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REPORT ON GEOTECHNICAL SITE INVESTIGATION

for

PROPOSED DEVELOPMENT

at

2A ALLEN AVENUE, BILGOLA BEACH, NSW

Prepared For

Wallhouse Holdings Pty Ltd On Behalf of
Wimbledon 1963 Pty Ltd

Project No.: 2021-086
June, 2022

Document Revision Record

Issue No	Date	Details of Revisions
0	27th May, 2021	DRAFT issue
1	28th June, 2022	Original issue

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER
FORM NO. 1 – To be submitted with Development Application

Development Application for ______________________________	
	Name of Applicant
Address of site	2a Allen Avenue, Bilgola Beach, NSW

Declaration made by geotechnical engineer or engineering geologist or coastal engineer (where applicable) as part of a geotechnical report

I, __***Troy Crozier***__ *on behalf of* ___***Crozier Geotechnical Consultants*** _on this the 28th June 2022_ certify that I am a ~~geotechnical engineer or~~ engineering geologist ~~or coastal engineer~~ as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above ~~organisation/~~company to issue this document and to certify that the ~~organisation/~~company has a current professional indemnity policy of at least $2million. I:

☐ have prepared the detailed Geotechnical Report referenced below in accordance with the Australia Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009

■ am willing to technically verify that the detailed Geotechnical Report referenced below has been prepared in accordance with the Australian Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009

☐ have examined the site and the proposed development in detail and have carried out a risk assessment in accordance with Section 6.0 of the Geotechnical Risk Management Policy for Pittwater - 2009. I confirm that the results of the risk assessment for the proposed development are in compliance with the Geotechnical Risk Management Policy for Pittwater - 2009 and further detailed geotechnical reporting is not required for the subject site.

☐ have examined the site and the proposed development/alteration in detail and I am of the opinion that the Development Application only involves Minor Development/Alteration that does not require a Geotechnical Report or Risk Assessment and hence my Report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements.

☐ have examined the site and the proposed development/alteration is separate from and is not affected by a Geotechnical Hazard and does not require a Geotechnical Report or Risk Assessment and hence my Report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements.

☐ have provided the coastal process and coastal forces analysis for inclusion in the Geotechnical Report

Geotechnical Report Details:

Report Title: Geotechnical Report for Proposed New Residential Development

Report Date: 28/06/2022 **Project No.:** 2021-086

Author: M. Lujan and T. Crozier

Author's Company/Organisation: Crozier Geotechnical Consultants

Documentation which relate to or are relied upon in report preparation:

Survey Drawing – by John Lowe and Assoc., Job No.: 98789 #38524, Plot Dated: 30th April 2020, Drawn By: J.L. & P.S.
Architectural Design for Appraisal and Definition – by Saota Architecture and Design, Rev.: 0, Date: 2022_02_23, Sheet: 1 of 1, Page No.: 1 to 80.

I am aware that the above Geotechnical Report, prepared for the abovementioned site is to be submitted in support of a Development Application for this site and will be relied on by Pittwater Council as the basis for ensuring that the Geotechnical Risk Management aspects of the proposed development have been adequately addressed to achieve an "Acceptable Risk Management" level for the life of the structure, taken as at least 100 years unless otherwise stated and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk.

Signature ....

Name ...**Troy Crozier**..........................

Chartered Professional Status...**RPGeo (AIG)**.........

Membership No. ...**10197**.......................

Company... **Crozier Geotechnical Consultants**

REGISTERED PROFESSIONAL GEOSCIENTIST
AUSTRALIAN INSTITUTE OF GEOSCIENTISTS
TROY CROZIER
10197

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1(a) - Checklist of Requirements For Geotechnical Risk Management Report for Development Application

Development Application for ____

Name of Applicant

Address of site __2a Allen Avenue, Bilgola Beach, NSW_______

The following checklist covers the minimum requirements to be addressed in a Geotechnical Risk Management Geotechnical Report. This checklist is to accompany the Geotechnical Report and its certification (Form No. 1).

Geotechnical Report Details:

Report Title: Geotechnical Report for Proposed New Residential Development
Report Date: 28/06/2022 **Project No.:** 2021-086
Author: M. Lujan and T. Crozier
Author's Company/Organisation: Crozier Geotechnical Consultants

Please mark appropriate box

- ■ Comprehensive site mapping conducted ___20th and 28th April, 2021_______ (date)
- ■ Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:200 (as appropriate)
- ■ Subsurface investigation required
  - ☐ No Justification ....
  - ■ Yes Date conducted: 20th and 28th April 2021............
- ■ Geotechnical model developed and reported as an inferred subsurface type-section
- ■ Geotechnical hazards identified
  - ☐ Above the site
  - ■ On the site
  - ☐ Below the site
  - ☐ Beside the site
- ■ Geotechnical hazards described and reported
- ■ Risk assessment conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
  - ■ Consequence analysis
  - ■ Frequency analysis
- ■ Risk calculation
- ■ Risk assessment for property conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ Risk assessment for loss of life conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ Assessed risks have been compared to "Acceptable Risk Management" criteria as defined in the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ Opinion has been provided that the design can achieve the "Acceptable Risk Management" criteria provided that the specified conditions are achieved.
- ■ Design Life Adopted:
  - ■ 100 years
  - ☐ Other .... specify
- ■ Geotechnical Conditions to be applied to all four phases as described in the Geotechnical Risk Management Policy for Pittwater - 2009 have been specified
- ■ Additional action to remove risk where reasonable and practical have been identified and included in the report.
- ☐ Risk assessment within Bushfire Asset Protection Zone.

I am aware that Pittwater Council will rely on the Geotechnical Report, to which this checklist applies, as the basis for ensuring that the geotechnical risk management aspects of the proposal have been adequately addressed to achieve an "Acceptable Risk Management" level for the life of the structure, taken as at least 100 years unless otherwise stated and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk.

Signature ....

Name ...**Troy Crozier**..

Chartered Professional Status...**RPGeo (AIG)** ....

Membership No. ...**10197**.......................................

Company... **Crozier Geotechnical Consultants**

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CROZIER
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Crozier Geotechnical Consultants is a division of PJC Geo-Engineering Pty Ltd

Date: 28th June 2022
No. Pages: 1 of 25
Project No.: 2021-086

GEOTECHNICAL REPORT FOR A PROPOSED NEW DEVELOPMENT AT 2A ALLEN AVENUE, BILGOLA BEACH, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for a proposed new development at No.2a Allen Avenue, Bilgola Beach, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of the Saota Architecture and Design for Wallhouse Holdings Pty Ltd on behalf of Wimbledon 1963 Pty Ltd.

The site is on the higher western side of Allen Avenue and it is currently occupied by a two storey dwelling on the rear half with a tennis court and driveway at the front.

The site is located within the H1 (highest category) landslip hazard zone as identified within Northern Beaches (Pittwater) Councils Geotechnical Risk Management Map (Map Sheet GTH_016). The site is also designated as Acid Sulfate Soils hazard Class 5 (Map-Sheet ASS_016).

This report includes a description of site and sub-surface conditions, a landslide risk assessment of the site and proposed works, plans, geological sections, an assessment of acid sulfate soils hazards and provides recommendations for construction and to ensure stability is maintained for a design life of 100 years.

The site assessment and reporting were generally undertaken as per the Proposal No.: P21-132, Dated: 30rd March 2021.

a) A detailed geotechnical inspection and mapping of the site and adjacent properties by a Geotechnical Engineer.
b) DBYD plan request and onsite clearing of test locations by an accredited service location contractor.
c) Concrete core at four locations by a contractor to allow drilling through existing concrete floor.
d) Drilling of six boreholes using a restricted access drill rig, the hand excavation of one Test Pit (TP1) adjacent to an existing retaining wall, along with six Dynamic Cone Penetrometer (DCP)

tests to determine subsurface geology, groundwater level, soil characteristics and depth to bedrock.

e) Collect soil samples and logging of material recovered from the boreholes as per "AS1726: 2017 Geotechnical Site Investigation".

f) All fieldwork was conducted under the full-time supervision of an experienced Geotechnical Professional.

The following plans and diagrams were supplied for the work.

- Architectural Design for Appraisal and Definition – by Saota Architecture and Design, Rev.: 0, Date: 2022_02_23, Sheet: 1 of 1, Page No.: 1 to 80.
- Survey Drawing – by John Lowe and Assoc., Job No.: 98789 #38524, Plot Dated: 30th April 2020, Drawn By: J.L. & P.S.
- Survey Plan of stormwater easement (received on site during site works) – by CMS Surveyors Pty Ltd, Ref No.: 1997 Cease, Date: 28-08-2013 and Title: Annexure "8" – Sketch showing proposed easement to drain water 1 wide within Lot 20 in DP11978 & Lot A in SP379490 being land contained in auto consol 6589-169.

2. PROPOSED WORKS:

It is understood that the proposed works involve demolition of the existing dwelling and construction of a new three storey dwelling with a basement carpark.

The proposed new basement will require bulk excavation of 7.0m depth within the higher western side of the block, reducing east to 3.50m to 0.50m depth within the eastern lower side of the block to achieve a Finished Floor Level (FFL) at RL= 3.24m. The distance between the basement excavation and the site boundaries and neighbouring properties is summarised in Section 5.3.2.

3. SITE FEATURES:

3.1. Description:

The site is a rectangular shaped block and is located on the high western side of Allen Avenue within gently to moderately south east dipping topography. The site contains an eastern front boundary of 18.288m, a west boundary of 18.141m, a north boundary of 48.033m and a south boundary of 45.72m, as referenced from the provided survey plan.

The site is located approximately 150m west from Bilgola Beach. The centre to eastern side of the block contains a near level ground surface formed as a tennis court with a low of approximately RL= 5.77m within the south-east corner of the block. The western side of the block rises and is higher than the rest of the site with a maximum of RL= 16.53m within the north-west corner of the block.

An aerial photograph of the site and its surrounds is provided below, as sourced from NSW Government Six Map spatial data system, as Photograph-1. General views of the site at the time of investigation are provided in Photograph-2 to Photograph-4.

Photograph-1: Aerial photo of site and surrounds

Photograph-2: Front view of the site. View looking west.

Photograph-3: Site-dwelling frontage. View looking north-west.

Photograph-4: Rear deck of the site. View looking north-west.

3.2. Geology:

Reference to the Sydney 1:100,000 Geological Series sheet indicates that the site is underlain by Quaternary Age coarse quartz sand with varying amounts of shell fragments.

This sand is likely to be underlaid by Newport Formation (Upper Narrabeen Group) rock (Rn) which rises above the Quaternary Age soils to the west. The Newport Formation is a middle Triassic Age typically comprises interbedded laminite, shale and quartz to lithic quartz sandstones and pink clay pellet sandstones.

Extract of Sydney: 1:100 000 – Geology underlying the site

4. FIELD WORK:

4.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site and adjacent properties on the 20th and 28th April, 2021 by a Geotechnical Engineer. It included a photographic record of site conditions as well as geological/geomorphological mapping of the site and inspection of neighbouring properties from the site and road reserve.

The investigation also included the drilling of six auger boreholes (BH1 – BH6) using a mini-drill rig employing solid stem, spiral flight augers to determine sub-surface geology and to collect soil samples for soil logging purposes. All boreholes were extended to auger refusal and to 6.0m depth in clay whist BH6 refused on fill (bricks), however the DCP test within BH6 (i.e. DCP6) was completed.

It also included the hand excavation of one Test Pit (TP1) adjacent to a retaining wall along the western boundary.

The boreholes BH6, BH1 and BH2 were drilled on the 20th of April and the boreholes BH3, BH4 and BH5 were drilled on the 28th of April along with the hand excavation of TP1.

