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REPORT ON GEOTECHNICAL SITE INVESTIGATION

for

PROPOSED NEW GARAGE

at

21 HERBERT STREET, MANLY

Prepared For

Andrew Vinson

Project No.: 2019-163

October, 2019

Document Revision Record

Issue No	Date	Details of Revisions
0	11th October 2019	Original issue

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GEOTECHNICAL REPORT FOR PROPOSED NEW GARAGE
21 HERBERT STREET, MANLY, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation undertaken for a proposed new garage at 21 Herbert Street, Manly, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of Baxter Jacobson Architects on behalf of Andrew Vinson.

The site is located within Landslip Risk Class óG4ô as identified within Northern Beaches (Manly) Councils ï Development Control Plan 2013 ï Schedule 1 Map C. A review of the preliminary slope stability assessment checklist and the development proposal identified that the Development Application is likely to require a full site stability (geotechnical) report. This full report must detail how the development may be achieved to ensure geotechnical stability and good engineering practice. The report must also include a risk assessment for existing/potential instability as per the AGS March 2007 publication.

This report therefore includes a detailed description of field work including site description, test results, a plan showing geotechnical mapping and test locations, a geotechnical assessment of the proposed works/site conditions, a risk assessment and provides preliminary design and construction recommendations.

The investigation was undertaken as per the Tender: P19-321, Dated: 12th September 2019 and comprised the following (requested) scope of work .

a) A detailed geotechnical inspection and mapping of the site and adjacent properties by a Senior Engineering Geologist.

b) Excavation of three test pits along and two hand auger boreholes using hand tools and six Dynamic Cone Penetrometer (DCP) tests to investigate subsurface conditions.

c) Onsite mark-out/clearance of test locations by an accredited service location contractor.

The following plans and drawings were supplied for the work:

- Site survey plan by CMS Surveyors Pty Ltd, Drawing Name: 18671detail, Dated: 04/07/2019.
- Architectural Drawings ï Concept drawings by Baxter & Jacobson Architects, Job No.: 411-01, Drawing No.: SK 01, Issue B, Dated: 2nd August 2019.

2. PROPOSED DEVELOPMENT:

It is understood that the proposed works involve the construction of a new garage and driveway which extends from the front (north) of the property to slightly under the mid-point of the existing site house. A total height of approximately 2.6m is required under the house and excavation up to approximately 2.0m depth will be required to achieve FFL24.5m. It is further understood that it is intended to underpin the existing sandstone subfloor walls, install internal vertical props, construct new block walls and replace/re-mortar some areas of the existing walls as required.

The excavation will be within 1.0m, 1.5m, and >20m of the west, east and south boundaries and up the front boundary to achieve level access with Herbert Street.

3. SITE FEATURES:

3.1. Site Description:

The site is a trapezoidal shaped block located on the high, south side of Herbert Street within gently north dipping topography. The site contains one and two storey dwelling with front and rear gardens and access paths.

It has front north and rear south boundaries of approximately 10.1m and 9.1m and side east and west boundaries of approximately 26.3m and 30.8m respectively and covers an area of approximately 261m^2 as referenced from supplied survey plan.

Ground surface levels within the site reduce from a high of approximately RL28.5m within the south west corner to a low of approximately RL25.0m near north east corner. An aerial image showing the site and immediate surrounds is shown in Photograph 2.

Photograph 1: Aerial view of the site and immediate surrounds

Herbert Street comprises an asphalt road which dips slightly to the west and is bound by a concrete kerb beyond which lies a grass easement and pedestrian pavement. The front of the site contains a gently north dipping lawn west of a concrete access pathway and to the south of a low sandstone wall on the front boundary. The site dwelling is constructed of brickwork formed off sandstone blockwork creating a sub-floor void. A concrete pathway provides access to the rear of the property which contains a paved patio and elevated lawn supported by a concrete block retaining wall. A general view of the site is provided in Photograph 2.

Photograph 2: View of the front of the site and immediate surrounds looking broadly south.

The NSW Governments Six Maps website indicates that the area had been developed by 1943 and the site and surrounding structures appear to have changed very little since that time.
The properties to the east and west (No.19 and No.23 Herbert Street respectively) contain one and two storey residential dwellings of brick (No.23) or clad and rendered (No.19) construction with front and rear gardens. To the south of the site lies No.24 Arthur Street which contains a residential dwelling.

3.2. Geology:

Reference to the Sydney 1: 100,000 Geological Series sheet (9130) indicates that the site is underlain by Hawkesbury Sandstone (Rh) which is of Triassic Age. The rock unit typically comprises medium to coarse grained quartz sandstone with minor lenses of shale and laminite. An extract of the 9130 Geological Series sheet is provided in Extract 1.

Extract 1: Geology underlying the site (circled and shown in red)

Morphological features often associated with the weathering of Hawkesbury Sandstone are the formation of near flat ridge tops with steep angular side slopes. These slopes often consist of sandstone terraces and cliffs with steep colluvial slopes below. The terraced areas above these cliffs often contain thin sandy (low plasticity) soil profiles with intervening rock (ledge) outcrops.

4. FIELD WORK:

4.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site and adjacent properties on the 26th September 2019 by a Senior Engineering Geologist. It included a photographic record of site conditions as well as geological/geomorphological mapping of the site and adjacent land with examination of outcrops, slopes and existing structures. It also included the excavation of three test pits (TP1 to TP3) and two boreholes (BH1 and BH2) using a hand tools to determine subsurface conditions. The investigation was limited to hand tools due to access conditions.

Dynamic Cone Penetrometer (DCP) testing was carried out within or adjacent to the test pit/borehole locations in accordance with AS1289.6.3.2 ï 1997, ñDetermination of the penetration resistance of a soil ï 9kg Dynamic Cone Penetrometerò to estimate near surface soil conditions and assess depths to bedrock.

The location of TP2 was moved to the south of the requested location to avoid a possible stormwater pipe within and to the north the original proposed location.

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

4.2. Field Observations:

The site contains a one and two storey brick house within gently north dipping topography. The existing dwelling is in good condition and no signs of cracking were observed within the external walls of the dwelling. A low retaining wall within the rear garden of the property displayed some signs of rotation/degradation however this is likely due to insufficient construction methods rather than a deep-seated stability issue as it appears the retaining wall is at least partially founded on bedrock (See Photograph 3). The bedrock observed in outcrop within the rear of the property comprised medium strength sandstone. Bedrock was also observed elsewhere adjacent to Herbert Street (Photograph 4) to the west and within the neighbouring property to the east (No.19).

Photographs 3 and 4: Bedrock outcrops within and adjacent to the site.

The property to the west of the site (No.23) contained a single storey brick residence which appeared in good condition and is at broadly the same elevation to the site immediately adjacent to shared boundary.

