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88 Republic of Gladys,
12-14 Gladys Ave,
Frenchs Forest, NSW 2086

Geotechnical Assessment for 12-14 Gladys Avenue, Frenchs Forest

This letter report details the results of a preliminary landslip assessment required by Northern Beaches Council to accompany all new Development or Building Certificate Applications. It is a review of the design plans followed by a walk over visual assessment of the stability of the existing property, no in-situ testing was undertaken.

The assessment follows the guidelines as set out in Section E10-Landslip Risk of Warringah Councils 2011 LEP Planning Rules. It follows previous investigations undertaken at the site by others - White Geotechnical Group, Project No. J4186, Dated: 19/05/2022 & 19/08/2022 and Martens Consulting Engineers, Project No.: P1806545JR01V04, Dated: 07/2020.

1. Landslip Risk Class:

According to Landslip Risk Map sheet _LSR008, the site is located within Landslip Risk Class Areas "A" – Slope <5º, "B" – Flanking Slopes 5° to 25° and "C" – Slopes >25º.

2. Site Location:

The site is located on the northwest side of the road within gentle to steep north dipping topography. It comprises two blocks (12 Gladys Avenue – Lot A/DP393276 and 14 Gladys Avenue – Lot B/DP393276). The combined lots form a roughly trapezoidal shaped block with a long driveway, with the driveway portion of the site approximately 9.40m wide and 31.205m long. The main portion of the site has an eastern boundary of 90.10m, a northern rear boundary sum of 57.91m, and a western boundary of 52.0m, with the site covering an area of approximately 4,704m^2 as referenced from the provided survey plan. The site elevations vary from a high of approximately RL156.73m at the street front to a low of RL130.60m at the northwest corner.

3. Proposed Development:

It is understood the proposed works involve the demolition of existing site structures, the amalgamation of the two lots and construction of a two-storey seniors living facility with a basement carpark. The proposed works will require bulk excavation to a maximum of approximately 10.0m depth that will extend to within 6.0m of the side boundaries.

4. Existing Site Description:

The site is located on the low northwest side of Gladys Avenue which comprises a bitumen pavement that is gently northeast dipping and contains low concrete gutters along the sides. Between the gutters and front site boundary the road reserve contains a grass lawn and a bitumen driveway. Cracking, ground movement or signs of underlying geotechnical issues were not observed within the road reserve which appeared in good condition.

The full length of the driveway is approximately 60m with grassed sides and some large mature trees adjacent to the driveway, with the area gently north dipping towards the bulk of the site. The driveway opens out to the main portion of the site which is gently north dipping towards the existing site structures.

The structures comprise a single-storey brick and rendered (No.12 Gladys Ave) and a one and two-storey masonry and timber clad (No. 14 Gladys Ave) residential dwellings of estimated construction age of ~60 years. The structures showed signs of superficial aging however there were no visible signs of any significant cracking or settlement to indicate underlying geotechnical issues.

To the rear (north) of the dwellings the sandstone bedrock outcrops in a cliff line that extends roughly east-west across the site and is estimated to be up to ~4.0m high in places. The sandstone was preliminarily assessed as low to medium strength with roughly horizontal bedding parts at approximately 2.0m vertical intervals, with overhangs at the base extending up to ~2.0m horizontally into the slope and 0.60m vertically at the face. The slope above is heavily vegetated, with a pool below the cliff line. Below this pool the site is heavily vegetated however appears to be steeply north sloping and contains boulders of various size up to approximately 1.50m maximum dimension.

5. Neighbouring Property Conditions:

The neighbouring properties to the southwest (No. 10, 10a and 10b Gladys Avenue) contain two-storey brick residential structures set within ~1.0m of the common boundary. The structures appeared to be in good condition without signs of cracking or excessive settlement and have an estimated construction age of ~30 years. There was limited visibility into these properties however these were no indications of geotechnical instability noted during the inspection.

The property to the east (No. 16A Gladys Avenue) comprises a battle axe block with a long driveway adjacent to the site's eastern boundary leading to the main portion of the property which contains a two-storey brick residence positioned northeast of the existing dwelling at No. 14. It was not possible to make a thorough assessment of the structures or ground levels due to the limited visibility however obvious signs of ground movement or underlying geotechnical issues were not observed at the property.

The neighbouring properties to the north (No. 4 Arden Place and No. 66 Epping Drive) were not able to be inspected due to the dense vegetation at the rear (north) of the site preventing visibility. These properties are set lower than the site with separation distances from the proposed works to the shared boundary in excessive of 25m.