Samples of soil were collected as per 'AS1726:2017 Geotechnical Site Investigations', for logging purposes and select samples were subsequently submitted to NATA accredited geotechnical and chemical testing laboratories for analysis of the corrosion potential of the site soils to provide durability classification for a new steel pile and concrete structures as per AS2159 and for analysis of moisture reactivity as per AS1289.3 Soil Classification and AS1289.7 Soil Reactivity.

Dynamic Cone Penetrometer (DCP) testing was carried out within the boreholes in accordance with AS1289.6.3.2 – 1997, "Determination of the penetration resistance of a soil – 9kg Dynamic Cone Penetrometer test" to estimate near surface soil conditions and confirm depth to bedrock.

Explanatory notes are included in Appendix: 1. Mapping information and test locations are shown on Figure: 1, along with detailed log sheets in Appendix: 2. Geological model/section are provided as Figure:2 to Figure:5 in Appendix 2.

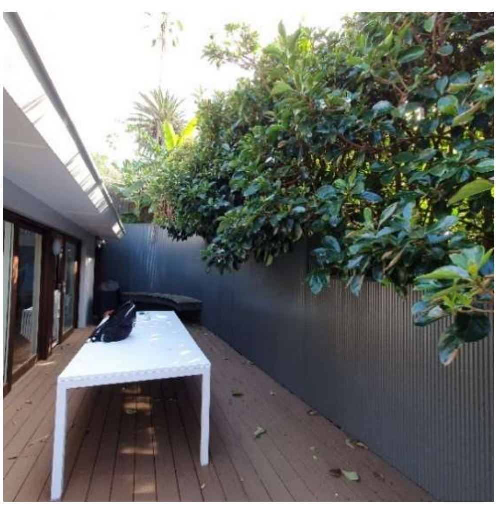
4.2. Field Observations:

The site is located on the high west side of Allen Avenue within gently and then moderately south east dipping topography. Allen Avenue contains a bitumen pavement and is gently south dipping where it passes the site. The bitumen pavement is adjacent to gardens which contain high trees (approximately ≤20.0m high) and does not contain gutters or kerbs along the sides. Vehicular access to the site is achieved via a concrete crossover driveway. The cross over driveway contains a stormwater drainage pit, directly adjacent to south-east corner of the block. Signs of cracking, ground movement, instability or underlying geotechnical issues were not observed within the road reserve.

The centre to eastern side of the site contains a concrete driveway and a ramp along the south boundary with a tennis court occupying most of the centre to eastern side to the north boundary. The ramp is supported by what appears to be a rendered brick wall (up to ≤ 3.0m high) along the northern side that extends north along the western side of the tennis court and supports the higher western side of the site (approximately ≤ 3.0m higher). The ramp is supported by a low brick wall (up to ≤ 1.0m high) along the southern side which is founded on a terrace that extends south (approximately ≤ 2.0m within the neighbouring property) and is supported by a dry stone retaining wall. Directly adjacent to the ramp is a relatively large tree (Photograph-5). Signs of instability, cracking or underlying geotechnical issues were not observed within the retaining structures, large tree or adjacent structures.

Photograph-5: Large tree located directly adjacent to the site boundary/concrete ramp. View looking south-east.

The western higher portion of the site contains a two-storey rendered residence with an upper timber deck at the north-west corner of the dwelling's First Floor. A timber deck is located to the rear of the dwelling at the south-west corner of the site with access available via a timber pathway along the south boundary. It appears that the dwelling's northern wall is also a retaining wall that supports the neighbouring ground to the north (No.4B Allen Avenue). The northern retaining wall increases in high towards the west and appears to continue to the north-west corner of the site, although this is unconfirmed as the wall is covered by a metal sheet wall (Photograph-6). The retaining wall along the north boundary comprises rendered brick (≤250mm wide, Photograph-7) and is up to ≤ 3.0m high adjacent to the tennis court and up to approximately 5.0m high at the north-west corner of the block.



Photograph-6: Retaining wall covered by metal sheet. View looking north-west.

Photograph-7: Brick retaining wall along the north boundary. View looking down east.

The site-dwelling is detached from the western boundary (approximately ≤1.0m to ≤4.0m). Along the western boundary is an approximately 4.0m to 5.0m high retaining wall. The southern portion of the wall is rendered (Photograph-4), whilst the northern portion is stone veneer cladded. Observations from the top of the wall identified what appears to be a concrete column/pier which is located on top of the wall (Photograph-8 and 9) (approx. ≤1.0m high and approximately 2.5m spaced along the western boundary). To the rear of the retaining wall (within south-east corner of No8 The Serpentine) a stormwater drainage pit which may be part of the water easement located along the south boundary of the site and a large tree (approximately ≤10m high) are located 1.0m from the retaining wall. The retaining wall contained a semi-vertical crack (up to ≤1mm wide and 3.0m long, Photograph-10), however it appeared stable with no signs of rotation, humidity, or underlying geotechnical issues and appeared in good condition.

Photograph-8: Brick retaining wall along the north boundary. View looking down east.

Photograph-9: Closer look to concrete pier. View looking south.

Photograph-10: Cracking within the retaining wall. View looking west.

Along the south side of the timber deck and timber pathway is a rendered retaining wall with a sandstone block retaining wall located to the rear (which appears to be supporting the rear of the property No. 6 Bilgola Avenue). Both retaining walls appeared in good condition.

The neighbouring property to the north (No.4B Allen Avenue) contains a two-storey weatherboard house with a pathway between the common boundary and the dwelling. The eastern front of the block contains a grass lawn and driveway within the northern and southern sides, respectively. The western portion of the property contains retained gardens that rise west, upslope. The pathway adjacent to the dwelling is at similar Ground Surface Level (GSL) to the site along the common boundary. The GSL of the neighbouring property rises west to approximately ≤3.0m above the western end of the tennis court. The remainder western portion of the property is approximately 1.0m to 5.0m above the site (or dwelling's Ground Floor Level, GFL) along the common boundary. The property dwelling extends to approximately 0.90m from the common boundary. The neighbouring property appeared in good condition, signs of instability or underlying geotechnical issues were not observed within the property dwelling and the raised gardens within the western end of the property.

Very limited observation was possible to the neighbouring property to the north-west of the site (No.10 The Serpentine). From available google maps satellite view it can be observed that the neighbouring property contains a dwelling with a suspended front concrete driveway within the western side. The eastern side appears to contain a pathway directly adjacent to the common boundary and a grass lawn approximately 5.0m from the common boundary. The neighbouring property is approximately 5.0m above the site level along the common boundary and the dwelling is approximately 15.0m from the common boundary. Observation from the site did not identify unstable trees or signs of excessive ground movement.

The neighbouring property to the west (No.8 The Serpentine) contains a one to two storey weatherboard house with a timber deck directly to the rear and with a carport directly to the south of the dwelling. The western front of the block contains gardens and a concrete carparking area. The eastern rear of the block contains a grass lawn, a timber decking terrace within the eastern end and the large palm tree (previously described) within the south-east corner of the block. The GSL of the property is approximately 4.0m to 5.0m above the site along the common boundary. The property dwelling is located approximately 15.0m from the common boundary. The timber decking terrace and large palm tree are located approximately 1.0m from the common boundary. Limited observation was possible to the neighbouring property, however observation within the decking terrace and adjacent gardens identified no signs of underlying geotechnical issues and structures appeared in good condition.

The neighbouring property to the south-west of the site (No. 6 Bilgola Avenue) contains a two-storey weatherboard dwelling with a carport directly to the east, which broadly occupies the centre of the block.

The front of the property contains a grass lawn and a stone flagging driveway within the northern and southern sides, respectively. The rear of the block contains a grass lawn (bounded by overgrown vegetation along the sides and an inground swimming pool within the western and eastern sides, respectively. The property dwelling is located approximately ≤10.0m from the common boundary, the pool ≤5.0m from the common boundary whilst the grass/lawn is approximately ≤2.0m from the common boundary. Limited observation was possible to the neighbouring property, however the sandstone block retaining wall directly adjacent to the common boundary appeared in good condition and no signs of ground instability were observed within the overgrown vegetation adjacent to the common boundary.

The neighbouring property to the south (No. 2 Allen Avenue) contains a two-storey rendered and sandstone residence with a shed to the north-west, located within the western portion of the block. The eastern portion of the block contains a grass lawn within the southern side and a concrete driveway located approximately ≤2.0m from the common boundary. Adjacent to the common boundary is an artificial grass lawn (with gardens and small sized trees) that rises in level towards the west and contains the large tree (previously described) approximately ≤0.50m from the common boundary. The ground adjacent to the site's concrete ramp extends south approximately ≤2.0m and is supported by a dry stone retaining wall forming a narrow pathway between the dwelling and the wall. The property dwelling is located approximately ≤2.5m from the common boundary, whilst the shed is located on the boundary. The dry stone retaining wall appeared old and not-engineered, however it appeared stable with no signs of ground movement. The property dwelling, shed and front driveway appeared in good condition and signs of underlying geotechnical issues were not observed.

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. Additional Site Information:

Information provided by the Architect (CMS Surveyors Pty Ltd, 28-08-2013) identified a 1.0m wide stormwater easement that is located along the south boundary of the site.

Information from available DBYD Sydney Water plans identified an east-west striking sewer main that is located adjacent and parallel to the north boundary (approximately 1m to 2m from the boundary).

Based on available google maps satellite view it appears that the centre of the site previously contained a concrete surface with a platform adjacent to the north boundary, whilst the eastern side contained a grass lawn (Photograph-11).

Photograph-11: Previous site plan. Extract from available Google Maps satellite view.

4.4. Ground Conditions:

Test Pit 1

TP1 (Photograph-12) was excavated below the existing retaining wall along the western boundary at the location as specified in Figure 1. TP1 was excavated to approximately 0.30m depth below the deck level and identified that the base of the retaining wall comprised of brick. TP1 is described in Figure 5.

The founding strata below the base of the wall comprised of very stiff clay, becoming hard below 0.50m depth (RL= 8.95m). The base of the retaining wall appeared in good condition, signs of soft soils or water accumulation were not encountered in the test pit.

Photograph-12: TP1 excavation. View looking down west.

The boreholes were drilled to varying depths between 1.50m (BH5) and 6.30m (BH4) with refusal encountered on potential bedrock or boulder within BH5 and on bedrock within BH4.

DCP test were carried out to varying depths between 1.35m (DCP4) and 2.55m (DCP1/6) with refusal encountered on interpreted hard clay and within extremely weathered bedrock (DCP1).

For a description of the sub-surface conditions encountered at each test location, the individual Borehole Log Report and Dynamic Penetrometer Test Sheets should be consulted. Based on the field borehole logs and DCP test results the subsurface conditions at the project site can be classified/summarised as follows:

- **FILL** – this unit was encountered in all the test locations to varying depths between 0.25m (BH5) and 1.50m (BH6). It was classified as loose, brown, fine to medium grained, sand with some fine to medium grained gravels and with some bricks within BH6.
- **SAND –** this unit was encountered underlying the fill within the lower portion of the site to 2.60m depth within BH1. It was classified as medium dense with a loose layer between 1.30m to 1.70m depth, light brown, fine to medium grained, moist, sand.
- **CLAY –** this unit was primarily encountered within BH2 to BH5 below the fill to varying depths between 1.50m (BH5) and 6.30m (BH4). It was classified as generally very stiff to hard, red mottled orange, moist; clay with zones of dry/moist extremely weathered bedrock.
- **SHALE (EW) –** this unit (0.20m thick) was encountered underlying the sand unit within BH1 to 2.80m depth, overlying interpreted shale bedrock of at least low strength. It was classified as fine to fine grained, pale yellow.
- **POTENTIAL BOULDER OR BEDROCK –** auger refusal was encountered at 1.50m depth within BH5 and it is considered at a relatively shallow depth when compared to the nearby auger refusal within BH4 (6.30m depth). The results within BH5 may suggest a boulder or significant bedrock variability.
- **INTERPRETED SHALE BEDROCK**– this unit may be encountered at a relatively high level of RL≈7.95m (BH5) within the north-western portion of the site but its surface of low strength rock is steeply south-east dipping to a low of RL≈ 2.45m (BH2) at the sites midpoint.

