GEOTECHNICAL SITE INVESTIGATION REPORT

FOR

PROPOSED ALTERATIONS & ADDITIONS

AT

19 BURNE AVENUE, DEE WHY NSW 2099

Report Prepared for: ROMINA ROJO DIAZ

Project No: SRE/1090/DW/24

Date: 01/07/2024

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The undersigned, on behalf of Soilsrock Engineering Pty Ltd, confirm that this document and all attached documents, drawings, and geotechnical results have been checked and reviewed for errors, omissions, and inaccuracies.

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1. INTRODUCTION

This report presents and interprets the result of a geotechnical investigation assessment carried out by Soilsrock Engineering Pty Ltd (SOILSROCK) of the existing property at 19 Burne Avenue, Dee Why NSW 2099. The work was carried out by the request of Ms. Romina Rojo Diaz who is the representative for the proposed development of the property. SOILSROCK conducted the work in general accordance as per letter proposal dated of 14th March 2024 and email acceptance on 11th May 2024y.

This assessment report comprised a detailed geotechnical inspection of the property and is based on the following documents supplied by the Ms. Romina Rojo Diaz on the email of 24th April 2024:

- Survey Drawing prepared by Hill & Blume Consulting Surveyors, "Chowing Selected Levels & Detail and Boundary Identification Survey of Lot 2 In DP 209386 being 19 Burne Avenue, Dee Why", Rev 'A – First Issue" dated 29th November 2021.
- Architectural Drawings prepared by ROMINO ROJO STUDIO, "ALTERATIONS & ADDITIONS TO AN EXISTING DWELLING, LANDSCAPING & POOL AT 19 BURNE AVENUE, DEE WHY NSW 2099", Job No: 2105, Dated 29/01/2024.

The purpose of this investigation was to assess the existing subsurface ground conditions and risks associated with the existing slope versus new development construction and to provide geotechnical recommendations and advice on excavation conditions, retaining walls and foundations design options and landslide risk assessment.

The following sections describe the proposed development, scope of works and factual results of this site investigation. Comments and recommendations on excavation and foundations conditions, including landslip risk assessment for the proposed dwelling is given in the last part of this report.

2. PROPOSED DEVELOPMENT

Based on the architectural drawings provided by the client, it is understood that for the existing dwelling is proposed the following additions & alterations:

- For the Main Dwelling - landscaping and planting areas, a garage, a swimming pool, a carport, a terrace, a driveway and a porch.
- For the Secondary Dwelling – a driveway and a terrace.
- Level 1-A Main Dwelling - will accommodate a dining, a living, a kitchen, a porch, a lounge, a LAU, a lawn, a pantry, four bedrooms, three bathrooms, two WIC and two terraces.

- Level 1-B Main Dwelling - will include a pool terrace, a deck, BBQ terrace, games room, a terrace, and a driveway.
- Level 2 Main Dwelling - will include a terrace, a master bedroom, an, one WC and two WIC.
- Level 1 Secondary Dwelling – will include a porch, a living, a kitchen a WC, terrace, a bedroom, a robe and bathroom.

Details of the proposed development are shown on the architectural drawings provided by “ROMINA ROJO STUDIO” as referred above.

3. SCOPE OF WORKS

The field work for investigation was carried on the 27th of May 2024 and consisted of the following:

- Dial Before You Dig (DBYD) – Conduct an online buried services search at the site before field works.
- Conduct an OH&S and walkover survey to assess local topography, geology, hydrology, and existing site conditions, including exposed soil/rock conditions, vegetation, and surface drainage.
- Conduct a geotechnical inspection of the site area and adjacent land.
- 4 x Dynamic Cone Penetrometer tests (DCP1 to DCP4) to maximum depth of 0.85m were carried out by using a 9kg Dynamic Cone Penetrometer specialised steel cone device. The testing followed the procedure as per AS 1289-1997, method 6.3.2.
- Photographic record of the site conditions.

The field work was conducted in presence of two geotechnical/civil engineers, from Soilsrock office, who observed visually the existing geotechnical conditions and recorded the in-situ test results.

4. SITE LOCATION AND DESCRIPTION

The subject site is located at 19 Burne Avenue, Dee Why NSW 2099. The site belongs to the Northern Beaches Council and is legally described as lot 2 DP 209286 with an area around 1,035 m^2.

The project site is situated within R2- Low Density Residential. It is delimited by 11 Burne Avenue at the South of the site, at East by residential house 17 Burne Avenue, at West by a residential house 26 Burne Avenue, and finally at North by 21 Burne Avenue, Dee Why.The site is a trapezoidal plus rectangular in shape. The surrounding land comprise mostly of residential dwellings.

The DCP's and photo's location are shown ***in Appendix B*** and photographs of the area are attached to this report ***in Appendix D***.

5. REGIONAL GEOLOGY

From the analysis of Geology of Sydney 1:100 000 Geological Series Sheet 9130, it is indicated that the site is located within a region of Triassic age, underlain by **Hawkesbury Sandstone (Rh)** which is comprised of medium to coarse-grained quartz sandstone, very minor shale and laminate lenses.

A reproduction of the geological map is shown on following ***Figure 1*** and is based on a portion of the geological map of Sydney 1:100 000 Geological Series Sheet 9130 (EDITION 1) 1983 (interactive resource provided by the Geological Survey of NSW), which depicts the site geological condition.

Figure 1 – Portion of the Sydney1:100,000 Geological Series Map 9130. Site area location is highlighted in a red/black sign.

6. RESULTS AND ANALYSES OF THE INVESTIGATION

6.1 Subsurface Investigation

Four Dynamic Cone Penetrometer (DCP) tests were carried out to complement the investigation of subsurface ground conditions. The following ***Table 1*** summarised the in-situ DCP test results and ***Table 2*** describes generically the principal strata sequentially observed and interpreted by the test results carried out on site.

Table 1 - Dynamic Cone Penetrometer tests results – DCP1 to DCP4.

Depth (m)	DCP1 (Blows/ 300mm)	DCP2 (Blows/ 300mm)	DCP3 (Blows/ 300mm)	DCP4 (Blows/ 300mm)
0.00 – 0.30	3	2	4	3
0.30 – 0.60	2 Bouncing @ 0.40m	2 Bouncing @ 0.35m	5	3 Bouncing @ 0.40m
0.60 – 0.90	–	–	7 Bouncing @ 0.85m	–

Equipment & Procedure Notes:

Equipment used: 9kg hammer, 510mm drop distance, conical tip: Standard used: AS1289.6.3.2 - 1997; the total number of blows are considered for 300mm penetration steps.

DCP Notes:

- 60 blows within 300mm soil interval defined as a “refusal”, which may indicates reaching into “Very Dense” sand layer or “hard Clay” or on top of bedrock.
- “Bouncing” indicates reached top of rock or in some cases can be due to presence of a hard obstacle like steel, rubble, flouters, boulders, cobbles, cement sand layers or hard materials.

Table 2 - Geotechnical subsurface interpretation by in-situ DCP results – DCP1 to DCP4.

Depth (m)	DCP1 (Blows/ 300mm)	DCP2 (Blows/ 300mm)	DCP3 (Blows/ 300mm)	DCP4 (Blows/ 300mm)
0.00 – 0.30	Very Loose Silty Sand	Very Loose Silty Sand	Loose Silty Sand	Very Loose Silty Sand
0.30 – 0.60	Very Loose Silty Sand 2 Bouncing @ 0.40m	Very Loose Silty Sand 2 Bouncing @ 0.35m	Loose Silty Sand	Very Loose Silty Sand 2 Bouncing @ 0.40m
0.60 – 0.90	-	-	Loose Silty Sand 7 Bouncing @ 0.85m	-

Notes: No samples were provided by DCP test, thus the geotechnical interpretation above is based only on the observation carried through the soil traces left attached to the rods and tip; this subsurface interpretation is based in DCP results obtained in table 1 and engineering judgement, it is only indicative, and some soils characteristics can be difficult to identify properly without samples. “Probably on top of rock” indicates reached top of rock or in some cases can be due to presence of hard obstacles such as steel, rubble, flouters, boulders, cobbles, cement sand layers or any other hard materials.

The ***Table 3*** below assesses the strength of the relevant materials crossed by the DCP tests, according to in-situ test results, soil classification, visual interpretation, and extrapolation.

