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GEOTECHNICAL ASSESSMENT REPORT

for

PROPOSED NEW HOUSE

at

95 GURNEY CRESCENT, SEAFORTH

Prepared For

Ms N van Gemert and Mr J Ball

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GEOTECHNICAL ASSESSMENT REPORT
95 GURNEY CRESCENT, SEAFORTH, NSW

1. INTRODUCTION:

This report details the results of a geotechnical assessment undertaken for a proposed four storey residential house at 95 Gurney Crescent, Seaforth, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of Mark Hurcum Design Practice Pty Ltd on behalf of the client Ms N van Gemert and Mr J Ball.

The site is located within Landslip Risk Class óG1ô as identified within Northern Beaches (Manly) Councils ï Development Control Plan 2013 ï Schedule 1 Map C. As such the Development Application requires a site stability (geotechnical) report. This report must detail the stability of the site and how the development may be achieved to ensure geotechnical stability and good engineering practice.

The investigation was undertaken as per the Tender: P19-429, Dated: 11th November 2019 and comprised the following scope of work:

a) A detailed geotechnical mapping of the entire site and adjacent land, with identification of geotechnical conditions including landslip related to the existing site and proposed structures by a Senior Geotechnical Professional and a Geotechnical Engineer.
b) A photographic record of site conditions and field observations.

The following documents, plans and drawings were supplied for the work:

<u>Previous Design Documents:</u>

- Construction Certificate approved Architectural Drawings ï by G Loupis, Title: Proposed New Residence, For: John & Robyn Tripolitis, Drawings: Site plan, carport plan, living level plan, bedroom level plan, pool & open entertainment area plan, section A-A; north, south, east and west elevation; boundary retaining wall fences, stormwater layout plan, agricultural pipe layout plan, landscaping plan, soil & water management plan.

- Alternate Retaining wall base for rock ï by D Mitsopoulos & Associated Pty Ltd, Project: Proposed new residence, Issue: Original, Details: Council Submission, Date: April 2000, Drawing No.: S00-0, Sheets: 1/2, 2/2.
- Geotechnical site investigation report ï by D.F. Dickson & Associates Pty Ltd, Report: 28106-G1, Titled: Geotechnical Site investigation for proposed residence, date: 28/02/2008.

Proposed Development and recent Survey Drawings:

- Architectural Drawings ï by Mark Hurcum Design Practice Architects, Dated: 9/10/2019, Drawing No.: A001A/P1, A100A/P1, A101A/P1 to A105A/P1, A201A/P1 to A203A/P1, SK01AP1 to SK04A/P1.
- Survey Floor plans of Lot 44 In DP 11214 Being 95 Gurney Crescent, Seaforth ï by Hill & Blume Consulting Surveyors, Survey Date: 14/08/2019, Drawing No.: 61539001A & 61539002A.

2. PROPOSED DEVELOPMENT:

It is understood from the provided architectural drawings that the proposed works involves the completion of a new four storey dwelling plus an enclosed garage and office space at the entry level. The works will incorporate the existing structure which consists of four slabs and limited walls for the previously approved development. The new dwelling will have a pool extension (wading depth) to the west requiring minor excavation (up to 1.2m depth) with all other works above/to existing structures.

3. SITE FEATURES:

3.1. Site Description:

The site is situated on the low west side of Gurney Crescent, within a steep west dipping (29°) topography. Based on the supplied survey drawing, the site is a long rectangular shaped block with a rear/foreshore west boundary defined by the Mean High Water Mark (MHWM) of 13.21m, an angled front boundary to Gurney Crescent of 12.60m, a north boundary of 42.06m and a south boundary of 44.8m.

Ground surface levels within the site reduce from a high of approximately RL27.35m adjacent to Gurney Crescent to a low of approximately RL0.0m at the foreshore boundary. An aerial image showing the site and immediate surrounds is shown in Photograph 1.

