Crozier Geotechnical Consultants
Unit 12/ 42-46 Wattle Road
Brookvale NSW 2100
Crozier Geotechnical Consultants, a division of PJC Geo-Engineering Pty Ltd
ABN: 96 113 453 624
Phone: (02) 9939 1882
Email: info@croziergeotech.com.au

REPORT ON GEOTECHNICAL SITE INVESTIGATION

for

ALTERATIONS AND ADDITIONS

at

66 BOWER STREET, MANLY, NSW

Prepared For

Vahuvu Pty Ltd

Project No.: 2019-138

December 2019

Document Revision Record

Issue No	Date	Details of Revisions
0	17th December 2019	Original issue

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GEOTECHNICAL CONSULTANTS

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Unit 12/ 42-46 Wattle Road Phone: (02) 9939 1882
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GEOTECHNICAL INVESTIGATION FOR ALTERATIONS AND ADDITIONS
66 BOWER STREET, MANLY, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for proposed alterations and additions at 66 Bower Street, Manly, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of the client Vahuvu Pty Ltd.

It is understood that the proposed works involve the demolition of two existing double garages and construction of two new double garages with a lower ground storage level below and a new entry foyer to the apartments. The works will require excavations of up to approximately 3.0m depth which will extend to the front south boundary and within 0.90m of the side east and west boundaries. The excavation will reduce to nil towards the north to the front of the apartment building due to the ground surface slope.

The site is located within Landslip Risk Class óG1/G2ô as identified within Northern Beaches (Manly) Councils ï Development Control Plan 2013 ï Schedule 1 Map C.

A review of the preliminary slope stability assessment checklist and the proposed works identified that the Development Application requires a full site stability (geotechnical) report. Therefore, this report is provided to meet Councils requirements. It includes a description of site and sub-surface conditions, a geotechnical assessment of the development and landslip risk assessment, site mapping/plan, geological section and provides recommendations for construction.

The investigation and reporting were undertaken as per the Tender P19-274, Dated: 26th July 2019.

The investigation comprised:

a) A detailed geotechnical inspection and mapping of the site and adjacent properties by a Geotechnical Engineer.
b) DBYD plan request and onsite service location by accredited contractor.

c) Drilling of two hand auger boreholes along with Dynamic Cone Penetrometer (DCP) testing to investigate the subsurface geology, depth to bedrock and identification of ground water conditions.

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

- Architectural drawings by Watershed Design, Drawing No. DA02 to DA14, Issue: E, Dated: 16/12/2019.
- Survey Plan by C.M.S Surveyors Pty Ltd, Drawing Name: 18125detail, Dated: 31/10/2018.

2. SITE FEATURES:

2.1. Description:

The site is a rhomboid shaped block located on the low north side of Bower Street. It has front south and rear north boundaries of 15.40m and side east and west boundaries of 45.72m as referenced from the provided survey plan.

An aerial view of the site and its surrounds is provided below in Photograph 1, as sourced from NSW Government Six Map spatial data system.

Photograph: 1 – Aerial photo of site and surrounds

The site contains a three and four storey rendered apartment building with two double garages along the front boundary, a swimming pool and a lawn at the rear north. The front view of the site is shown in Photograph: 2. The apartment building and the garage in the east appear in good condition with no signs of cracking or settlement on their external walls. The garage in the west was damaged by a car crash which resulted in partial wall fail and cracks on the walls.

Photograph 2: Front view of the site, facing south

The ground surface of the site displays a gentle dip towards the north from a high of approximately RL26.20 at the southeast corner of the site to a low of approximately RL17.00 near the north boundary.

A sandstone outcrop/cliff up to 5.00m in height is located on the south side of Bower Street, as shown in Photograph: 3. The site appears to have been excavated into sandstone bedrock to accommodate the existing garage and ground floor level. Sandstone has also been excavated along the east boundary as shown in Photograph: 4 & 5. It appears that the east boundary wall is founded directly off bedrock of low to medium strength.

Photograph 3: Sandstone outcrop/cliff on the south side of Bower Street

Photograph 4: Sandstone excavation along east boundary, south portion

Photograph 5: Sandstone excavation along east boundary, middle portion

The neighbouring property to the west (No. 68) contains a two and three storey rendered house with a rendered garage and studio located at the southwest corner of the property. The building structures appear to be recently built/renovated and in good condition with no signs of cracking or settlement on the external walls. The house building is located within 0.80m of the common boundary, whilst the garage is located approximately 7.50m off the common boundary with on grade driveway access through the south-east corner from the street. The ground surface level of the property could not be confirmed due to the high fence wall. However, it is level with the site at the front south boundary and appears to drop to Ò1.00m below at the rear of the site garage floor.

The neighbouring property to the east (No. 64) contains a two and three storey rendered house with a rendered garage located at the front centre of the property with entry terrace and garden at the south-west corner adjacent to the site garage. The building structures appear to be recently built/renovated and in good condition with no signs of cracking or settlement on the external walls. The house is located within 1.30m of the common boundary, whilst the garage is located approximately 3.20m off the common boundary. The ground surface level of the property could not be confirmed due to the high fence wall. However, the garden and terrace adjacent to the site are at similar level to the garage.

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

2.2. Geology:

Reference to the Sydney 1: 100,000 Geological Series sheet (9130) indicates that the site is underlain by Hawkesbury Sandstone (Rh) which is of Triassic Age. The rock unit typically comprises medium to coarse grained quartz sandstone with minor lenses of shale and laminite. This rock unit was identified in surface exposures within and adjacent to 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 that consist of sandstone terraces and cliffs in part covered with sandy colluvium. The terraced areas often contain thin sandy clay to clayey sand residual soil profiles with intervening rock (ledge) outcrops. The outline of the cliff areas are often rectilinear in plan view, controlled by large bed thickness and wide spaced near vertical joint patterns. The dominant defect orientations are sub-horizontal to gently west dipping bedding and subvertical south-east and north-east striking joints. Many cliff areas are undercut by differential weathering along sub-horizontal to gently west dipping bedding defects or weaker sandstone/siltstone/shale horizons. Slopes are often steep (15º to 23°) and are randomly covered by sandstone boulders.

3. FIELD WORK:

3.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site and adjacent properties on the 29th August 2019 by a Geotechnical Engineer. It included a photographic record of site conditions with examination of existing structures and surrounding conditions. It also included the drilling of two auger boreholes (BH1 and BH2) using hand tools, due to limited access, to investigate sub-surface geology.

Dynamic Cone Penetrometer testing was carried out from ground surface adjacent to the boreholes and at a selected location, in accordance with AS1289.6.3.2 ï 1997, ñDetermination of the penetration resistance of a soil ï 9kg Dynamic Cone Penetrometerò to estimate near surface soil conditions and depth to bedrock.