The geotechnical parameters interpretation and extrapolation is based and limited to DCP tests carried on site, which are only indicative for design proposes. For detailed description of the subsurface conditions, explanation sheets about geotechnical parameters are presented in ***Appendix A***.

Table 3 - Allowable Bearing Pressure and Strength Interpreted and Extrapolated by in-situ tests.

Depth Range (m)	Material Conditions	Allowable Bearing Pressure (kPa)	Strength (ϕ, UCS)
Based on DCP1 Test Results			
0.00 - 0.40	Very Loose Silty Sand	NR	NR
Based on DCP2 Test Results			
0.00 –0.35	Very Loose Silty Sand	NR	NR
Based on DCP3 Test Results			
0.00 – 0.85	Loose Silty Sand	50	25°
Based on DCP4 Test Results			
0.00 –0.40	Very Loose Silty Sand	NR	NR

Notes:

- The geotechnical parameters interpretation and extrapolation is based and limited to the DCP test carried on site, which are only indicative for design proposes.
- The depth ranges of geological units as shown in the table are average thickness based on DCP test results obtained. It is understood that the subsurface conditions can vary from places to places.
- NR – Not Recommended.

As indicated within the table above, all of the DCP's tests recorded "bouncing" (DCP1, DCP2, DCP3, and DCP4), the DCP rods were bouncing at the end of the tests which indicate that the top of the rock was reached.

The DCP tests indicates that the site is underlying by silty sandy soils which directs probably to sandstone as indicated within the Regional Geology referred above as well as the visual inspection on the side, therefore the following ***Table 4*** indicates the interpreted and inferred geotechnical parameters for sandstone rock if encountered during excavations for construction. The following rock parameters are given for the lowest rock quality; regarding the hand methods by DCP tests are not able to investigate the rock in deep.

In addition, the below geotechnical parameters should not be used if it is confirmed the presence of rock boulders and floaters within the site, further geotechnical inspections and testing must be undertaken to confirm properly the geotechnical parameters for rock at the specific locations.

Table 4 – Recommended Geotechnical Parameters for Rock

Foundation Stratum	Allowable End Bearing Pressure (kPa)	Ultimate End Bearing Pressure (kPa)	Ultimate Shaft Adhesion (kPa)	Typical Elastic Modulus (MPa)
Class V	1,000	3,000	150	50

Notes:

- Rock Classification and bearing pressures based on P.J.N Pells "Substance and Mass Properties for The Design of Engineering Structures in The Hawkesbury Sandstone" AGM Vol No. 39 September 2004
- Ultimate end bearing pressures values occur at large settlements (>5% of minimum footing dimensions)
- Ultimate shaft adhesion values to depend on clean socket of roughness category R2 or better. Values may have to be reduced because of smear.
- Shaft adhesion applicable to the design of CFA or bored piles, uncased over the rock socket length, where adequate sidewall cleanliness and roughness are achieved.

To clarify the rock quality and to determine if rock boulder and floaters are present within the site, to assist retaining walls and foundations design, it is recommended that an additional geotechnical investigation by rock core drilling to core the rock and permit carry strength tests such as "IS50 - Point Load Tests" should be carried out.

6.2 Groundwater

According to the Geotechnical investigation groundwater was not recorded on the DCP tests rods when extracted from the ground. Groundwater can be investigated properly by further geo-hydrological assessment using a proper drilling and standpipe installation to monitor groundwater if required.

7. LANDSLIP RISK ASSESSMENT

The site is mostly located within an "Area B ", accordingly with the Warringah Landslide Risk Map from Northern Beaches online Mapping.

A reproduction of the Warringah Landslide Risk Map is shown in ***Figure 2*** and is based on a portion of the Landslide Risk Mapping from Northern Beaches Mapping, which shows the site geological condition as follow:

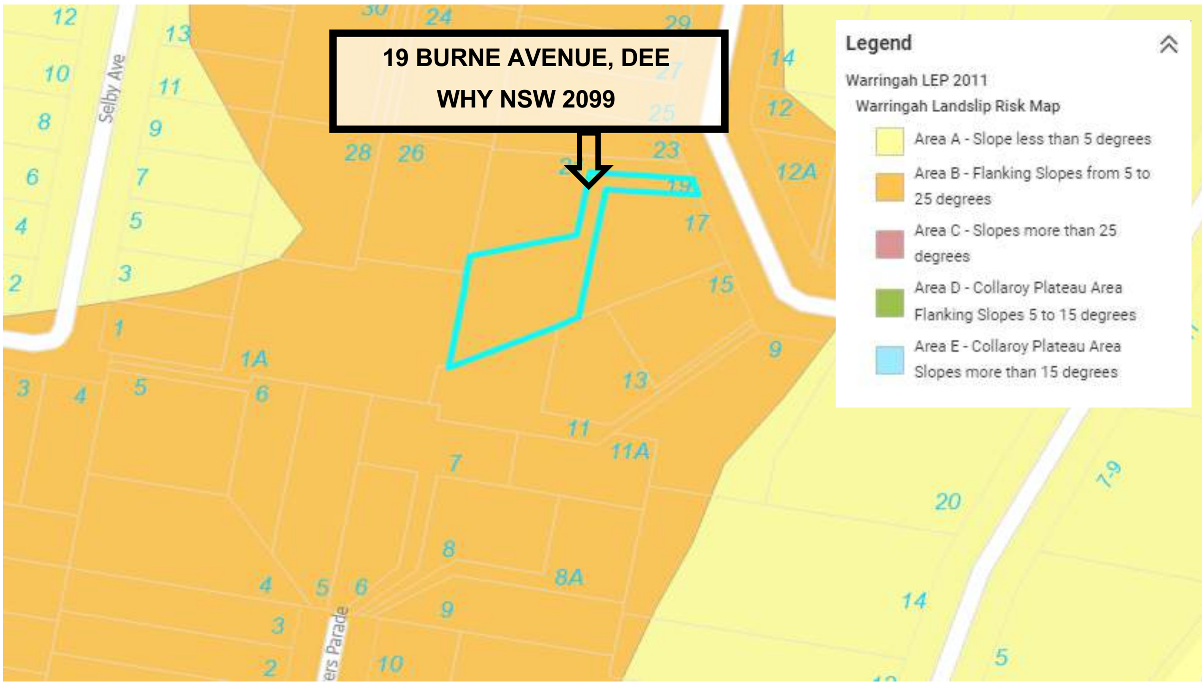

Figure 2 – Portion of the Warringah Landslide Risk Map. Site area is highlighted in Blue.

Nevertheless, some hazards have been identified and assessed for risk to property and life using the general methodology outline by the Australian Geomechanics Society (Landslide Risk Management AGS Subcommittee 2007), the risk assessment is outlined on the following ***Table 5***.

Table 5 – Geotechnical Hazards Summary Risk Analyses

HAZARDS	*Qualitative Measures of likelihood	*Qualitative Measures of Consequences to Property	*Risk to Property	*Risks To Life	*Level Risk Implications
Soil creek Ground movements causing cracking on the existing residential buildings and structures when heavy rain events occur	**Rare - (annual probability $P_{(H)} = 10^{-5}$)	Minor (5%)	Very Low ($2.5x10^{-05}$)	$1.5x10^{-7}$/annum	***Risk Acceptable
Soil erosion weakens tree roots and causes trees falling	**Rare - (annual probability $P_{(H)} = 10^{-5}$)	Minor (5%)	Very Low ($1.3x10^{-05}$)	$1.0x10^{-7}$/annum	***Risk Acceptable

Soil erosion exposes rock boulders and outcrops and causes potential rockfall.	**Unlikely – (annual probability $P_{(H)} = 10^{-4}$)	Minor (5%)	Low ($1.1x10^{-04}$)	$3.6x10^{-6}$	***Risk Acceptable
Rapid Collapse of excavated batters with no more than 1V:1.5H inclination	**Unlikely – (annual probability $P_{(H)} = 10^{-4}$)	Minor (5%)	Low ($1.1x10^{-05}$)	$3.8x10^{-6}$/annum	***Risk Acceptable
Slow failure of building foundations	** Unlikely – (annual probability $P_{(H)} = 10^{-4}$)	Minor (5%)	Low ($4x10^{-05}$)	$5x10^{-6}$/annum	***Risk Acceptable

Note: *Refer to Australian Geo-Mechanics Vol. 42 No. 1 March 2007, for full explanation of terms above.

