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GEOTECHNICAL REPORT

for

REMEDIATION DESIGN

at

60 ALEXANDER STREET, COLLAROY

Prepared For

SP81259

Project No.: 2020-097

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GEOTECHNICAL REPORT FOR REMEDIATION DESIGN
60 ALEXANDER STREET, COLLAROY, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for proposed retaining wall remediation/replacement works at 60 Alexander Street, Collaroy. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of Michael Kelett of MTK consulting on behalf of the owner Strata Plan SP81259.

It is understood that it is intended to remediate or replace a retaining wall (‘the retaining wall’) near the rear, south boundary of 60 Alexander Street (‘No.60’) which is displaying signs of distress and that geotechnical investigation is required to determine the possible options available for remediation/replacement. It was proposed in CGC Fee Proposal No. P20-161 Dated 1st April 2020 that the movement of the wall was likely due to one or a combination of the following:

1. Additional (unanticipated) surcharge on the wall via over-compacted fill, elevated water pressures or new loads added to the wall than originally designed for,
2. The original design was not sufficient to retain the height/load of material,
3. Inadequate foundation material has led to movement of the subgrade and rotation of the wall.

To investigate the cause of the movement it was proposed to undertake an intrusive field investigation and walkover assessment and provide a geotechnical report presenting the findings of the investigation. The field investigation was undertaken as proposed and comprised:

a) A detailed geotechnical inspection and mapping of the site by a Senior Engineering Geologist including a photographic record of site conditions.
b) Two hand auger boreholes, one hand excavated test pit and eight Dynamic Cone Penetrometer (DCP) tests to investigate the subsurface/retaining wall founding conditions.

The site is located within landslip risk Class ‘E’ within the Landslip Risk Map Sheet 009. A review of the preliminary checklist and the nature of the works identified that the Development Application (DA) involves works which exceed the preliminary assessment guidelines due to the height of the existing retaining wall. Therefore, a full geotechnical report in support of the DA will be required including a stability assessment of the proposed works. At the time of writing the proposed works (either remediation of replacement) had not been confirmed. Once the proposed remediation/replacement works are confirmed (and DA drawings supplied) this report will likely require revision prior to DA submission to fulfill Council requirements. However, for the purposes of the report the AGS stability risk assessment has been undertaken based on the existing conditions at the site i.e. where no remediation works are undertaken.

1.1 Proposed Development

Preliminary design drawings have not been provided however based on communications with the Structural Engineer for the development (MTK Consulting) it is understood that it is intended to remove the existing retaining wall as well as a portion of the supported fill and construct a new bored pile retaining wall. The replacement wall is understood to be proposed further north of the existing distressed wall location and within the rear garden of No.60 as the existing retaining structure is within the neighbouring property to the south.

2. THE SITE:

2.1. Site Description, Location and Topography:

The site is (defined herein as the retaining wall constructed near the south end of No.60 and garden) is located at the rear of 60 Alexander Street which is situated on the low south side of the road positioned above a nature reserve to the south. Based on survey drawings provided it appears that the retaining wall has been constructed approximately 0.6m into the existing nature reserve to the south (No.22 Homestead Avenue).

No.60 Alexander Street comprises a three storey rendered, concrete and brick residential structure which is understood to have undergone alterations and additions in the last 10-15 years including the addition of a lower level at the rear of the site residence, a new retaining wall and (presumably) placement of fill to create a new level rear garden at the rear. An aerial view of the site obtained from Google Earth is shown in Photograph 1.

Photograph 1: Aerial view of the site (outlined red) obtained from Google Earth

The rear garden of No.60 comprises a synthetic turf lawn between the retaining wall and the site dwelling which is accessed via a pathway running along the western side of the dwelling. Within the south west corner of No.60 a level concrete platform has been constructed which is approximately 1.5m above the garden level. It is supported by a sandstone block retaining which appears to be older than the distressed retaining wall and appears to be in good condition.

Concrete piers support the site dwelling and are understood to have been constructed as part of the previous alterations and additions to support the additional load and appear in good condition. A view of the rear garden is provided in Photograph 2.

Photograph 2: View of the rear garden of No.70 looking south east.

The distressed retaining wall is approximately 12.0m in length and approximately 0.4m in height above the neighbouring land to the south at the west end increasing to 2.5m in height at the east end. The wall is comprised of blockwork masonry with roughly finished cement mortar. The wall supports the rear garden of No.60 which comprises a synthetic grass lawn and extends to the main residential site structure located approximately 5.0m to the north. A metal fence approximately 2.0m in height extends along the length of the wall. A view of the retaining wall is provided in Photograph 3a and 3b.

Photograph 3a

Photograph 3b

View of the south side and east end of the retaining wall at the rear garden of No.60

A second retaining wall has been constructed on the boundary with the property to the east (No.58) and appears to be similar in construction to that of the distressed retaining wall and likely constructed at the same time by the same builder. The wall is up to approximately 1.0m in height increasing to 3.0m where it connects to the distressed retaining wall at the south end of No.60, however this wall appeared in good condition.