The neighbouring property to the east (No.19) contained a two storey rendered/clad dwelling broadly at a similar level to the site and did not display any indications of distress.

The neighbouring property to the south (No.24 Arthur Street) contains a single storey clad dwelling which appeared in good condition. The property is at a similar level to the site immediately adjacent to the shared boundary.

Herbert Street comprised an asphalt pavement with a concrete kerb and was gently west dipping were it passed the site and appeared in good condition.

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

4.3. Ground Conditions:

For a description of the subsurface conditions encountered at the test locations, the Borehole/Test Pit Log sheets should be consulted. However, a broad summary of the subsurface conditions encountered at the site is given below.

- **FILL** ï this layer was encountered at all test pit/borehole locations to a maximum depth of 0.65m (BH2) and predominantly comprised sandy gravel or gravelly sand containing fragments of brick, tile and sandstone. Within TP1 a possible boulder or irregularly shaped intact bedrock was observed between 0.5m and 1.15m depth within the southern side of the test pit.
- **SILTY SAND** ï Predominately loose to medium dense silty sand was encountered in TP1 and TP3 to a maximum depth of 1.20m.
- **SANDY CLAY** ï Underlying the silty sand in TP1 only, soft grey sandy clay was encountered between 0.80m and the upper surface of the underlying bedrock.
- **SANDSTONE BEDROCK** ï Sandstone of at least low strength was encountered within the test pits/boreholes undertaken directly adjacent to the site house and similar strength bedrock has been interpreted as being encountered within the DCP tests undertaken in the front garden. On the west side of the site dwelling bedrock was encountered at a depth of 0.20m (TP2) and 0.60m within DCP2 a short distance outside the test pit location, suggesting the presence of a steeply inclined ledge.

Groundwater was encountered in TP3 only at a depth of 0.6m underlying the existing ground surface. Groundwater was not encountered within any other borehole or test pit and was not observed on the DCP rods on retrieval.

4.4 Existing Footings

Within TP1 the base of the house external sandstone wall was founded at approximately 0.5m below the existing path level and bedrock was encountered at approximately 1.15m depth. Within the southern side of the test pit the wall was founded off what appeared to be a boulder however it may represent an irregular surface of in-situ bedrock. Within the northern side of the test pit the sandstone wall was founded off natural soils comprising of loose clayey sand.

Within TP2 the sandstone wall was founded off low strength bedrock at 0.20m depth however it appears the bedrock surface may contain steep sided ledges as the bedrock surface appeared to reduce to 0.60m depth below the ground surface based on the results on DCP2.

Within TP3 the sandstone wall was founded off sandstone bedrock of at least low strength at approximately 1.0m depth.

5. COMMENTS:

5.1. Ground Model

The site investigation identified the presence of granular fill to a maximum depth of 0.65m (BH2) underlain by loose/medium dense clayey sand and/or soft to stiff sandy clay to 0.80m depth. Very soft grey clay was encountered within TP1 only below 0.80m depth and was underlain by at least low strength bedrock which was encountered at all test locations. A possible boulder or irregular in-situ bedrock was encountered below 0.50m in TP1. The boulder may represent a shelf feature as observed elsewhere however this is unconfirmed. Based on the results of the test pits it appears that sections of the housesôsandstone block wall will be founded off bedrock whilst other sections will be founded within clayey/sandy soils.

The depth to sandstone bedrock varies from approximately 0.20m (TP2) to 1.15m (TP1) and it is considered that the upper surface will vary in elevation as a result of steeply sided ledges. The transition from soft/loose soils to low strength sandstone appears to be rapid with a limited thickness of overlying residual or extremely low to very low strength bedrock.

Where excavation occurs below the site house it is anticipated the excavation side walls will consist of sections of natural soil comprising clayey sand/sandy clay, sandstone bedrock and sections of the existing perimeter walls. The exposed soil will be unlikely to remain stable in the short term (vertically) and will require battering back supported prior to bulk excavation. Preliminary safe batter slopes are provided in Section 5.3.2 however where the house walls found on soils, underpinning will be required however this will need to be confirmed on site. Where insufficient space exists for construction of safe batter slopes, soils will need to be retained prior to bulk excavation.

It is anticipated that up to approximately 1.2m height/depth of fill/natural soils will be exposed in the garage excavation under which at least low strength sandstone bedrock will be encountered. The sandstone will likely grade very quickly to medium or stronger based on a tactile assessment of bedrock exposures within and surrounding the site.

The excavation of medium to high strength bedrock will require the use of rock excavation equipment, which can produce damaging ground vibrations. Since the excavation work is to be carried out within approximately

2.0m of neighbouring residential buildings, it is recommended that either the size of excavation equipment is limited (specifically rock hammers) or vibration monitoring be undertaken to confirm that proposed excavation equipment will not result in vibrations which exceed the preliminary threshold limits provided in Section 5.3.3. Based on the site conditions it is anticipated that equipment selection will likely be dictated by access however excavation contractors/builders should be made aware of any potential machinery restrictions to enable accurate tendering (and assessment of) as well as assisting in project planning.

Preliminary allowable bearing pressures appropriate for the sandstone encountered underlying the site are provided in Section 5.3.1 based on assessment of observed bedrock strength. However, the bearing capacity of sandstone bedrock is critically controlled by the presence of any defects, clay seams or zones of intensely weathered bedrock. Where the bedrock supporting the existing house footings is free of any defects/weaker zones to the full depth of garage excavation, higher allowable bearing pressures than those provided in Section 5.3.1 may be available. Conversely, where weaker zones are encountered within the sandstone bedrock underlying the existing footings, allowable bearing pressures may require downgrading to less than those provided in Section 5.3.1.

New footings to support the proposed retaining new retaining structures in the front garden of the property will also be required. The results of the investigation indicated that fill was present to a depth of approximately 0.5m and new footings should extend below this depth. DCP5 and DCP3a indicated very soft/loose soils were present at approximately 0.75 and 0.90m depth which are not considered suitable to bear even very lightly loaded footings unless the potential for some differential movement can be tolerated. Where movement cannot be tolerated all footings should extend to the sandstone bedrock encountered at approximately 1.0m depth below the existing ground surface level.

In addition to the determination of appropriate maximum allowable bearing pressures, the presence of defects within the rock mass will also determine whether the maximum preliminary batter slopes provided in Section 5.3.2 are appropriate for the exposed bedrock or whether the presence or un-favourable jointing /clay seams etc. may require the installation of additional support to the excavation. Therefore, it is recommended (and will likely be necessary to fulfil Council requirements) that inspection of the excavation be undertaken as excavation progresses under the house in order to confirm the appropriate bearing pressures and assess whether additional support or underpinning is required.