**Likelihood assumes appropriate engineering design and construction methodologies and on-site assessment and approval by a geotechnical engineer.

Level of Risk Acceptable: AGS Suggested Tolerable loss of life individual risk = 10^{-4} /annum for existing slope/ existing development (Appendix F***). Risk level is acceptable provided the comments and recommendations on this report are followed.

Following the above, it is considered that the current site meets “Acceptable Risk Management” criteria with respect to both property and life under current and foreseeable conditions. As indicated by the DCP tests results, it is also noted the soils consists of silty sands present on the proposed development area to be at maximum shallow depths of 0.85m.

Prior to start any excavations and foundations structural elements to construct the proposed alterations and additions to the existing building, further additional geotechnical investigations by rock coring should be undertaken to confirm the quality and strength of the rock for the foundations of the additions of the two storeys above the existing one storey building, and to investigate the possibility of the presence within the foundation ground materials of rock boulders and floaters.

Upon knowing the rock quality foundations, a proper foundations design must be undertaken properly to ensure that all necessary excavations (to check if retaining walls are necessary) and new foundations will be constructed without any issues for the new additions and alterations to existing building and surrounding properties.

Batters slope excavation could be considered if there is enough space to excavate by batters with maximum inclination of 1V:1.5H as the excavation progresses, to stabilise and retain ground adjacent to the excavation, to permit to excavate safely any areas that need to be excavated. When vertical excavations are required, temporary or permanent retaining walls could be required depending on the excavation depths.

Providing the batters slope excavation inclination are not over 1V:1.5H and or shoring walls are constructed accordingly when required and the foundations materials are socket into solid rock (without the presence of boulders or floaters) it is considered that the proposed development will meet "Acceptable Risk Management" criteria with respect to both property and life upon appropriate application of geotechnical recommendations in this section, proper engineering design and construction methodology, and adequate on-site supervision and assessment by a professional chartered registered geotechnical engineer.

To maintain a good hillside construction practice, the following are recommended for the proposed development (refer to ***Appendix F***):

- Appropriate surface water drainage must be installed to avoid excessive water infiltration through the ground.
- Appropriate roof water piped and connected properly to the stormwater street systems to avoid excess water infiltration through the ground.
- Piles and footings must be socket into competent rock to allow for landslide risk.
- Cutting and filling should be minimized to reduce site disturbance within a landslide risk area.
- Sewage effluent pumped out or connected to sewer tanks shall be adequately founded and watertight.

8. COMMENTS AND RECOMMENDATIONS

8.1 Excavation Conditions

The supplied architectural drawing plans indicate that to construct the swimming pool and secondary dwelling, could require excavation depths of maximum approximately 2.0m. Based on the in situ testing the cuts are expected to be carried out through the very loose to loose silty sandy soils and rock materials. In addition, excavation for deep footings or piles could be required to minimum 1.0m socket into solid rock regarding the landslide risk and good engineering practice on hillside construction (refer to ***Appendix F***).

Excavation of the soil profile and weak rock can be completed using conventional earthworks equipment such excavators equipped with bucket to cut the silty sandy soils and extremely weathered, and very low strength rock will be suitable. If better strength and quality rock above medium strength rock are encountered at shallow depths, regarding the proximity of the neighbour buildings, it is recommended to cut the rock by sawing methods, this will reduce considerably the vibration and minimise noise levels to the surrounding building structures.

It is recommended that the demolition of the existing structures, excavation and construction techniques be adopted without causing more than 5mm/sec vibrations limit (Peak Particle Velocity) to the existing neighbouring buildings. If the existing neighbours' houses are constructed in weak conditions vibration limits could be necessary to be reduced to 3mm/sec vibrations limit (Peak Particle Velocity). If necessary, a vibration monitoring plan should be implemented to control vibration levels. Vibration monitoring plan must be carefully planned by the builder and will depending on the rock cutting methods by saw cutting, small rippers, small hammers to detach rock, and small size of excavators employed prior start demolition and excavation works. Dilapidation survey report for the neighbouring residential buildings could also be required to record the current building conditions prior to the commencement of site works including excavation and any necessary shoring/retaining walls on site. These will document any defects within the building(s) so that any claims for damage due to vibration can be properly assessed.

A Waste Classification should be carried for all the excavated materials to be disposed in accordance with NSW Environment Protection Authority (EPA) Waste Classification Guidelines Nov 2014, and under the Protection of the Environment Operations Act 1997 (POEO Act). Environmental sampling and chemical laboratory testing will need to be carried out to classify the spoil resulted from the excavation prior to disposal. This includes filling and excavated natural materials (GSW/VENM/ENM), if it is intended to be removed from the site. The type and extent of testing undertaken will depend on the final use or destination of the spoil, and requirements of the site.

8.2 Excavation Support & Retentions Systems.

As mentioned above, maximum excavation depth expected is 2.0m for the swimming pool and 1.5m for the secondary dwelling construction. If there is enough space between the excavations face and site boundary, excavation by batters with maximum inclination of 1V:1.5H can be considered, otherwise ground support or retaining structures are required.

Special attention must be undertaken to not excavate vertically below close or adjacent to the footings of the existing residential building, otherwise underpinning or piling walls could be required.

Regarding the swimming pool construction area adjoined to the boundary fence of the neighbouring property 11 Bourne Avenue, it will be necessary to undertake vertical excavations, in this case an additional borehole investigation by rock coring must be undertaken to say 3m deep, to permit to design a proper retaining wall, if necessary, prior to the commencement of the excavation. Additionally, boreholes to 3-4m deep will need also to be undertaken for the foundations of the new three storey building (after the addition of two storeys to the existing one storey building; **Note:** the existing one storey building footings must not be used to take the loads of the additional new two storeys above, news footings are to be design and constructed or reinforcing the existing ones, this needs to be further investigated), to determine the quality and strength of the rock foundations to permit to design a proper deep footings or piles.

Overall, when vertical excavations are necessary and slope excavation batters are not possible to be considered, to the swimming pool and secondary dwelling, a shotcrete wall combined with temporary ground anchors are recommended. For shotcrete wall combined with temporary ground anchors, excavation drops must be not more than 1.2m deep in soils to ensure stability of the excavation during the drilling to install ground anchors and preparation of the steel mesh for further shotcrete. All excavation drops are subject to an inspection of a chartered registered professional geotechnical engineer. If anchors are not allowed for ground support/retaining wall solutions, piling shoring walls by concrete soldier piles with 300mm to 450mm diameter combined with shotcrete in cantilever are required.

8.3 Foundations – Footings & Piles

The foundations conditions across the footprint of the alteration three storey building development are expected to intersect very loose to loose silty sands materials to maximum depth of 0.85m deep underlying by rock materials.

Further to the results of the investigation and architectural drawings described above, it is recommended for the foundations of the alteration three storey main building, two storey secondary building, swimming pool and any other structure, to install deep footings or footings supported by concrete piles socket minimum 1.0m into solid good quality rock. Carefully must be taken to ensure that the foundations are not installed within and above rock boulders and/or floaters. Prior to the foundations construction it is recommended to undertake an additional geotechnical investigation by rock coring to say minimum 3-4m deep into rock to determine the rock quality and strength and confirm if there is any boulder or floaters within the foundations location.

It is also recommended that the footings to be founded in an appropriated ground allowable bearing pressures determined by the footing/pile design, depending on the loads considered, size and type of footings/piles. Either way, the foundations of the entire buildings (main and secondary) must be installed and socket into solid rock minimum 1m to ensure stability of the footing/pile in competent solid rock materials (loose or debris materials must be removed prior to footing construction) to prevent against landslide regarding the property is on the Landslide Risk Risk Map Area B.

The founding depths must be adjusted and confirmed by the structural loads and foundations type required for the project. During the excavation to install the footings/piles, it is essential the ground foundations materials to be inspected and approved by a qualified professional registered geotechnical engineer to ensure ground materials and bearing pressures are as expected.

All piles/footings excavation base should be dewatered, cleaned, and be free of any loose material prior to pouring. Time between footing/piles excavation and concrete pour must be kept to the minimum, and delays are anticipated, it is recommended that the base of the footings be protected by a blinding layer of concrete with minimum strength of 25Mpa immediately after excavation to reduce any potential "loosening" effects.

All foundations should be designed and constructed in accordance with AS 2870 – 2011 – "Residential Slabs and Footing".