Assessment:

Based on the above items and on Councils flow chart check list (Page: 2 of 2 in Section E10), i.e., does the present site or proposed development contain:

- History of Landslip — No
- Proposed Excavation/Fill >2m — Yes
- Site developed — Yes
- Existing Fill >1m — No
- Site Steeper than 1V:4H — Yes – To the rear (north) of existing structures
- Existing Excavation >2m — No
- Natural Cliffs >3m — Yes

It is considered that a due to the nature of proposed DA submission and existing site stability, a detailed Landslip Risk Assessment for this Development Application is required and presented below.

6. Site Specific Landslip Risk Assessment

Based on our site investigation we have identified the following geological/geotechnical landslip hazards which need to be considered in relation to the existing site and the proposed works. The main hazards are:

A. Landslip (earth slide/rock collapse) due to collapse of proposed excavation (<$10m^3$);
B. Boulder impact from dislodged boulder in steep, vegetated area at rear of site during or following proposed works.

A qualitative assessment of risk to life and property related to these hazards is presented in **Tables A and B**, Appendix: 1, and is based on methods outlined in Appendix: C of the Australian Geomechanics Society (AGS) Guidelines for Landslide Risk Management 2007. AGS terms and their descriptions are provided in Appendix: 2.

The Risk to Life from Hazard **A** was estimated to be up to **5×10^{-5}** for any person while the Risk to Property was considered to be '**Moderate**', which is unacceptable without treatment. Detailed investigation, planning and implementation of treatment options are required to reduce risk to Low.

The Risk to Life from Hazard **B** was estimated to be up to **2.50×10^{-5}** for any person while the Risk to Property was considered to be '**Moderate**', which may be tolerated in certain circumstances but requires investigation, planning and implementation of treatment options to reduce the risk to Low. However, the assessments were based on excavations with no support or underpinning or planning.

The assessments were based on excavations with no support, planning or implementation of engineered retention and with no consideration of vibration limits.

It is considered likely that excavation into the existing cliff line below the residential structures may improve the overall stability of the site, as the overhangs and any boulders within that area would be removed.

As such the project is considered suitable for the site provided the recommendations of this report and future assessment/reporting are implemented.

7. Geotechnical Assessment and Recommendations

The proposed works involve the demolition of existing site structures and construction of a two-storey seniors living facility with a basement carpark. The proposed works will require bulk excavation to approximately 10.0m depth that will extend to within 6.0m of the side boundaries.

The excavation is anticipated to extend through shallow fill and residual soils to depths up to approximately 2.10m, with the excavation also extending into the very low to medium strength sandstone bedrock.

Pre-excavation support is expected to be required along the southern portion of the excavation perimeter to prevent deflection in the neighbouring property as well as anywhere else safe batter slopes provided below are not possible within the site boundaries. This support should comprise soldier piles which may then be incorporated into the completed structure, with shotcrete infill panels. The retaining structure will need to be designed and constructed in accordance with AS4678-2002 Earth Retaining Structures using the parameters provided in the White (2022) Section 14, an excerpt of which is provided below. It is recommended that 'At Rest' (K_o) values are adopted in the design.

Unit	Earth Pressure Coefficients		
	Unit weight (kN/m^3)	'Active' K_a	'At Rest' K_0
Fill, Topsoil, Sand	20	0.40	0.55
Residual Clays	20	0.35	0.45
Low Strength Sandstone	24	0.20	0.34
Medium Strength Sandstone	24	0.00	0.01

For rock classes refer to Pells et al "Design Loadings for Foundations on Shale and Sandstone in the Sydney Region". Australian Geomechanics Journal 1978.

Excerpt 1: Table 1 of White Geotechnical Group Report (2022)

It is expected that safe batter slopes as outlined below will be possible around the much of the excavation perimeter given the 6.0m separation to the site boundaries, however this will need to be confirmed following clearing of the site.

The temporary safe batter slopes are 1V:2H for fill and natural soils, 1V:1H for natural clay soils and 1V:0.5H for low strength and better sandstone, pending geotechnical inspection of the rock mass.

It is possible that temporary support will be required in some locations where the safe batter slopes are marginally possible and therefore an allowance should be made for this, in the form of I-beams concreted into the sandstone with walers.