Explanatory notes are included in Appendix: 1. Mapping information and test locations are shown on Figure: 1, along with detailed log sheets in Appendix: 2. A geological model/section is provided as Figure: 2, Appendix: 2. Landslip risk assessments are included in Appendix 3 along with explanatory notes in Appendix 4.

3.2. Field Testing:

BH1 was drilled approximately at the centre of the front south boundary with refusal encountered on interpreted sandstone bedrock at 0.45m depth. BH2 was drilled within the garden bed in front of the apartment building with refusal encountered on interpreted sandstone bedrock at 0.55m depth. DCP testing encountered refusal at depths from 0.26m (DCP1b) to 1.02m (DCP1).

Based on the borehole logs and DCP test results, the sub-surface conditions at the project site can be classified as follows:

- **TOPSOIL/FILL –** this layer was encountered at both test locations to depths varying from 0.45m (BH1) to 0.55m (BH2). It is classified as dark grey, fine to medium grained silty sand.
- **SANDSTONE BEDROCK** ï based on the DCP testing results, sandstone bedrock of at least very low strength is located at varying depths from 0.26m (DCP1b) to 1.02m (DCP1). Outcrops within the site and adjacent land indicate the bedrock will grade to low to medium strength over shallow depth.

A freestanding ground water table or significant signs of water seepage were not identified within the boreholes or outcrops.

4. COMMENTS:

4.1. Geotechnical Assessment:

The site investigation identified the presence of shallow sandy topsoil/fill (Ò0.55m) directly overlying sandstone bedrock. It appears that the sandstone bedrock underlying the site was excavated to accommodate the existing garages and apartment building and is of low to medium strength from shallow depth. A ground water table or significant signs of water seepage were not identified during the investigation.

The proposed works involve the demolition of two existing double garages and construction of two new double garages with a lower ground storage level below and a new entry foyer to the apartments. The works will require excavations of up to approximately 3.0m depth which will extend to the front south boundary and within 0.90m of the side east and west boundaries. The excavation will reduce to nil towards the north to the front of the apartment building.

Based on the investigation results, the proposed excavation is anticipated to encounter granular topsoil to 0.55m depth, under which sandstone bedrock of at least very low strength, rapidly grading to low to medium strength is anticipated with a potential for high strength bands.

Whilst excavation of soils and very low to low strength bedrock can be completed using conventional bucket systems with ripper. The excavation of medium to high strength rock will require the use of rock excavation equipment. Rock excavation equipment can produce ground vibrations of a level which can potentially cause damage to neighbouring structures. Therefore selection of suitable equipment and a sensible methodology are critical. The need for full time vibration monitoring will be determined based upon the type of rock excavation equipment proposed for use, and Crozier Geotechnical Consultants should be consulted for assessment prior to its use. It is recommended that a rock saw and small (Ò250kg) rock hammers be proposed for use at this site.

The proposed excavation will extend to the front south boundary and within 0.90m of the east and west side boundaries. The recommended safe temporary batter slopes provided in Section 4.3.2 are marginably achievable along the front south boundary and protection of the front boundary and footpath stability will need to be considered during initial excavation works. Where safe batters cannot be constructed, support prior to or directly following excavation will be required. it appears likely that some form of minor temporary support (i.e. sandbags) may be required to support the soils at the front boundary. The boundary wall along the east boundary appears to found off sandstone bedrock of at least low strength, therefore, safe batters are not required. The boundary wall along the west boundary is also expected to found off sandstone bedrock. However, this has to be confirmed prior to bulk excavation.

The exact strength of the bedrock that will be encountered during the excavation is unconfirmed. Where exposed, low to medium strength bedrock can be excavated at steep (0.25H:1.0V) to vertical batter slopes, provided it is unfractured by the excavation works and does not contain unfavorable defects. Where these are encountered then support systems (i.e. rock bolts/shotcrete) can be implemented as excavation works progress. As such, geotechnical inspection of the site following demolition and removal of topsoil to expose the bedrock surface and at regular intervals during rock excavation is recommended.

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 tools. This test equipment provides limited data from small isolated test points across the entire site with limited penetration into rock, therefore some minor variation to the interpreted sub-surface conditions is possible, especially between test locations. However provided the recommendations of this report are implemented in the design and construction phases the proposed development is considered suitable for the site.

4.2. Site Specific Risk Assessment:

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

A. Landslip (rock slide $<3m^3$) from excavation through rock extending to front boundary and within 0.90m of side boundaries
B. Landslip (earth slide $<1m^3$) from excavation through soils adjacent to front boundary and within 0.90m of side boundaries

A qualitative assessment of risk to life and property related to this hazard is presented in Table A and B, Appendix: 3, 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: 4.

The **Risk to Life** from **Hazard A and B** was estimated to be up to **2.60 x 10^{-7}** for a single person, whilst the **Risk to Property** was considered to be **'Low'** in all situations.

The assessments were based on excavations with no support or planning. The assessments were also made using ground conditions anticipated in adjacent properties. Provided the recommendations of this report are implemented including regular detailed geotechnical mapping of the excavation and installation of determined support systems in timely manner the likelihood of any failure becomes óRareô and as such the consequences and risk further reduce within óAcceptableô levels when assessed against the criteria of the AGS. As such the project is considered suitable for the site provided the recommendations of this report are implemented.

4.3. Design & Construction Recommendations:

Design and construction recommendations are tabulated below:

4.3.1. New Footings:	
Site Classification as per AS2870 ï 2011 for new footing design	Class óAô for footings founded on bedrock at base of excavation
Type of Footing	Strip/pad or slab at base of excavation, pads/strip external to the excavation
Maximum Allowable Bearing Capacity	- Weathered, VLS Sandstone: 700kPa - Weathered LS Sandstone: 1000kPa* - Weathered MS Sandstone: 2000kPa*
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	B_e ï rock site
Remarks: * Subject to confirmation by geotechnical professional including further investigation if required. All permanent structure footings should be founded off bedrock of similar strength to prevent differential settlement unless designed for by the structural engineer. All new footings must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity and the in-situ nature of the founding strata. This is mandatory to allow them to be ócertifiedô at the end of the project.	