8.4 Subgrade Preparation for Slab on Ground and Pavements

Depending on the loads required, slab-on-grade construction is feasible for the buildings, garage, carport and driveway, depending on the ground conditions encountered after excavation, subgrade preparation could be required.

Slab on Ground

Following bulk excavation, when the subgrade encountered comprises soil or residual soils a well compacted granular course material (with maximum particle size of 37.5mm) subgrade with maximum 150mm thick layers of crushed recycled concrete or crushed sandstone (DGB20 or similar) layers it is recommended to install and be properly compacted. The subgrade layers should be compacted using a vibratory roller to target density ratio of 98% of SMDD. Moistening of each layer will facilitate compaction. Density/compaction tests should be carried out on each layer to confirm the above specification has been achieved in accordance with AS3798 Guidelines on Earthworks for Commercial and Residential Developments. A qualified geotechnical chartered engineering should supervise on site the subgrade preparation at minimum Level 2 Inspection and Testing as defined in AS3798. Final thickness of slab on ground should be determined by the structural engineer design.

Pavements

For pavement design, minimum CBR values of the subgrade material must be determined by the design engineer depending on the pavement design type considered. Depending on the pavement type design, the subgrade depth shall be compacted to achieve minimum relative compaction of minimum dry density ratio of 100% obtained from Standard Compactive Effort "SMDD – Standard Maximum Dry Density", following the same compaction methodology described for slab on ground subgrade preparation.

Above the well compacted subgrade materials a subbase granular course material layer with minimum 150mm thickness by crushed concrete or crushed sandstone (DGB20 or similar) should be installed. Subbase layers should be also compacted using the same compaction methods described above. Final thickness of subbase should be determined by the pavement design.

All pavements subgrade and subbase preparation geotechnical inspection and testing level 2 geotechnical inspection and testing should be allowed for all pavements accordingly with AS3798 Guidelines on Earthworks for Commercial and Residential Developments.

8.5 Engineering Fill

If backfill is to support landscaped areas and backfill walls, an engineered fill should be carried comprising 'clean' sandy soils, free of organic matter and contain a maximum particle size of 37.5mm. The engineered fill should be placed in a controlled and engineered manner compacted using a vibrating plate compactor and/or trench roller in layers not more than 150mm for non-sand materials not containing gravel-sized, or not more than 300mm for sand materials for controlled fill following AS2870-2011. Compaction should achieve minimum density index (ID) of 70%, to be proof tested by "DCP" tests Dynamic Cone Penetrometer as per AS1289.6.3.3.

8.6 Final Comments and Conclusions

Further to the above, additional geotechnical input is required and summarized as follow:

- Undertake additional geotechnical investigations by rock coring to 3-4m deep for the main and secondary buildings and swimming pool and any other relevant areas within the property development that requires deep excavation and foundations.

- Develop and concept a batter slope excavation and/or ground support/retaining wall design solution for the excavation of the swimming pool and secondary dwelling building.
- Geotechnical monitoring program to control and ensure low noise and vibrations to neighbour residence buildings prior start and during the excavation works if required.
- Dilapidation reports to neighbouring residential buildings regarding the excavation works.
- Geotechnical site inspections during excavation works, retaining structures, footings, and piles to confirm soil and rock bearing capacities.
- Geotechnical site inspections for anchoring installation and testing if required.
- Density tests to control all engineered fill material if required.
- Geotechnical site inspections and compaction tests to confirm density targets for subgrade preparation and subbase installation below slab-on-grade and pavements.

Further to the results of the investigations and geotechnical recommendations above, providing the works are carried accordingly with this report, and good engineering and building construction practice on hillside construction is maintained the proposed development is suitable for the site.

The geotechnical bearing pressures recommended on this report are based on the testing locations and on the in-situ soils investigated in deep. However, the geotechnical bearing capacities could vary across the site outside of those locations, the founding depth for foundations to be constructed could also vary as a result. Therefore, it is recommended that during the excavation and foundation's installation, specialised personnel such as an experienced professional registered chartered geotechnical engineer should inspect and approve the excavation works and founding levels.

9. LIMITATIONS

The site geotechnical investigation undertaken for the present report is an interpretation and estimation of the characteristics of the soil and or rock of subsurface conditions encountered during the test locations points investigated. No matter how comprehensive the investigation is, site ground conditions in other test locations investigated can differ and geological/geotechnical conditions can be unpredictable or can reveal unforeseen conditions.

The present report analyses form an engineering model interpretation and opinion of the actual subsurface conditions of the locations points where the tests were carried. The selected in-situ tests results are indicative of actual conditions encountered on the location points investigated. Recommendations are given based on the data testing results and visual interpretation carried by professional geotechnical and geological engineers from this office. Interpretation of the present report by others may differ from the interpretation given, there is the risk the report may be misinterpreted and Soilsrock cannot be held responsible for this.

Geotechnical reports rely on factual interpreted, and judgement of information based on professional visual interpretation of soils and rock samples, in situ and sampling tests, which can have some uncertainty due to unexpected natural and normal changing ground conditions. Soilsrock Engineering accepts no responsibility if different unexpected ground conditions occur in locations where the investigations were not carried out.

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APPENDIX A

GEOTECHNICAL EXPLANATORY NOTES

APPENDIX A – GEOTECHNICAL EXPLANATORY NOTES

The following geotechnical notes are provided, to give a better understanding of the description and classification methods and field procedures used for the interpretation and compilation of this report which is entirely based on the AS 1726-1993 – Geotechnical Investigations.

INVESTIGATIONS METHODS

Test Pits

Test pits are usually excavated with a backhoe or an excavator, allowing close examination of the in-situ soil if it is safe to enter into the pit. The depth of excavation is limited to about 3m for a backhoe and up to 6m for a large excavator. A potential disadvantage of this investigation method is the larger area of disturbance to the site. Samples can be taken from the test pits for soils testing and analyses.

Large Diameter Augers

Boreholes can be drilled using a rotating plate or short spiral auger, generally 3000mm or large in diameter commonly mounted on a standard piling rig. The cuttings are returned to the surface at intervals (generally 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 samples.

Continuous Spiral Flight Augers

The borehole is advanced using 90-125mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and sands above the water table. Samples are returned to the surface, or may be mixed with soils from the sides of the hole. Information from the drilling (as a distinct from specific sampling by SPTs or undisturbed samples) is of relatively low reliability, due to the remoulding, possible mixing or softening of samples by groundwater.

Dynamic Cone Penetromer Tests

Dynamic penetrometer tests (DCP) are carried out by driving a steel rod into the ground using a standard weight of hammer falling a specified distance. As the rood penetrates the soil the number of blows required to penetrate each successive 300mm depth are recorded. Normally there is a depth limitation of 1.2m, but this may be extended in certain conditions by the use of extension rods. A 16mm diameter rod with a 20mm diameter cone end is driven using a 9kg hammer dropping 510mm (AS 1289, Test 6.3.2). This test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various road authorities. Also Correlations with SPT tests can be made for Cohesion less and cohesive soils.

Standard Penetration Tests

Standard penetration tests (SPT) are used as a means of estimating the density or strength of soils and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, Methods of Testing Soils for Engineering Proposes – 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 equal to 450mm in total. The first 150mm increment it not considered for the so-called “N” value (standard penetration resistance), which is taken from the number of blows of the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm may not be practicable and the test will be discontinued. The results are represented in the following example:

- In the case where full penetration is obtained with successive blow counts for each 150mm as follow:
  - 1st Increment (150mm) = 2 blows
  - 2nd Increment (150mm) = 8 blows
  - 3rd Increment (150mm) = 15 blows
  - Representation – 2,8,15 “N” Value = 23
- In the case where the test is discontinued before the full penetration:
  - 1st Increment (150mm) = 20 blows
  - 2nd Increment (100mm) = 40 blows – test interrupted
  - 3rd Increment (150mm) = not carried – test refusal
  - Representation – 20, 40/100 mm “N” Value = 40

The results of the SPT tests can be related empirically to the engineering properties of the soils.