A risk assessment undertaken for the proposed development (See Section 5.2) indicated that there were limited stability hazards identified in the investigation which could potentially impact neighbouring properties, however there is a potential for soil instability to result in localized soil landslip which has potential impact to the work site or the existing paved/access areas adjacent to the excavation. Through selection of suitable excavation equipment, geotechnical inspection and mapping during the excavation works along with the installation of support measures as determined necessary by the inspections, the risk from the proposed works can be maintained within óAcceptableô levels for all situations.

The recommendations and conclusions in this report are based on an investigation utilising only surface observations and hand tools due to access limitations. This test equipment provides limited data from small isolated test points across the entire site with limited penetration into bedrock, therefore some minor variation to the interpreted subsurface conditions is possible, especially between test locations. The results of the investigation provide a reasonable basis for the DA analysis and preliminary design.

5.2. Site Specific Risk Assessment:

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

A. Landslip of unsupported soils in garage excavation walls (<$3m^3$)
B. Rockslide/topple failure in bedrock due to defects (<$0.5m^3$)

A qualitative assessment of risk to life and property related to these hazards 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 **2.08×10^{-6} for persons within the excavation and varies from 1.04×10^{-7} to 5.20×10^{-8} for the neighbouring properties, whilst the Risk to Property was considered to be 'Very Low'** in all situations.

The Risk to Life from Hazard B was estimated to be up to **5.20×10^{-7} for persons within the excavation and was not considered a risk to persons within neighbouring properties, whilst the Risk to Property was considered to be 'Very Low'**.

The works are considered to have an óAcceptableô risk level when assessed against the criteria of the AGS 2007 and would be expected to further reduce where geotechnical inspection during the construction phase is undertaken.

5.3. Design & Construction Recommendations:

Design and construction recommendations are tabulated below:

5.3.1. New Footings:	
Site Classification as per AS2870 ï 2011 for new footing design	Class ôAô for footings on bedrock at the base of the excavation, Class ôPô for all other footings.
Type of Footing	Strip/Pad or Slab at base of excavation, piers outside the excavation.
Recommended Founding Strata and Maximum Allowable Bearing Capacity	- Low Strength Sandstone: 1,000kPa - Medium Strength Sandstone: 2,000kPa
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	B_e ï rock site
Remarks: The strength of the bedrock with depth is unconfirmed therefore there is a potential for the bedrock to be more deeply weathered and of lesser or higher strength than interpreted. All footings should be founded off bedrock of similar strength to reduce the potential for differential settlement. Higher pressures based on the Pells et al (1988) system are possible however further investigation assessment would be required utilizing core drilling techniques. All new footings must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity and the in-situ nature of the founding strata. This is mandatory to allow them to be ócertifiedô at the end of the project.	

5.3.2. Excavation:	
Depth of Excavation	New garage and driveway ï Up to 2.60m (total height) at the rear of the garage
Distance to Neighbouring Properties/Structures	No.19 Herbert Street ï 1.5m to the common boundary, building a further 0.5m away, No.23 Herbert Street ï 1.0m to the common boundary, building a further 1.5m away, No.24 Arthur Street ï >10m to the common boundary, North boundary ï Up to Herbert Street to create near level access.

Type of Material to be Excavated	Up to 1.15m	Granular fill and natural sand/clay.
	Below 1.15m	LS-MS/HS bedrock

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

Material	Safe Batter Slope (H:V)*	
	Short Term/ Temporary	Long Term/ Permanent
Fill and natural soils	1:1.5	2:1
Low strength bedrock	1:1	1.25:1
Medium strength (MS), defect free bedrock	Vertical	Vertical

*Dependent on defects and assessment by engineering geologist.

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
	LS ï MS/HS bedrock	Rock hammer and saw

LS ï low strength, MS ï medium strength, HS ï High Strength

Remarks:
Based on previous testing of ground vibrations created by various rock excavation equipment within medium strength bedrock, to achieve a low level of vibration as recommended by the Australian Standards then the below hammer weights to approximate buffer distances are required:

Buffer Distance from Structure	Maximum Hammer Weight
2.0m	200kg
4.0m	500kg
5.0m	800kg
8.0m	1000kg

Vibration generation is dependent on rock strength, defects, hammer weight and power/weight ratio of any particular hammer. Onsite calibration will provide accurate vibration levels to the site specific conditions and will generally allow for larger excavation machinery or smaller buffers to be used. Calibration of rock excavation machinery will need to be carried out prior to commencement of rock excavation works if rock

hammers heavier than approximately 100kg are proposed which will determine the need for full time monitoring. Rock sawing of the excavation perimeter is recommended as it has several advantages. It often reduces the need for rock bolting as the cut faces generally remain more stable and require a lower level of rock support than hammer cut excavations, ground vibrations from rock saws are minimal and the saw cuts will provide a slight increase in buffer distance for use of rock hammers.	
Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	Buildings in surrounding properties = 5mm/s
Vibration Calibration Tests Required	Required where hammers heavier than approximately 100kg proposed.
Full time vibration Monitoring Required	Pending proposed equipment and vibration calibration testing results
Geotechnical Inspection Requirement	Yes, recommended that these inspections be undertaken as per below mentioned sequence: ● Following cleaning of soils from bedrock surface ● At 1.50m depth interval of excavation where unsupported ● At completion of the excavation
Dilapidation Surveys Requirement	On neighbouring structures or parts thereof within 10m of the excavation perimeter prior to site work to allow assessment of the recommended vibration limit and protect the client against spurious claims of damage.
Remarks: Water ingress into exposed excavations can result in erosion and stability concerns in both soil and rock portions. Drainage measures will need to be in place during excavation works to divert any surface flow away from the excavation crest and any batter slope.	

5.3.3. Retaining Structures:	
Required	The clayey/sand soils encountered will require support prior to excavation if batter slope can not be constructed. Low to medium strength sandstone bedrock is self-supporting however this will need to be confirmed via geotechnical inspection.
Types	Steel reinforced concrete/concrete block wall designed in accordance with Australian Standard AS 4678-2002 Earth Retaining Structures.

	Pre excavation support may be required where the possibility of instability cannot be tolerated or managed.

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	ű' = 32°	0.30	0.50	N/A
LS or fractured bedrock	23	ű' = 40°	0.10	0.15	400kPa

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 backfilled with free-draining granular material (preferably not recycled concrete) which is only lightly compacted in order to minimize horizontal stresses.