Photograph 1: Aerial view of the site and immediate surrounds

Gurney Crescent (Photograph-2 and 3) comprises an asphalt road which dips to the north at approximately 3-4° which continues north to a narrower road reserve and has no pedestrian pavement. To the east of the road pavement is a low gutter and kerb followed by a bedrock cliff face (approximately 5m height) underlying a dense vegetated area, occupied by trees, plants and boulders that extends upslope to the east. To the west of the road pavement is a concrete driveway which provides access from Gurney Crescent to the existing upper level of the development.

The NSW Government Six Maps website indicates that the site and the neighbouring properties were undeveloped in 1943. Information provided from Mark Nurcum Design Practice Architects states that the existing building received its original approval for construction 20 years ago (in 1999), it received a S96 approval in 2008 and then was put on hold by Council till date, hence the existing partially constructed four storey structure is at least 20 years old.

The properties to the north and south (No.97 and No.93 Gurney Crescent respectively) contains a four storey brick rendered residential dwelling (No.97) and brick and timber frame/fiberboard (No.93) structure, with limited garden areas. A pool is located at the rear west side of No.97 at ground floor level whilst some grass terraces along with small trees exist within No.93.

Photograph-2: Gurney Crescent road reserve, view looking north.

Photograph-3: Gurney Crescent road reserve, view looking south.

3.2. 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 medium to coarse grained quartz sandstone with minor lenses of shale and laminite. An extract of the 9130 Geological Series sheet is provided in Extract 1.

Extract 1: Geology underlying the site

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.

4. FIELD WORK:

4.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site and adjacent properties on the 20th November 2019 by a Senior Engineering Geologist and 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, slopes and structures.

4.2. Field Observations:

Outside of the site, to the front of the eastern boundary, the bedrock cliff-face comprises of sandstone of at least medium strength and displayed an apparent gently dipping bedding plane at the top of the cliff. The cliff-face also contained apparent joints, these joints were supported by hazard reduction measures such as rock bolts/anchors and shotcrete. The sandstone cliff-face appears to be stable with no sign of rock instability, in the portion in front of the site.

Photograph-4: Sandstone cliff-face, view looking east.

Entrance to the site (Level 4) is via a concrete driveway. Level 4 is an unroofed carport which contains unfinished concrete block walls (approximately 1m height). A concrete staircase adjacent to the south boundary permits access to the lower levels. The concrete floor and staircase appear to be in reasonable condition with no signs of cracks or deformation, whilst the walls contained minor vertical cracks (approximately 0.5mm width).

Based on architectural drawing (dated: 25/05/2000) and site investigation, Level 3 contains a concrete floor with an approximate R.L. of 20.50m. It is an unroofed level with concrete floor and contains temporary timber framed walls (approximately 1.5m height) surrounding the floor level. The concrete floor appears to be in reasonable condition with no signs of cracks or deformation. Adjacent to the south boundary a concrete block retaining wall (Photograph-5) is visible and it appears to support the road reserve (Gurney Street). The concrete block retaining wall appears to be in reasonable condition with no sign of tilting or deformation, however it presented minor vertical cracks in a similar style as the walls observed in Level 4.

The provided original architectural drawings (dated:25/05/2000), indicate that Level 3 and Level 4 were to be founded onto an unexcavated rock surface, whilst rock excavation was required for Level 2 to Level 1, to accommodate construction. This was confirmed in the site investigation, it appears that Level 3 and Level 4 are formed on brick and concrete columns, which are founded directly onto unexcavated bedrock (Photograph ï 6) to the east of an excavation into the bedrock. The bedrock comprises of sandstone of at least medium strength containing horizontal bedding planes with minor defects and no destabilizing defects were observed. There were no signs of underlying geotechnical issues within the supporting columns of the existing structure.

Photograph-5: Concrete block retaining wall, view looking east into No.93.

Photograph-6: Level four supported in brick columns directly on to rock, view looking east.