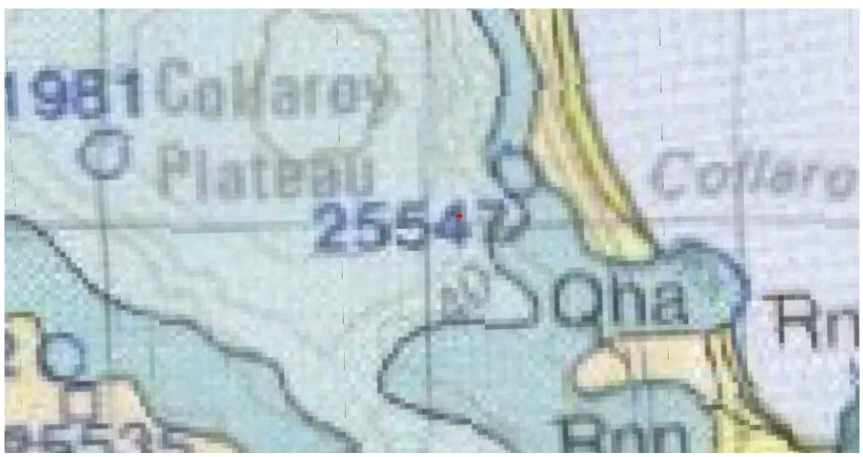
The neighbouring property to the east of the site, No.58 Alexander Street (ôNo.58ö), contains a brick and clad house approximately 8.0m from the common boundary. The ground surface level with this property is generally similar to that of No.60 however adjacent to the distressed retaining wall it is up to 3.0m below the level of the lawn.

The neighbouring property to the west of the site, No. 62 Alexander Street, (ôNo.62ö), contains a rendered house approximately 12.0m from the common boundary. The ground surface level with this property is similar to No.60 immediately adjacent to the boundary.

It is understood that an existing Sydney Water asset lies below the rear garden however details on depth or construction has not been confirmed. Based on information provided by the client and on DBYD information it is understood that the asset may comprise a vitreous clay pipe. An invert pipe level of 1.6m below a manhole cover near the south east corner of No.60 is shown on available plans which would indicate it may be below the base of the existing distressed retaining wall.

2.2. Geology:

Reference to the Sydney 1 : 100,000 Geological Series sheet indicates that the site is underlain by Hawkesbury Sandstone which is of Triassic Age. The rock unit typically comprises medium to coarse grained quartz sandstone with minor lenses of shale and laminite. An extract of the Sydney 1:100,000 series sheet is provided below with the site indicated in red.



Extract of Sydney 1:100 000 Geological Survey Sydney Sheet

3. FIELD WORK

3.1 Methods

The fieldwork comprised a geotechnical inspection of the site and a subsurface investigation which were both undertaken on 18th May 2020 and supervised/undertaken by a Senior Engineering Geologist.

The geotechnical inspection comprised an examination of existing structures, slopes and vegetation within and surrounding the site to assist in determination of retaining wall distress/potential stability issues. Photographs of relevant observations were taken at the time of inspection.

The sub-surface investigation comprised eight Dynamic Cone Penetrometer tests undertaken within the rear garden and within the area at the base of the distressed retaining wall. One auger borehole and one test pit were completed within the area near the base of the retaining wall to assess ground/retaining wall footing conditions. A second borehole was undertaken within the rear garden. The ground conditions restricted the capabilities of the hand auger to provide sufficient information, therefore additional DCP tests were undertaken than originally proposed. The investigation was limited to the use of hand tools due to site access limitations.

Dynamic Cone Penetrometer (DCP) testing was carried out in accordance with AS1289.6.3.2 ï 1997, ñDetermination of the penetration resistance of a soil ï 9kg Cone Penetrometerò to estimate ground conditions.

Explanatory notes are included in Appendix: 1. Mapping information and test locations are shown on Figure: 1, along with detailed Borehole/Test Pit Log sheets and Dynamic Penetrometer Test Sheet in Appendix: 2. A geological model/section is provided as Figure: 2, Appendix: 2.

3.2 Ground Conditions

For a description of the ground conditions encountered at the borehole/DCP test locations the relevant log sheets provided in Appendix 1 should be consulted however the subsurface conditions underlying the site can be broadly classified as follows:

- **Fill –** Fill was encountered in both the test pit and boreholes and has been interpreted to have been encountered at all DCP test locations. It appears that the fill is thickest under the existing garden and was encountered to at least 1.35m below ground surface level in DCP6. It comprised very loose sand underlain by what is thought to be building rubble which was observed where displacement of the wall had occurred at the south east corner of the site. It was noticeable that the depth to the interpreted building rubble increased progressively toward the east end of the wall from 0.60m (DCP7) to 1.35m (DCP6). Within DCP8, natural soils have been interpreted as being present from around 0.75m depth.

  Fill was also encountered to the south of the retaining wall and comprised a layer of topsoil under which, a discontinuous layer of concrete was encountered which appeared to be an óoverpourô from the original construction (see Section 3.2). Below the topsoil/concrete layer, building refuse comprising coarse brick/concrete etc. was encountered within BH1 however auger advancement was hindered by the nature of the fill and the base of this layer has been estimated to be between around 0.6m to 1.2m based on DCP1 to DCP4.

- **Clayey Sand/ Sandy Clay –** this layer was only observed in TP1 and comprised moist to wet sandy clay/clayey sand from approximately 0.50m below the ground surface level at the base of the wall. The test pit indicated that the base of the retaining wall footing may be at or around 0.6m depth below ground surface level. The results of DCP1 and DCP2 undertaken near TP1 indicated that at or below this depth zones of very weak soils/fill are present. Below around 1.35m to 1.65m the soils apparently became 'stronger'. It should be noted that it is feasible that the retaining wall has become detached from the footing however this could not be determined based on the level of investigation undertaken to date.

- **Bedrock –** Extremely to very low strength bedrock has been interpreted as being encountered in DCP1 to DCP4 at around 2.0m depth below the ground surface level on the south side of the wall.

A free-standing ground water table was not encountered however with soils with the base of TP1 appeared wet, and of higher moisture content than anticipated. No signs of ground water were observed after the retrieval of the DCP rods.

3.3 Indications of Instability:

There are several indications of stability problems displayed within the distressed wall and in the immediate surrounding area which are discussed below.

