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REPORT ON GEOTECHNICAL SITE INVESTIGATION

for

PROPOSED NEW HOUSE

at

2075 PITTWATER ROAD, BAYVIEW

Prepared For

Mr. JOHN SIMMONDS

Project: 2012-176

January, 2016

Document Revision Record

Issue No	Date	Details of Revisions
0	7th September 2012	Original issue
A	2nd May 2013	2nd Stage Geotechnical Investigation
1	25th January 2016	Updated Architectural plans

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1 – To be submitted with Development Application

Development Application for____
Name of Applicant
Address of site 2075 Pittwater Road, Bayview

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 ______________________________ 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:

Please mark appropriate box

- ☐ 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 HOUSE
Report Date: 25/01/2016 PROJECT NO. 2012-176.1
Author: J. BUTCHER / T. CROZIER
Author's Company/Organisation: CROZIER GEOTECHNICAL CONSULTANTS

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

- PLANS BY CLAYTON ORSZACZKY, PROJECT NO. 11.15
SK001 TO SK009, SK12 TO SK15, REV: C TO G
DATED: 4/12/2015

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 ......T. Cz..

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

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

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

Company... **Crozier Geotechnical Consultants**

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 2075 Pittwater Road, Bayview

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 HOUSE PROJECT NO. 2012-176.1
Report Date: 25/01/2016
Author: J. BUTLER / T. CROZIER
Author's Company/Organisation: CROZIER GEOTECHNICAL CONSULTANTS

Please mark appropriate box

- ☑ Comprehensive site mapping conducted 24/08/2012 (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 24/08/2012, 15-16/04/2013
- ☑ 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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APPENDICES

Crozier Geotechnical Consultants *ABN: 96 113 453 624*
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Date: 25th January 2016

Project No.: 2012-176

GEOTECHNICAL REPORT FOR PROPOSED NEW HOUSE
2075 PITTWATER ROAD, BAYVIEW, NSW.

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for a proposed new house at 2075 Pittwater Road, Bayview, NSW. The investigation was undertaken by Crozier Geotechnical Consultants at the request of the owner Mr. John Simmonds.

Previous geotechnical investigation was carried out at the site (Project No. 2012-176, August 2012) for a proposed new house. A second stage of investigation was carried out at the site including cored boreholes drilled by a specialist contractor (Project No. 2012-176A, April 2013). It is understood that the design has now changed and geotechnical assessment of the new design in regard to the site is required. The previous reporting should be read in conjunction with this report.

It is understood that the proposed works involve demolition of the existing house and construction of a new three storey house with swimming pool. Most of the house will be constructed at existing ground levels however some excavation will be required for the new swimming pool and for the western end of the proposed first floor. It is expected that the excavation works will extend to approximately 2.50m depth.

The site is located within the H1 landslip hazard zone as identified within Pittwater Councils Geotechnical Risk Management Policy Map and requires geotechnical investigation and landslide risk assessment in line with the requirements of Pittwater Council Geotechnical Risk Management Policy It is also classified as Acid Sulfate Soils hazard Class □2 and 5□. Therefore an assessment of these type of soils is required to accompany the development submission.

The geotechnical investigation comprised:

- A detailed geotechnical inspection and mapping of the site and adjacent land by a Principal Engineering Geologist and Geotechnical Engineer.
- Photographic record of site conditions.
- Drilling of boreholes to determine subsurface geology and depth to bedrock along with Dynamic Penetrometer testing.

The following plans and diagrams were supplied for the new development submission;

- Architectural Plans by Clayton Orszaczky Architects, Project No. 11.15, Drawings:

SK001, SK002, SK003, SK004, □all revision C, all dated: 4/12/15

SK005 □revision F, SK006 □revision G, SK007 □revision F, SK008 □revision C, all dated: 4/12/15

SK009 □Revision E, SK010 □Revision B, all dated: 4/12/15

SK12, SK13, SK14, all revision E, all dated 1/12/15

SK15 □revision B dated 1/12/15, SK019 □revision C dated: 4/12/15

2. SITE FEATURES:

2.1. Description:

The site is located on the high west side of Pittwater Road within steeply south-east sloping topography adjacent to the western shore of Pittwater. It is located at the base of a very steep south-east dipping slope which is formed as part of a steep sided drainage gully that passes to the south of the site. This gully flows down the east side of a steeply sided north striking ridge line which extends to Church Point, Sydney NSW.