Correlation between DCP vs SPT for Cohesionless Soils

DCP (Blows/300mm)	SPT Value (Blows/300mm)	RELATIVE DENSITY
0-3	0-4	Very Loose
3-9	4-10	Loose
9-24	10-30	Medium Dense
24-45	30-50	Dense
>45	>50	Very Dense

Correlation Between DCP vs SPT for Cohesive Soils

DCP (Blows/300mm)	SPT Value (Blows/300mm)	CONSISTENCY
0-3	0-2	Very Soft
3-6	2-5	Soft
6-9	5-10	Medium/Firm
9-21	10-20	Stiff
21-36	20-40	Very Stiff
>36	>40	Hard

Continuous Diamond Core Drilling

A continuous core sample can be obtained using a diamond tipped core barrel, usually with a 50mm internal diameter. Provided full core recovery is achieved (which is not always possible in weak rocks and granular soils), this technique provides a very reliable method of investigation.

Sampling

Sampling is carried out during drilling or test pitting to allow engineering examination (and laboratory testing where required) of the soil rock.

Disturbed samples taken during drilling provide 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 it to obtain 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 affective only in cohesive soils.

DESCRIPTION AND CLASSIFICATIONS METHODS FOR SOILS AND ROCK

Descriptions include strength or density, colour, structure, soil or rock type and inclusions.

SOIL DESCRIPTIONS

Soil types are described according to the predominant particle size, qualified by the grading of other particles present:

Type	Particle size (mm)
Boulder	>200
Cobble	63 – 200
Gravel	0.6 – 63
Sand	0.075 – 0.6
Silt	0.002 – 0.075
Clay	<0.002

Type	Sand & Gravel Particle size
Coarse gravel	36mm – 19mm
Medium gravel	19mm – 6.7mm
Fine gravel	6.7mm – 2.36mm
Coarse sand	2.36mm – 600µm
Medium sand	600µm – 212µm
Fine sand	212µm – 75µm

The proportions of secondary constituents of soils are described as:

Coarse grained soils		Fine grained soils	
%Fines	**Modifier**	**%Coarse**	**Modifier**
≤5	Omit, or use 'trace'	≤15	Omit, or use 'trace'
>5 - ≤12	Describe as 'with clay/silt' as applicable	>15 - ≤30	Describe as 'with clay/silt' as applicable
>12	Describe as 'with silty/clayey' as applicable	>30	Describe as 'with silty/clayey' as applicable

Definitions of grading terms used are:

- Well graded – a good representation of all particle sizes.
- Poorly graded – an excess or deficiency of particular sizes within specified range.
- Uniformly graded – an excess of a particular particle size.
- Gap graded – a deficiency of a particular particle size with the range.

Cohesive Soils

Cohesive soils, such as clays, are classified on the basics of undrained shear strength. The strength may be measured by laboratory testing, or estimated by field tests or engineering examination. The strength terms are defines as follows:

Description	Abbreviation	Undrained shears strength (kPa)
Very soft	vs	≤12
Soft	s	>12 – ≤25
Firm	f	>25 – ≤50
Stiff	st	>50 – ≤100
Very stiff	vst	>100 – ≤200
Hard	h	>200

Cohesionless Soils

Cohesionless soils, such as clean sands, are classified on the basics of relative density, generally from the results of standard penetration tests (SPT), cone penetration tests (CPT), or dynamic penetrometers (PSP). The relative density terms are given below:

Relative density	Abbreviation	Density index %
Very loose	vl	≤15
Loose	l	>15 – ≤35
Medium dense	md	>35 – ≤65
Dense	d	>65 – ≤85
Very dense	vd	>85

Soil Origin

It is often difficult to accurately determine the origin of a soil. Soils can generally be classified as:

- Residual soil – derived from in-situ weathering of the underlying rock.
- Transported soils – formed somewhere else and transported by nature to the site.
- Filling – moved by man.

Transported soils may be further subdivided into:

- Alluvium – river deposits.
- Lacustrine – lake deposits.
- Aeolian – wind deposits.
- Littoral – beach deposits.
- Estuarine – tidal river deposits.
- Talus – coarse colluvium.
- Slopwash or Colluvium – transported downslope by gravity assisted by water. Often includes angular rock fragments and boulders.

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ROCK DESCRIPTIONS

Rock Strength

Rock strength is defined by the Point Load Strength (Is50) and refers to the strength of the rock substance and not the strength of the overall rock mass, which may be considerably weaker due to defects. The test procedure is described by Australian Standards 1726. The terms used to describe rocks strength are as follow:

Term	Abbreviation	Point Load Index $Is_{(50)}$ MPa	Approx. Unconfined Compressive Strength MPa*
Extremely low	EL	≤0.03	<0.6
Very low	VL	>0.03 – ≤0.1	0.6 – 2
Low	L	>0.1 – ≤0.3	2 – 6
Medium	M	>0.3 – ≤1.0	6 – 20
High	H	>1 – ≤3	20 – 60
Very high	VH	>3 – ≤10	60 – 200
Extremely high	EH	>10	>200

*Assumes a ratio of 20:1 for UCS to Is(50)

Degree of Weathering

The degree of weathering of rocks is classified as follows:

Term	Abbreviation	Description
Residual	RS	Soil developed on extremely weathered rock; the mass structure and substance are no longer evident.
Extremely weathered	XW	Rock is weathered to such an extent that it has 'soil' properties, i.e. it either disintegrates or can be remoulded in water, but the texture of the original rock is still evident.
Distinctly weathered	DW	Staining and discolouration of rock substance has taken place.
Slightly weathered	SW	Rock substance is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh	FR	No signs of decomposition or staining.

Degree of Fracturing

The following classification applies to the spacing of natural fractures in diamond drill cores. It includes bedding plane partings, joints and other defects, but excludes drilling breaks.

Term	Description
Fragmented	Fragments of <20mm
Highly fragmented	Core lengths of 20 – 40mm with some fragments
Fractured	Core lengths of 40 – 200mm with some shorter and longer sections
Slightly Fractured	Core lengths of 200 – 400mm with some shorter and longer sections
Unbroken	Core lengths mostly >1000mm

Rock Quality Designation

The quality of the cored rock can be measured using the Rock Quality Designation (RQD) index, defined as:

$$RQD\,\% = \frac{cumulativelengthof\,'sound'coresections\ \geq 100mmlong}{totaldrilledlenghtofsectionbeingassessed}$$

Where 'sound' rock is assessed to be rock of low strength or better. The RQD applies only to natural fractures. If the core is broken by drilling or handling (i.e. drilling breaks) then the broken pieces are fitted back together and are not included in the calculation or RQD.

Rock Quality Designation

For sedimentary rocks the following terms may be used to describe the spacing of bedding partings:

Term	Separation of Stratification Planes
Thinly laminated	< 6mm
Laminated	6mm to 20mm
Very thinly bedded	20mm to 60mm
Thinly bedded	60mm to 0.2m
Medium Bedded	0.2m to 0.6m
Thickly bedded	0.6m to 2m
Very thickly bedded	> 2m

LOG SYMBOLS

Moisture Condition - Cohesive Soils:

MC > PL – Moisture content estimated to be greater than plastic limit
MC = PL - Moisture content estimated to be approximately equal to plastic limit
MC < PL - Moisture content estimated to be less than plastic limit

Moisture Condition - Cohesionless Soils:

D – Dry – Runs freely through fingers
M – Moist – Does not run freely but no free water visible on soil surface
W – Wet – Free water visible on soil surface

Strength (Consistency) - Cohesive Soils:

VS – Very Soft – Unconfined compressive strength less than 25 kPa
S – Soft – Unconfined compressive strength 25-50 kPa
F – Firm – Unconfined compressive strength 50-100 kPa
St – Stiff – Unconfined compressive strength 100-200 kPa
VSt – Very Stiff – Unconfined compressive strength 200-400 kPa
H – Hard - Unconfined compressive strength greater than 400 kPa

Density Index/Relative Density - Cohesionless Soils

Symbol	Density Index (ID)	Range %	SPT "N" Value Range (Blows/300mm)
VL	Very Loose	<15	0-4
L	Loose	15-35	4-10
MD	Medium Dense	35-65	10-30
D	Dense	65-85	30-50
VD	Very Dense	>85	>50

