.25	0.5	0.1	**6.25E-05**
		b) excavation base	Likely 0.01	0.30	0.33	0.5	1.0	**5.00E-04**
	Following construction	c) undermining footpath	Rare 0.00001	0.10	0.50	0.25	0.1	**1.25E-08**

* hazards considered in current condition and/or without suitable remedial/stabilisation measures
* likelihood of occurrence for design life of house (considered 100years)
* considered for person most at risk
* evacuation scale from Almost Certain to not evacuate (1.0), Likely (0.75), Possible (0.5), Unlikely (0.25), Rare to not evacuate (0.01)
* vulnerability assessed using Appendix F - AGS Practice Note Guidelines for Landslide Risk Management 2007

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip (earth/debris slide <10m³) from proposed excavation	a) undermining footpath above	Likely	Event will probably occur under adverse circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required, no impact to neighbouring properties.	Low
		b) excavation base	Likely	Event will probably occur under adverse circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required, no impact to neighbouring properties.	Low
		c) undermining footpath	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required, no impact to neighbouring properties.	Very Low

* hazards considered in current condition, without remedial/stabilisation measures and during construction works.
* qualitative expression of likelihood incorporates both frequency analysis estimate and spatial impact probability estimate as per AGS guidelines.
* qualitative measures of consequences to property assessed per Appendix C in AGS Guidelines for Landslide Risk Management.
* Indicative cost of damage expressed as cost of site development with respect to consequence values: Catastrophic : 200%, Major: 60%, Medium: 20%, Minor: 5%, Insignificant: 0.5%.