The most obvious sign of distress is the displacement of the wall which appears to have separated from the retaining wall aligned along the east boundary of the rear garden and moved approximately 0.4m to the south (Photograph 4). The displacement appears to be far less at the western end of the wall compared to the east and in addition to outward displacement, the wall also appears to have undergone minor displacement towards the west (Photograph 5).

Photograph 4 and 5:
Views of the corner of the distressed retaining wall looking east from within the site (4) and west from outside the site (5)

Where the retained soils could be observed behind the retaining wall (Photograph 5), the materials comprised building rubble including bricks concrete etc.

Considering the degree of displacement from the east boundary wall observed, the overall condition on the wall is relatively good and, although rotated outwards, is relatively intact. The exception to this is are broadly vertical cracks which extend through the mortar and individual blocks (Photograph 6 and 7) visible on the south side of the retaining wall.

Photographs 6 and 7: Near vertical cracking in retaining wall taken from the southern side looking broadly north.

Directly to the south of the retaining wall and exposed at the ground surface or under a thin veneer of topsoil, a layer of concrete was observed which appeared to be present along the entire section of wall. Where sections of concrete were observed it appears that they are related to an óoverflowô of concrete from the original construction of the retaining wall. The concrete appeared to vary between approximately 0.3m and 0.1m in thickness where observed.

This concrete indicates that the ground surface level was previously higher at the base of the wall than it is currently (Photograph 8).

Photograph 8: Probable original ground line (dashed red) on south side of retaining looking broadly west from the east end of the wall.

It was also observed that drainage from the wall via an outflow pipe did not appear to be functioning and was dry despite recent rainfall. Based on photographs provided by the client it appears water flow from behind the wall resulting in loss of sand fill has occurred in the past (Photograph 9).

Also shown in Photograph 9 is a pier footing casing liner which appears to be deformed. The movement appears to have occurred within the lower section of the pile and to the south. Based on survey information provided by the client and assuming the top of the pier is in its original position, the top and the bottom of the wall appear to have moved as well as the lower section of the pier footing.

Photograph 9: View of the deformed pier at the corner of the site and outwash sand

With the exception of the distressed retaining structure/pier footing near the rear of No.60, no signs of distress were observed elsewhere within the site. Similarly, the neighbouring properties and structures generally inspected from the site showed no indications of geotechnical hazards.

4. GEOTECHNICAL DISCUSSION:

4.1. Causes of Wall Movement:

The cause of the movement observed in the wall is likely to be a combination of several mechanisms and determining a single initial ‘trigger’ (if one exists) is of limited usefulness and would require additional investigation. However, for the purposes of this investigation and to identify economic remediation strategies the mechanisms inducing wall movement proposed in Section 1.0 are addressed below.

4.1.1 Additional Surcharge Load

No indications of over-compaction of the fill soils where observed within the retained sand fill to a maximum depth of 1.35m. Below the sand fill, the interpreted building refuse would not be expected to be over compacted and likely only marginally denser than moderately compacted soil. In addition, no additional structure loads were evident near the distressed retaining wall.

It is also understood that the wall movement has accelerated or been noticed in the last 1-2 years and it appears that water flow from behind the wall is currently unimpeded based on-site observations and discussions with the client.

Potentially elevated water pressures may have been present behind the retaining wall previously and surcharged the lower section of the wall however deformation of the wall would be expected (although not guaranteed) to occur until a flow pathway (potentially only 1-2mm wide) had developed allowing water to drain from behind the wall removing elevated pressure conditions. The displacement observed within the eastern end of the wall does not appear to reflect this behaviour with approximately 0.4m movement observed indicating continued rotation/displacement post opening of the flow path.

The layer of concrete observed at the toe of the wall appears to have previously been attached to the retaining wall and probably became detached following settlement of the underlying building refuse. The additional concrete will have increased the load on the lower sections of the wall (prior to detachment) however without knowing the extent of concrete present and history of detachment, it is difficult determine potential surcharge loads based on field work completed to date.

Where the thickness of concrete was able to be determined in a limited number of locations it appeared to be up to 0.3m thick adjacent to the wall (where it would be likely be thickest) and around 0.1m where visible elsewhere. The concrete layer appears to extend around 2.0m from the wall (on average) and extends most of the length of the wall. Using these dimensions an estimated increase in load of approximately 10kN/m is estimated along the full length of the retaining wall.

It is therefore considered that additional, unanticipated surcharge loads resulting from the concrete overpour *may* be a factor in the distress observed in the retaining wall however it is unlikely to be currently affecting the wall as it appears to have become detached.

4.1.2 Structural/Construction Deficiency of the Retaining Wall

It appears that the wall itself is constructed relatively well with limited cracking or bulging evident in the mortar/brickwork considering the amount of displacement observed. The design of the wall is not known and if structural drawings are available, they should be reviewed by the Structural Engineer allowing for elevated water pressures behind the wall and the overpour concrete surcharge.

Obvious signs of distress where not observed within any other structures within No.60 (either within the new additions or within what is thought to represent pre-existing structures) which would be expected if deep seated instability was occurring.

Based on the above it is considered that the structural design and construction quality of the wall itself is unlikely to be responsible for the deformation observed.

4.1.3 Inadequate Founding Strata

The existing retaining wall is in relatively good condition and it has been interpreted that the wall is/was moving as an intact óblockô. The base of the wall footing has been interpreted to be at a depth of 0.6m below the existing ground surface level and material below this depth appeared to include sections of loose fill or potentially óweakô natural soils. As such settlement from downslope could have reduced founding support if footing are at a shallow depth.

The deformation of the pile observed at the south eastern corned may also indicate that the pier founding strata is insufficient to resist downslope movement.