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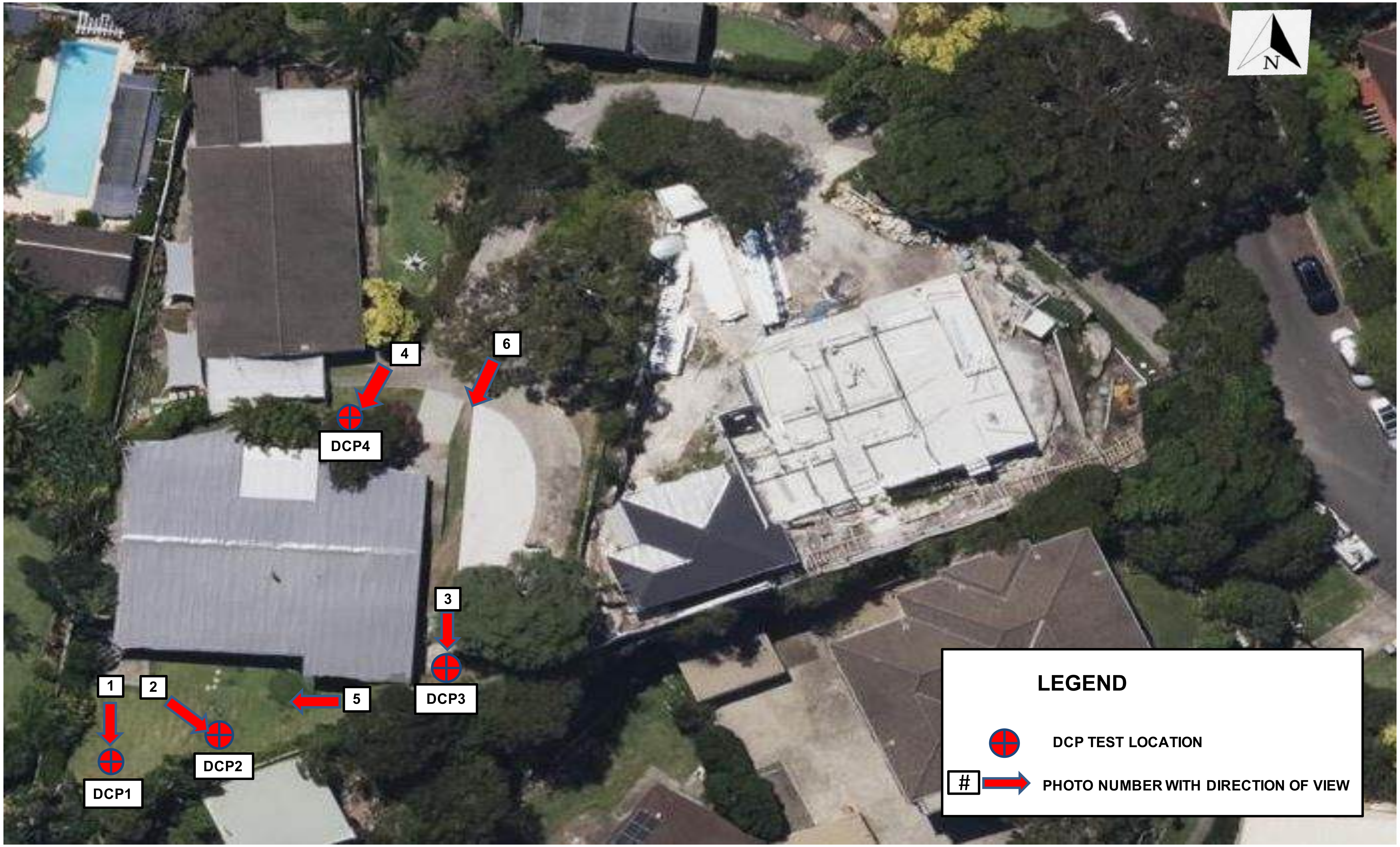
APPENDIX B

DCP TESTS & SITE PHOTOS LOCATION PLAN

19 BURNE AVENUE, DEE WHY NSW 2099



SOURCE: : SIX MAPS. Dated 17/06/2024.

			Revision	Date			
SOILSROCK ENGINEERING PTY LTD 2A/32 Fisher Road, Dee Why NSW 2099 M: 0457 115 044 | T: (02) 9982 2629 Email: info@soilsrock.com.au www.soilsrock.com.au	TITLE:	DCP TESTS & SITE PHOTOS LOCATION PLAN			Date: 27/06/2024	By:	SS
	SITE ADDRESS:	19 BURNE AVENUE, DEE WHY NSW 2099			Scale: NTS	Approved:	JC
	CLIENT:	ROMINA ROJO DIAZ			Project No.: SRE/1090/DW/24	Appendix:	B

APPENDIX C

DCP TESTS GRAPHIC

IN-SITU DCP TESTS RESULT SUMMARY (DYNAMIC CONE PENETROMETER TEST)

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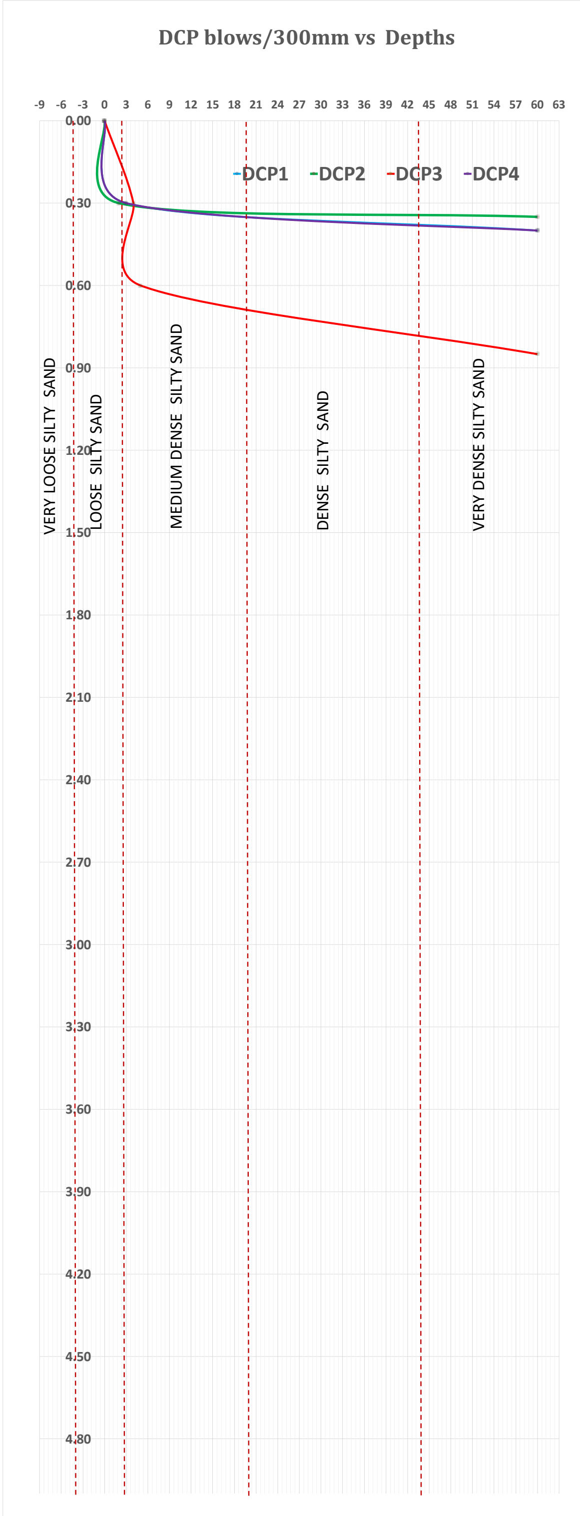
CLIENT:	ROMINA ROJO DIAZ	**PAGE:**	1 of 1
PROJECT:	GEOTECHNICAL SITE INVESTIGATION REPORT	**TESTING DATE:**	27/05/2024
LOCATION:	19 BURNE AVENUE, DEE WHY NSW 2099	**LOGGED/CHECKED BY:**	SS/JC
DATE:	17/06/2024	**Standards:**	AS 1289.6.3.2 - 1997
PROJECT NO.:	SRE/1090/DW/24		

Equipment: 9kg Dynamic Cone Penetrometer

Soil Type: SILTY SAND

Item	Depth (m)	Np (blows/300mm) - Interpretation					
		DCP1	DCP2	DCP3	DCP4		
1	0.0 - 0.3	3	2	4	3		
2	0.3 - 0.6	2 Bouncing @ 0.40m	2 Bouncing @ 0.35m	5	3 Bouncing @ 0.40m		
3	0.6 - 0.9			7 Bouncing @ 0.85m			
4	0.9 - 1.2						
5	1.2 - 1.5						
6	1.5 - 1.8						
7	1.8 - 2.1						
8	2.1 - 2.4						
9	2.4 - 2.7						
10	2.7 - 3.0						
11	3.0 - 3.3						
12	3.3 - 3.6						
13	3.6 - 3.9						
14	3.9 - 4.2						
15	4.2 - 4.5						
16	4.5 - 4.8						
17	4.8 - 5.1						
18							
19							
20							
21							
22							
23							
24							
25							



Comments: By conducting in-situ Dynamic Cone Penetration (DCP), the blow number (Np) per 300mm has been recorded and shown on the table above.