A free-standing ground water table was not encountered in the investigation; however seepage was encountered within BH2 and BH3 at RL ≈ 3.20m to RL ≈ 3.30m, respectively.

4.5 Laboratory Testing:

Geotechnical and chemical testing has been undertaken at NATA accredited Chemical testing laboratory (Envirolab) and the results are summarised and discussed in the following sections. The laboratory test report sheets are included in Appendix: 3

Two select soil samples recovered boreholes were tested to determine the corrosion potential of the site soils to provide durability classification for a new steel pile and concrete structures as per AS2159. The reported results are summarised below and Eurofins Certificate of Analysis is provided in Appendix: 3.

Table a: Summary of reported Chemical Analysis

Sample Location	pH	Electrical Conductivity (µS/cm)	Chloride, Cl (mg/kg)	Sulphate, SO4 (mg/kg)
BH1 5.5 – 5.7	5.2	64	25	98
BH1 5.3 – 5.5	5.3	55	10	95

5. COMMENTS:

5.1. Geotechnical Assessment:

The site investigation the presence of fill underlaid by a sand unit to a maximum depth of 2.60m (BH1) within the eastern lower portion and underlaid by a residual clay unit at varying depths between 1.50m (BH5) and 6.30m (BH4) within the western upper portion. Due to difference in auger refusal levels within the rear of the site (4.80m difference in height), the underlying bedrock geological boundary has been interpreted into two different scenarios detailed below.

Scenario 1

Below or within the clay unit within BH5, the presence of a boulder/floater is a possibility as seen in BH5. Where this is the case, the floater has the potential to be underlaid by a clay unit to a similar level as BH4.

Scenario 2

Below the clay unit within BH5, the presence of shale or sandstone bedrock of at least low strength at RL= 7.95m. This bedrock would then be present at similar or higher levels towards the north to north west.

The presence of a free-standing groundwater table was not encountered in the investigation; however seepage was encountered within the clay/bedrock interface within BH2, at 5.50m depth within BH3 and within BH4 from 2.50m (RL 7.00m) to 3.0m (RL 6.50m) depth.

Due to the variable geological conditions and lack of access to the north west corner of the site, further geotechnical investigation is required post demolition of the existing dwelling and prior to any bulk excavation. It is recommended that an additional two to three boreholes be drilled within the western portion of the site to re-assess the geological model to at least 3.0m below the basement level. Also, due to the early auger refusal on fill (bricks) within BH6, additional investigation nearby BH6 is also recommended.

Based on the proposed works, the excavation conditions encountered will depend on the geological scenario. Scenario 1 excavation will consist entirely of soils (fill/clay/sand) with bedrock only towards the base of the basement. Scenario 2 excavation will consist of bedrock within the north-western portion of the block with soils (fill/clay/sand) within the remainder of the basement. Both scenarios will require soil and bedrock excavation, however more rock excavation will be required in Scenario 2.

The fill, sand/clay and extremely to very low strength bedrock can be excavated using conventional earthmoving equipment, however low to high strength bedrock will require the use of the rock breaking equipment (e.g. rock hammers). The use of rock hammers can create ground vibrations which could damage the neighbouring structures including nearby services (e.g. sewer to the north of the site). Care will be required during the demolition, construction and excavation works to ensure the neighbouring properties, structures and services are not adversely impacted by ground vibrations. Small scale equipment (i.e. rock hammer <250kg) along with rock saw and a good excavation methodology can be used to maintain low vibration levels and avoid the need for full time vibration monitoring. However, this will result in slow excavation progress and it is anticipated that larger scale rock hammers will be preferred (especially where Scenario 2 is proven). As such Crozier Geotechnical Consultants (CGC) should be consulted regarding the size and type of demolition/excavation equipment proposed and demolition/excavation methodology prior to works.

The adjacent Sydney Water (SW) sewer main is within the Zone of Influence of the proposed site excavation. Sydney Water should be consulted regarding design and construction well prior to start of the works to prevent delays in approvals. It is anticipated that a Specialist Engineering Assessment report will be required.

The excavation is proposed to extend close to the locations of two large trees in the neighbouring properties (No.8 The Serpentine and No.2 Allen Avenue). It is recommended that an experienced arborist assess the proposed excavation and the potential risk of undermining or affecting the roots/stability of the adjacent trees.

Based on the proposed excavation depths and the recommended safe batter slopes for the geological conditions anticipated (as per Section 5.3.2), safe batter slopes are not achievable along the northern, western and southern sides of the excavation. As such, support prior to excavation will be required along these sides of the new excavation. The construction of closely spaced soldier pile wall or similar with the base of the pile wall socketed into bedrock with adequate strength and depth as determined by the structural engineer may be a viable option where access to the crest of the site can be created for a piling rig. It appears unlikely that cantilever support will be possible, therefore lateral support (where necessary) could be provided by anchoring, internal bracing or propping. Where anchoring outside the property boundaries is to be avoided, boundary support via propping/bracing could be designed to maintain all support internal to the site boundaries. Where anchors are proposed, it is recommended that approvals for anchoring across site boundaries and beneath adjoining properties or structures be obtained as far in advance of construction as possible, to enable finalization of excavation support design. More information regarding supporting structure will be provided following additional geotechnical investigation.

Regardless of support design, some relaxation of soils/rock external to excavation will occur resulting in potential settlement on adjacent ground and may impact the adjacent structures.

Wall design needs analysis (i.e. Wallap) as part of design to determine deflection and a monitoring program will be required in construction to assess deflection against estimated/anticipated values to allow implementation of additional support if required.

Where Scenario 2 is proven then excavation may be possible via a staged excavation and support methodology, though the risk of minor instability is increased and anchoring will be necessary.

It is recommended that all new footings extend through the fill and residual soil units and bear onto/within the bedrock of similar strengths to reduce the potential risk of differential settlement. Preliminary allowable bearing pressures appropriate for the bedrock encountered underlying the site are provided in Section 5.3.1.

After the completion of BH2, the borehole was left open for a period of 1 hour to monitor seepage level coming into the open borehole. The seepage level after 1hr was determined to be RL≈ 3.20m. Similarly, seepage was determined at RL≈ 3.30m within BH3. This seepage level is above the Basement Excavation Level (BEL). Therefore, the stormwater engineer should consider these results and allow for waterproofing in the design of the basement. The proposed works will be expected to encounter seepage within the excavation side walls.

The site investigation did not identify signs of potential or actual acid sulfate soils. The likelihood of intersecting Acid Sulfate Soils on this site or the proposed works impacting the water table external to the site is considered ‘Very Low’ therefore further investigation is not necessary.

Provided further geotechnical investigation is undertaken and the recommendations of this reports are implemented in the design and construction phases the proposed works are considered suitable for the site and may be completed with negligible impact to existing nearby structures within the site or on neighbouring properties.

5.1.1 Corrosion Resistance

The results of the soil chemical testing undertaken on the soil samples were compared against the Australian Standard AS 2159-2009 Pile Design and Installation.

The results were compared against Table 6.4.2 (C) Exposure Classification for Concrete Piles – Piles in Soil. The results indicate that the soils are ‘non-aggresive’ to concrete from pH, chloride and sulphate.

The results were also compared against Table 6.5.2 (C) Exposure Classification for Steel Piles – Piles in Soil. The results indicate that the soil is ‘non-aggressive’ to steel with regard to pH, chloride and sulphate.

5.2. Site Specific Risk Assessment:

Based on our site investigation we have selected Scenario 1 (worst case scenario) to identify the following geological/geotechnical landslip hazard which needs to be considered in relation to the existing site and the proposed works. The hazard is:

A. Landslip (earth slide <20³) from soils due to excavation within the site.

A qualitative assessment of risk to life and property related to these hazards is presented in Tables A and B, Appendix: 3, and is based on methods outlined in Appendix: C of the Australian Geomechanics Society (AGS) Guidelines for Landslide Risk Management 2007. AGS terms and their descriptions are provided in Appendix: 4.

The **Risk to Life** from **Hazard A** was estimated to be up to **1.01×10^{-3}** for a single person, whilst the **Risk to Property** was considered to be up to **‘Very High’.**

The Risk to Property and Person for Hazard A is considered to be ‘Unacceptable’. However, the assessments were based on excavations with no support or planning. Provided further geotechnical investigation is undertaken and the recommendations of this report are implemented including installation of an engineer designed retaining wall prior to bulk excavation or in stages during excavation. The likelihood of any failure becomes 'Rare' and as such the consequences reduce and risk becomes within 'Acceptable' levels when assessed against the criteria of the AGS. As such the project is considered suitable for the site provided the recommendations of this report are implemented.

The proposed works are considered suitable for the site and may be completed with negligible impact to existing nearby structures within the site or on neighbouring properties provided the recommendations of this report are implemented in the design and construction phases.

The recommendations and conclusions in this report are based on an investigation utilizing only surface observation and a limited number of auger boreholes. This test equipment provides limited data from small isolated test points across the entire site. Therefore, some minor variation to the interpreted sur-face conditions is possible, especially between test locations.

5.3. Design & Construction Recommendations:

Design and the construction recommendations are tabulated below:

5.3.1. New Footings:	
Site Classification as per AS2870 – 2011 for new footing design	Class 'A' for footings on bedrock at the base of the excavation.
Type of Footing	Piers/Piles or Strip/Pad
Sub-grade material and Maximum Allowable Bearing Capacity	- Bedrock (EW-VLS): 800kPa - Bedrock (LS): 1000kPa - Bedrock (MS): 2000kPa*
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	B_e – rock site
Remarks: * Higher bearing pressures likely subject to the results of core drilling of bedrock All new footings must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify the preliminary maximum bearing capacities provided above and the in-situ nature of the founding strata. This is mandatory to allow them to be 'certified' at the end of the project. Individual structures should not be founded on materials with varying bearing and settlement characteristics unless the potential for differential movement has been allowed for in structural design.	

5.3.2. Excavation:

Basement Excavation

Table 1: Property Separation Distances

Boundary	Adjacent Property	Structure	Bulk Excavation Depth (m bgl)	Separation Distances (m)	
				Boundary (m)	Structure
North	No.4B Allen Avenue	Pathway, Dwelling and rear gardens	7.0m depth decreasing east to 3.5m depth	1.0m off the boundary	Pathway, directly adjacent; Dwelling, a further 0.90m; Rear gardens, directly adjacent.
North	No. 10 The Serpentine	Pathway, lawn and dwelling	7.0m depth	1.0m off the boundary	Pathway, directly adjacent; Lawn, a further 2.0m; Dwelling, a further 5.0m.
North	SW Sewer Main		7.0m depth decreasing east to 3.5m depth	Sewer approximately ≥1.0m from the boundary	
South	No.2 Allen Avenue	Tree, pathway, shed, dwelling and driveway	7.0m depth decreasing east to 0.5m depth	0.0m off the boundary	Tree, a further 0.50m; Pathway, a further 2.0m; Shed, directly on the boundary; Dwelling, a further 2.5m; Driveway, a further 2.0m.
South	No.6 Bilgola Avenue	Lawn, pool and dwelling	7.0m depth	1.0m off the boundary	Lawn, a further 2.0m; Pool, a further 5.0m; Dwelling, a further 10.0m.
East	Allen Avenue	Road pavement	0.50m depth within the driveway	Main excavation >10.0m off boundary; Ramp excavation extends to the boundary	Road pavement a further 3.0m
West	No. 8 The Serpentine	Tree, Decking terrace, lawn and dwelling	7.0m depth	1.0m to 3.0m off the boundary	Tree and deck a further 1.0m, lawn, a further 5.0; Dwelling, a further 15.0m.