If water ponding/flow has occurred through/under the wall it is likely to have to softened the founding soils and potentially removed sand from any granular fill on the south side of the wall which could result in a reduction in strength of founding strata.

It is not known if the poor founding conditions have been created following the construction of the retaining wall (e.g. via leaking pipework or ponding of water behind the wall prior to development of crack) or whether the conditions were present at the time of construction. As the movement has apparently increased recently it is probable that any softening/loosening of the soils has occurred relatively recently.

The source of the water would likely be ruptured pipework however whether the pipework ruptured first or was ruptured as a result of the movement is difficult to determine.

It is considered that insufficient founding strata is likely to have played a significant role in the damage observed.

4.2 AGS Landslip Assessment

Following confirmation of proposed works and submission of DA drawings an additional Risk Assessment will be required to satisfy Council, however for the purposes of this report a risk assessment has been completed for current site conditions.

The existing Sydney water asset has not been included in this assessment as it is understood that, based on the available service plans, the asset is below the base of the wall however when proposed design is confirmed an additional assessment will be necessary.

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

A. Rapid collapse of retaining wall and downslope slide of retained fill.

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 was estimated to be up to **1.50 x 10^{-6} for persons within the rear garden and 5.00 x 10^{-10} for the neighbouring property downslope, whilst the Risk to Property was considered to be 'Very Low'**. Therefore, the site without any remediation works undertaken would be considered to have an óAcceptableô risk level when assessed against the criteria of the AGS 2007 due to limited occupancy of the areas affected however where additional movement is observed this estimate will require further assessment.

4.3 Remediation Options

Various options are available to remediate the wall and the most suitable will be determined by the adjacent property owners, access/construction conditions, economic viability, Sydney Water and Council requirements however the options are briefly outlined below. It is envisaged that a survey of the existing sewer will be required whichever option is selected and it is anticipated that the asset may be defective and potentially the original source of the retaining wall problems however further investigation would be necessary to determine.

Option 1: Full Removal and Reconstruction

It is understood the current proposal is to remove the existing retaining wall and retained fill and construct and new bored pile retaining wall within the rear garden. This solution is likely to be the most robust however it is also likely to be the highest cost and will be further complicated by the Sydney Water asset in the vicinity of the wall.

As access is limited it is envisaged that machinery size will be limited to very small machines or hand tools.

The presence of building rubble may make the installation bored piers difficult and prospective contractors should be consulted as to the capabilities of proposed equipment.

Where bored piers are undertaken, they will need to be founded and socketed within at least very low strength bedrock which is anticipated at around 3.0 - 4.0m below the depth of the existing synthetic lawn.

Previous experience indicates that Sydney Water requirements may dictate the design and construction methodology of any new works near the asset.

Option 2: Survey Monitoring

This option may represent the most economical solution as the results may indicate that the movement has ceased and that the cause of the 0.4m of displacement observed was due to a mechanism no longer active (e.g. softening of soils via water ponding, water pressure increase or surcharge loading). Survey should be undertaken at monthly intervals to an accuracy of no less than 1mm and the results reviewed as they become available. Where the wall displacement has ceased, alternate design may be available.

Option 3: Additional Retaining Wall

Consideration could be given to construction of an entirely new retaining wall to the south of the existing distressed wall and within the adjacent property (No.22 Homestead Avenue). This would require permission from the owners of the property to the south however this may be a more economic option than Option 1. It is envisaged that nested micropiles (or similar) installed to very low strength bedrock using hand tools would be required.

Option 4: Remediation and Underpinning

As the wall itself appears in relatively good condition and it appears that it is/has been moving as an intact structure, if the wall itself can be remediated and adequate drainage installed it may be a more economic option than Option 1. Specialist retaining wall contractors should be consulted if this option is to be considered. It is considered that part of the remediation strategy will include underpinning of the existing footings/wall to appropriate strata.

4.4 Design and Construction Recommendations

The proposed design has not been confirmed however the preliminary parameters below could be used for preliminary design.

4.4.1. New Footings:	
Site Classification as per AS2870 ï 2011 for new footing design	Class óPô due to potential landslip risk.
Type of Footing	
Sub-grade material and Maximum Allowable Bearing Capacity	Weathered, XLS-VLS Bedrock: 700kPa
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	C_e ï shallow soil site
Remarks: All new footings must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity against the structural engineers design requirements. This is mandatory to allow them to be ócertifiedô at the end of the project. Instability of the fill and potential natural soils will need to be considered where/if new footings are proposed adjacent to existing footings founded at shallow depth.	

4.4.2. Excavation:	
Depth of Excavation	Unknown-potentially to 3.0m to remove existing fill
Distance to Neighbouring Properties/Structures	No.22 Homestead-The existing retaining wall is within this property, No.58 Alexander Street - Within 1.0m of the common boundary (subject to design), property residence >10.0m away, No.62 Alexander Street ï Approximately 7.0m to the common boundary (subject to design), property residence >15m away.

Type of Material to be Excavated (subject to design)	0.00 up to 3.0m	Fill

Guidelines for batter slopes for this site are tabulated below:

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

*Dependent on defects and assessment by engineering geologist during excavation.

Remarks:

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/Fill	Excavator with bucket
Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	Buildings in surrounding properties = Unlikely to apply	
Vibration Calibration Tests Required	Unlikely	
Full time vibration Monitoring Required	Unlikely	
Geotechnical Inspection Requirement	To be confirmed following preliminary design	
Dilapidation Surveys Requirement	Unlikely to be required.	

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.