APPENDIX D

SITE PHOTOS

	CLIENT:	ROMINA ROJO DIAZ	**PAGE:**	1 of 1
	PROJECT:	GEOTECHNICAL SITE INVESTIGATION REPORT	**DATE RECORD:**	27/05/2024
	LOCATION:	19 BURNE AVENUE, DEE WHY NSW 2099		
	DATE:	17/06/224	**LOGGED BY:**	SS
	PROJECT NO.:	SRE/1090/DW/24	**CHECKED BY:**	JC

SITE PHOTOGRAPHS

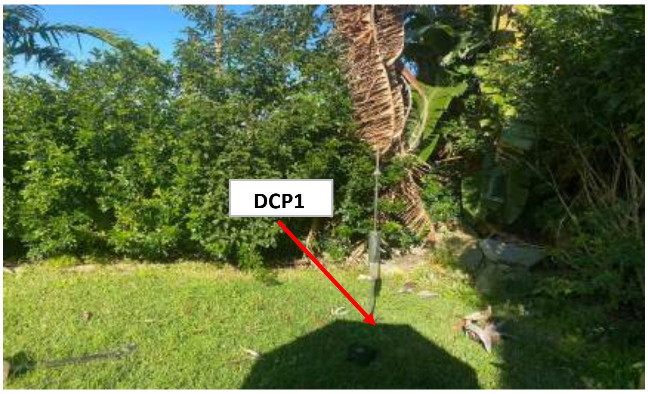


Photo 1 - South view of DCP1 test location.

Photo 2 - Southeast view of DCP2 test location.

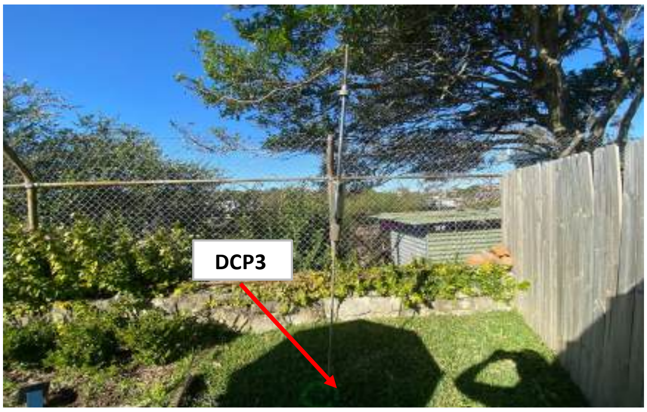


Photo 3 - South view of DCP3 test location.

Photo 4 - Southwest view of DCP4 test Location.

Photo 5 - West view of the front of the property.

Photo 6 - Southwest view of the property

APPENDIX E

LANDSLIDE RISK ASSSSMENT TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

(PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT 2007 AGS

(AUSTRALIAN GEOMECANICS SOCIETY)

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}		10 years		The event is expected to occur over the design life.	ALMOST CERTAIN	A
	5×10^{-2}		20 years			
10^{-2}		100 years		The event will probably occur under adverse conditions over the design life.	LIKELY	B
	5×10^{-3}		200 years			
10^{-3}		1000 years		The event could occur under adverse conditions over the design life.	POSSIBLE	C
	5×10^{-4}		2000 years			
10^{-4}		10,000 years		The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
	5×10^{-5}		20,000 years			
10^{-5}		100,000 years		The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
	5×10^{-6}		200,000 years			
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%		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
	100%			
60%		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
	40%			
20%		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
	10%			
5%		Limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works.	MINOR	4
	1%			
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.

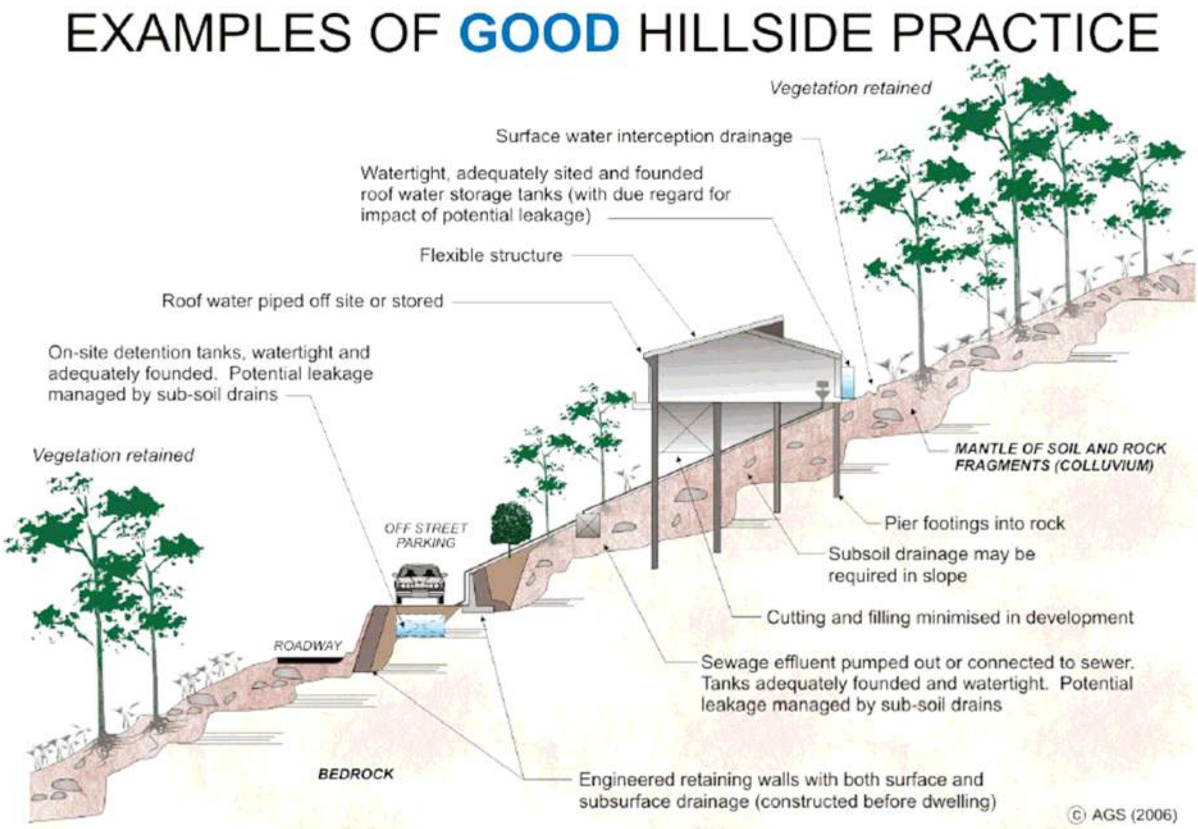
Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