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

5.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		Seepage encountered in TP3 at 0.6m depth.
Excavation likely to intersect	Water Table	No
	Seepage	Minor (≤0.50 L/min), on defects and at soil/rock interface
Site Location and Topography		High south side of the road within gently north sloping topography
Impact of development on local hydrogeology		Negligible
Onsite Stormwater Disposal		Not considered necessary

Remarks: As the excavation faces are expected to encounter some seepage, an excavation trench should be installed at the base of excavation cuts to below floor slab levels to reduce the risk of resulting dampness issues.

Trenches, as well as all new building gutters, down pipes and stormwater intercept trenches should be connected to a stormwater system designed by a Hydraulic Engineer which discharges to the Councilôs stormwater system off site.

5.4. Conditions Relating to Design and Construction Monitoring:

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

1. Review and approve the structural drawings and stormwater system plans for compliance with the recommendations of this report,
2. Review excavation Methodology and Equipment prior to hard rock excavation,
3. Inspect excavation at 1.50m depth intervals where unsupported,
4. Inspect all new footings to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and the stability prior to the placement of steel or concrete,
5. Inspect the completed development to ensure all stormwater systems are complete and connected and that construction activity has not created any new landslip hazards.

The client and builder should make themselves familiar with the Councils Policy and the requirements spelled out in this report for inspections during the construction phase. Crozier Geotechnical Consultants cannot complete the certification if it has not been called to site to undertake the required inspections.

6. CONCLUSION:

The site investigation identified the presence of granular fill of shallow thickness (<0.5m) overlying soft/loose soils to a maximum of 1.15m depth which was underlain by low strength sandstone bedrock. This low strength bedrock is expected to grade very quickly to medium strength and likely stronger.

The proposed works involve excavation under the existing house and construction of a new garage which will result in an excavation face of up to 2.6m in depth/height.

The risk from geological/geotechnical hazards which were identified in relation to the proposed works is limited to excavation failure and can be considered ôAcceptableö, when assessed against the criteria of the AGS 2007, provided the recommendations of this report and future geotechnical directive during excavation are implemented.

Inspection on excavated faces will be required during construction to confirm preliminary bearing pressures, excavation stability and the need for any underpinning of existing footings.

The excavation of medium to high strength bedrock will require the use of rock excavation equipment, which can produce damaging ground vibrations. Since the excavation work is to be carried out within approximately 2.0m of surrounding structures, it is recommended that the geotechnical engineer or engineering geologist assess the excavation methodology and equipment prior to commencement to determine the need for vibration monitoring of neighbouring structures.

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

Kieron Nicholson

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7. REFERENCES:

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

Appendix 1

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NOTES RELATING TO THIS REPORT

Introduction

These notes have been provided to amplify the geotechnical report in regard to classification methods, specialist field procedures and certain matters relating to the Discussion and Comments section. Not all, of course, are necessarily relevant to all reports.

Geotechnical reports are based on information gained from limited subsurface test boring and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Description and classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, Geotechnical Site Investigation Code. In general, descriptions cover the following properties - strength or density, colour, structure, soil or rock type and inclusions.

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (eg. Sandy clay) on the following bases:

Soil Classification	Particle Size
Clay	less than 0.002 mm
Silt	0.002 to 0.06 mm
Sand	0.06 to 2.00 mm
Gravel	2.00 to 60.00mm

Cohesive soils are classified on the basis of strength either by laboratory testing or engineering examination. The strength terms are defined as follows:

Classification	Undrained Shear Strength kPa
Very soft	Less than 12
Soft	12 - 25
Firm	25 – 50
Stiff	50 – 100
Very stiff	100 - 200
Hard	Greater than 200

Non-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT) as below:

Relative Density	SPT "N" Value (blows/300mm)	CPT Cone Value (Qc – MPa)
Very loose	less than 5	less than 2
Loose	5 – 10	2 – 5
Medium dense	10 – 30	5 -15
Dense	30 – 50	15 – 25
Very dense	greater than 50	greater than 25

Rock types are classified by their geological names. Where relevant, further information regarding rock classification is given on the following sheet.

Sampling

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

Disturbed samples taken during drilling to allow information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Drilling Methods

The following is a brief summary of drilling methods currently adopted by the company and some comments on their use and application.

Test Pits – these are excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils if it is safe to descent into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (eg. Pengo) – the hole is advanced by a rotating plate or short spiral auger, generally 300mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling – the hole is advanced by pushing a 100mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling soils, since moisture content is unchanged and soil structure, strength, etc. is only marginally affected.

Continuous Spiral Flight Augers – the hole is advanced using 90 – 115mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPT's or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

Non-core Rotary Drilling - the hole is advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from 'feel' and rate of penetration.

Rotary Mud Drilling – similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. From SPT).

Continuous Core Drilling – a continuous core sample is obtained using a diamond-tipped core barrel, usually 50mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample. The test procedures is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" – Test 6.3.1.

The test is carried out in a borehole by driving a 50mm diameter split sample tube under the impact of a 63kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150mm of say 4, 6 and 7 as 4, 6, 7 then N = 13
- In the case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm then as 15, 30/40mm.

The results of the test can be related empirically to the engineering properties of the soil. Occasionally, the test method is used to obtain samples in 50mm diameter thin wall sample tubes in clay. In such circumstances, the test results are shown on the borelogs in brackets.

Cone Penetrometer Testing and Interpretation

Cone penetrometer testing (sometimes referred to as Dutch Cone – abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in Australia Standard 1289, Test 6.4.1.

In tests, a 35mm diameter rod with a cone-tipped end is pushed continually into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the friction resistance on a separte 130mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected buy electrical wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20mm per second) their information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises: -

- Cone resistance – the actual end bearing force divided by the cross-sectional area of the cone – expressed in MPa.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio - the ratio of sleeve friction to cone resistance, expressed in percent.

There are two scales available for measurement of cone resistance. The lower scale (0 – 5 MPa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main scale (0 – 50 MPa) is less sensitive and is shown as a full line. The ratios of the sleeve friction to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios 1% - 2% are commonly encountered in sands and very soft clays rising to 4% - 10% in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range: -

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

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

Q_c = (12 to 18) C_u

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

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

Dynamic Penetrometers

Dynamic penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150mm increments of penetration. Normally, there is a depth limitation of 1.2m but this may be extended in certain conditions by the use of extension rods.

Two relatively similar tests are used.

- Perth sand penetrometer – a 16mm diameter flattened rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test 6.3.3). The test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.
- Cone penetrometer (sometimes known as Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS 1289, Test 6.3.2). The test was developed initially for pavement sub-grade investigations, and published correlations of the test results with California bearing ratio have been published by various Road Authorities.

Laboratory Testing

Laboratory testing is generally carried out in accordance with Australian Standard 1289 “Methods of Testing Soil for Engineering Purposes”. Details of the test procedure used are given on the individual report forms.