4.4.3. Retaining Structures:

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

Material	Unit Weight (kN/m3)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
			Active (Ka)	At Rest (K_0)	
Fill and Natural soils	20	$\phi' = 32°$	0.30	0.50	N/A
Extremely low strength bedrock	23	$\phi' = 35°$	0.20	0.30	300kPa

Remarks:

In suggesting these parameters it is assumed that the retaining walls will be fully drained with suitable subsoil drains provided at the rear of the wall footings. 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.

4.4.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		Yes
Excavation likely to intersect	Water Table	No
	Seepage	Minor (Ò0.50 L/min), possibly at fill/soil interface, possibly due to leaking pipework.
Site Location and Topography		Low south side of the road within moderate to steeply south sloping topography
Impact of development on local hydrogeology		Negligible
Onsite Stormwater Disposal		Not considered feasible

Remarks:

The condition of all pipework will require verification as part of the works to ensure that adequate long-term drainage can be constructed or repaired and be disposed of well away from the base of the retaining wall.

5. CONCLUSION:

The existing retaining wall near the south end of No.60 Alexander Street appears to be in distress and has been displaced up to approximately 0.4m downslope however appears to be moving as an intact structure.

The investigation indicated that insufficient founding strata and potentially elevated water levels behind the retaining wall (possibly from ruptured pipework adjacent) may be the source of the movement observed in the retaining wall.

Where full replacement is proposed significant cost may be incurred due to existing ground conditions, access restrictions and the Sydney Water asset in the vicinity. Prior to design DA submission Sydney Water should be consulted to determine what conditions they may impose for the remediation/replacement works.

Survey monitoring should be undertaken to better identify the cause and whether movement is still occuring which may have significant implications for remediation options.

A drain survey should be undertaken of all nearby pipework to determine their condition prior to confirmation of remediation works.

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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: -

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

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

Qc = (12 to 18) Cu

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

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

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

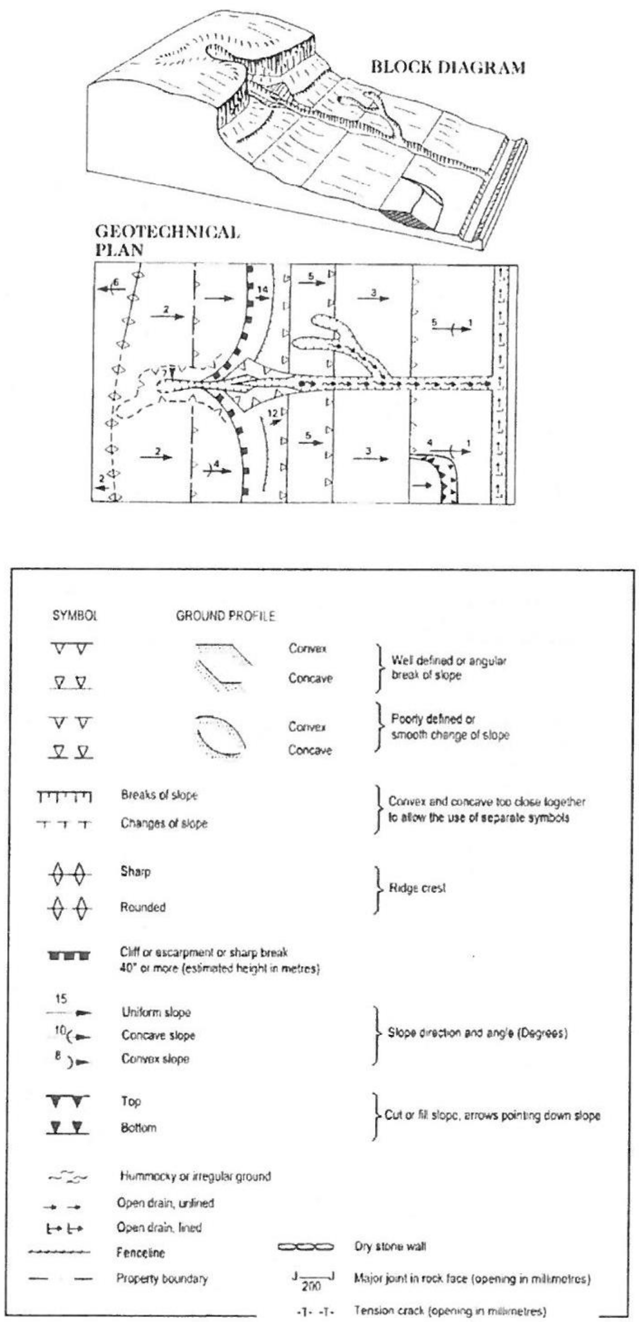
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