4.3.2. Excavation:	
Depth of Excavation	Up to 3.00m depth
Distance of Excavation to Neighbouring Properties/structures	No. 68 Bower Street ï 0.90m to boundary, garage another 7.50m No. 64 Bower Street ï 0.90m to boundary, garage another 3.20m Road reserve ï on boundary
Type of Material to be Excavated	Topsoil/fill up to 0.55m depth

	Interpreted VLS ï LS/MS bedrock below 0.26m to 1.02m depth

Guidelines for <u>unsurcharged</u> batter slopes are tabulated below:

Material	Safe Batter Slope (H:V)	
	Short Term/Temporary	Long Term/Permanent
Granular topsoil/fill	1.5:1	2:1
Very low to low strength sandstone bedrock or fractured bedrock	0.5:1*	0.75:1*
Medium strength sandstone bedrock, defect free	Vertical*	Vertical*

Remarks:

*Dependent on assessment by geotechnical engineer.

Seepage at the bedrock surface or along defects in the soil/rock can also reduce the stability of batter slopes and invoke the need to implement additional support measures. Where safe batter slopes are not implemented the stability of the excavation cannot be guaranteed until the installation of permanent support measures. This should also be considered with respect to safe working conditions.

Equipment for Excavation	Topsoil and ELS rock	Excavator with bucket
	VLS bedrock	Excavator with bucket and ripper
	LS ï MS bedrock	Rock hammer and saw

ELS ï extremely low strength, VLS ï very low strength, LS ï low strength, MS ï medium strength

Remarks:

Rock sawing of the hard rock excavation perimeters is recommended as it has several advantages. It often reduces the need for rock bolting as the cut faces generally remain more stable and require a lower level of rock support than hammer cut excavations, ground vibrations from rock saws are minimal and the saw cuts will provide a slight increase in buffer distance for use of rock hammers. It also reduces deflection across boundary of detached sections of bedrock near surface.

Based on previous testing of ground vibrations created by various rock excavation equipment within medium strength Hawkesbury Sandstone bedrock, to achieve the specified low level of vibration the below hammer weights and buffer distances are required:

Maximum Hammer Weight	Required Buffer Distance from Structure
300kg	2.00m
400kg	3.00m
600kg	6.00m
Ó1 tonne	20.00m

It is recommended that smaller scale hammers (<250kg) be used due to the proximity of the excavation to the neighbouring properties Onsite calibration will provide accurate vibration levels to the site specific

conditions and will generally allow for larger excavation machinery or smaller buffers to be used. Inspection of proposed equipment and review of dilapidation surveys and excavation location will determine the need for calibration and full time monitoring.	
Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	Neighbouring residential dwellings = 5mm/s Road Reserve = 5mm/s Services = 3mm/s
Vibration Calibration Tests Required	If Ó250kg rock hammer proposed for use
Full time vibration Monitoring Required	Pending proposed equipment and vibration calibration testing results
Geotechnical Inspection Requirement	Yes, recommended that these inspections be undertaken as per below mentioned sequence: • Following demolition and removal of topsoil to expose the bedrock surface, and at 1.50m depth intervals within bedrock • Where unexpected ground conditions are identified, or any other concerns are held. • At completion of the excavation • Following footing excavations to confirm founding material strength
Dilapidation Surveys Requirement	Recommended on neighbouring structures or parts thereof within 5m of the excavation perimeter prior to site work to allow assessment of the recommended vibration limit and protect the client against spurious claims of damage.
Remarks: 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, whilst any groundwater seepage must be controlled within the excavation and prevented from ponding or saturating slopes/batters.	

4.3.3. Retaining Structures:	
Required	New retaining structures/excavation support wall will be required as part of the proposed development
Types	Temporary (sandbags/blocks). steel reinforced concrete/ concrete block walls where excavation stability can be ensured during excavation and construction phases, designed in accordance with Australian Standards AS4678-2002 Earth

	Retaining Structures.
Parameters for calculating pressures acting on retaining walls for the materials likely to be retained:	

Material	Unit Weight (kN/m^3)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
			Active (Ka)	At Rest (K_0)	
Topsoil	18	$\phi' = 28°$	0.35	0.52	N/A
LS or fractured bedrock	23	$\phi' = 40°$	0.10	0.15	200kPa

Remarks:

Where medium strength, defect free sandstone bedrock is exposed in the excavation, then it will be self supporting and does not require retention. This will require geotechnical inspection and confirmation during site works.

In suggesting these parameters, it is assumed that the retaining walls will be fully drained with suitable subsoil drains provided to allow release of groundwater seepage. If this is not done, then the walls should be designed to support full hydrostatic pressure in addition to pressures due to the soil backfill. It is suggested that the retaining walls should be back filled with free-draining granular material (preferably not recycled concrete) which is only lightly compacted in order to minimize horizontal stresses.

Retaining structures near site boundaries or existing structures should be designed with the use of at rest (K_0) earth pressure coefficients and incorporate surcharge loading to reduce the risk of movement in the excavation support and resulting surface movement in adjoining areas. Backfilled retaining walls within the site, away from site boundaries or existing structures, that may deflect can utilize active earth pressure coefficients (Ka).

4.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		No
Excavation likely to intersect	Water Table	No
	Seepage	Minor (<2L /min) estimated
Site Location and Topography		Low north side of the road, within gently north sloping topography
Impact of development on local hydrogeology		Negligible
Onsite Stormwater Disposal		Not required or recommended

Remarks:

As the excavation faces are expected to encounter some seepage, an excavation trench should be installed at the base of excavation cuts to below floor slab levels to reduce the risk of resulting dampness issues. Trenches, as well as all new building gutters, down pipes and stormwater intercept trenches should be connected to a stormwater system designed by a Hydraulic Engineer which preferably discharges to the Councilôs stormwater system off site.

4.4. Conditions Relating to Design and Construction Monitoring:

To allow certification at the completion of the project it will be necessary for Crozier Geotechnical Consultants to:

1. Review and approve the structural design drawings for compliance with the recommendations of this report prior to construction,
2. Inspection of site and works as per Section 4.3.2 of this report,
3. Inspect all new footings and earthworks to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and the stability prior to the placement of steel or concrete,
4. Inspect completed works to ensure construction activity has not created any new hazards and that all retention and stormwater control systems are completed.

Crozier Geotechnical Consultants cannot provide certification for the Occupation Certificate if it has not been called to site to undertake the required inspections.

5. CONCLUSION:

The site investigation identified the presence of granular topsoil to a maximum depth of 0.55m, underlain by sandstone bedrock that appears likely to be generally low to medium strength from adjacent outcrops within the area of the proposed development.

The proposed works involve the demolition of two existing double garages and construction of two new double garages with a lower ground storage level and a new entry foyer to the apartments. The works will require excavations of up to approximately 3.0m depth which will extend to the front south boundary and within 0.90m of the side east and west side boundaries.