It is a long rectangular shaped block with a two storey brick and weatherboard house located on the front half of the property with a small single storey studio located up slope to the rear. There is a gentle sloping lawn and gardens at the front with moderately sloping landscaped gardens extending across the rear western half of the block. The site has a southern side boundary of 76.65m length and a front eastern street front boundary of 15.24m length as referenced from the provided survey plan.

2.2. Geology:

Reference to the Sydney 1:100,000 Geological Series sheet (9130) indicates that the site is located near the boundary between the Hawkesbury Sandstone (Rh) and Upper Narrabeen Group rocks (Rnn). Hawkesbury Sandstone which is of Triassic Age typically comprises medium to coarse grained quartz sandstone with minor lenses of shale and laminite and commonly forms a capping to the ridges in this area.

Newport Formation rocks (Upper Narrabeen Group) are slightly older and found lower in the stratigraphy than the Hawkesbury Sandstone. They comprise interbedded laminite, shale and quartz to lithic quartz sandstone and pink clay pellet sandstone. It is considered based on our experience in the area that the majority of the site is entirely underlain by Upper Narrabeen Group sandstone, shale and laminate. Estuarine sediments are expected to the east of the site, as seen in the Pittwater foreshore.

3. FIELD WORK

3.1 Methods:

The investigation was carried out in two stages. The initial investigation was carried out on the 24th August 2012 by Senior Engineering Geologist. It involved geological/geomorphological mapping of the site with examination of soil and garden slopes, retaining walls, the house and associated structures for stability.

It included the drilling of four hand auger boreholes (BH1 to BH4) to investigate sub-surface geology along with one test pit to identify the footing of the existing house structure. Penetrometer testing (DCP1 to DCP4b) was conducted in accordance with AS1289.6.3.2 □ 1997, □Determination of the penetration resistance of a soil □9kg dynamic cone penetrometer□to estimate soil properties and confirm depths to bedrock.

The second stage of investigation was carried out on the 15th and 16th April 2013. This investigation involved the excavation of test pits and boreholes at the rear of the house to expose existing footings and foundation conditions along with two boreholes in the front of the site to determine bedrock depth. The test pits were excavated by hand due to site access limitations however the boreholes at the front of the site were undertaken by Terratest Drilling Services using an XC Drill rig. These boreholes were undertaken using 100mm augers (CFA) with Tungsten Carbide bit from surface with Standard Penetration Tests (SPT) at 1.0m depth intervals. Upon refusal the boreholes were extended using NMLC diamond core techniques.

A visual inspection of the site conditions was also carried out by a Senior Geotechnical Engineer on 25th January 2016 to assess changes in site conditions since the previous stages of investigation.

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

3.2. Field Observations:

The road reserve adjacent to the site is gently sloping to near level before a rendered brick wall extends around the front of the property. A level lawn and garden are located in the south-east corner of the site. A gently sloping concrete driveway enters the site near the north-east corner and swings around to a concrete car space along the front edge of the existing house. This driveway and car space is raised above the level of the lawn and garden.

The existing house is a two storey brick and weatherboard structure on the front half of the block which extends to within 1.0m of both side boundaries. The rear three quarters of the house appears formed at the base of an excavation into the hill slope. On the northern boundary adjacent to the house entrance is a 2.0-3.0m high vertical concrete block retaining wall which supports the base of the slope within the neighbouring property (No. 2079). To the rear of this wall and extending around the entire western side of the existing house is a 2.0 to 3.0m high steeply sloping concrete retaining wall which appears to have been constructed of sprayed concrete. This wall contains numerous seepage holes and appeared in good condition with a concrete path formed at its base on the western side of the house at ground floor level. Extending down the southern side boundary is a low timber sleeper retaining wall which supports a garden in the neighbouring property (No. 2073) above a paved pathway along the southern wall of the sites house at ground floor level. The existing house is constructed with brick external walls and weatherboard and timber structure internally.

The original house appears to be approximately 40-50 years of age and shows no obvious signs of cracking or settlement in external masonry walls. The structure appears in a relatively well maintained condition. The timber retaining wall extending along the southern boundary shows significant rotation into the site however the sloping concrete and concrete block retaining walls both appear in good condition.

At the rear north-west corner of the existing house a set of stairs extend up from the first floor level to a small weatherboard room located above the crest of the sloping concrete retaining wall. Adjacent to the rear western edge of this small weatherboard room is another low (1.0m) sloping concrete retaining wall of similar construction which supports the slope above. This wall is then replaced to the south of the room by a brick retaining wall of up to 1.8m in height that extends across to the southern property boundary and retains a patio.