Type of Material to be Excavated	Topsoil /Fill > 1.50m depth
	Clay > 0.25m depth within western upper portion, to the southern portion of the basement excavation
	Sand > 0.50m depth within eastern lower portion
	Potential Shale or Sandstone Bedrock VLS/LS below >1.50m depth. Possible LS to MS to the base of the northern portion of the basement excavation (although unconfirmed)

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	1.5:1	2:1
Clay/ Sandy Clay and ELS bedrock	1.1:1	2:1
Very Low to Low strength or fractured bedrock	0.5:1	1.5:1*
Medium strength (MS), defect free bedrock	Vertical	0.25:1.0

*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	Fill and soil	Excavator with bucket
	VLS bedrock	Excavator with bucket and ripper
	LS-HS bedrock	Rock hammer and saw

ELS – extremely low strength, VLS – very low, LS – low, MS – medium, HS – high strength

Remarks:

Based on previous testing of ground vibrations created by various rock excavation equipment within medium strength bedrock, to maintain a vibration level below 5mm/s PPV the below hammer weights and buffer distances are required:

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

<table>
<tr><td colspan="2">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 should be carried out prior to commencement of rock excavation works where ≥250kg rock hammers are proposed for use.
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, the saw cuts will provide a slight increase in buffer distance for use of rock hammers whilst also reducing deflection of separated rock across boundaries.
The strength of bedrock below the maximum depth achieved during the investigation is unconfirmed and requires cored boreholes using specialist restricted access drilling equipment unless demolition of existing structures can occur prior to final design.
Excavation of soils to ELS will not create excessive vibrations provided it is undertaken with medium scale (<20 tonne excavator) excavation equipment in a sensible manner.</td></tr>
<tr><td>Recommended Vibration Limits
(Maximum Peak Particle Velocity (PPV))</td><td>Road Reserve = 5mm/s
Adjacent residential Developments = 5mm/s
SW sewer main = 3mm/s (Subject to Sydney Water conditions)
Nearby services and water easement within the site = 3mm/s</td></tr>
<tr><td>Vibration Calibration Tests Required</td><td>Yes, recommended for any rock hammer >250kg weight</td></tr>
<tr><td>Full time vibration Monitoring Required</td><td>Pending proposed equipment and vibration calibration testing results</td></tr>
<tr><td>Geotechnical Inspection Requirement</td><td>Yes, recommended that these inspections be undertaken as per below mentioned sequence:
• During construction of the contiguous pile wall or similar
• During the first 1.0m excavation and at 1.5m depth intervals of unsupported rock excavation
• Any excavation where unsupported.
• At completion of the excavation.
• During construction of new footings</td></tr>
<tr><td>Dilapidation Surveys Requirement</td><td>Recommended 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.</td></tr>
</table>

Remarks:

Water ingress into exposed excavations can result in erosion and stability concerns in both soil and rock portions. Drainage measures will need to be in place during excavation works to divert any surface flow away from the excavation crest and any batter slope, whilst any groundwater seepage must be controlled within the excavation and prevented from ponding or saturating slopes/batters.

5.3.3. Retaining Structures:

Required	Retaining structure are required within the northern, western and southern sides of the proposed excavation within the site.
Types	- Soldier pile (with maximum of 0.50m spacing) or similar where support prior to excavation is required. - Steel reinforced concrete walls or conventional gravity walls where temporary batters are achievable and support post to excavation is required.

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/ Sand	18	$\phi' = 28°$	0.35	0.52	N/A
Clay (very stiff to hard)	20	$\phi' = 35°$	0.27	0.50	N/A
ELS to VLS bedrock	22	$\phi' = 38°$	0.15	0.20	200kPa
LS to MS bedrock	23	$\phi' = 40°$	0.05	0.10	400kPa

Remarks:

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

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	Groundwater table not encountered; however, seepage was encountered at similar level within BH2 and BH3 at RL≈ 3.20m to RL ≈ 3.30m and at RL≈ 7.0m within BH4.
Site Location and Topography	On the higher western side of the road within gently south-east dipping topography.
Impact of development on local hydrogeology	Negligible
Onsite Stormwater Disposal	Not applicable
Remarks: All new structure gutters, down pipes and stormwater intercept trenches should be connected to a stormwater system design by a Hydraulic Engineer which preferably discharges to the Council's 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. Additional boreholes post demolition of the existing structures and prior to any bulk excavation, including a rock core drilling investigation as recommended in this report.
2. Review and approve the structural design drawings and construction methodology, for compliance with the recommendations of this report prior to construction,
3. Inspection of site and works as per Section 5.3 of this report,
4. Inspect all new footings to confirm compliance to design assumptions with respect to allowable bearing pressure prior to the placement of steel or concrete.

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

5.5. Design Life of Structure:

We have interpreted the design life requirements specified within Council's Risk Management Policy to refer to structural elements designed to support the existing structures, control stormwater and maintain the risk of instability within acceptable limits. Specific structures and features that may affect the maintenance and stability of the site in relation to the proposed and existing development are considered to comprise:

- stormwater and subsoil drainage systems,
- retaining walls and instability,
- maintenance of trees/vegetation on this and adjacent properties.

Man-made features should be designed and maintained for a design life consistent with surrounding structures (as per AS2870 – 2011 (100 years)). It will be necessary for the structural and geotechnical engineers to incorporate appropriate design and inspection procedures during the construction period. Additionally, the property owner should adopt and implement a maintenance and inspection program.

If this maintenance and inspection schedule are not maintained the design life of the property cannot be attained. A recommended program is given in Table: C in Appendix: 3 and should also include the following guidelines.

- The conditions on the block don't change from those present at the time this report was prepared, except for the changes due to this development.
- There is no change to the property due to an extraordinary event external to this site
- The property is maintained in good order and in accordance with the guidelines set out in;
  a) CSIRO sheet BTF 18
  b) Australian Geomechanics "Landslide Risk Management" Volume 42, March 2007.
  c) AS 2870 – 2011, Australian Standard for Residential Slabs and Footings

Where changes to site conditions are identified during the maintenance and inspection program, reference should be made to relevant professionals (e.g. structural engineer, geotechnical engineer or Council). Where the property owner has any lack of understanding or concerns about the implementation of any component of the maintenance and inspection program the relevant engineer should be contacted for advice or to complete the component. It is assumed that Council will control development on neighbouring properties, carry out regular inspections and maintenance of the road verge, stormwater systems and large trees on public land adjacent to the site so as to ensure that stability conditions do not deteriorate with potential increase in risk level to the site. Also, individual Government Departments will maintain public utilities in the form of power lines, water and sewer mains to ensure they don't leak and increase either the local groundwater level or landslide potential.

6. CONCLUSION:

The site investigation identified the presence of fill, underlaid by sand within the lower eastern side and by clay within the upper western side of the block. The presence of a sandstone boulder or bedrock of at least low strength was indicated within the western side whilst interpreted shale bedrock was encountered at lower levels within the south and eastern sides of the block. A free-standing groundwater table was not encountered within the site, however seepage was encountered at RL ≈ 3.30m depth.

Further investigation is required within the site including the north-western side of the site to finalize the geological model and support required. Where bedrock is encountered at higher levels (Scenario 2), then a core drilling investigation at a minimum of two locations is recommended to assess the strength of the bedrock.

The proposed excavation within the eastern side of the site will mainly comprise of soil (fill/sand/clay) with minor bedrock within the base of the basement. The excavation within the western side of the site will comprise of soil (fill/clay) with bedrock (Scenario 2) or boulders with clay (Scenario 1). Therefore, conventional earth moving excavation machinery will be suitable for large parts of the works along with minor rock excavation equipment (e.g. rock hammers, rock saw, ripper). CGC should be consulted to assess size and excavation methodology.

Extreme care must be exercised not to damage the adjacent SW sewer main and the adjacent trees due to the excavation works and related ground vibrations. Sydney Water must be contacted regarding construction and design purposes to prevent delays in approval.

The construction of support either prior to or during excavation will be necessary for retaining structures along the northern, western and southern sides of the proposed excavation.

It is recommended that all new footings be constructed onto bedrock of similar bearing capacity. It is also recommended that the construction of the footings be inspected by an experienced geotechnical consultant, to assess the competency of the founding material.

Provided further investigation is undertaken and the recommendations of this report are implemented in the design and construction phases of the development, it is considered that the works can be carried out with negligible impact to the site and neighbouring properties and as such are considered suitable for the site.

The potential risks associated with the proposed development will be within 'Unacceptable' levels where insufficient/unsuitable support systems are implemented. However, where suitable engineer designed systems are implemented the risks will be reduced and can be maintained within 'Acceptable' risk criteria for the design life of the development, taken as 100 years.

Prepared by:	Reviewed by:
Marvin Lujan	Troy Crozier
Engineer	Principal
	MIEAust, MAIG, RPGeo – Geotechnical and Engineering

7. REFERENCES:

1. Australian Geomechanics Society 2007, "Landslide Risk Assessment and Management", Australian Geomechanics Journal Vol. 42, No 1, March 2007.
2. Geological Society Engineering Group Working Party 1972, "The preparation of maps and plans in terms of engineering geology" Quarterly Journal Engineering Geology, Volume 5, Pages 295 - 382.
3. C. W. Fetter 1995, "Applied Hydrology" by Prentice Hall. V. Gardiner & R. Dackombe 1983, "Geomorphological Field Manual" by George Allen & Unwin
4. Pells et. al. Design loadings for foundations on shale and sandstone in the Sydney region. Australian Geomechanics Society Journal, 1978.
5. Australian Standard AS 2870 – 2011, Residential Slabs and Footings.

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 1

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NOTES RELATING TO THIS REPORT

Introduction

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

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

Description and classification Methods

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

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

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

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

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

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

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

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

Sampling

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

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

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

Drilling Methods

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

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

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

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

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

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

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

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

Standard Penetration Tests

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

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

The test results are reported in the following form.

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

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

Cone Penetrometer Testing and Interpretation

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

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

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

The information provided on the plotted results comprises: -

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

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

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

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

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

Q_c = (12 to 18) C_u

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

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

Dynamic Penetrometers

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

Two relatively similar tests are used.