The excavation is expected to intersect granular topsoil below the existing ground surface. Underlying the soil, variably weathered sandstone of interpreted very low to low strength, potentially grading quickly to medium strength is anticipated. Considering the identified shallow bedrock depth and footings of boundary walls founded on bedrock, it appears the recommended safe temporary batter slopes are generally achievable; however some temporary support may be required. The excavation of bedrock may also be self supporting. As such, geotechnical inspections are required to determine excavation stability and the need for support systems.

The proposed excavation is expected to extend below the bedrock surface level. As such, it will be necessary to assess proposed rock excavation equipment (e.g. rock hammers) prior to use to determine whether vibration monitoring will be required during excavation.

The risks associated with the proposed development can be maintained within óAcceptableô levels with negligible impact to the neighbouring properties or structures provided the recommendations of this report and any future geotechnical directive are implemented. As such the site is considered suitable for the proposed construction works provided that the recommendations outlined in this report are followed.

Prepared by:	Reviewed by:
Jun Yan	Troy Crozier
Geotechnical Engineer	Principal
	MEng, BSc, Dip. Civ. Eng
	MAIG, RPGeo ï Geotechnical and Engineering
	Registration No.: 10197

6. REFERENCES:

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2. Woollahra Municipal Councilôs óGuidelines for Preparation of Geotechnical and Hydrogeological Reportsô Annexure 3, September 2002.
3. 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.
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Appendix 1

CROZIER
GEOTECHNICAL CONSULTANTS

Crozier Geotechnical Consultants ABN: 96 113 453 624
Unit 12/ 42-46 Wattle Road Phone: (02) 9939 1882
Brookvale NSW 2100 Email: info@croziergeotech.com.au
Crozier Geotechnical Consultants, a division of PJC Geo-Engineering Pty Ltd

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

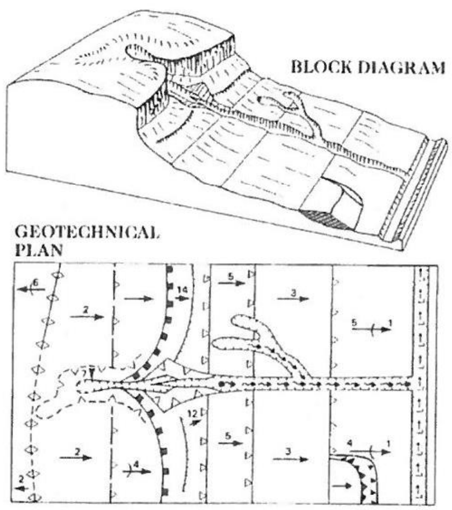
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

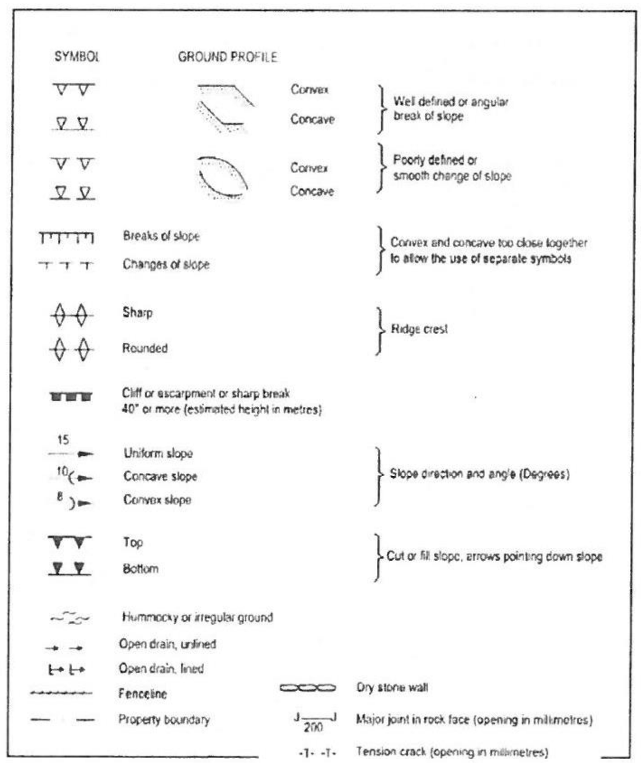
Reproduction of Information for Contractual Purposes

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

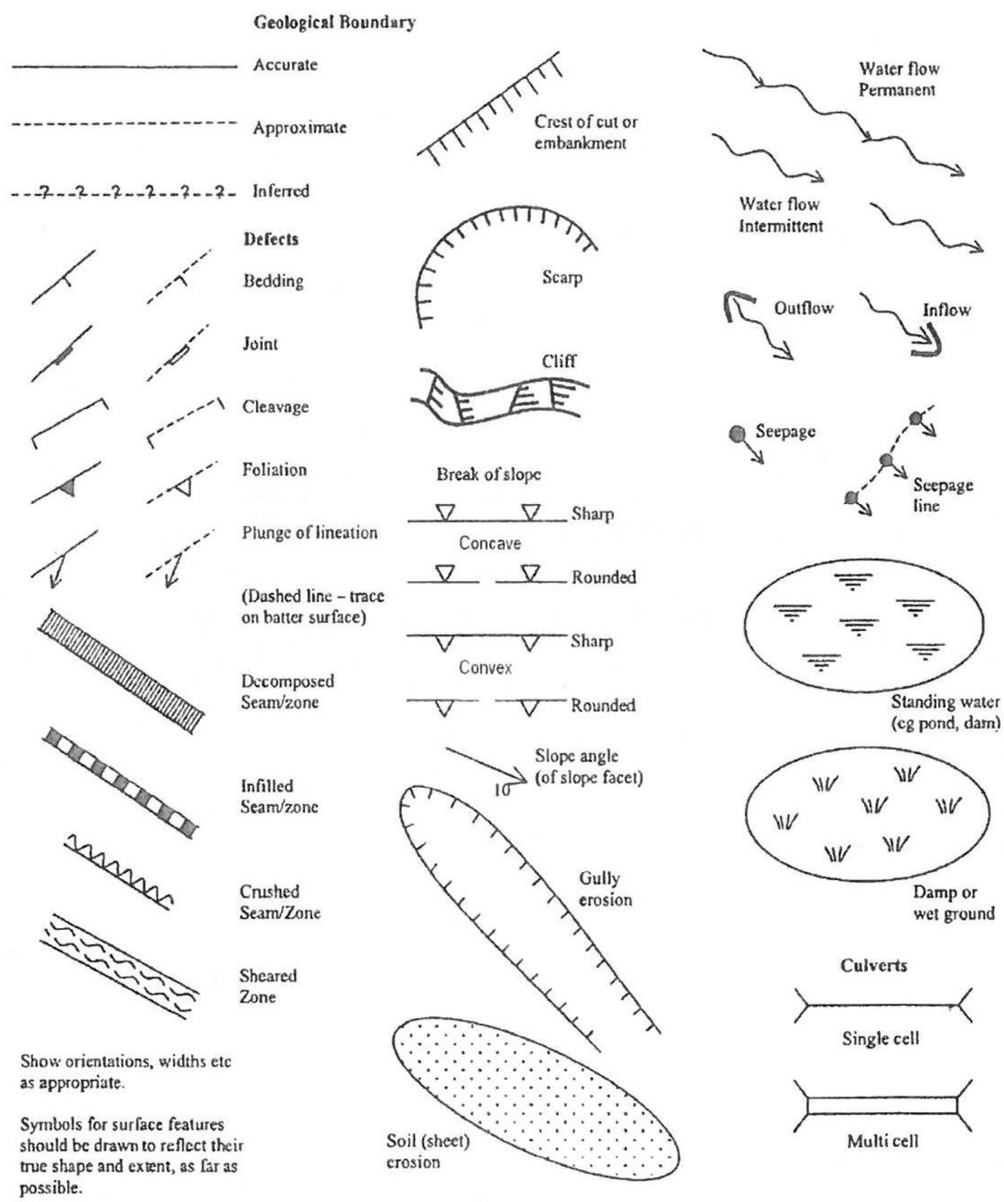
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.