Borehole Logs

The bore logs presented herein are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable, or possible to justify on economic grounds. In any case, the boreholes represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes, the frequency of sampling and the possibility of other than ‘straight line’ variations between the boreholes.

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

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

Ground Water

Where ground water levels are measured in boreholes there are several potential problems:

- In low permeability soils, ground water although present, may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report.
- The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water observations are to be made. More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be interference from a perched water table.

Engineering Reports

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. A three-storey building), the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty-storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface condition, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- unexpected variations in ground conditions – the potential for this will depend partly on bore spacing and sampling frequency,
- changes in policy or interpretation of policy by statutory authorities,
- the actions of contractors responding to commercial pressures,

If these occur, the Company will be pleased to assist with investigation or advice to resolve the matter.

Site Anomalies

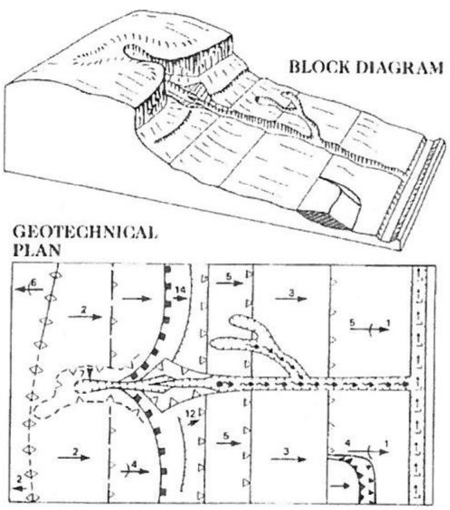
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

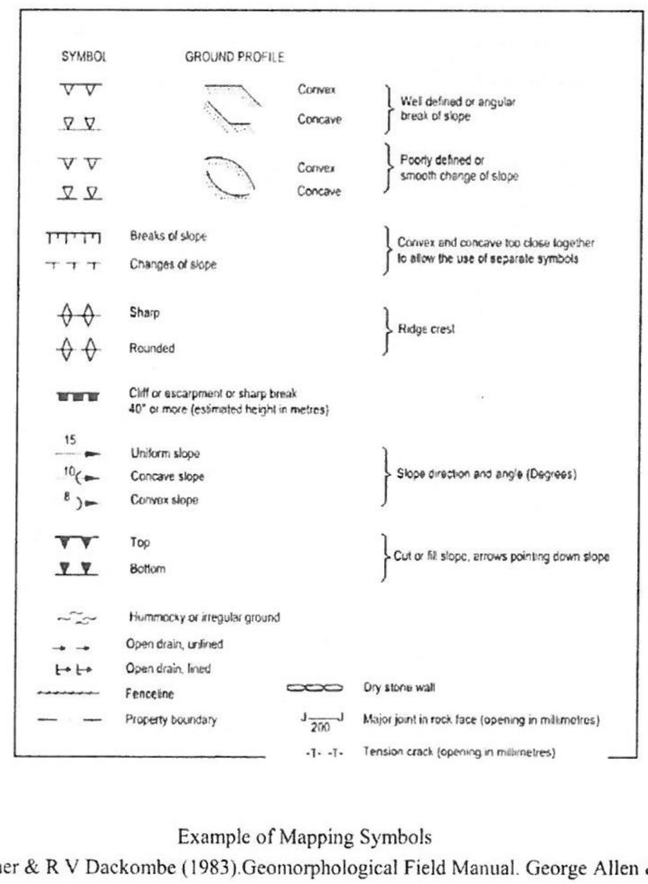
Reproduction of Information for Contractual Purposes

Attention is drawn to the document "Guidelines for the Provision of Geotechnical Information in Tender Documents", published by the Institution of Engineers Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a special ally edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

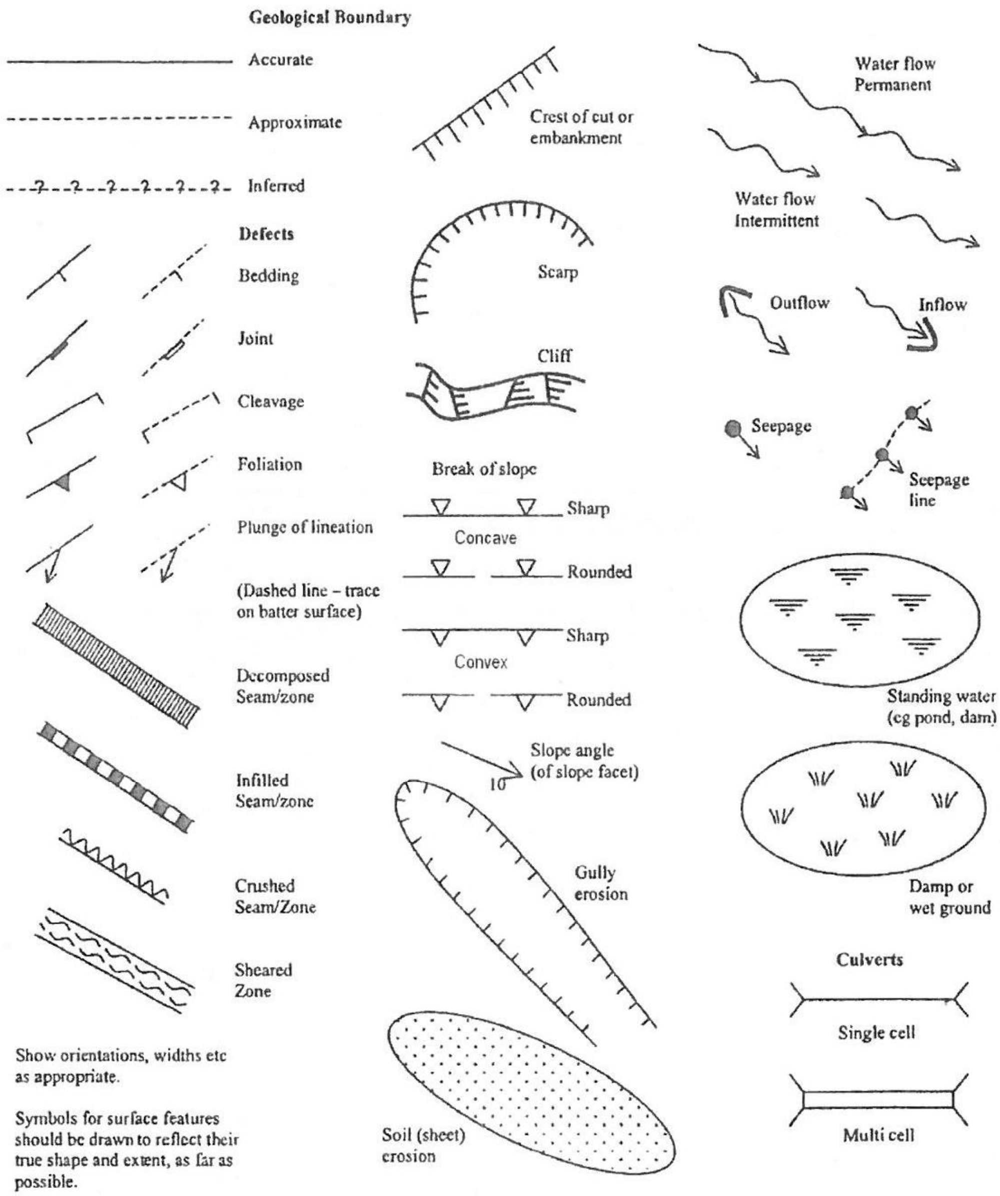
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.