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

Laboratory Testing

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

Borehole Logs

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

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

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

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

Ground Water

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

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

Engineering Reports

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

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

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

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

Site Anomalies

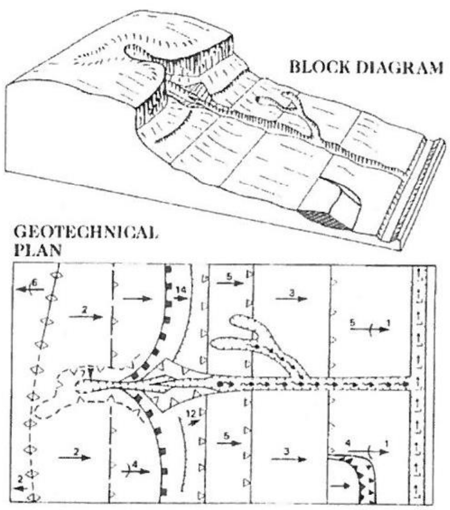
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

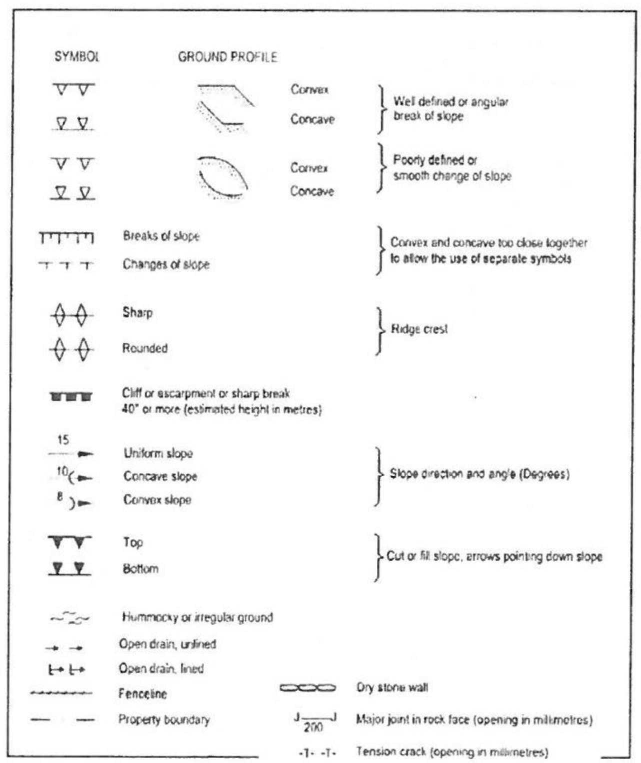
Reproduction of Information for Contractual Purposes

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

Site Inspection

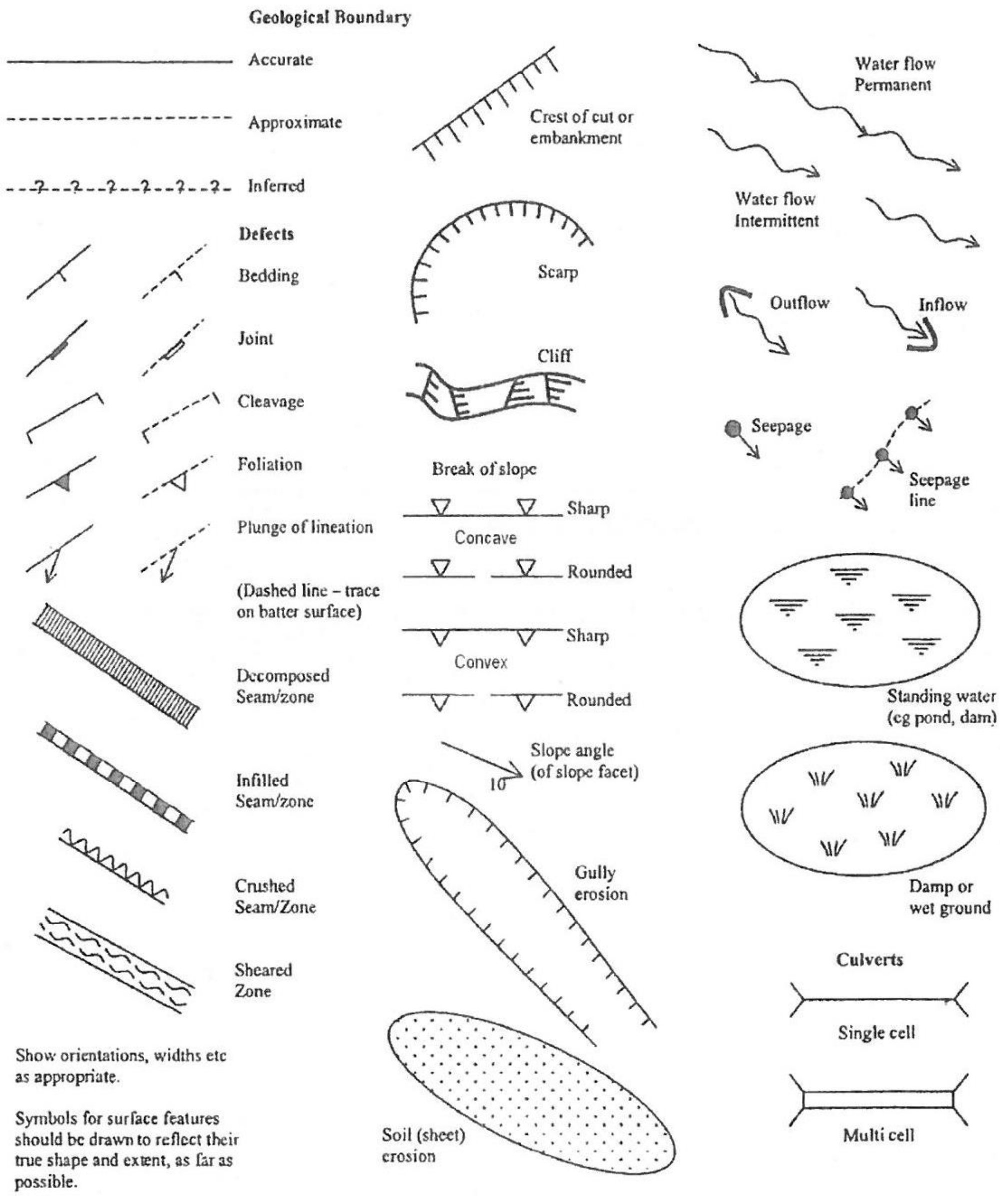
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.





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

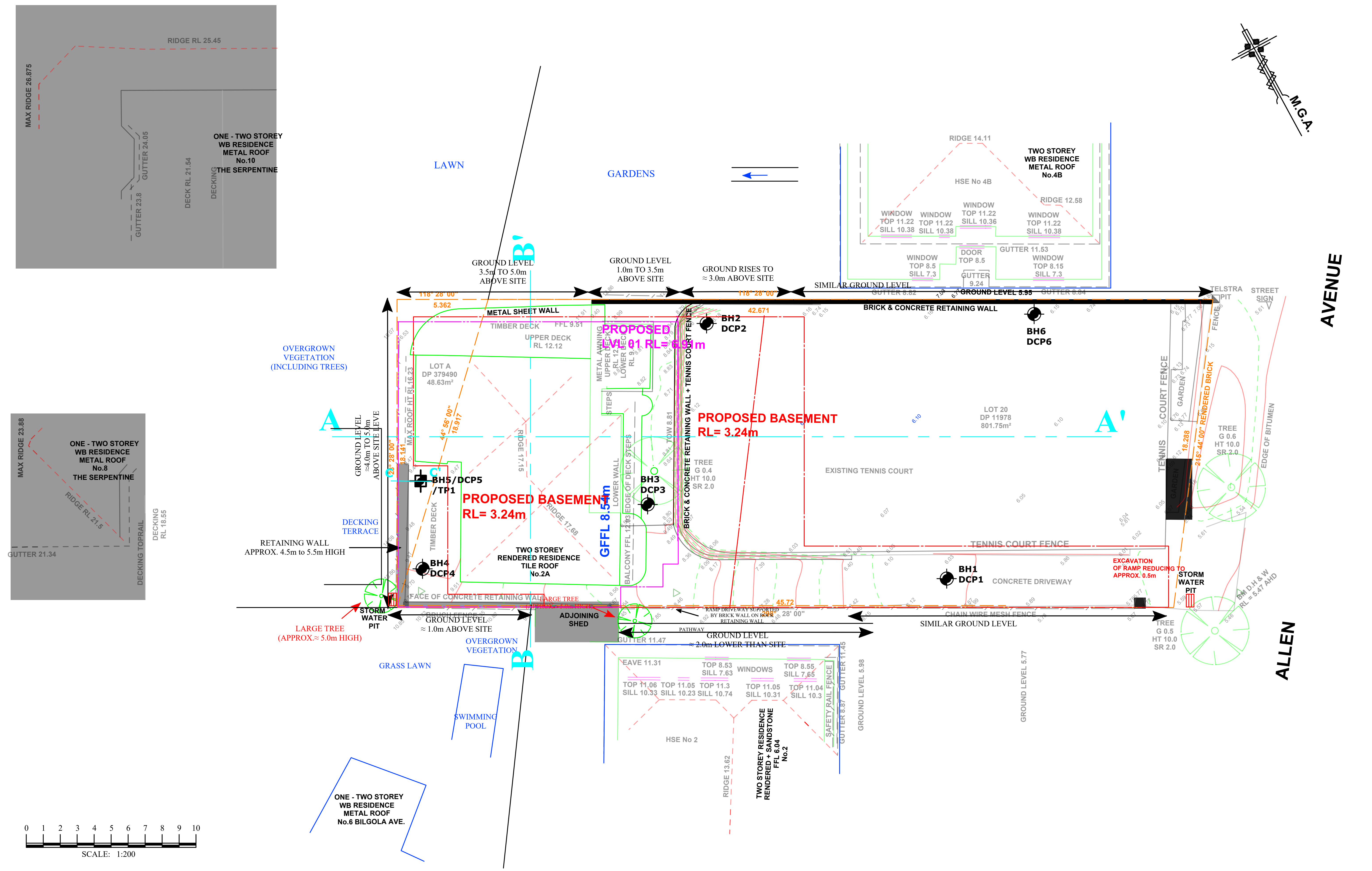
APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY



Examples of Mapping Symbols (after Guide to Slope Risk Analysis Version 3.1 November 2001, Roads and Traffic Authority of New South Wales).

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 2



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

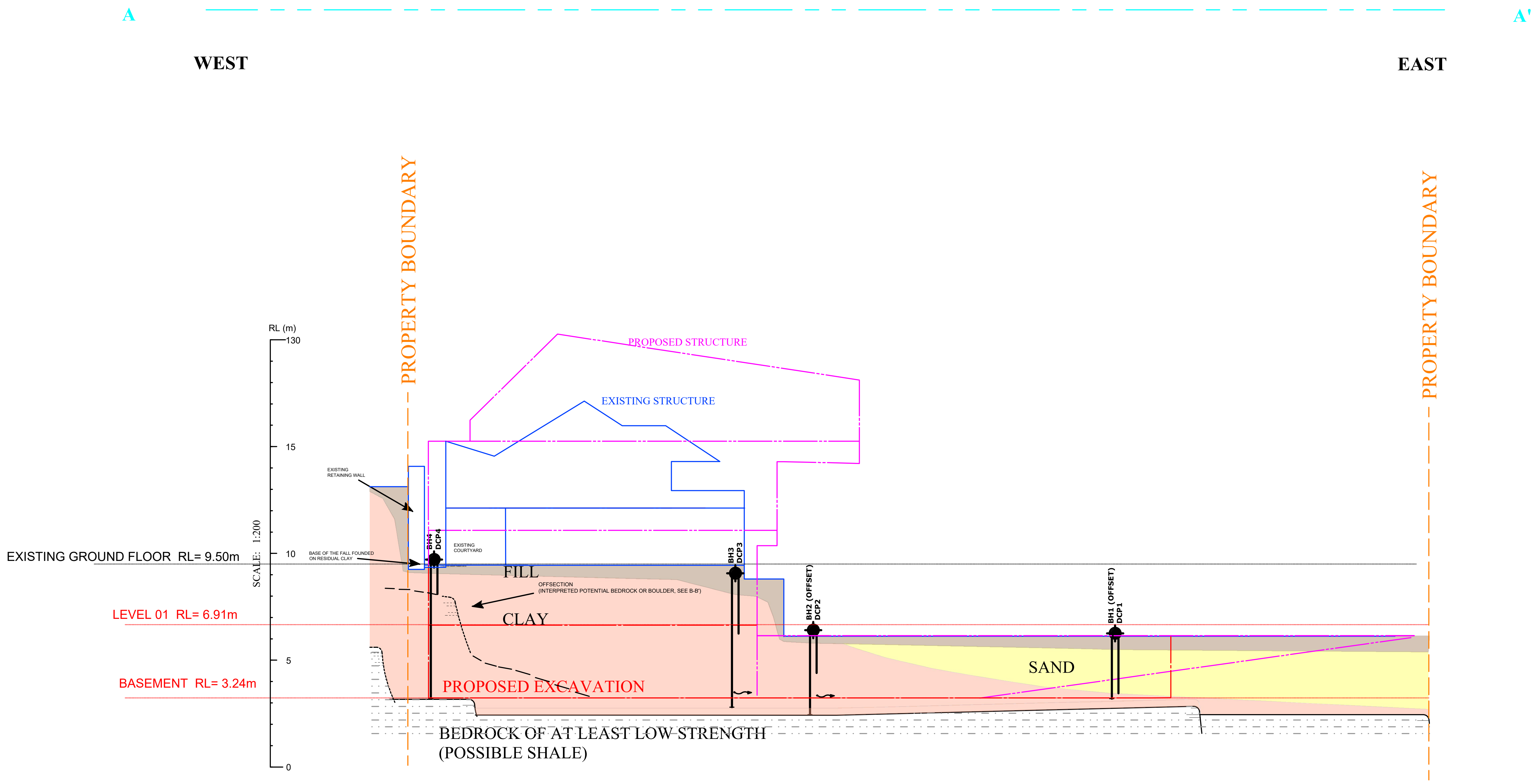
LEGEND

BH DCP	AUGER / DYNAMIC CONE PENETROMETER LOCATION
TP DCP	TEST PIT / DYNAMIC CONE PENETROMETER LOCATION
A — A'	CROSS-SECTION REFERENCE LINE
(orange dashed line)	PROPERTY BOUNDARY
(blue line)	EXISTING STRUCTURE
(magenta line)	PROPOSED STRUCTURE

SCALE:	1:200 @ A3
DRAWING:	FIGURE 1
DATE:	22/06/2022
APPROVED BY:	TMC
DRAWN BY:	ML
PROJECT:	2021-086

PREPARED FOR:
WALLHOUSE HOLDINGS

ADDRESS:
2A ALLEN AVENUE,
BILGOLA BEACH



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.