A single storey brick studio is located directly to the west of the concrete retaining wall and up slope of the weatherboard portion of the house. This studio which is of brick construction appears to be a more recent addition to the original house and is formed at the base of a shallow (<1.2m) excavation along its northern and western sides. Concrete block retaining walls support the excavated slope above the level of the studio floor and also a set of concrete steps that lead to the backyard.

A tiled patio is located to the south of the studio, supported along its eastern edge by the brick retaining wall. The patio has a low concrete block retaining wall along its rear western edge before sloping gardens extend up to the ground surface slope of the backyard.

The rear half of the site contains landscaped garden beds with paths and roughly formed steps leading up to the rear property boundary. This portion of the site contains several very low timber log garden bed walls and several large to medium sized trees and low vegetation.

The neighbouring property to the north (No. 2079) is currently under development at the time of the most recent inspection. The previous site structures have been demolished and excavation works have been carried out across the site to □ 6.0m depth at the rear. The excavation extends to within 0.50m of the common boundary with the front half of the excavation not supported along the boundary and the western half along this perimeter supported by a steel and concrete support wall (see Photo: 1 below).

Photo: 1 – Development underway in No. 2079

The neighbouring property to the south (No. 2073) contains a gently to moderately south-east sloping garden and driveway at the front with a three storey brick and weatherboard house on the centre of the block. A steep slope, potentially natural, extends around the rear north-west corner of this house before a moderately sloping lawn covered backyard with garden surrounds extends to the rear boundary.

The neighbouring property to the west (No. 38 Kananook Ave) is located upslope and contains moderately to gently sloping landscaped garden beds that extend up to a residential house structure.

A limited inspection did not identify any signs of excess surface stormwater flow, erosion or landslip instability within either of these two neighbouring properties (No. 2073 and No. 38) that may impact on the site.

3.3. Field Testing:

The past borehole information is included below from the two stages of investigation in 2012 and 2013.

First Stage of Investigation (August 2012)

Borehole 1 was drilled through the pathway along the southern side of the existing house. This bore was attempted several times due to the numerous service lines exposed during drilling. Borehole 1d was drilled through the location of test pit 1 and identified pavers over brown sandy fill to 0.14m depth before brown and red-brown sand, clay and gravel, uncontrolled fill was intersected. At 0.70m depth brown and red-brown clay fill was identified to 0.85m depth where black, silty clay to clayey silt with plant roots was intersected. This horizon contained fine grained sand and silt and extended to 1.20m depth where a thin horizon of dark brown moist clayey sand was identified. At 1.30m brown with grey and orange-brown moist plastic sandy clay with ironstone gravel was intersected. Horizons of moist to wet clayey sand to sandy clay were then intersected to 2.20m depth before auger refusal occurred on an ironstone band.

A test pit was conducted at borehole 1d to expose the existing house footing. This test pit identified a concrete footing at 350mm depth below the surface of the paved pathway. The concrete footing extended 220mm out from the external wall of the house and extended to 0.65m depth below the adjacent path. Attempts to determine the exact width of the footing were unsuccessful.

DCP tests conducted adjacent to the boreholes identified variable density and consistency within the fill and clay soils. DCP 1a was conducted adjacent to the existing house footing, through borehole 1d. This test identified the fill and silty clay near the base of the house footing as being of firm consistency however the black silty clay/clayey silt is soft. The clay below 1.05m depth is of stiff consistency to 1.80m depth and then very stiff to hard to 2.85m depth.

Borehole 2 was drilled at the rear of the house, close to the crest of the sloping concrete retaining wall. This bore identified silty sand fill to 0.30m depth overlying yellow-brown and orange-brown clay with ironstone gravel. This horizon extended to 1.10m depth before auger refusal occurred on an ironstone band.

DCP tests undertaken adjacent to and through the base of the borehole identified the clay soils as being of stiff consistency to 1.05m then very stiff to hard. The results below 1.20m depth are indicative of weathered bedrock.

Borehole 3 was drilled in the garden slope at the rear of the existing brick studio structure. This bore intersected silty sand topsoil and garden soil fill to 0.45m depth then grey-brown sand with a trace of clay that was interpreted as colluvium. Below 0.60m depth the bore intersected yellow-brown, dry to damp clay before auger refusal occurred at 0.75m depth on an ironstone band.

DCP testing undertaken adjacent to the bore identified the surficial natural soils as being dense with the clays below 0.60m depth being very stiff to 1.80m depth. The results below 1.80m are indicative of extremely low strength, extremely weathered bedrock.

Borehole 4 was drilled in the rear garden slope, in the location of the proposed swimming pool. This bore was attempted several times with refusal on an unknown object at 0.50m depth on all occasions. DCP testing was then also undertaken throughout this area with the test refusing on solid material at approximately 1.0m depth.