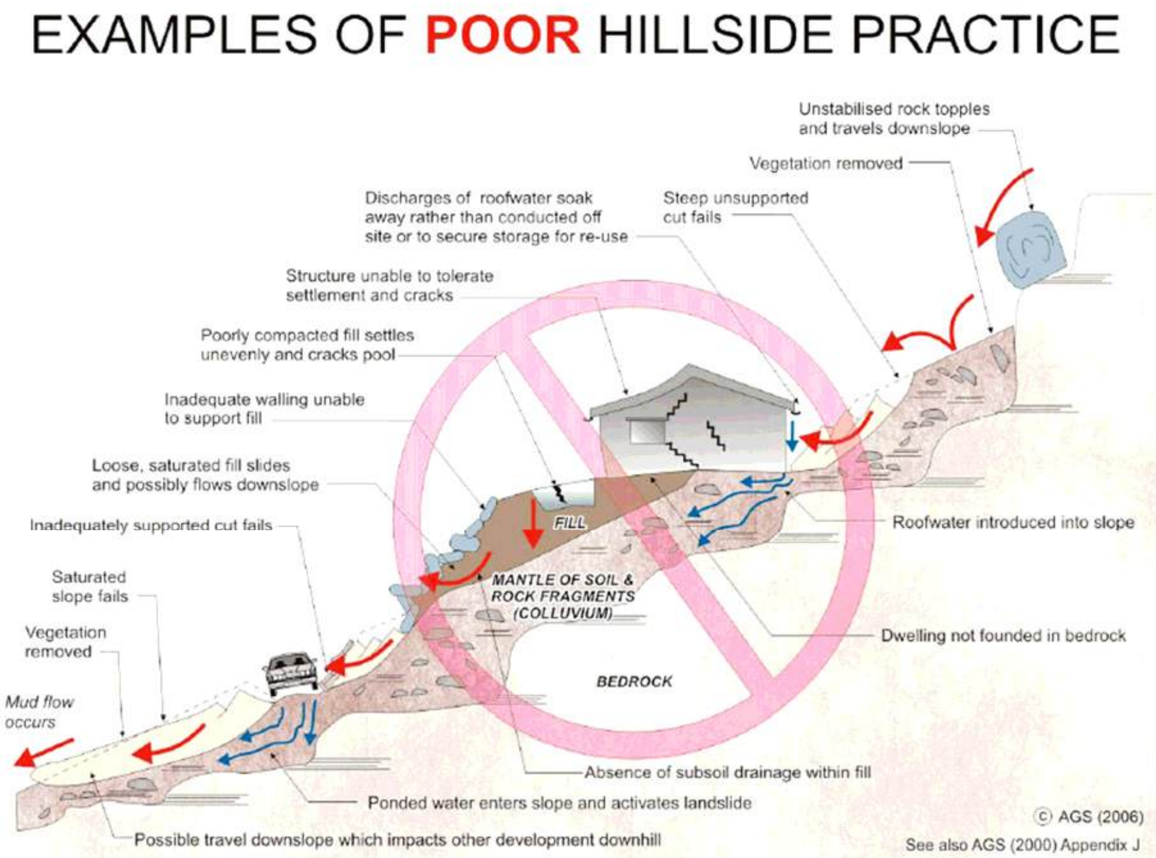
APPENDIX F

PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT AGS 2007 AGS – APPENDIX G – SOME GUIDELINES FOR HILLSIDE CONSTRUCTION INTRODUCTION

APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

	GOOD ENGINEERING PRACTICE	POOR ENGINEERING PRACTICE
ADVICE		
GEOTECHNICAL ASSESSMENT	Obtain advice from a qualified, experienced geotechnical practitioner at early stage of planning and before site works.	Prepare detailed plan and start site works before geotechnical advice.
PLANNING		
SITE PLANNING	Having obtained geotechnical advice, plan the development with the risk arising from the identified hazards and consequences in mind.	Plan development without regard for the Risk.
DESIGN AND CONSTRUCTION		
HOUSE DESIGN	Use flexible structures which incorporate properly designed brickwork, timber or steel frames, timber or panel cladding. Consider use of split levels. Use decks for recreational areas where appropriate.	Floor plans which require extensive cutting and filling. Movement intolerant structures.
SITE CLEARING	Retain natural vegetation wherever practicable.	Indiscriminately clear the site.
ACCESS & DRIVEWAYS	Satisfy requirements below for cuts, fills, retaining walls and drainage. Council specifications for grades may need to be modified. Driveways and parking areas may need to be fully supported on piers.	Excavate and fill for site access before geotechnical advice.
EARTHWORKS	Retain natural contours wherever possible.	Indiscriminatory bulk earthworks.
CUTS	Minimise depth. Support with engineered retaining walls or batter to appropriate slope. Provide drainage measures and erosion control.	Large scale cuts and benching. Unsupported cuts. Ignore drainage requirements
FILLS	Minimise height. Strip vegetation and topsoil and key into natural slopes prior to filling. Use clean fill materials and compact to engineering standards. Batter to appropriate slope or support with engineered retaining wall. Provide surface drainage and appropriate subsurface drainage.	Loose or poorly compacted fill, which if it fails, may flow a considerable distance including onto property below. Block natural drainage lines. Fill over existing vegetation and topsoil. Include stumps, trees, vegetation, topsoil, boulders, building rubble etc in fill.
ROCK OUTCROPS & BOULDERS	Remove or stabilise boulders which may have unacceptable risk. Support rock faces where necessary.	Disturb or undercut detached blocks or boulders.
RETAINING WALLS	Engineer design to resist applied soil and water forces. Found on rock where practicable. Provide subsurface drainage within wall backfill and surface drainage on slope above. Construct wall as soon as possible after cut/fill operation.	Construct a structurally inadequate wall such as sandstone flagging, brick or unreinforced blockwork. Lack of subsurface drains and weepholes.
FOOTINGS	Found within rock where practicable. Use rows of piers or strip footings oriented up and down slope. Design for lateral creep pressures if necessary. Backfill footing excavations to exclude ingress of surface water.	Found on topsoil, loose fill, detached boulders or undercut cliffs.
SWIMMING POOLS	Engineer designed. Support on piers to rock where practicable. Provide with under-drainage and gravity drain outlet where practicable. Design for high soil pressures which may develop on uphill side whilst there may be little or no lateral support on downhill side.	
DRAINAGE		
SURFACE	Provide at tops of cut and fill slopes. Discharge to street drainage or natural water courses. Provide general falls to prevent blockage by siltation and incorporate silt traps. Line to minimise infiltration and make flexible where possible. Special structures to dissipate energy at changes of slope and/or direction.	Discharge at top of fills and cuts. Allow water to pond on bench areas.
SUBSURFACE	Provide filter around subsurface drain. Provide drain behind retaining walls. Use flexible pipelines with access for maintenance. Prevent inflow of surface water.	Discharge roof runoff into absorption trenches.
SEPTIC & SULLAGE	Usually requires pump-out or mains sewer systems; absorption trenches may be possible in some areas if risk is acceptable. Storage tanks should be water-tight and adequately founded.	Discharge sullage directly onto and into slopes. Use absorption trenches without consideration of landslide risk.
EROSION CONTROL & LANDSCAPING	Control erosion as this may lead to instability. Revegetate cleared area.	Failure to observe earthworks and drainage recommendations when landscaping.
DRAWINGS AND SITE VISITS DURING CONSTRUCTION		
DRAWINGS	Building Application drawings should be viewed by geotechnical consultant	
SITE VISITS	Site Visits by consultant may be appropriate during construction/	
INSPECTION AND MAINTENANCE BY OWNER		
OWNER'S RESPONSIBILITY	Clean drainage systems; repair broken joints in drains and leaks in supply pipes. Where structural distress is evident see advice. If seepage observed, determine causes or seek advice on consequences.	


EXAMPLES OF GOOD HILLSIDE PRACTICE
Vegetation retained
Surface water interception drainage
Watertight, adequately sited and founded roof water storage tanks (with due regard for impact of potential leakage)
Flexible structure
Roof water piped off site or stored
On-site detention tanks, watertight and adequately founded. Potential leakage managed by sub-soil drains
Vegetation retained
MANTLE OF SOIL AND ROCK FRAGMENTS (COLLUVIUM)
OFF STREET PARKING
Pier footings into rock
Subsoil drainage may be required in slope
ROADWAY
Cutting and filling minimised in development
Sewage effluent pumped out or connected to sewer. Tanks adequately founded and watertight. Potential leakage managed by sub-soil drains
BEDROCK
Engineered retaining walls with both surface and subsurface drainage (constructed before dwelling)
© AGS (2006)


EXAMPLES OF POOR HILLSIDE PRACTICE
Unstabilised rock topples and travels downslope
Vegetation removed
Discharges of roofwater soak away rather than conducted off site or to secure storage for re-use
Steep unsupported cut fails
Structure unable to tolerate settlement and cracks
Poorly compacted fill settles unevenly and cracks pool
Inadequate walling unable to support fill
Loose, saturated fill slides and possibly flows downslope
FILL
Roofwater introduced into slope
Inadequately supported cut fails
MANTLE OF SOIL & ROCK FRAGMENTS (COLLUVIUM)
Saturated slope fails
Dwelling not founded in bedrock
Vegetation removed
BEDROCK
Mud flow occurs
Absence of subsoil drainage within fill
Ponded water enters slope and activates landslide
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Possible travel downslope which impacts other development downhill
See also AGS (2000) Appendix J