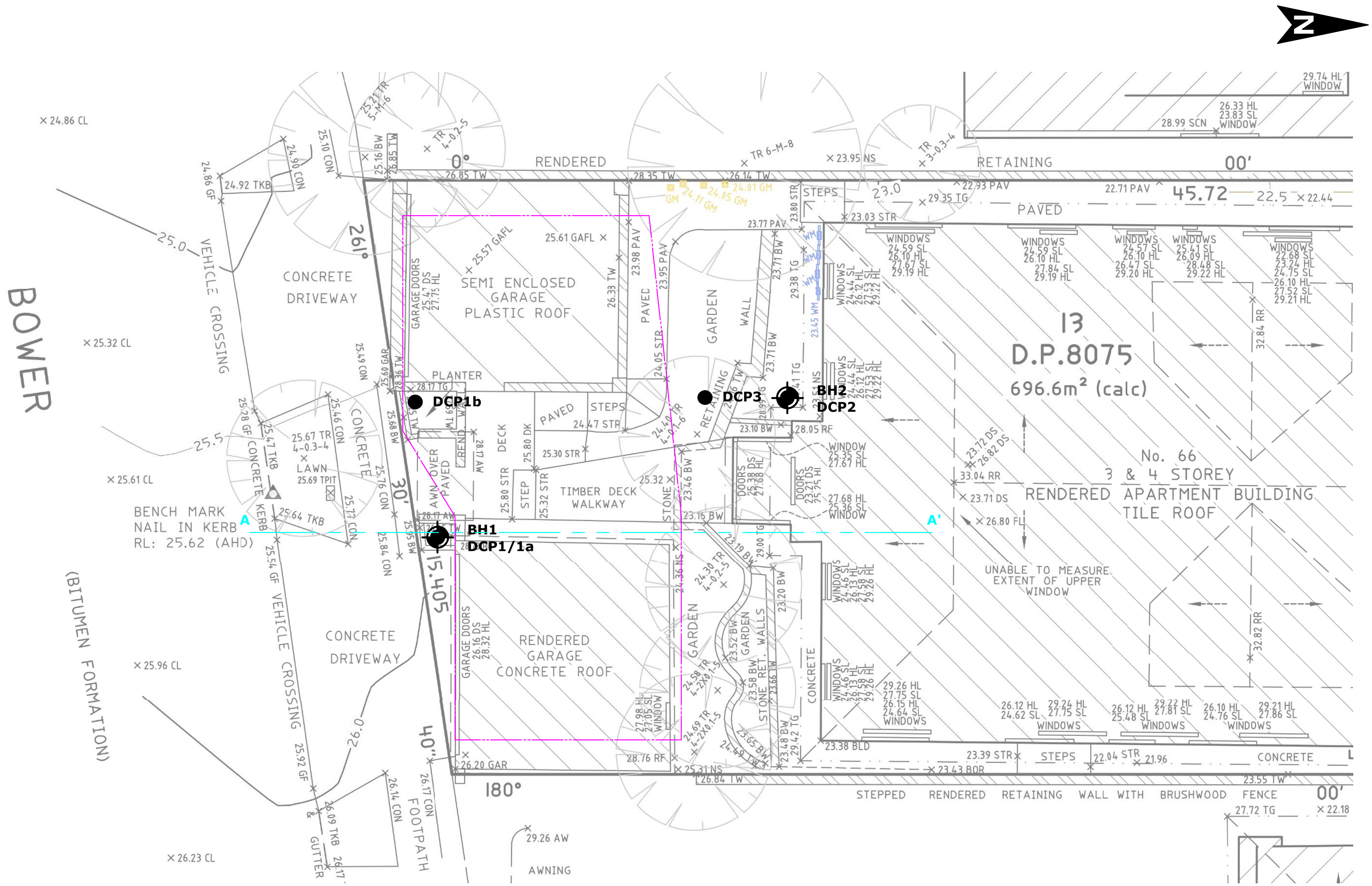
Example of Mapping Symbols
(after V Gardiner & R V Dackombe (1983).Geomorphological Field Manual. George Allen & Unwin).

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



0 1 2 3 4 5

SCALE: 1:100

VL - Very Loose	VS - Very Soft	ELS - Extremely Low Strength	EW - Extremely Weathered	fg - Fine Grained
L - Loose	S - Soft	VLS - Very Low Strength	HW - Highly Weathered	mg - Medium Grained
MD - Medium Dense	F - Firm	LS - Low Strength	DW - Distinctly Weathered	cg - Coarse Grained
D - Dense	St - Stiff	MS - Medium Strength	MW - Moderately Weathered	MAS - Massive
VD - Very Dense	VSt - Very Stiff	HS - High Strength	SW - Slightly Weathered	BD - Bedded
	H - Hard	VHS - Very High Strength	FR - Fresh	OC - Outcrop

SITE PLAN & TEST LOCATIONS FIGURE 1.