Level 2 comprises of a concrete floor, concrete block walls and steel beam. Signs of rock excavation were found in the north east corner of the slab (Photograph-7). The bedrock outcrop comprises of sandstone of at least medium strength, it presented an irregular outcrop face due to the excavation, however it did not contain destabilizing defects, where visible. There were no signs of underlying geotechnical issues/destabilization within the bearing walls, slab and columns. Vertical cracks (approximately 2.5m height and 5mm width) were observed within the internal walls (Photograph 8&9), however no evidence indicated they were due to a geotechnical issue. Photograph 10 shows a timber framed door, which appeared to be in good condition, the frame is vertically straight with no sign of tilting/dislocation.

Photograph-7: Rock excavation in the north east corner. View looking east.

Photograph-8: Vertical crack. View looking south

Photograph-9: Vertical crack. View looking south

Photograph-10: Straight timber framed door. View looking south.

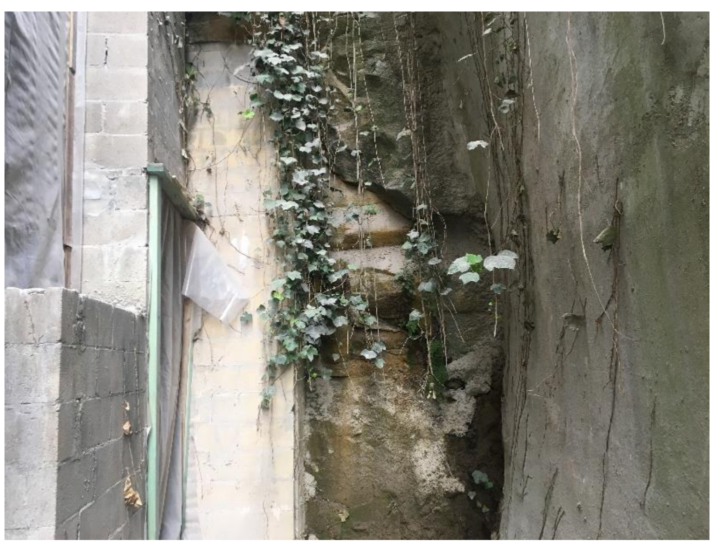
The northern boundary contains a concrete block boundary/retaining wall which is covered or underlain by a shotcrete wall/face from Level 2 (Photograph-11) down to Level 1 (Photograph-12). The wall appears to be in good condition with no sign of cracking, tilting or deformation. Similarly, the southern boundary contains a shotcrete boundary/retaining wall. The wall also appears to be in good condition with no sign of cracking, tilting or deformation.

Photograph-11: Retaining wall. View looking east.

Photograph-12: Retaining wall. View looking east.

Level 1 comprises of a concrete floor and concrete block columns. Access to the sandstone cliff-face to the east was possible via a pathway between the existing structure and the north and south boundary. The sandstone cliff-face at the south east corner of the site appears to be supported and underpinned by a reinforced brick/concrete retaining wall (Photograph-13&14). The retaining wall supporting the cliff-face appears to be in good condition with no signs of cracks, deformation or rotation. The exposed sandstone cliff-face contained some sub-horizontal bedding defects along with some steeply defect joints (Photograph-15), however it did not present any destabilizing defects and appeared stable.

Photograph-13: Reinforced concrete/brick retaining wall.



Photograph-14: Reinforced concrete/brick retaining wall. View looking east.

Photograph-15: Steeply defect joints from behind the wall. View looking east.

To the west of Level 1 is an excavation for the existing incomplete pool (that was to be founded on piers) (Photograph 16). To the west of the pool is a terrace with concrete floor that extends west approximately 5m. This terrace is supported by a concrete block retaining wall (approximately 5m height) around its western edge. The concrete block retaining wall (Photograph-17) appears to be founded on bedrock (Photograph-18), the bedrock extends approximately 2m west. The concrete block retaining wall appeared to be in good condition with no sign of cracks, deformation or rotation. The same bedrock composition was observed to the north, in the northern boundary line and in the neighbourŝ property (No.97). It comprises of sandstone of at least medium strength and no destabilizing defects were observed. To the west of the sandstone outcrop is a dense vegetated slope and further inspection was not possible in this part of the site.