Inspections of excavations undertaken through sandstone bedrock will need to be undertaken by a geotechnical engineer in order to assess the rock mass and determine the need for additional stabilisation such as rock bolts or shotcrete.

Footings should all extend to the low strength or better sandstone bedrock to reduce the risk of differential settlement, with only ancillary structures founded within the residual sandy clay soil. Footings may be designed for an allowable bearing pressure of 1,000kPa for low strength sandstone and 2,000kPa for medium strength sandstone, although this will require confirmation following further geotechnical investigation including coring of the bedrock to below basement levels.

Provided the recommendations outlined below as well as those outlined in White (2022) are implemented including the installation of the recommended engineered retention of the excavation and consideration of vibration limits and survey of boulders the likelihood of any failure becomes 'Rare' and as such the consequences reduce and risk becomes within 'Acceptable' levels when assessed against the criteria of the AGS 2007.

- Additional geotechnical investigation at CC stage including cored boreholes to below the proposed depth of excavation to confirm bedrock characteristics.
- A survey of the lower, northern section of the site for boulders by a geotechnical engineer or engineering geologist, with stabilization where required.
- Installation of engineered retaining structures where required, designed and constructed in accordance with AS4678-2002 Earth Retaining Structures, with the design reviewed by a geotechnical engineer.
- An assessment of excavation machinery and methodology by a qualified geotechnical engineer to ensure vibration levels do not impact neighbouring properties or boulders.

8. Date of Assessment: 28th June 2024

9. Assessment by:

Ben Taylor
Senior Geotechnical Engineer

10. References:

- Architectural Drawings – Smith & Tzannes, Project No.: 24_041, Drawing No.: DA-A-010, DA-A-013, DA-A-014, DA-A-100 to DA-A-106, DA-A-200 to DA-A-205, DA-A-800 to DA-A-808, DA-A-850 to DA-A-855, Rev 08 08 2024
- Geotechnical Reports – White Geotechnical Group, Project No. J4186, Dated: 19/05/2022 & 19/08/2022
- Geotechnical Report – Martens Consulting Engineers, Project No.: P1806545JR01V04, Dated: 07/2020

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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 (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: -

Q_c = (12 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

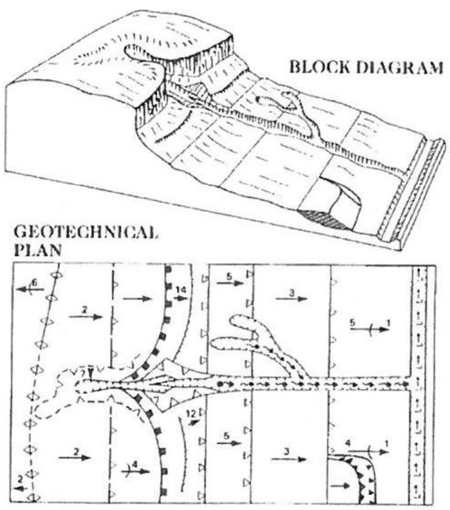
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

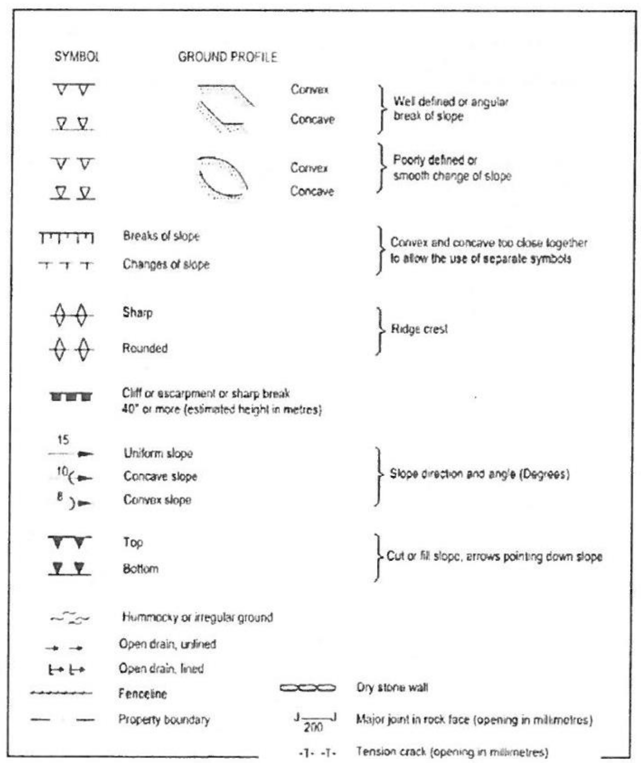
Reproduction of Information for Contractual Purposes