LEGEND

A — A' CROSS-SECTION REFERENCE LINE

PROPOSED WORK

BH DCP AUGER / DYNAMIC CONE PENETROMETER LOCATION

CROZIER
GEOTECHNICAL CONSULTANTS

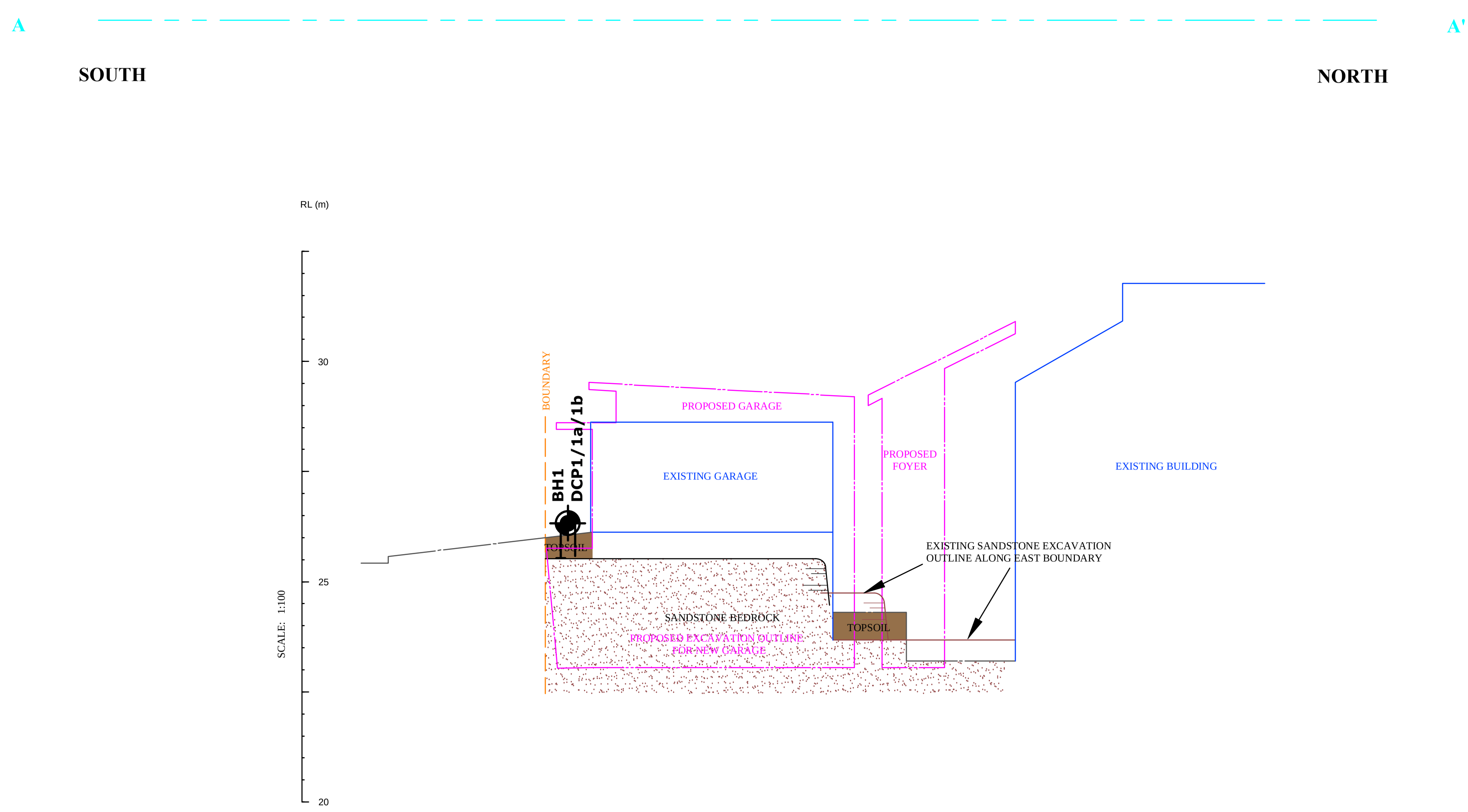
Crozier Geotechnical
Unit 12, 42-46 Wattle Road
Brookvale NSW 2100
ABN: 96 113 453 624
Phone: (02) 9939 1882
Fax: (02) 9939 1883
Crozier Geotechnical is a division of PJC Geo-Engineering Pty Ltd

SCALE: 1:100 @ A3
DRAWING: FIGURE 1
DATE: 03/09/2019

APPROVED BY: TMC
DRAWN BY: JY
PROJECT: 2019-138

PREPARED FOR:
Vahuvu Pty Ltd

ADDRESS:
66 Bower Street, Manly



0 1 2 3 4 5 6 7 8 9 10

SCALE: 1:200

VL - Very Loose	VS - Very Soft	ELS - Extremely Low Strength	EW - Extremely Weathered	fg - Fine Grained
L - Loose	S - Soft	VLS - Very Low Strength	HW - Highly Weathered	mg - Medium Grained
MD - Medium Dense	F - Firm	LS - Low Strength	DW - Distinctly Weathered	cg - Coarse Grained
D - Dense	St - Stiff	MS - Medium Strength	MW - Moderately Weathered	MAS - Massive
VD - Very Dense	VSt - Very Stiff	HS - High Strength	SW - Slightly Weathered	BD - Bedded
	H - Hard	VHS - Very High Strength	FR - Fresh	OC - Outcrop

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

GEOLOGICAL MODEL FIGURE 2.

LEGEND

A — A' CROSS-SECTION REFERENCE LINE

BH DCP AUGER / DYNAMIC CONE PENETROMETER LOCATION

PROPERTY BOUNDARY

SOIL/FILL

SANDSTONE BEDROCK

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

SCALE: 1:100 @ A3
DRAWING: FIGURE 2
DATE: 03/09/2019
APPROVED BY: TMC
DRAWN BY: JY
PROJECT: 2019-138

PREPARED FOR:
Vahuvu Pty Ltd

ADDRESS:
66 Bower Street, Manly

BOREHOLE LOG

CLIENT:	**Vahuvu Pty Ltd**	**DATE:**	29/08/2019	**BORE No.:**	1
PROJECT:	**Alterations and Additions**	**PROJECT No.:**	2019-138	**SHEET:**	1 of 1
LOCATION:	**66 Bower Street, Manly**	**SURFACE LEVEL:**	RL26.15		

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
			Type	Tests	Type	Results
0.00		TOPSOIL/FILL: Dark grey, fine to medium granied, silty sand				
0.45		Auger refusal at 0.45m on interpreted sandstone bedrock				
1.00						
2.00						

RIG: NA

DRILLER: TJ

METHOD: Hand Auger

LOGGED: JY

GROUND WATER OBSERVATIONS: No freestanding groundwater found

REMARKS:

CHECKED:

BOREHOLE LOG

CLIENT: Vahuvu Pty Ltd | **DATE:** 29/08/2019 | **BORE No.:** 2

PROJECT: Alterations and Additions | **PROJECT No.:** 2019-138 | **SHEET:** 1 of 1