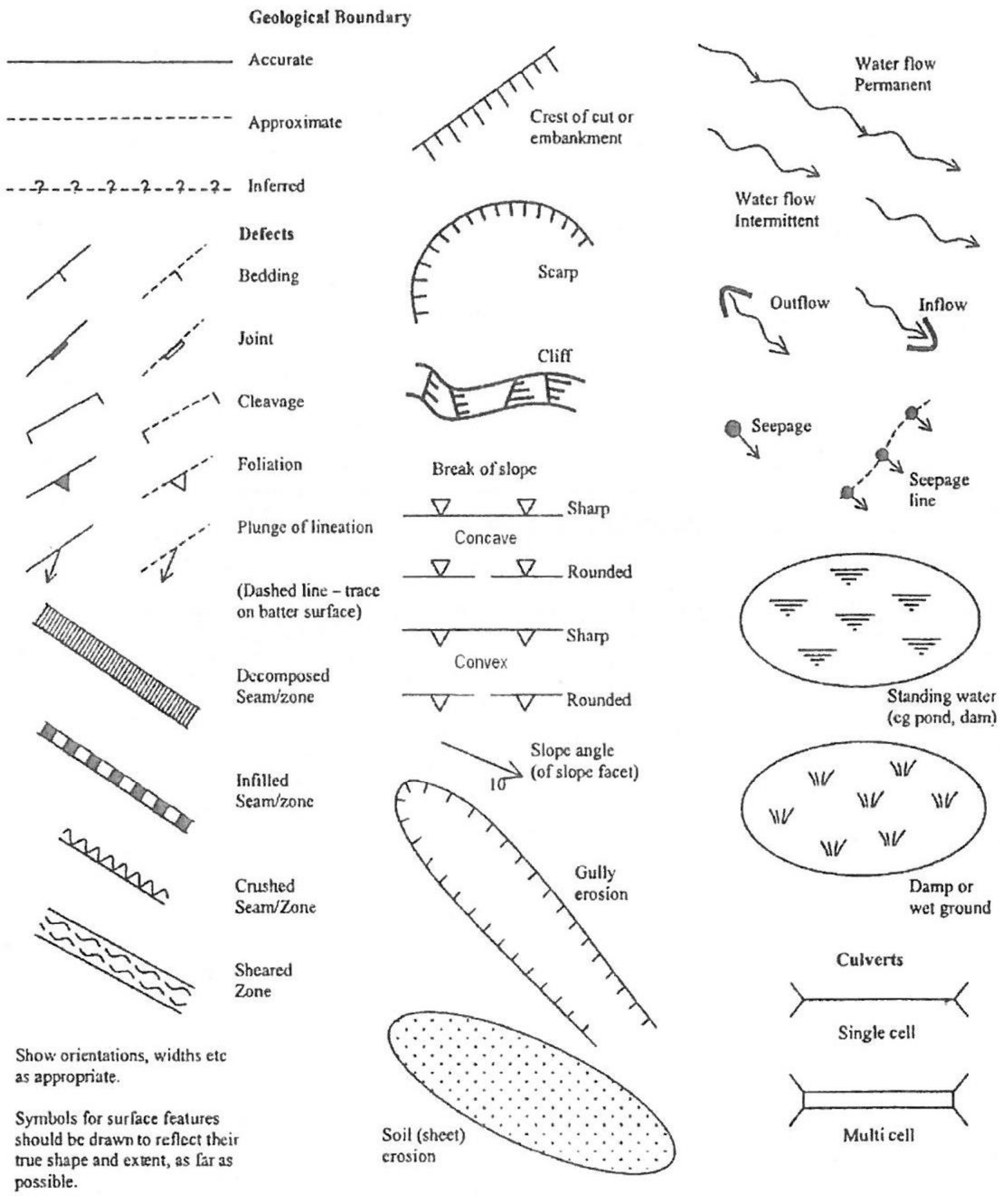
Site Inspection

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).

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Appendix 2

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

SITE PLAN & TEST LOCATIONS

FIGURE 1.

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

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

LEGEND

A — A' CROSS-SECTION REFERENCE LINE

TP / DCP: TEST PIT / DYNAMIC CONE PENETROMETER LOCATION

BH / DCP: AUGER / DYNAMIC CONE PENETROMETER LOCATION

SCALE: 1:200 @ A3
DRAWING: FIGURE 1
DATE: 29/06/2020

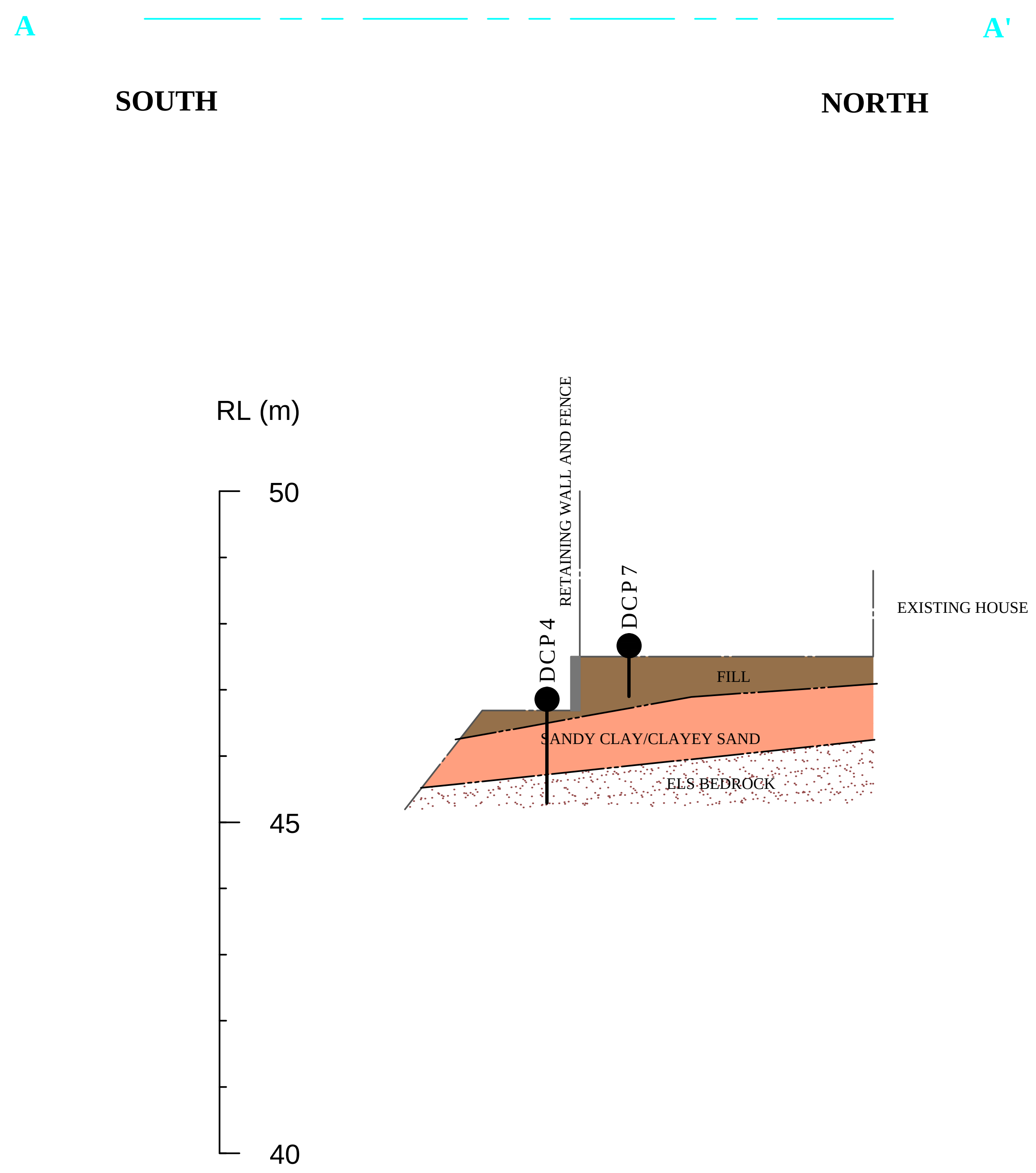
APPROVED BY: TMC
DRAWN BY: JY
PROJECT: 2020-097