CROZIER
GEOTECHNICAL CONSULTANTS

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

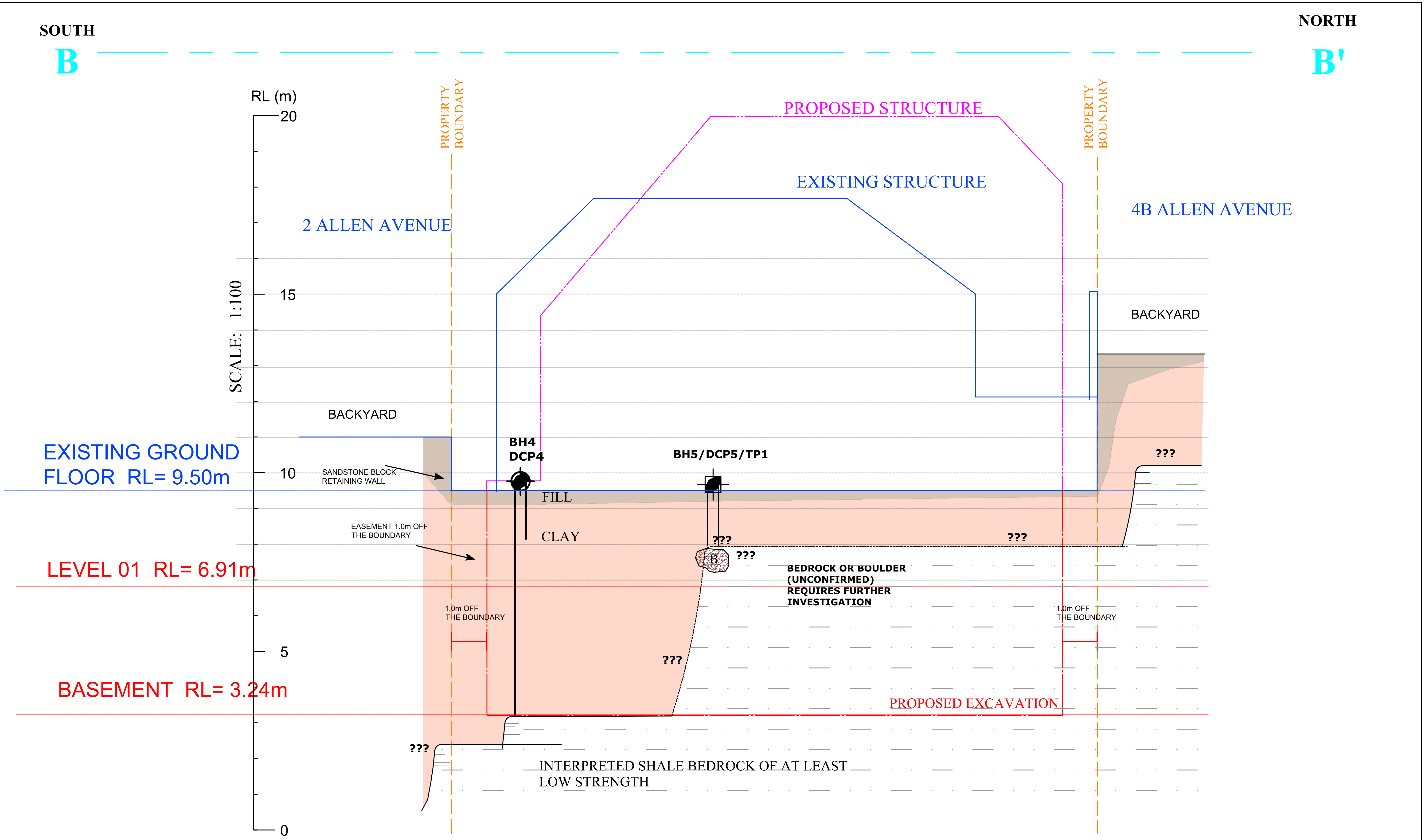
LEGEND

BH DCP	AUGER / DYNAMIC CONE PENETROMETER LOCATION
A — A'	CROSS-SECTION REFERENCE LINE
	EXISTING STRUCTURE
	PROPERTY BOUNDARY
	PROPOSED STRUCTURE
	PROPOSED EXCAVATION
	FILL
	SAND
	CLAY
	SHALE BEDROCK
	GEOLOGICAL BOUNDARY
	SEEPAGE

SCALE:	1:200 @ A3
DRAWING:	FIGURE 2
DATE:	22/06/2022
APPROVED BY:	TMC
DRAWN BY:	ML
PROJECT:	2021-086

PREPARED FOR:
WALLHOUSE HOLDINGS

ADDRESS:
2A ALLEN AVENUE,
BILGOLA BEACH



0 1 2 3 4 5
SCALE: 1:100

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

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

GEOLOGICAL MODEL FIGURE 3.

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

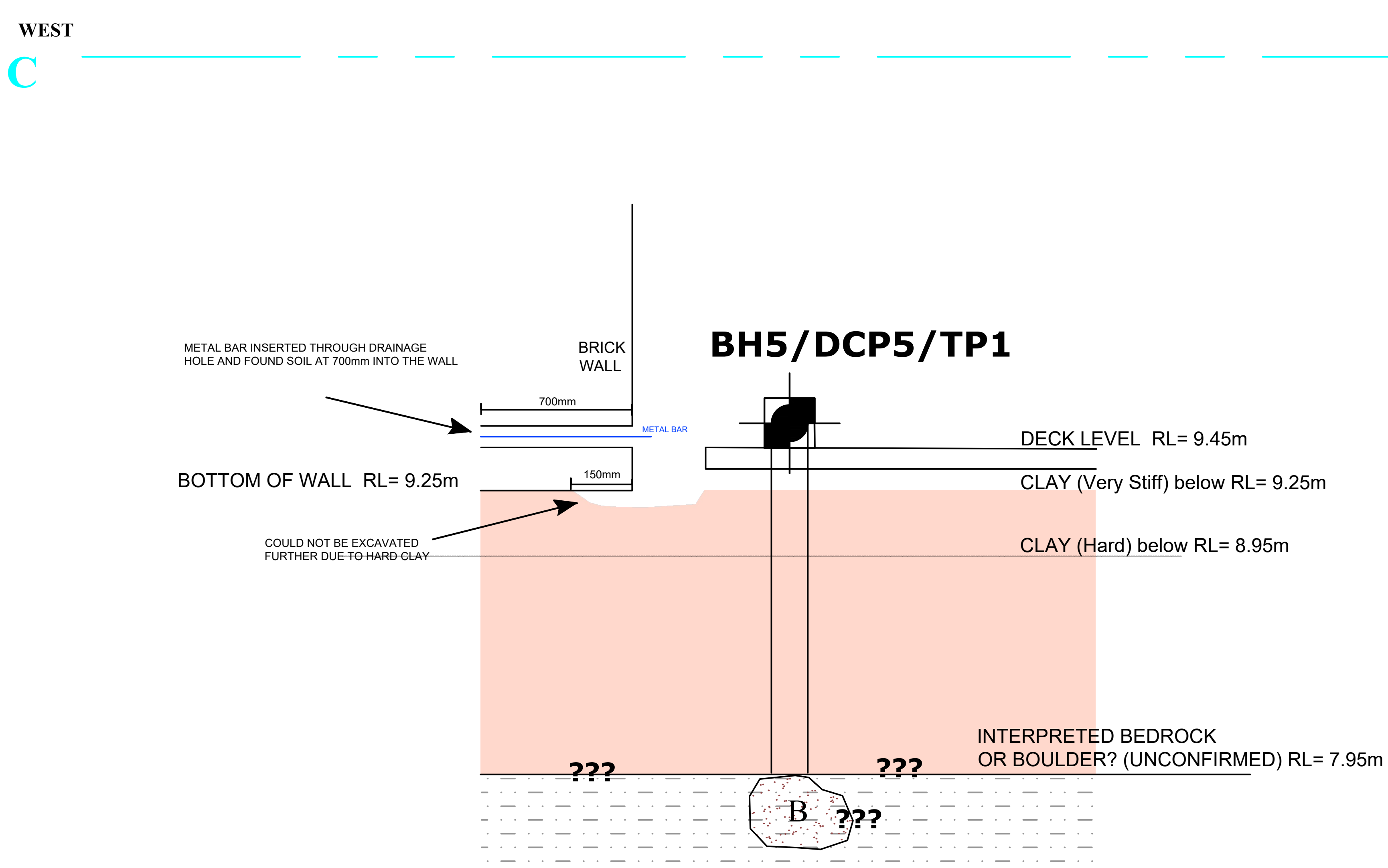
LEGEND

- BH DCP – AUGER / DYNAMIC CONE PENETROMETER LOCATION
- TP DCP BH – TEST PIT/ DYNAMIC CONE PENETROMETER/ DCP
- EXISTING STRUCTURE
- PROPERTY BOUNDARY
- PROPOSED STRUCTURE
- PROPOSED EXCAVATION
- FILL
- SHALE BEDROCK
- CLAY
- A — A' CROSS-SECTION REFERENCE LINE
- GEOLOGICAL BOUNDARY

SCALE:	1:100 @ A3
DRAWING:	FIGURE 3
DATE:	22/06/2022
APPROVED BY:	TMC
DRAWN BY:	ML
PROJECT:	2021-086

PREPARED FOR: WALLHOUSE HOLDINGS

ADDRESS: 2A ALLEN AVENUE, BILGOLA BEACH



0 0.5 1
SCALE: 1:20

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 D-D', PLEASE REFER TO FIGURE 1. SITE PLAN AND TEST LOCATIONS

FIGURE 4.