LOCATION: 66 Bower Street, Manly | **SURFACE LEVEL:** RL23.70

Depth (m)	Classification	Description of Strata PRIMARY SOIL - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
			Type	Tests	Type	Results
0.00		TOPSOIL/FILL: Dark grey, fine to medium granied, silty sand				
0.55		Auger refusal at 0.55m on interpreted sandstone bedrock				
1.00						
2.00						

RIG: NA

DRILLER: TJ

METHOD: Hand Auger

LOGGED: JY

GROUND WATER OBSERVATIONS: No freestanding groundwater found

REMARKS:

CHECKED:

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Vahuvu Pty Ltd **DATE:** 29/08/2019

PROJECT: Alterations and additions **PROJECT No.:** 2019-138

LOCATION: 66 Bower Street, Manly **SHEET:** 1 of 1

	Test Location							
Depth (m)	DCP1	DCP1a	DCP1b	DCP2	DCP3			
0.00 - 0.15	0	0	1	1	1			
0.15 - 0.30	0	0	7 (B) ref at 0.26m	2	1			
0.30 - 0.45	0	6 (B) ref at 0.45m		9	4			
0.45 - 0.60	12			5	5 (B) ref at 0.57m			
0.60 - 0.75	16			12 (B) ref at 0.62m				
0.75 - 0.90	5							
0.90 - 1.05	30 (B) ref at 1.02m							
1.05 - 1.20								
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								
3.00 - 3.15								
3.15 - 3.30								
3.30 - 3.45								
3.45 - 3.60								
3.60 - 3.75								
3.75 - 3.90								
3.90 - 4.05								

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

Appendix 3

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 (rock slide <3m³) from excavation through rock extending to front boundary and within 0.90m of side boundaries		Excavation up to 2.45m depth through bedrock **Condition A**: Insufficient retention - **Condition B**: Geotechnical inspection, engineer designed and implemented support Condition A	Condition B	a) Boundary wall 0.90m off edge of excavation, impact 10% of the wall b) Boundary wall 0.90m off edge of excavation, impact 10% of the wall c) Footpath on edge of excavation, impact 10% of footpath		a) Person in terraces and driveway 1hrs/day avge. b) Person in terraces 0.5hr/day avge. c) Person in footpath 1hrs/day avge.	a) Possible to not evacuate b) Possible to not evacuate c) Possible to not evacuate	a) Person in open space, partly buried b) Person in open space, partly buried c) Person in open space, minor damage	**Condition A**	**Condition B**
		a) front of neighbouring property (No. 68)	**Possible** 0.001	**Rare** 0.00001	**Prob. of Impact** 0.25	**Impacted** 0.10	0.0417	0.5	0.50	**2.60E-07**	**2.60E-09**
		b) front of neighbouring property (No. 64)	**Possible** 0.001	**Rare** 0.00001	0.25	0.10	0.0208	0.5	0.50	**1.30E-07**	**1.30E-09**
		c) footpath	**Possible** 0.001	**Rare** 0.00001	1.00	0.10	0.0417	0.5	0.10	**2.08E-07**	**2.08E-09**
B	Landslip (earth slide <1m³) from excavation through soils adjacent to front boundary and within 0.90m of side boundaries		Excavation up to 0.55m depth through granular topsoil		a) Footpath on edge of excavation, impact 10% of footpath		a) person in footpath 1hrs/day	a) Possible to not evacuate	a) Person in open space, minor damage only/not buried		
		a) footpath	**Possible** 0.001	**Rare** 0.00001	**Prob. of Impact** 0.50	**Impacted** 0.10	0.0417	0.5	0.10	**1.04E-07**	**1.04E-09**

* hazards considered in current condition and without pool and retention systems proposed in design
* likelihood of occurrence for design life of 100 years
* Spatial Impact - Probaility of Impact referes to slide impacting structure/area expressed as a % (1.00 = 100% probability of slide impacting area if it occurs), Impacted refers to % of area/strucure impacted if slide occurred
* neighbouring houses considered for bedroom impact unless specified
* considered for person most at risk
* considered for adjacent premises/buildings founded via shallow footings unless indicated
* evacuation scale from Almost Certain to<u>not</u> evacuate (1.0), Likely (0.75), Possible (0.5), Unlikely (0.25), Rare to not evacuate (0.01). Based on likelihood of person knwoing 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 (rock slide $<3m^3$) from excavation through rock extending to front boundary and within 0.90m of side boundaries	a) front of neighbouring property (No. 68)	Unlikely	The event might occur under very adverse circumstances 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.	Low
		b) front of neighbouring property (No. 64)	Unlikely	The event might occur under very adverse circumstances 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.	Low
		c) footpath	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Low
B	Landslip (earth slide $<1m^3$) from excavation through soils adjacent to front boundary and within 0.90m of side boundaries	a) footpath	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Low

* hazards considered in current condition, without pool and retention systems proposed in designing.

* 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%.

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 4

APPENDIX A

DEFINITION OF TERMS

INTERNATIONAL UNION OF GEOLOGICAL SCIENCES WORKING GROUP ON LANDSLIDES, COMMITTEE ON RISK ASSESSMENT

Risk – A measure of the probability and severity of an adverse effect to health, property or the environment. Risk is often estimated by the product of probability x consequences. However, a more general interpretation of risk involves a comparison of the probability and consequences in a non-product form.

Hazard – A condition with the potential for causing an undesirable consequence (*the landslide*). The description of landslide hazard should include the location, volume (or area), classification and velocity of the potential landslides and any resultant detached material, and the likelihood of their occurrence within a given period of time.

Elements at Risk – Meaning the population, buildings and engineering works, economic activities, public services utilities, infrastructure and environmental features in the area potentially affected by landslides.

Probability – The likelihood of a specific outcome, measured by the ratio of specific outcomes to the total number of possible outcomes. Probability is expressed as a number between 0 and 1, with 0 indicating an impossible outcome, and 1 indicating that an outcome is certain.

Frequency – A measure of likelihood expressed as the number of occurrences of an event in a given time. See also Likelihood and Probability.

Likelihood – used as a qualitative description of probability or frequency.

Temporal Probability – The probability that the element at risk is in the area affected by the landsliding, at the time of the landslide.

Vulnerability – The degree of loss to a given element or set of elements within the area affected by the landslide hazard. It is expressed on a scale of 0 (no loss) to 1 (total loss). For property, the loss will be the value of the damage relative to the value of the property; for persons, it will be the probability that a particular life (the element at risk) will be lost, given the person(s) is affected by the landslide.