Second Stage of Investigation (April 2013)

Borehole 11 (BH 11) was undertaken near the front north-east corner of the existing house, through the 100mm thick driveway slab. This bore intersected light grey, sandy clay fill to 0.70m then very loose, brown and grey, clay fill with a trace of sand to 1.30m. Below the fill soft, dark brown, moist to wet silty clay with some potential marine sediments was encountered. At 2.30m depth grey, low plasticity, moist sandy clay was intersected to 2.50m before brown, moist to wet silty clay with some sand was identified. At 3.0m depth extremely to very low strength, light grey and red-brown, sandstone was intersected before coring was begun at 3.15m depth.

Extremely weathered, extremely low strength sandstone with iron rich low strength sandstone bands were returned to 3.60m before highly weathered, very low to low strength, fine grained sandstone bedrock was intersected. At 5.04m highly to moderately weathered, low strength sandstone bedrock was indentified to 6.60m overlying medium strength, moderately to slightly weathered shale bedrock. The borehole was discontinued at 6.95m depth within this shale horizon.

Borehole 12 (BH 12) was undertaken near the front south-east corner of the house, through the concrete car parking bay. This bore identified a 0.13m thick slab over clay and then sand and sandy clay fill horizons to 1.10m depth. Below 1.10m soft, dark grey, silty clay was encountered to 1.90m where medium dense, wet sand was identified. The wet condition of the sand is considered to represent the natural groundwater table at 1.90m depth. At 2.50m extremely weathered, extremely low strength sandstone was intersected to 4.45m depth where coring started.

No core recovery occurred until 5.0m depth where extremely low strength, sandstone was returned until 6.25m. The core loss between 4.50m and 5.0m depth is interpreted to have occurred through extremely low strength, extremely weathered bedrock. At 6.25m extremely low strength shale was intersected to 8.10m before a low to medium strength sandstone horizon was identified to 8.34m. Below this sandstone unit a thin very low to low strength shale horizon was encountered before low to medium strength sandstone was intersected from 8.50m to 10.15m where the borehole was discontinued.

Test Pit 1 (TP 1) was undertaken adjacent to the rear north-west corner of the house, at the rear of the laundry, at first floor level. This test pit required excavation through the concrete pathway formed at the rear of the house that also extends upslope as a concrete retaining wall. This wall and path are are interpreted to have been formed via sprayed concrete (shotcrete) over excavated soil and weathered bedrock. Below the path a 600mm deep void was identified before moist clayey sand to sandy clay fill was intersected to 1.30m depth. Below 1.30m sand fill with an ironstone gravel band at 1.90m was intersected to 3.0m where sandy gravel fill was encountered.

Auger refusal occurred within the fill at 3.65m depth, with the fill below 3.50m being wet. This fill was interpreted as backfill with the existing house wall acting as retaining wall. The void below the path is considered to be the result of compaction/settlement of the fill soils after spraying of the concrete pathway.

As the foundation conditions in this portion of the house were still unresolved a borehole (BH 13) was undertaken through the storeroom in the north-west corner of the ground floor level of the house. This borehole intersected a 60mm thick reinforced concrete slab over sand and gravel fill to 0.32m depth. Below the fill extremely weathered, extremely low strength sandstone was intersected before auger and steel bar refusal occurred at 0.46m depth on red, iron rich medium strength sandstone interpreted as bedrock.

Test Pit 2 (TP 2) was undertaken through the concrete pathway at the rear south-west corner of the house. This pit identified wet yellow brown, sand with gravel fill to 0.38m then silty to slightly clayey sand fill to 0.45m. Below this material moist sand fill with bricks was intersected to 1.15m depth. At 1.10m the top of a concrete footing was identified below the existing house wall. At 1.15m blue metal gravel fill was intersected to 1.25m before grey and orange-brown, wet sandy clay was encountered. At 1.40m extremely weathered, extremely low strength sandstone was intersected until hand auger refusal occurred at 1.60m depth on very low to low strength sandstone interpreted as bedrock. The concrete footing was identified as being 300mm thick (1.10m to 1.40m depth) and founded on the extremely low strength bedrock.

4. COMMENTS:

4.1. Geotechnical Assessment:

There were no signs of existing or previous, deep seated or large scale landslip instability identified within the site or adjacent properties. The existing retaining walls around the northern and western edges of the house are in good condition. The timber log retaining wall along the southern boundary is rotated and will continue to deteriorate until collapse. This wall only supports a gently sloping garden of bamboo therefore collapse does not present a significant hazard.