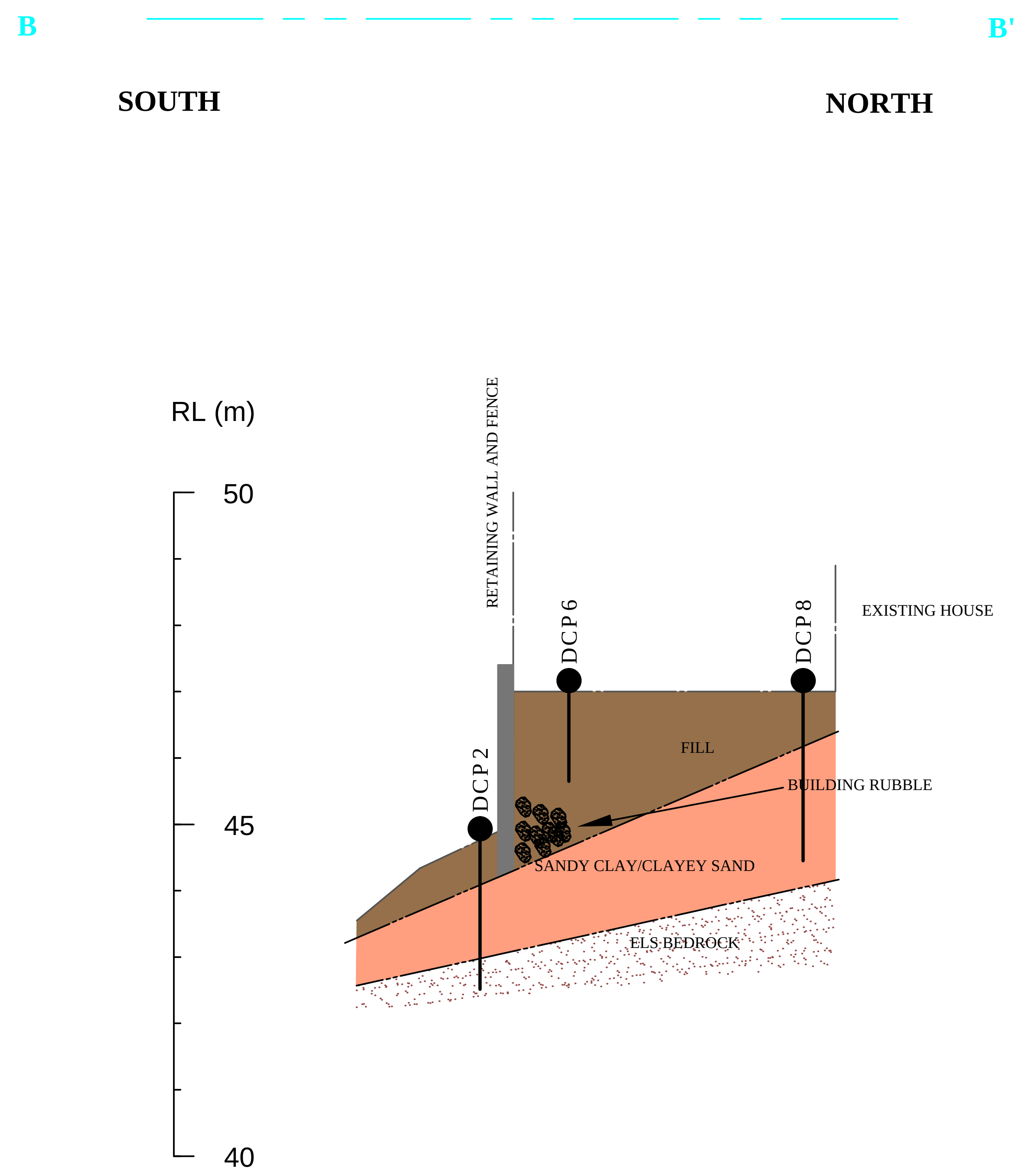
PREPARED FOR:

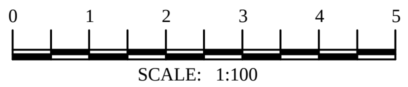
SP81259

ADDRESS:

60 Alexander Street, Collaroy







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.