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

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY



Show orientations, widths etc as appropriate.

Symbols for surface features should be drawn to reflect their true shape and extent, as far as possible.

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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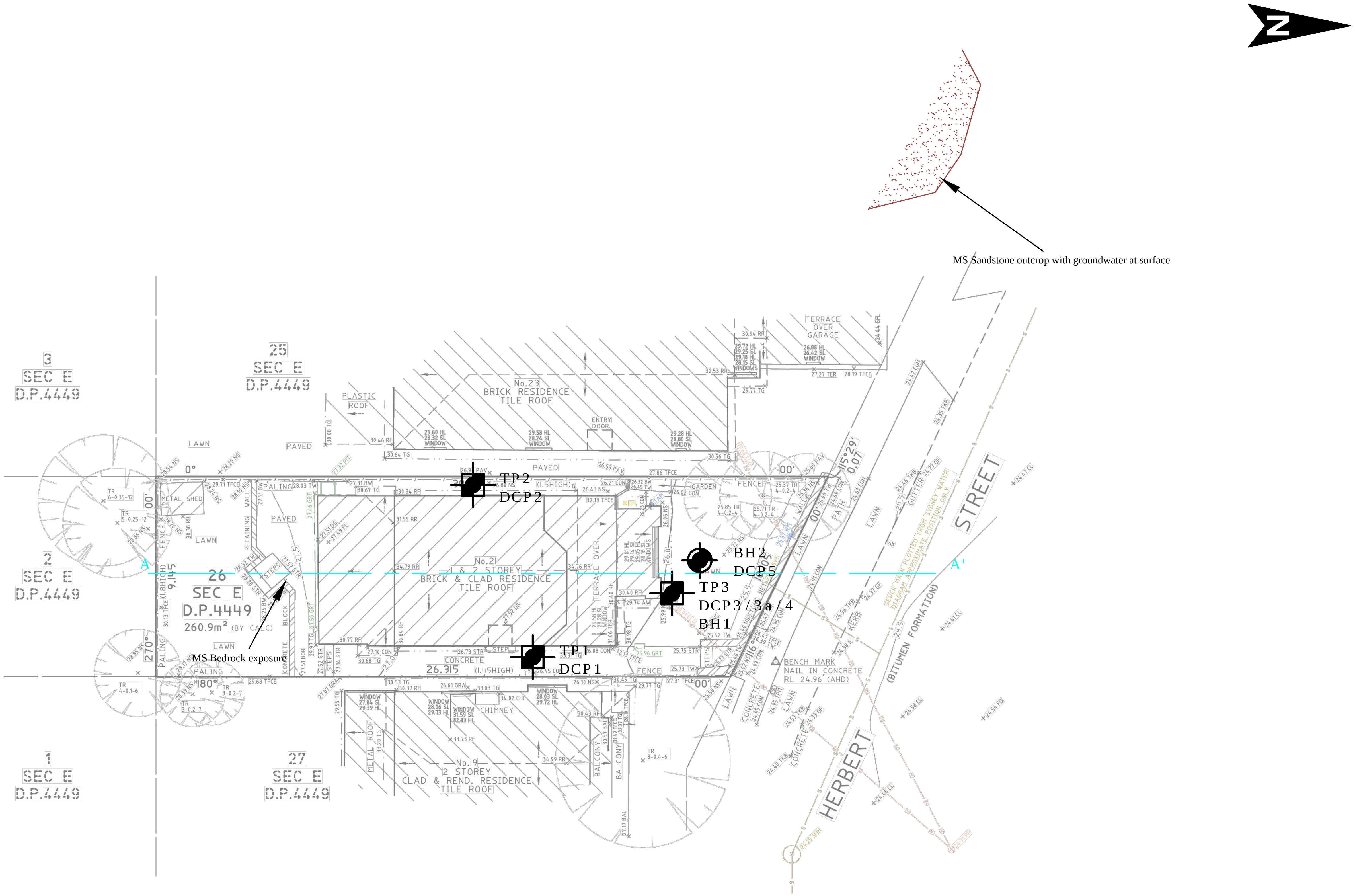
Appendix 2



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.