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

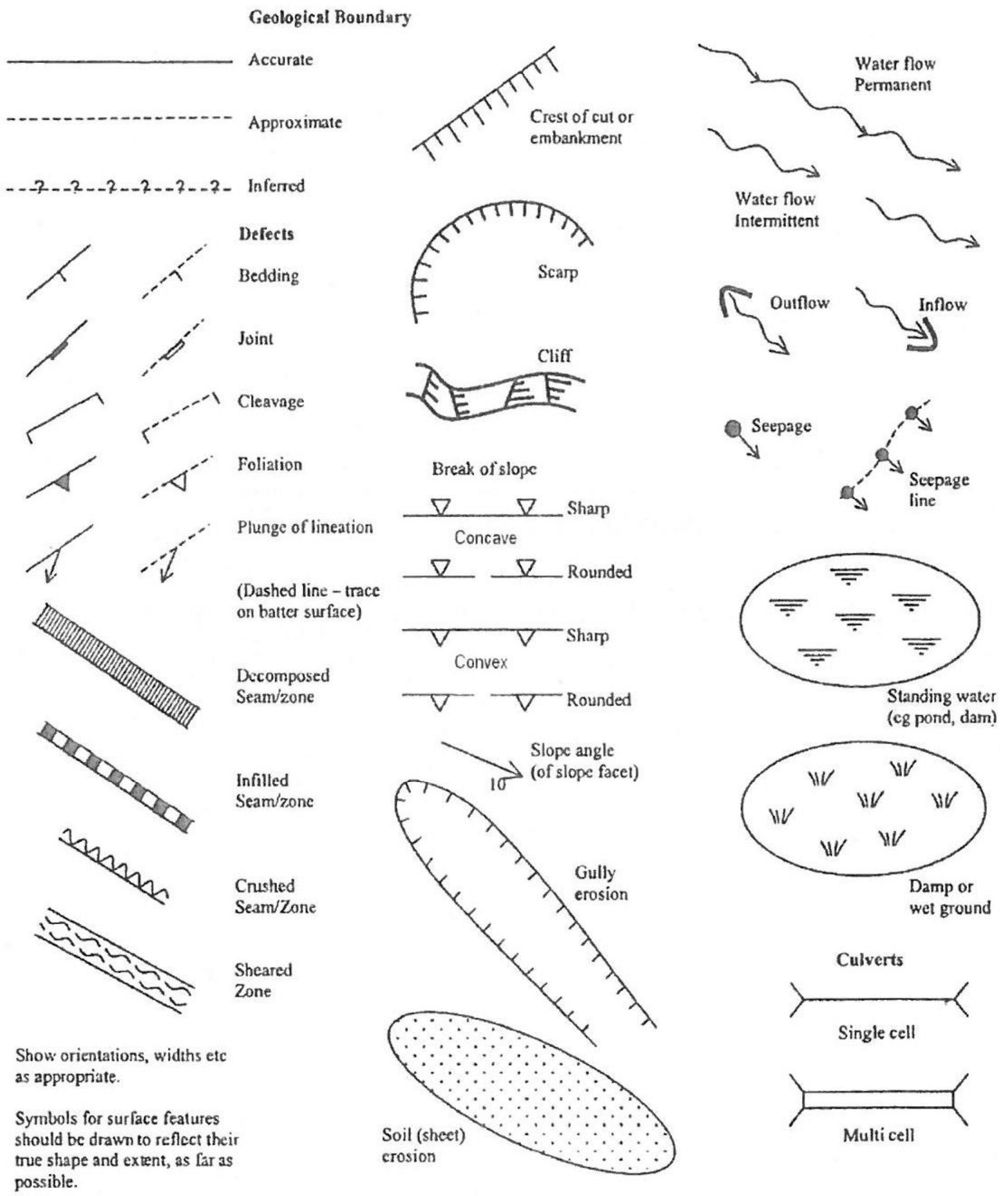
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.





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).