Photograph-16: Proposed pool location

Photograph-17: Concrete block retaining wall. View looking east

Photograph-18: Sandstone bedrock View looking east

5. ASSESSMENT:

Based on the above items the present site contains no obvious history of landslip instability. Based on the proposed architectural drawing, the proposed works require only excavation for the proposed pool extension (wading depth) that is expected to have a maximum depth of up to 1.2m and is expected to intersect fill only behind the existing retaining wall. It is likely that fill material might extent to depths greater than 1.2m, behind this wall.

The site appears to be dipping steeper than 1V:4H and has been extensively excavated for this existing development. The existing excavation appears to have been in existence for 20 years and is in good condition with no indication of potential instability, whilst all retaining walls appear in good condition.

There were no existing landslip hazards identified within the site or adjacent properties and the proposed works will not create any new landslip hazards. Therefore, it is considered that a detailed Geotechnical Report with Landslip Risk Assessment is not required for this Development.

5.1. Design & Construction Recommendations for proposed pool:

5.1.1. Excavation:	
Depth of Excavation	Up to 1.2m depth.
Distance of Excavation to Neighbouring Properties	No.97 Gurney Crescent ï 2m from boundary, neighborôs pool another 5m. No.93 Gurney Crescent ï 2.5m from boundary, building another 1m. Road reserve- 25m from boundary, road pavement another 1.5m
Type of Material to be Excavated	Fill, up to approximately 1.2m.

Guidelines for un-surcharged batter slopes for this site are tabulated below:

Material	Safe Batter Slope (H: V)	
	Short Term/Temporary	Long Term/Permanent
Fill and natural soils	1.5:1 to 1:1	2:1

Remarks:
Where safe batter slopes are not implemented, the stability of the excavation cannot be guaranteed until permanent support measures are installed. This should also be considered with respect to safe working conditions. Batter slopes should not be left unsupported without geotechnical inspection and approval.

Equipment for Excavation	Fill/natural soils	Bucket

Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	Not Applicable
Vibration Calibration Tests Required	Not Applicable
Full time vibration Monitoring Required	Not Applicable
Geotechnical Inspection Requirement	Subject to the finding of subsurface investigation
Dilapidation Surveys Requirement	Not considered critical

5.1.2. Retaining Structures:	
Required	The new pool shell will permanently support the required excavation for the pool extension. It is recommended that the pool extension and any new footings extend to found on sandstone bedrock of at least low strength.

5.2. Design Life of Future Development:

A recommended maintenance program is given in Table: 1 below and should also include the following guidelines:

- The conditions on the block donĉt change from those present at the time this report was prepared, except for the changes due to new development.
- There is no change to the property due to an extraordinary event external to this site, and the property is maintained in good order and in accordance with the guidelines set out in;
  a) CSIRO sheet BTF 18
  b) Australian Geomechanics ñLandslide Risk Managementò Volume 42, March 2007.
  c) AS 2870 ï 2011, Australian Standard for Residential Slabs and Footings

Table 1: Recommended Maintenance and Inspection Program for Future Developments

Structure	**Maintenance/ Inspection Item**	**Frequency**
Stormwater Drains.	Owner to inspect to ensure that the drains and pipes are free of debris & sediment build-up. Clear surface grates and litter.	Every year or following each major rainfall event
Retaining Walls or remedial measures	Owner to inspect walls for deviation from as constructed condition or for excess deterioration/rotation or signs of soil settlement/erosion or significant cracking adjacent to crest.	Every two years or following major rainfall events.
Excavation and bedrock outcrops and slopes	Geotechnical inspections	10 to 15 years interval

N.B. Provided the above schedule is maintained the design life of the property should conform AS2870 and Councils 100 years stability criteria

Where changes to site conditions are identified during the maintenance and inspection program, reference should be made to relevant professionals (e.g. structural engineer, geotechnical engineer or Council). It is assumed that Council will control development on neighbouring properties, carry out regular inspections and maintenance of the road verge, stormwater systems and large trees on public land adjacent to the site so as to ensure that stability conditions do not deteriorate with potential increase in risk level to the site. Also individual Government Departments will maintain public utilities in the form of power lines, water and sewer mains to ensure they donôt leak and increase either the local groundwater levels or landslide potential.