CROZIER GEOTECHNICAL CONSULTANTS

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

LEGEND

A —— A' CROSS-SECTION REFERENCE LINE

TP DCP TEST PIT / DYNAMIC CONE PENETROMETER LOCATION

BH DCP AUGER / DYNAMIC CONE PENETROMETER LOCATION

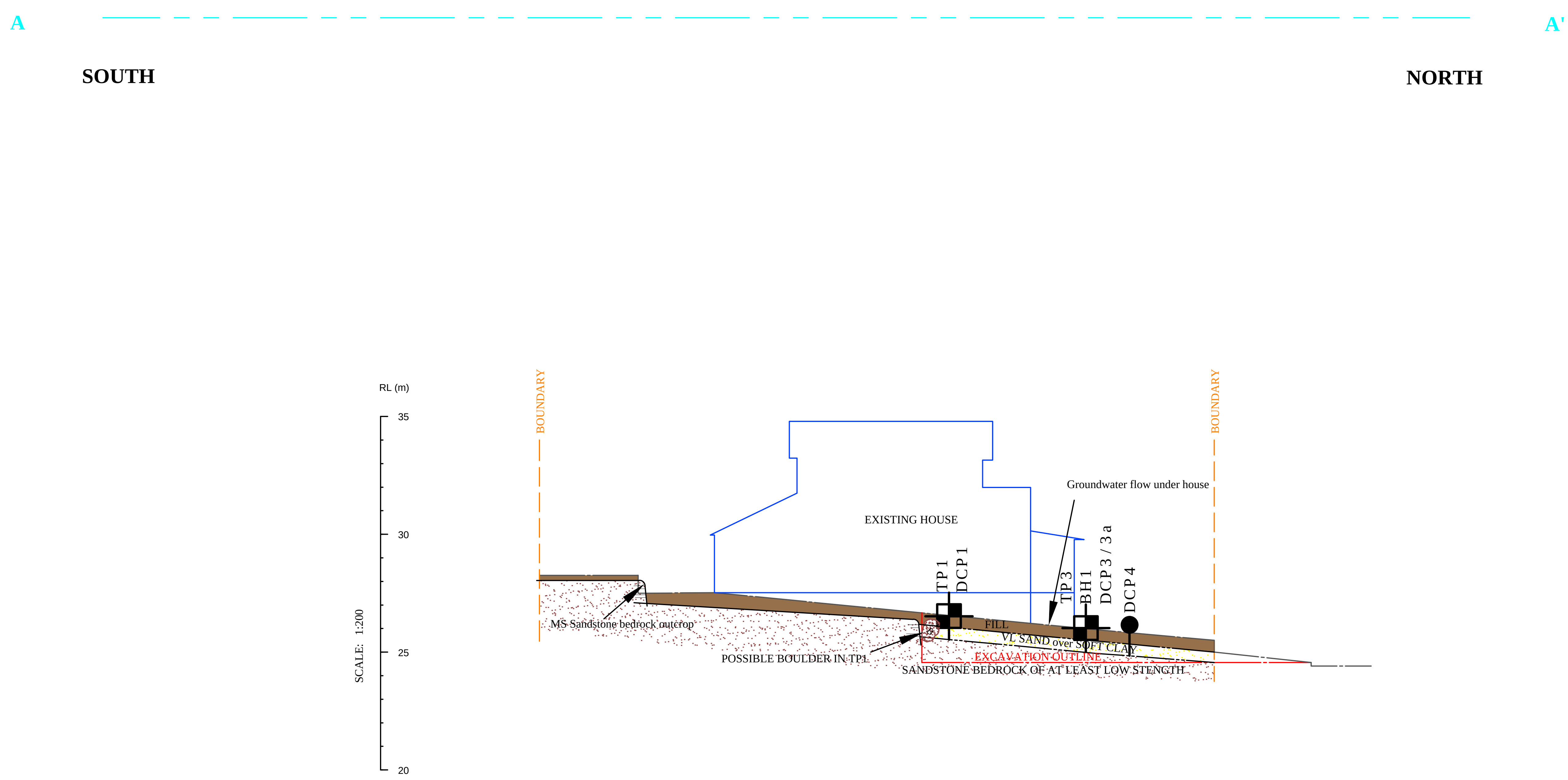
SCALE:	1:200 @ A3
DRAWING:	FIGURE 1
DATE:	04/10/2019
APPROVED BY:	TMC
DRAWN BY:	JY
PROJECT:	2019-163

PREPARED FOR:

Andrew Vinsen

ADDRESS:

21 Herbert Street, Manly, NSW



0 1 2 3 4 5 6 7 8 9 10

SCALE: 1:200

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

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

GEOLOGICAL MODEL FIGURE 2.

LEGEND

A — A' CROSS-SECTION REFERENCE LINE	TP / DCP TEST PIT / DYNAMIC CONE PENETROMETER LOCATION	SAND/CLAY
PROPERTY BOUNDARY	FILL	SANDSTONE BEDROCK

CROZIER GEOTECHNICAL CONSULTANTS
Crozier Geotechnical ABN: 96 113 453 624
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

SCALE:	1:200 @ A3
DRAWING:	FIGURE 2
DATE:	04/10/2019
APPROVED BY:	TMC
DRAWN BY:	JY
PROJECT:	2019-163

PREPARED FOR:
Andrew Vinsen

ADDRESS:
21 Herbert Street, Manly, NSW

TEST PIT REPORT

CLIENT: Andrew Vinson | **DATE:** 26/09/2019 | **TP No.:** 1

PROJECT: Alterations and Additions-New Garage Under House | **PROJECT No.:** 2019-163 | **SHEET:** 1 of 1

LOCATION: 21 Herbert Street, Manly | **SURFACE LEVEL:** RL å 26.4m

Depth (m)	Description of Strata PRIMARY SOIL - strength/density, colour, grainsize/plasticity, moisture, soil type incl. secondary constituents, other remarks	Sampling		In Situ Testing	
		Type	Depth (m)	Type	Results
0.00					
0.50	CONCRETE (50mm) over brown sandy gravel fill of brick and sandstone				
0.80	SILTY SAND (SM) Loose, brown, fine grained silty sand, moist (Possible fill)				
1.15	SANDY CLAY (CL) Very soft, grey sandy clay, fine grained sand, moist	D	0.90 1.10		
	End of test pit at 1.15m depth-Low strength bedrock				

RIG: None | DRILLER: AC | LOGGED: KN

METHOD: Hand Tools

GROUND WATER OBSERVATIONS: GW not encountered

REMARKS:

CHECKED:

BOREHOLE REPORT

CLIENT: Andrew Vinson **DATE:** 26/09/2019 **BH No.:** 2

PROJECT: Alterations and Additions-New Garage Under House **PROJECT No.:** 2019-163 **SHEET:** 1 of 1

LOCATION: 21 Herbert Street, Manly **SURFACE LEVEL:** RL å 25.8m

Depth (m)	Description of Strata PRIMARY SOIL - strength/density, colour, grainsize/plasticity, moisture, soil type incl. secondary constituents, other remarks	Sampling		In Situ Testing		
		Type	Depth (m)	Type	Results	
0.00						
	FILL: Brown silty sand					
0.65	CLAYEY SAND/SANDY CLAY (SC/CL) Medium dense/Stiff, brown, fine to medium grained clayey sand/sandy clay, moist					
1.05	Refusal at 1.05m depth on interpreted low strength bedrock					
1.00						

RIG: None DRILLER: AC LOGGED: KN

METHOD: Hand Tools

GROUND WATER OBSERVATIONS: GW not encountered

REMARKS: CHECKED:

TEST PIT/BOREHOLE REPORT

CLIENT: Andrew Vinson **DATE:** 26/09/2019 **TP/BH No.:** TP3/BH1

PROJECT: Alterations and Additions-New Garage Under House **PROJECT No.:** 2019-163 **SHEET:** 1 of 1

LOCATION: 21 Herbert Street, Manly **SURFACE LEVEL:** RL å 26.00m

Depth (m)	Description of Strata PRIMARY SOIL - strength/density, colour, grainsize/plasticity, moisture, soil type incl. secondary constituents, other remarks	Sampling Type	Sampling Depth (m)	In Situ Testing Type	Results	
0.00						
	FILL: Dark grey silty sand with decaying wood/charcoal.					
0.20	SILTY SAND: Loose, grey brown, fine to medium grained, wet	D	0.20 0.40			
0.50	é brown					
0.60	é saturated	D	0.50 0.60			
		D	0.60 0.70			
1.20	Test Pit refusal 1.2m depth-Low strength bedrock					
2.00						