Appendix 2

TABLE : A

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood of Slide	Spatial Impact of Slide		Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip (earth slide/rock collapse) due to collapse of proposed excavation (<10m^3)		Excavation up to 3.0m depth through minor fill soils and bedrock	Expected to impact small portion of structure and garden above		a) Person on the lower floor 24hr/day average; b) person in the upper garden area 2hr/day average;	a) Almost certain to not evacuate b) Possible to not evacuate	a) Person in the building and the debris striking portion of the building only b) Person in open area, not buried	
			Possible	**Prob. of Impact**	**Impacted**				
		a) Lower level of proposed dwelling below excavation	0.001	1.00	0.20	1.0000	1.00	0.25	**5.00E-05**
		b) Garden area above excavation	0.001	1.00	0.20	0.0833	0.50	1.00	**8.33E-06**
B	Boulder impact from dislodged boulder in steep, vegetated area at rear of site during or following proposed works		Boulders up to ~1.50m maximum dimension in rear of site may be dislodged due to vibration, impact or due to changed surface water conditions eroding around base of boulder	Extent and size of boulders unknown due to dense vegetation a) and b) Boulder may impact small portion of either propoerty c)Boulder may impact small portion of rear garden area		a) and b) Person in rear of properties 4hr/day average; c) Person in the rear of site, 1hr/day average;	Likely to not evacuate	Person in open space, likely to be impacted	
			Possible	**Prob. of Impact**	**Impacted**				
		a) Outdoor areas and structure No. 4 Arden Place	0.001	1.00	0.20	0.1667	0.75	1.00	**2.50E-05**
		b) Outdoor areas and structure in No. 66 Epping Drive	0.001	1.00	0.20	0.1667	0.75	1.00	**2.50E-05**
		c) Persons in rear of site	0.001	0.25	0.10	0.0417	0.75	1.00	**7.81E-07**

* hazards considered in current condition and/or without remedial/stabilisation measures or poor support systems
* likelihood of occurrence for design life of 100 years
* Spatial Impact - Probaility of Impact refers to slide impacting structure/area expressed as a % (i.e. 1.00 = 100% probability of slide impacting area if slide occurs).
Impacted refers to expected % of area/structure damaged if slide impacts (i.e. small, slow earth slide will damage small portion of house structure such as 1 bedroom (5%), where as large boulder roll may damage/destroy >50%)

* neighbouring houses considered for impact of slide to bedroom unless specified, due to high occupancy and lower potential for evacuation.
* considered for person most at risk, where multiple people occupy area then increased risk levels
* for excavation induced landslip then considered for adjacent premises/buildings founded off shallow footings, unless indicated
* 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). Based on likelihood of person knowing of landslide and completely evacuating area prior to landslide impact.
* 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 slide/rock collapse) due to collapse of proposed excavation (<10m3)	a) Lower level of proposed dwelling below excavation	Possible	The event could occur under adverse conditions over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works **or** MINOR damage to neighbouring property.	Moderate
		b) Garden area above excavation	Possible	The event could occur under adverse conditions over the design life.	Insignificant	Little Damage, no significant stabilising required or no impact to neighbouring properties.	Very Low
B	Boulder impact from dislodged boulder in steep, vegetated area at rear of site during or following proposed works	a) Outdoor areas and structure No. 4 Arden Place	Possible	The event could occur under adverse conditions over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	Moderate
		b) Outdoor areas and structure in No. 66 Epping Drive	Possible	The event could occur under adverse conditions over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	Moderate
		c) Persons in rear of site	Possible	The event could occur under adverse conditions over the design life.	Insignificant	Little Damage, no significant stabilising required or 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%.
* Cost of site development estimated at $2,000,000

TABLE: 2

Recommended Maintenance and Inspection Program

Structure	Maintenance/ Inspection Item	Frequency
Stormwater drains.	Owner to inspect to ensure that the open drains, and pipes are free of debris & sediment build-up. Clear surface grates and litter. Owner to check and flush retaining wall drainage pipes/systems	Every year or following each major rainfall event. Every 7 years or where dampness/moisture
Retaining Walls. or remedial measures	Owner to inspect walls for deveation from as constructed condition and repair/replace. Replace non engineered rock/timber walls prior to collapse	Every two years or following major rainfall event. As soon as practicable
Large Trees on or adjacent to site	Arborist to check condition of trees and remove as required. Where tree within steep slopes (>18°) or adjacent to structures requires geotechincal inspection prior to removal	Every five years
Slope Stability	Geotechnical Engineering Consultant to check on site stability and maintenance	Five years after construction is completed.

N.B. Provided the above shedule is maintained the design life of the property should conform with Councils Risk Management Policy.