CROZIER
GEOTECHNICAL CONSULTANTS

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

LEGEND

- A — A' CROSS-SECTION REFERENCE LINE
- BH / DCP AUGER / DYNAMIC CONE PENETROMETER LOCATION
- SANDY CLAY/ CLAYEY SAND
- PROPERTY BOUNDARY
- FILL
- ELS BEDROCK

SCALE:	1:100 @ A3
DRAWING:	FIGURE 2
DATE:	29/06/2020
APPROVED BY:	TMC
DRAWN BY:	JY
PROJECT:	2020-097

PREPARED FOR:
SP81259

ADDRESS:
60 Alexander Street, Collaroy

BOREHOLE LOG

CLIENT: SP81259 **DATE:** 19/05/2020 **BORE No.:** 1

PROJECT: Remediation of Retaining Wall **PROJECT No.:** 2020-097 **SHEET:** 1 of 1

LOCATION: 60 Alexander Street, Collaroy **SURFACE LEVEL:** 44.5m

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		FILL: Brown sandy clay (TOPSOIL)				
	0.10	..brown gravelly clay/sand with brick/concrete				
		Borehole refusal on concrete blocks/rubble.				
1.00						
2.00						

RIG: Not Applicable DRILLER: AC

METHOD: Hand Auger LOGGED: KN

GROUND WATER OBSERVATIONS: Not encountered

REMARKS: CHECKED:

BOREHOLE LOG

CLIENT: SP81259 **DATE:** **BORE No.:** BH2

PROJECT: Remediation of Retaining Wall **PROJECT No.:** 2020-097 **SHEET:** 1 of 1

LOCATION: 60 Alexander Street, Collaroy **SURFACE LEVEL:** 47.0m

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		FILL: Orange brown silty sand				
0.75		End of borehole at 0.75m depth-refusal on interpreted building rubble.				
1.00						

RIG: Not Applicable DRILLER: AC

METHOD: Hand Auger LOGGED: KN

GROUND WATER OBSERVATIONS: Not observed

REMARKS: CHECKED:

Crozier Geotechnical Consultants

BOREHOLE LOG

CLIENT: SP81259 **DATE:** 19/05/2020 **TEST PIT:** 1

PROJECT: Remediation of Retaining Wall **PROJECT No.:** 2020-097 **SHEET:** 1 of 1

LOCATION: 60 Alexander Street, Collaroy **SURFACE LEVEL:** 44.7m

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		FILL: Brown sandy clay, gravel/cobbles of brick				
	0.50	SANDY CLAY: Firm brown grey, sandy clay, fine grained sand, moist ..wet,				
		End of borehole/test pit @ 0.80m depth-footing base encountered at 0.60m depth				
1.00						

RIG: Not Applicable DRILLEF AC

METHOD: Hand Tools LOGGEI KN

GROUND WATER OBSERVATIONS:

REMARKS: Test pit using hand tools to expose existing footing base CHECKE

DYNAMIC PENETROMETER TEST SHEET

CLIENT: SP81259 **DATE:** 19/05/2020

PROJECT: Remediation of Retaining Wall **PROJECT No.:** 2020-097

LOCATION: 60 Alexander Street, Collaroy **SHEET:** 1 of

	Test Location							
Depth (m)	DCP1	DCP2	DCP3	DCP4	DCP5	DCP6	DCP7	DCP8
0.00 - 0.15	--	2	--	--	0	0	0	1
0.15 - 0.30	1	5	--	--	0	0	1	0
0.30 - 0.45	1	2	4	--	0	1	2	1
0.45 - 0.60	1	1	7	--	0	0	1	2
0.60 - 0.75	2	4	2	1	1	0	B@0.60m	4
0.75 - 0.90	2	3	8	5	B@0.75m	1		7
0.90 - 1.05	3	2	4	2		0		5
1.05 - 1.20	4	1	5	4		1		9
1.20 - 1.35	6	3	4	2		0		4
1.35 - 1.50	6	6	6	2		B@1.35m		7
1.50 - 1.65	7	10	5	3				6
1.65 - 1.80	10	8	8	9				6
1.80 - 1.95	11	8	15	16				7
1.95 - 2.10	12	16	25	12				7
2.10 - 2.25	19	25	End	12				7
2.25 - 2.40	19	End		14				7
2.40 - 2.55	End			End				7
2.55 - 2.70								End
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	Full collapse of existing retaining wall resulting in downslope movement of fill material behind wall			On-going mechanisms which may result in full collapse has not been identified	a) Would likely impact 2.0m-3.0m of rear garden. b) Would likely impact <5.0m downslope of wall		a) Person in garden 0.25hrs/day avge. b) Persons below retaining is likely to be very rare wuithin nature reserve, 5 minutes per day.	a) Unlikely to not evacuate b) Likely to not evacuate	a) Person in open, likely engulfed b) Person in open, likely engulfed	
				Unlikely	Prob. of Impact	Impacted				
		a)	Rear garden of No.60	0.0001	1.00	0.30	0.1000	0.5	1.00	**1.50E-06**
		b)	Property to the south (No.22 Homestead Avenue)	0.0001	0.10	0.01	0.1000	1	0.05	**5.00E-10**

* hazards considered in current condition and/or without remedial/stabilisation measures
* 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), Imapcted 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 not 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	Full collapse of existing retaining wall resulting in downslope movement of fill material behind wall	Rear garden of No.60	Unlikely	The event might occur under very adverse circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required or no impact to neighbouring properties.	Very Low
		Property to the south (No.22 Homestead Avenue)	Unlikely	The event might occur under very adverse circumstances 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%.

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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.