RIG: None DRILLER: AC LOGGED: KN

METHOD: Hand Tools

GROUND WATER OBSERVATIONS: Groundwater inflow at 0.50m depth-inflow from under house, unable to reduce water level below 0.60m

REMARKS: CHECKED:

TEST PIT

CLIENT: Andrew Vinson **DATE:** 26/09/2019 **TP No.:** 2

PROJECT: Alterations and Additions-New Garage Under House **PROJECT No.:** 2019-163 **SHEET:** 1 of 1

LOCATION: 21 Herbert Street, Manly **SURFACE LEVEL:** RL å 26.8m

Depth (m)	Description of Strata PRIMARY SOIL - strength/density, colour, grainsize/plasticity, moisture, soil type incl. secondary constituents, other remarks	Sampling		In Situ Testing		
		Type	Depth (m)	Type	Results	
0.00						
0.20	FILL: Brown gravelly sand.					
	Refusal at 0.20m depth-Low strength sandstone					
1.00						

RIG: None DRILLER: AC LOGGED: KN

METHOD: Hand Tools

GROUND WATER OBSERVATIONS: GW not encountered

REMARKS:

CHECKED:

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Andrew Vinson **DATE:** 26/09/2019

PROJECT: Alterations and Additions-New G **PROJECT No.:** 2019-163

LOCATION: 21 Herbert Street, Manly **SHEET:**

	Test Location						
Depth (m)	DCP1	DCP2	DCP3	DCP3a	DCP4	DCP5	
0.00 - 0.15	--	--	1	2	1	1	
0.15 - 0.30	--	4	0	1	1	2	
0.30 - 0.45	--	1	2	3	3	3	
0.45 - 0.60	--	B@0.58m	2	2	4	4	
0.60 - 0.75	--		2	2	6	3	
0.75 - 0.90	10		4	5	2	1	
0.90 - 1.05	2		1	0	3	2	
1.05 - 1.20	2		4	4	B@1.0m	B@1.05m	
1.20 - 1.35	B@1.22m		B@1.2m	B@1.2m			
1.35 - 1.50							
1.50 - 1.65							
1.65 - 1.80							
1.80 - 1.95							
1.95 - 2.10							
2.10 - 2.25							
2.25 - 2.40							
2.40 - 2.55							
2.55 - 2.70							
2.70 - 2.85							
2.85 - 3.00							

TEST METHOD: AS 1289. F3.2, CONE PENETROMETER
AS 1289. F3.3, PERTH SAND PENETROMETER

REMARKS:
(B) Test hammer bouncing upon refusal on solid object
-- No test undertaken at this level due to prior excavation of soils

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Appendix 3

TABLE : A

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood of Slide	Spatial Impact of Slide		Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip of unsupported soils in garage excavation walls (<3m^3)		Up to 1.2m of very soft/loose soil, possibility for perched water table	a) Site side access directly adjacent to excavation, impact up to 20% b) Boundary approximately 1.5m from excavation, up to 1.15m of soil anticipated. c) Boundary approximately 1.0m from excavation, less than 1.0m of soil anticipated. d) Narrow section of base impacted		a) Person using site access path 0.25hrs/day avge. b) Person on side access 0.25hrs/day avge. c) Person on side access 0.25 hrs/day avge. d) Person in garage 0.25hrs/day avg.	a) Possible to not evacuate b) Possible to not evacuate c) Possible to not evacuate d) Likely to not evacuate	a) Person in open space, possibly engulfed b) Person in open space, unlikely engulfed c) Person in open space, unlikely engulfed d) Person in garage, possibly crushed	
			Possible	**Prob. of Impact**	**Impacted**				
		a) Pavement/side access to site dwelling.	0.001	1.00	0.20	0.0208	0.5	1.0	**2.08E-06**
		b) Side access to No.19 (to the east).	0.001	0.10	0.10	0.0208	0.5	0.5	**5.20E-08**
		c) Side access to No.21 (to the west).	0.001	0.20	0.10	0.0208	0.5	0.5	**1.04E-07**
		d) Base of excavation	0.001	1.00	0.1	0.0208	1	1	**2.08E-06**
B	Rockslide/topple failure in bedrock due to defects (<0.5m^3)		Approximately 1m of rock likely to be encountered at base of garage excavation, may contain unfavourable defects	a) Small block may topple onto floor of basement		a) Person in garage 0.25hrs/day avg.	a) Likely to not evacuate	a) Person in open space, unlikely crushed	
			Possible	**Prob. of Impact**	**Impacted**				
		a) Base of excavation.	0.001	1.00	0.05	0.0208	1	0.1 .	**5.20E-08**

* hazards considered in current condition and/or without remedial/stabilisation measures or poor support systems

* likelihood of occurrence for design life of 100 years

* Spatial Impact - Probaility of Impact refers to slide impacting structure/area expressed as a % (i.e. 1.00 = 100% probability of slide impacting area if slide occurs).
Impacted refers to expected % of area/structure damaged if slide impacts (i.e. small, slow earth slide will damage small portion of house structure such as 1 bedroom (5%), where as large boulder roll may damage/destroy >50%)

* neighbouring houses considered for impact of slide to bedroom unless specified, due to high occupancy and lower potential for evacuation.

* considered for person most at risk, where multiple people occupy area then increased risk levels

* for excavation induced landslip then considered for adjacent premises/buildings founded off shallow footings, unless indicated

* evacuation scale from Almost Certain to not evacuate (1.0), Likely (0.75), Possible (0.5), Unlikely (0.25), Rare to not evacuate (0.01). Based on likelihood of person knowing of landslide and completely evacuating area prior to landslide impact.

* vulnerability assessed using Appendix F - AGS Practice Note Guidelines for Landslide Risk Management 2007

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip of unsupported soils in garage excavation walls (<3m3)	a) Pavement/side access to site dwelling.	Possible	The event could occur under adverse conditions over the design life.	Insignificant	Little Damage, no significant stabilising required or no impact to neighbouring properties.	Very Low
		b) Side access to No.19 (to the east).	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
		c) Side access to No.21 (to the west).	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
		d) Base of excavation	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
B	Rockslide/topple failure in bedrock due to defects (<0.5m3)	a) Base of excavation.	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%.

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}		100 years		The event will probably occur under adverse conditions over the design life.	LIKELY	B
10^{-3}	$5x10^{-3}$	1000 years	200 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10^{-4}	$5x10^{-4}$	10,000 years	2000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10^{-5}	$5x10^{-5}$	100,000 years	20,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
10^{-6}	$5x10^{-6}$	1,000,000 years	200,000 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.