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6. REFERENCES:

1. Australian Geomechanics Society 2007, ñLandslide Risk Assessment and Managementò, Australian Geomechanics Journal Vol. 42, No 1, March 2007.
2. Geological Society Engineering Group Working Party 1972, ñThe preparation of maps and plans in terms of engineering geologyò Quarterly Journal Engineering Geology, Volume 5, Pages 295 - 382.
3. E. Hoek & J.W. Bray 1981, ñRock Slope Engineeringò By The Institution of Mining and Metallurgy, London.
4. C. W. Fetter 1995, ñApplied Hydrologyò by Prentice Hall. V. Gardiner & R. Dackombe 1983, ñGeomorphological Field Manualò by George Allen & Unwin
5. Pells et. al. Design loadings for foundations on shale and sandstone in the Sydney region. Australian Geomechanics Society Journal, 1978.
6. Australian Standard AS 2870 ï 2011, Residential Slabs and Footings.

Appendix 1

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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.

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 then 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 then 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: -

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

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.

Dynamic Penetrometers

Dynamic 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 6.3.3). 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 generally 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.

Borehole 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.

Details of the type and method of sampling are given in the report and the following sample codes are on the borehole logs where applicable:

D	Disturbed Sample	E	Environmental sample	DT	Diatube
B	Bulk Sample	PP	Pocket Penetrometer Test		
U50	50mm Undisturbed Tube Sample	SPT	Standard Penetration Test		
U63	63mm “ “ “ “ “	C	Core		

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

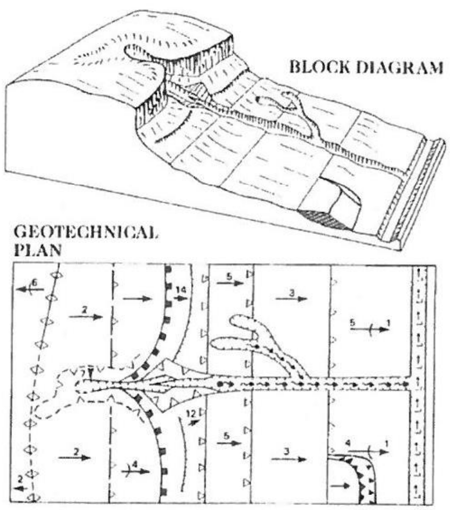
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some later stage, well after the event.

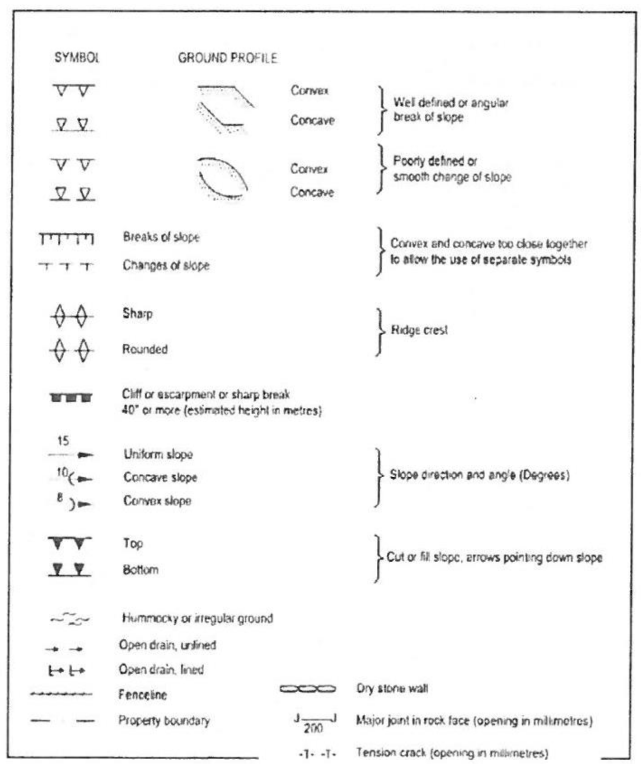
Reproduction of Information for Contractual Purposes

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.