CROZIER GEOTECHNICAL CONSULTANTS

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

LEGEND

BOULDER | TEST PIT/ DYNAMIC CONE PENETROMETER/ DCP (TP DCP BH) | GEOLOGICAL BOUNDARY | CLAY | SHALE BEDROCK

SCALE: 1:20 @ A3
DRAWING: FIGURE 5
DATE: 22/06/2022

APPROVED BY: TMC
DRAWN BY: ML
PROJECT: 2021-086

PREPARED FOR: WALLHOUSE HOLDINGS

ADDRESS: 2A ALLEN AVENUE, BILGOLA BEACH

BOREHOLE LOG

CLIENT: Wallhouse Holdings | **DATE:** 20/04/2021 | **BORE No.:** BH 1

PROJECT: Demolition and Construction of new dwelling | **PROJECT No.:** 2021-086 | **SHEET:** 1 of 1

LOCATION: 2a Allen Avenue, Bilgola Beach | **SURFACE LEVEL:** R.L.= 6.00m

Depth (m)	Classification	Description of Strata PRIMARY SOIL - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.00		SLAB				
0.15		FILL: Loose, brown, fine to medium grained, moist, sand with some fine to medium grained gravels				
0.50	SM	SAND: Medium dense, light brown, fine to medium grained, moist, sand				
1.30		… loose				
1.50						
1.70		… medium dense				
2.60		SHALE (EX): Fine to medium grained, pale yellow				
2.80		AUGER REFUSAL at 2.80m depth on shale or sandstone bedrock of at least low strength				
3.00						
4.50						
6.00						

RIG: Dingo | DRILLER: AC

METHOD: Solid Stem spiral flight auger, tungsten carbide bit | LOGGED: JD

GROUND WATER OBSERVATIONS: None

REMARKS: | CHECKED: TMC

BOREHOLE LOG

CLIENT: Wallhouse Holdings **DATE:** 20/04/2021 **BORE No.:** BH 2

PROJECT: Demolition and Construction of new dwelling **PROJECT No.:** 2021-086 **SHEET:** 1 of 1

LOCATION: 2a Allen Avenue, Bilgola Beach **SURFACE LEVEL:** R.L.= 6.15m

Depth (m)	Classification	Description of Strata PRIMARY SOIL - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.00						
0.15		SLAB				
0.25		Layer of loose brown sand base				
0.35		Slag black, compacted with bluestone gravels				
	CI	CLAY: Very stiff, grey, medium plasticity, moist, clay with iron rich gravels				
0.80		… hard, red mottled orange				
1.50						
1.70		… zones of extremely weathered sandstone bedrock, red/brown, dry/moist				
1.85		… pale red/brown				
2.30		… with some clay				
3.00						
3.20		… moist/wet				
3.70						
		AUGER REFUSAL at 3.70m depth on shale or sandstone bedrock of at least low strength				
4.50						
6.00						

RIG: Dingo DRILLER: AC

METHOD: Solid Stem spiral flight auger, tungsten carbide bit LOGGED: JD

GROUND WATER OBSERVATIONS: bottom 750mm of the borehole saturated after 1hr

REMARKS: CHECKED: TMC

BOREHOLE LOG

CLIENT: Wallhouse Holdings **DATE:** 20/04/2021 **BORE No.:** BH 3

PROJECT: Demolition and Construction of new dwelling **PROJECT No.:** 2021-086 **SHEET:** 1 of 1

LOCATION: 2a Allen Avenue, Bilgola Beach **SURFACE LEVEL:** R.L.= 8.80m

Depth (m)	Classification	Description of Strata PRIMARY SOIL - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.00						
0.15		PEBBLES SURFACE				
		FILL: Loose, dark brown, fine to medium grained, moist, silty sand				
0.60		… orange mottled pale grey, becomig clayey				
0.75	CI	CLAY: Firm, pale grey mottled pale orange, medium plasticity, moist, clay		0.80		
			D	0.90		
0.90		… very stiff, dry/moist				
1.20		… hard, moist				
1.50						
2.50		… pale grey				
3.00						
3.50		… becoming orange red				
				3.80		
			D	4.00		
4.50						
5.00		… pink mottled pale grey				
5.50		… moist/wet, pale orange mottled grey clay with sub-agular sandstone gravel				
6.00		END OF BOREHOLE at 6.0m depth on moist/wet gravelly clay				

RIG: Dingo DRILLER: AC

METHOD: Solid Stem spiral flight auger, tungsten carbide bit LOGGED: ML

GROUND WATER OBSERVATIONS: Seepage encountered at 5.50m depth

REMARKS: CHECKED: TMC

BOREHOLE LOG

CLIENT: Wallhouse Holdings **DATE:** 20/04/2021 **BORE No.:** BH 4

PROJECT: Demolition and Construction of new dwelling **PROJECT No.:** 2021-086 **SHEET:** 1 of 1

LOCATION: 2a Allen Avenue, Bilgola Beach **SURFACE LEVEL:** R.L.= 9.50m

Depth (m)	Classification	Description of Strata PRIMARY SOIL - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.00						
0.10		VOID UNDER TIMBER DECK				
0.20		SLAB				
0.30		WASHED SOIL FORMING A VOID				
0.40	CI	CLAY: Stiff, grey, low to medium plasticity, moist, clay				
0.50		… very stiff				
0.90		… hard				
1.50		… grey mottled orange red, with some ironstone gravel (≤3cm diameter),				
2.00		… pink mottled pale grey				
2.50		… 0.50m layer of moist/wet clay	D	2.50 2.80		
3.00		… moist				
4.50						
			D	5.30 5.50		
6.00						
6.30		AUGER REFUSAL at 6.30m depth on shale or sandstone bedrock of at least low strength				

RIG: Dingo DRILLER: AC

METHOD: Solid Stem spiral flight auger, tungsten carbide bit LOGGED: ML

GROUND WATER OBSERVATIONS: None

REMARKS: CHECKED: TMC

BOREHOLE LOG

CLIENT: Wallhouse Holdings **DATE:** 20/04/2021 **BORE No.:** BH 5

PROJECT: Demolition and Construction of new dwelling **PROJECT No.:** 2021-086 **SHEET:** 1 of 1

LOCATION: 2a Allen Avenue, Bilgola Beach **SURFACE LEVEL:** R.L.= 9.45m

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
			Type	Tests	Type	Results
0.00						
0.10		VOID UNDER TIMBER DECK				
0.25		FILL: Loose, orange mottled grey, fine to medium grained, moist, silty sand with some ironstone gravels				
	CL	CLAY: Very stiff, pale grey, low plasticity, moist/dry, clay		0.60		
0.50		… hard	D	0.70		
				1.30		
1.50			D	1.50		
		AUGER REFUSAL at 1.5m depth on possibly bedrock of at least low strength or boulder				
3.00						
4.50						
6.00						

RIG: Dingo DRILLER: AC

METHOD: Solid Stem spiral flight auger, tungsten carbide bit LOGGED: ML

GROUND WATER OBSERVATIONS: None

REMARKS: CHECKED: TMC

BOREHOLE LOG

CLIENT: Wallhouse Holdings

DATE: 20/04/2021

BORE No.: BH 6

PROJECT: Demolition and Construction of new dwelling

PROJECT No.: 2021-086

SHEET: 1 of 1

LOCATION: 2a Allen Avenue, Bilgola Beach

SURFACE LEVEL: R.L.= 6.15m

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.00		SLAB				
0.15		Layer of loose brown sand sub-base				
0.25		Layer of black slag with bluestone gravels sub-base				
0.45		FILL: Loose, brown, fine to medium grained, moist, sand with some fine to medium grained gravels				
1.50		AUGER REFUSAL at 1.5m depth on fill (bricks)				
3.00						
4.50						
6.00						

RIG: Dingo

DRILLER: AC

METHOD: Solid Stem spiral flight auger, tungsten carbide bit

LOGGED: JD

GROUND WATER OBSERVATIONS: None

REMARKS:

CHECKED: TMC

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Wallhouse Holdings **DATE:** 20/04/2021

PROJECT: Demolition and Construction of new dwelling

PROJECT No.: 2021-086

LOCATION: 2a Allen Avenue, Bilgola Beach **SHEET:** 1 of 1

	Test Location							
Depth (m)	DCP1 RL=6.00m	DCP2 RL=6.15m	DCP3 RL=8.80m	DCP4 RL=9.50m	DCP5 RL=9.45m	DCP6 RL=6.15m		
0.00 - 0.10	-	-	1	-	-	-		
0.10 - 0.20	-	-	2	-	2	2		
0.20 - 0.30	-	3	5	-	5	4		
0.30 - 0.40	-	8	2	-	6	8		
0.40 - 0.50	-	8	2	3	7	3		
0.50 - 0.60	2	10	3	6	9	7		
0.60 - 0.70	3	6	5	7	13	7		
0.70 - 0.80	2	6	3	7	18	14		
0.80 - 0.90	2	12	3	7	15	26		
0.90 - 1.00	3	11	7	9	15	11		
1.00 - 1.10	3	9	5	9	19	23		
1.10 - 1.20	2	9	9	11	18	10		
1.20 - 1.30	2	20	14	20	16	9		
1.30 - 1.40	1	15	15	17 (B) @1.35m depth	18	10		
1.40 - 1.50	1	13	12		30 (B) @1.50m depth	10		
1.50 - 1.60	2	25	14			6		
1.60 - 1.70	2	17 (B) @1.75m depth	10			5		
1.70 - 1.80	3		12			6		
1.80 - 1.90	3		22			5		
1.90 - 2.00	3		27			5		
2.00 - 2.10	3		18			7		
2.10 - 2.20	3		20 (B) @2.20m depth			7		
2.20 - 2.30	6					7		
2.30 - 2.40	18					8		
2.40 - 2.50	12					9		
2.50 - 2.60	10 (B) @2.55m depth					20(B) @2.55m depth		
2.60 - 2.70								

TEST METHOD: AS 1289. F3.2, CONE PENETROMETER

REMARKS:

(B) Test hammer bouncing upon refusal on solid object

-- No test undertaken at this level due to prior excavation of soils

Appendix 3

TABLE : A

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood of Slide	Spatial Impact of Slide		Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip of soils from basement excavation (<20m³)		Excavation (≤3.5m depth increasing west to ≤7.0m depth) of sand and clay	a) Pathway, impact 50%, ≥1.0m from the excavation b) Dwelling, impact 15%, ≥1.90m from excavation c) Rear gardens, impact 15%, ≥1.0m from the excavation d) Pathway, impact 20%, ≥1.0m from excavation e) Lawn, impact 10%, ≥3.0m from excavation f) Dwellig, impact 10%, ≥6.0m from excavation g) Terrace, impact 50%, ≥2.0m from the excavation h) Lawn, impact 10%, ≥6.0m from the excavation i) Lawn, impact 10%, ≥3.0m from the excavation j) Pool. impact 50%, ≥6.0m from the excavation k) Shed, impact 90%, ≥1.0m from the excavation l) Garden, impact 50%, directly adjacent to ramp excavation m) Dwelling, impact 10%, ≥3.5m from the excavation n) Driveway, impact 10%, ≥3.0m from the excavation r) Lawn, impact 5%, ≥1.0m from the excavation		a) Person in the pathway, 1hr/day avge. b) Person in the dwelling, 23hrs/day avge. c) Peron in the rear garden, 1hr/day avge. d) Person in the pathway, 1hr/day e) Person in the lawn, 1hr/day f) Person in the dwelling, 23hrs/day g) Person in the terrace, 1hr/day h) Person in the lawn, 1hr/day i) Person in the lawn, 1hr/day j) Person in the Pool, 1hr/day k) Person in the Shed, 10hrs/day l) Person in the garden, 1hr/day m) Person in the dwelling, 23hrs/day n) Person in the driveway, 1hr/day r) Person in the lawn, 1hr/day	a) Likely to not evacuate b) Almost certain to not evacuate c) Likely to not evacuate d) Almost certain to not evacuate e) Almost certain to not evacuate f) Almost certain to not evacuate g) Almost certain to not evacuate h) Almost certain to not evacuate i) Almost certain to not evacuate j) Almost certain to not evacuate k) Almost certain to not evacuate l) Likely to not evacuate m) Almost certain to not evacuate n) Likely to not evacuate r) Possible to not evacuate	a) Person in open space, likely engulfed into the excavation b) Person in the house, likely injured by ground collapsing or structure failure c) Person in open space, likely engulfed into the excavation d) Person in open space, likely engulfed into the excavation e) Person in open space, likely engulfed into the excavation f) Person in the house, likely injured by ground collapsing or structure failure g) Person in open space, likely engulfed into the excavation h) Person in open space, likely engulfed into the excavation i) Person in open space, likely engulfed into the excavation j) Person in open space, likely engulfed into the excavation k) Person in the shed, likely injured by ground collapsing or structure failure l) Person in open space, likely engulfed into the excavation m) Person in the dwelling, likely injured by ground collapsing or structure failure n) Person in open space, likely engulfed into the excavation r)Person in open space, likely engulfed into the excavation	
			Likely	**Prob. of Impact**	**Impacted**				
		a) Pathway (No.4B Allen Av.)	0.01	0.50	0.50	0.04	0.75	0.90	**7.03E-05**
		b) Dwelling (No.4B Allen Av.)	0.01	0.50	0.15	0.96	0.9	0.90	**5.82E-04**
		c) Rear gardens (No.4B Allen Av.)	0.01	0.50	0.15	0.04	0.75	0.90	**2.11E-05**
		d) Pathway (No.10 The Serpentine)	0.01	0.50	0.20	0.04	0.9	0.90	**3.38E-05**
		e) Lawn (No.10 The Serpentine)	0.01	0.30	0.10	0.04	0.9	0.90	**1.01E-05**
		f) Dwelling (No.10 The Serpentine)	0.01	0.20	0.10	0.96	0.9	0.90	**1.55E-04**
		g) Terrace (No.8 The Serpentine)	0.01	0.50	0.50	0.04	0.9	0.90	**8.44E-05**
		h) Lawn (No.8 The Serpentine)	0.01	0.30	0.10	0.04	0.9	0.90	**1.01E-05**
		i) Lawn (No.8 Bilgola Ave.)	0.01	0.50	0.10	0.04	0.9	0.90	**1.69E-05**
		j) Pool (No.8 Bilgola Ave.)	0.01	0.50	0.50	0.04	0.9	0.90	**8.44E-05**
		k) Shed (No.2 Allen Ave.)	0.01	0.50	0.60	0.42	0.9	0.90	**1.01E-03**
		l) Gardens (No.2 Allen Ave.)	0.01	0.20	0.50	0.04	0.75	0.15	**4.69E-06**
		m) Dwelling (No.2 Allen Ave.)	0.01	0.30	0.10	0.96	0.9	0.90	**2.33E-04**
		n) Driveway (No.2 Allen Ave.)	0.01	0.10	0.10	0.04	0.75	0.15	**4.69E-07**
		r) Allen Ave. Lawn	0.01	0.10	0.05	0.04	0.5	0.15	**1.56E-07**

* evacuation scale from Almost Certain to not evacuate (1.0), Likely (0.75), Possible (0.5), Unlikely (0.25), Rare to not evacuate (0.01). Based on likelihood of person knwoing of landslide and completely evacuating area prior to landslide impact.
* vulnerability assessed using Appendix F - AGS Practice Note Guidelines for Landslide Risk Management 2007

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip of soils from basement excavation (<20m³)	a) Pathway (No.4B Allen Av.)	Likely	Event will probably occur under adverse circumstances over the design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Very High
		b) Dwelling (No.4B Allen Av.)	Likely	Event will probably occur under adverse circumstances over the design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Very High
		c) Rear gardens (No.4B Allen Av.)	Likely	Event will probably occur under adverse circumstances over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	High
		d) Pathway (No.10 The Serpentine)	Likely	Event will probably occur under adverse circumstances over the design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Very High
		e) Lawn (No.10 The Serpentine)	Possible	The event could occur under adverse conditions over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	Moderate
		f) Dwelling (No.10 The Serpentine)	Unlikely	The event might occur under very adverse circumstances over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	Low
		g) Terrace (No.8 The Serpentine)	Likely	Event will probably occur under adverse circumstances over the design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Very High
		h) Lawn (No.8 The Serpentine)	Possible	The event could occur under adverse conditions over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	Moderate
		i) Lawn (No.8 Bilgola Ave.)	Likely	Event will probably occur under adverse circumstances over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	High
		j) Pool (No.8 Bilgola Ave.)	Likely	Event will probably occur under adverse circumstances over the design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Very High
		k) Shed (No.2 Allen Ave.)	Likely	Event will probably occur under adverse circumstances over the design life.	Catastrophic	Site structures completely destroyed, significant stabilising or MAJOR damage to neighbouring property.	Very High
		l) Gardens (No.2 Allen Ave.)	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Moderate
		m) Dwelling (No.2 Allen Ave.)	Possible	The event could occur under adverse conditions over the design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Very High
		n) Driveway (No.2 Allen Ave.)	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Moderate
		r) Allen Ave. Lawn	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required or no impact to neighbouring properties.	Very Low

* 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%.

TABLE: C

Recommended Maintenance and Inspection Program

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

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

Appendix 4

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CERTIFICATE OF ANALYSIS 271103

Client Details

Client	Crozier Geotechnical Consultants
Attention	Marvin Lujan
Address	Unit 12/42-46 Wattle Rd, Brookvale, NSW, 2100

Sample Details

Your Reference	**2021-086, 2a Allen Avenue, Bilgola Beach**
Number of Samples	2 soil
Date samples received	30/04/2021
Date completed instructions received	08/06/2021

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Please refer to the last page of this report for any comments relating to the results.

Report Details

Date results requested by	16/06/2021
Date of Issue	16/06/2021

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Accredited for compliance with ISO/IEC 17025 - Testing. **Tests not covered by NATA are denoted with ***

Results Approved By
Diego Bigolin, Team Leader, Inorganics

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil			
Our Reference		271103-1	271103-2
Your Reference	UNITS	BH1	BH5
Depth		5.5-5.7	5.3-5.5
Date Sampled		20/04/2021	28/04/2021
Type of sample		soil	soil
Date prepared	-	10/06/2021	10/06/2021
Date analysed	-	10/06/2021	10/06/2021
pH 1:5 soil:water	pH Units	5.2	5.3
Electrical Conductivity 1:5 soil:water	µS/cm	64	55
Chloride, Cl 1:5 soil:water	mg/kg	25	10
Sulphate, SO4 1:5 soil:water	mg/kg	98	95
Resistivity in soil*	ohm m	160	180

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25°C in accordance with APHA latest edition 2510 and Rayment & Lyons.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity (non NATA). Resistivity (calculated) may not correlate with results otherwise obtained using Resistivity-Current method, depending on the nature of the soil being analysed.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			10/06/2021	[NT]	[NT]	[NT]	[NT]	10/06/2021	[NT]
Date analysed	-			10/06/2021	[NT]	[NT]	[NT]	[NT]	10/06/2021	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	100	[NT]
Electrical Conductivity 1:5 soil:water	µS/cm	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	104	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	96	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	108	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]

Result Definitions	
NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.

The recommended maximums for analytes in urine are taken from “2018 TLVs and BEIs”, as published by ACGIH (where available). Limit provided for Nickel is a precautionary guideline as per Position Paper prepared by AIOH Exposure Standards Committee, 2016.

Guideline limits for Rinse Water Quality reported as per analytical requirements and specifications of AS 4187, Amdt 2 2019, Table 7.2

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals (not SPOCAS); 60-140% for organics/SPOCAS (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Measurement Uncertainty estimates are available for most tests upon request.

Analysis of aqueous samples typically involves the extraction/digestion and/or analysis of the liquid phase only (i.e. NOT any settled sediment phase but inclusive of suspended particles if present), unless stipulated on the Envirolab COC and/or by correspondence. Notable exceptions include certain Physical Tests (pH/EC/BOD/COD/Apparent Colour etc.), Solids testing, total recoverable metals and PFAS where solids are included by default.

Samples for Microbiological analysis (not Amoeba forms) received outside of the 2-8°C temperature range do not meet the ideal cooling conditions as stated in AS2031-2012.

Report Comments

MISC_INORG: pH/EC Samples were out of the recommended holding time for this analysis.

Appendix 5

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}	5×10^{-2}	10 years	20 years	The event is expected to occur over the design life.	ALMOST CERTAIN	A
10^{-2}	5×10^{-3}	100 years	200 years	The event will probably occur under adverse conditions over the design life.	LIKELY	B
10^{-3}	5×10^{-4}	1000 years	2000 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10^{-4}	5×10^{-5}	10,000 years	20,000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10^{-5}	5×10^{-6}	100,000 years	200,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
10^{-6}		1,000,000 years		The event is inconceivable or fanciful over the design life.	BARELY CREDIBLE	F

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

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

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

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

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

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

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

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

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

Notes: (5) For Cell A5, may be subdivided such that a consequence of less than 0.1% is Low Risk.
(6) When considering a risk assessment it must be clearly stated whether it is for existing conditions or with risk control measures which may not be implemented at the current time.

RISK LEVEL IMPLICATIONS

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

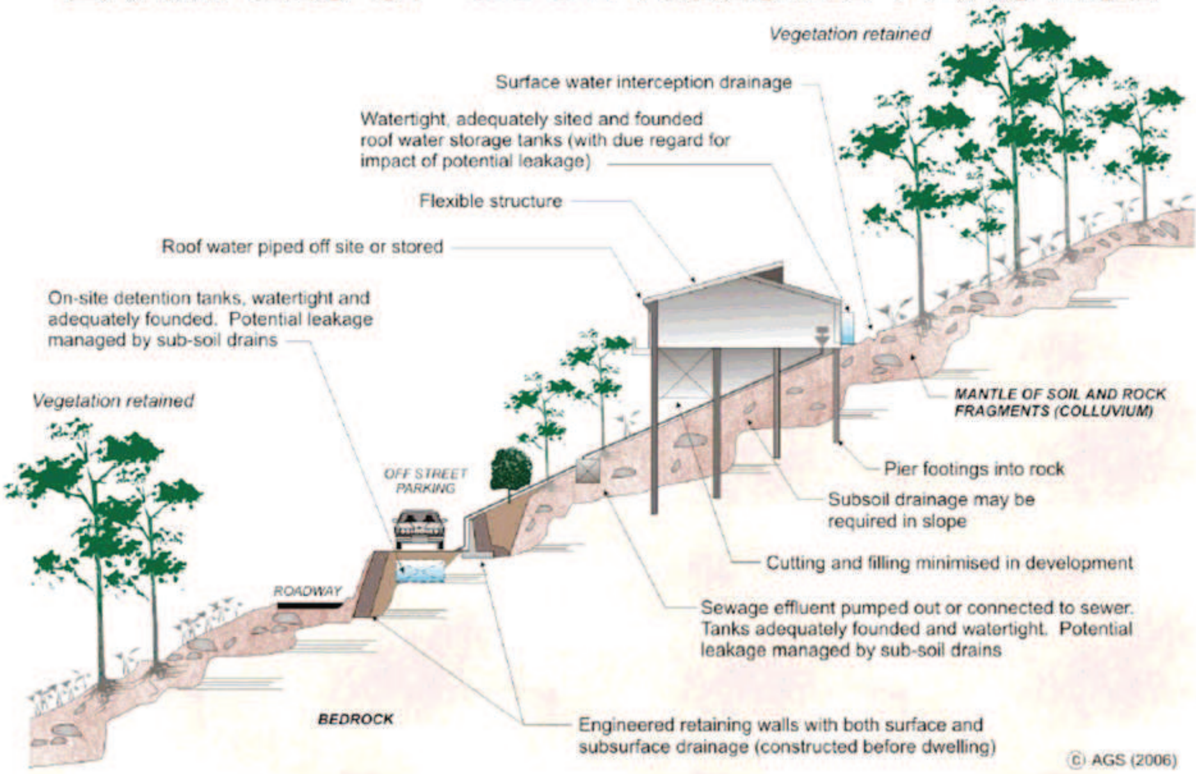
Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

Appendix 6

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



EXAMPLES OF POOR HILLSIDE PRACTICE