Consequence – The outcomes or potential outcomes arising from the occurrence of a landslide expressed qualitatively or quantitatively, in terms of loss, disadvantage or gain, damage, injury or loss of life.

Risk Analysis – The use of available information to estimate the risk to individuals or populations, property, or the environment, from hazards. Risk analyses generally contain the following steps: scope definition, hazard identification, and risk estimation.

Risk Estimation – The process used to produce a measure of the level of health, property, or environmental risks being analysed. Risk estimation contains the following steps: frequency analysis, consequence analysis, and their integration.

Risk Evaluation – The stage at which values and judgements enter the decision process, explicitly or implicitly, by including consideration of the importance of the estimated risks and the associated social, environmental, and economic consequences, in order to identify a range of alternatives for managing the risks.

Risk Assessment – The process of risk analysis and risk evaluation.

Risk Control or Risk Treatment – The process of decision making for managing risk, and the implementation, or enforcement of risk mitigation measures and the re-evaluation of its effectiveness from time to time, using the results of risk assessment as one input.

Risk Management – The complete process of risk assessment and risk control (*or risk treatment*).

Individual Risk – The risk of fatality or injury to any identifiable (named) individual who lives within the zone impacted by the landslide; or who follows a particular pattern of life that might subject him or her to the consequences of the landslide.

Societal Risk – The risk of multiple fatalities or injuries in society as a whole: one where society would have to carry the burden of a landslide causing a number of deaths, injuries, financial, environmental, and other losses.

Acceptable Risk – A risk for which, for the purposes of life or work, we are prepared to accept as it is with no regard to its management. Society does not generally consider expenditure in further reducing such risks justifiable.

Tolerable Risk – A risk that society is willing to live with so as to secure certain net benefits in the confidence that it is being properly controlled, kept under review and further reduced as and when possible.

In some situations risk may be tolerated because the individuals at risk cannot afford to reduce risk even though they recognise it is not properly controlled.

Landslide Intensity – A set of spatially distributed parameters related to the destructive power of a landslide. The parameters may be described quantitatively or qualitatively and may include maximum movement velocity, total displacement, differential displacement, depth of the moving mass, peak discharge per unit width, kinetic energy per unit area.

Note: Reference should also be made to Figure 1 which shows the inter-relationship of many of these terms and the relevant portion of Landslide Risk Management.

APPENDIX C: LANDSLIDE RISK ASSESSMENT

QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

QUALITATIVE MEASURES OF LIKELIHOOD

Approximate Annual Probability		Implied Indicative Landslide Recurrence Interval		Description	Descriptor	Level
Indicative Value	Notional Boundary					
10^{-1}	$5x10^{-2}$	10 years	20 years	The event is expected to occur over the design life.	ALMOST CERTAIN	A
10^{-2}	$5x10^{-3}$	100 years	200 years	The event will probably occur under adverse conditions over the design life.	LIKELY	B
10^{-3}	$5x10^{-4}$	1000 years	2000 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10^{-4}	$5x10^{-5}$	10,000 years	20,000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10^{-5}	$5x10^{-6}$	100,000 years	200,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
10^{-6}		1,000,000 years		The event is inconceivable or fanciful over the design life.	BARELY CREDIBLE	F

Note: (1) The table should be used from left to right; use Approximate Annual Probability or Description to assign Descriptor, not *vice versa*.

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

Approximate Cost of Damage		Description	Descriptor	Level
Indicative Value	Notional Boundary			
200%	100%	Structure(s) completely destroyed and/or large scale damage requiring major engineering works for stabilisation. Could cause at least one adjacent property major consequence damage.	CATASTROPHIC	1
60%	40%	Extensive damage to most of structure, and/or extending beyond site boundaries requiring significant stabilisation works. Could cause at least one adjacent property medium consequence damage.	MAJOR	2
20%	10%	Moderate damage to some of structure, and/or significant part of site requiring large stabilisation works. Could cause at least one adjacent property minor consequence damage.	MEDIUM	3
5%	1%	Limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works.	MINOR	4
0.5%		Little damage. (Note for high probability event (Almost Certain), this category may be subdivided at a notional boundary of 0.1%. See Risk Matrix.)	INSIGNIFICANT	5

Notes: (2) The Approximate Cost of Damage is expressed as a percentage of market value, being the cost of the improved value of the unaffected property which includes the land plus the unaffected structures.

(3) The Approximate Cost is to be an estimate of the direct cost of the damage, such as the cost of reinstatement of the damaged portion of the property (land plus structures), stabilisation works required to render the site to tolerable risk level for the landslide which has occurred and professional design fees, and consequential costs such as legal fees, temporary accommodation. It does not include additional stabilisation works to address other landslides which may affect the property.

(4) The table should be used from left to right; use Approximate Cost of Damage or Description to assign Descriptor, not vice versa

APPENDIX C: – QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY (CONTINUED)

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

LIKELIHOOD		**CONSEQUENCES TO PROPERTY (With Indicative Approximate Cost of Damage)**				
	Indicative Value of Approximate Annual Probability	**1: CATASTROPHIC 200%**	**2: MAJOR 60%**	**3: MEDIUM 20%**	**4: MINOR 5%**	**5: INSIGNIFICANT 0.5%**
A – ALMOST CERTAIN	10^{-1}	VH	VH	VH	H	M or **L** (5)
B - LIKELY	10^{-2}	VH	VH	H	M	**L**
C - POSSIBLE	10^{-3}	VH	H	M	M	**VL**
D - UNLIKELY	10^{-4}	H	M	**L**	**L**	**VL**
E - RARE	10^{-5}	M	**L**	**L**	**VL**	**VL**
F - BARELY CREDIBLE	10^{-6}	**L**	**VL**	**VL**	**VL**	**VL**

Notes: (5) For Cell A5, may be subdivided such that a consequence of less than 0.1% is Low Risk.

(6) When considering a risk assessment it must be clearly stated whether it is for existing conditions or with risk control measures which may not be implemented at the current time.

RISK LEVEL IMPLICATIONS

Risk Level		**Example Implications** (7)
VH	VERY HIGH RISK	Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low; may be too expensive and not practical. Work likely to cost more than value of the property.
H	HIGH RISK	Unacceptable without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to Low. Work would cost a substantial sum in relation to the value of the property.
M	MODERATE RISK	May be tolerated in certain circumstances (subject to regulator's approval) but requires investigation, planning and implementation of treatment options to reduce the risk to Low. Treatment options to reduce to Low risk should be implemented as soon as practicable.
L	LOW RISK	Usually acceptable to regulators. Where treatment has been required to reduce the risk to this level, ongoing maintenance is required.
VL	VERY LOW RISK	Acceptable. Manage by normal slope maintenance procedures.

Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.