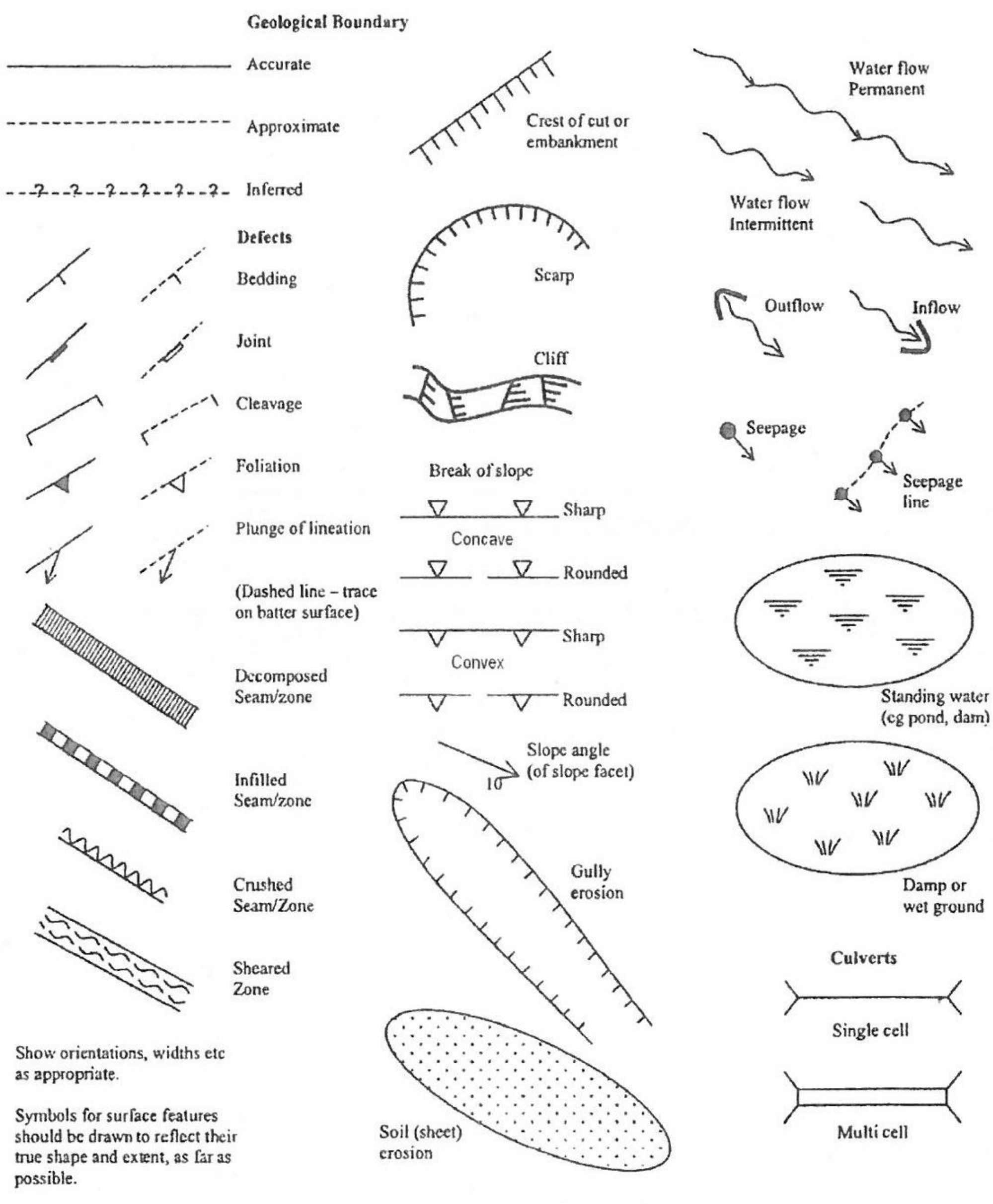




Example of Mapping Symbols

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

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).