The rear half of the site is underlain by minor fill overlying colluvium and then residual clay soils that appear to grade to weathered bedrock at shallow depth. These clay soils are generally over consolidated likely as a result of the trees within this area. A horizon of medium strength sandstone identified in nearby properties may also pass through this location at shallow depth.

The front south-east corner of the site appears to be founded over the natural gully base with clayey fill placed over natural silty soils of low strength. These silty clay soils are organic rich and may be of alluvial origin or related to the foreshore sediments and could have acid sulfate issues. Clayey sand to sandy clay, interpreted to be residual, is located below this material. The deep cored boreholes at the front of the site intersected extremely weathered bedrock from 2.50m to 3.0m depth which consisted of sandstone with shale bands. The bedrock was deeply weathered with extremely low strength bedrock extending up to 7.60m depth at the front south-east corner. The existing house footing along the southern wall appears to be a shallow concrete strip footing founded within the highly variable clay fill and is overlying the low strength silty soil.

It is understood that the proposed works involve demolition of the existing house and construction of a new three storey house with swimming pool. Most of the house will be constructed at existing ground levels however some excavation will be required for the new swimming pool and for the western end of the proposed first floor. It is expected that the excavation works will extend to approximately 2.50m depth.

The recommendations and conclusions in this report are based on an investigation utilising only surface observations and a limited number of test boreholes. This provides limited data from small isolated test points across the entire site; therefore some variation to the interpreted sub-surface conditions is possible, especially between test locations. The results of the investigation provide a reasonable basis for the analysis and subsequent design of the proposed works.

The site is situated within Class '5' and Class '2' Acid Sulfate Soils hazard zone under Pittwater Councils LEP2014. The soils encountered during the investigation showed no obvious signs of actual or potential acid sulfate characteristics and the works will not lower any water table that may be present below the site. The proposed excavation for the pool and upper levels will be located upslope to the west within a residual soil and weathered bedrock setting. Therefore Acid Sulfate Soils are not expected to be impacted by the proposed works. Where deeper pier footings are proposed at the front of the site then supervision of the pier drilling, on site assessment or laboratory confirmation for acid sulfate characteristics may be required with neutralization of excavated spoil where acid sulfate soils are identified.

4.2. Site Specific Risk Assessment:

Based on our site mapping we have identified the following geological/geotechnical landslip hazards which need to be considered in relation to the existing site and proposed works (see Figure: 2), these hazards are:

A. Landslip (earth/debris slide <$5m^3$) from proposed excavation
B. Landslip (earth slide <$5m^3$) due to collapse of timber retaining wall along southern boundary.

The site has been assessed in accordance with the methods of the Australian Geomechanics Society (Landslide Risk Management, AGS Subcommittee, May 2002 and March 2007) and Pittwater Council's Geotechnical Risk Management Policy for Pittwater – July 2009. The Australian Geomechanics Society Qualitative Risk Analysis Matrix is enclosed in Appendix 2 along with relevant AGS notes and figures.

The frequency of failure was interpreted from existing site conditions and the values from MacGregor et. al. (AGS 2007), due to a lack of evidence of previous instability within the site.

The risk assessment identified that Hazard A achieves a Risk to Life of <1.07×10^{-5} and a Risk to Property of 'Moderate' and therefore is within 'Tolerable' risk levels. Hazard B achieves a Risk to Life of <2.98×10^{-7} and a Risk to Property of 'Low' and therefore is within 'Acceptable' risk criteria. Provided careful design and construction practice as outlined in the recommendations of this report including engineered excavation support are implemented then all risks can be reduced to, and remain within the 'Acceptable' risk criteria for the life of the development.

4.3. Design & Construction Recommendations:

4.3.1. New Footings:

The proposed building will require some excavation below existing ground levels towards the rear which are expected to extend in to, or close to the surface, of the weathered bedrock. At the front of the site the deeper boreholes intersected very loose fill and soft clays overlying extremely weathered bedrock from 2.50m to 3.0m depth. These low density soils are not suitable for founding of the new structure and it is recommended that the entire new building be founded on at least extremely low strength bedrock. It is recommended that all footings be founded within similar strength bedrock to reduce the risk of differential settlement within the structure. This will require the use of piers where the excavation does not expose bedrock of at least very low strength or where the new development extends out of the excavation works.

The site is considered a Class ‘P’ site as per the Australian Standard for Residential Slabs and Footings AS2870 – 2011 due to the depth of fill and potential for landslip.

Under the Australian Standard Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia the site Sub-soil classification would be C_e – shallow soil site.

Footings founded on weathered, extremely low strength rock should be designed for a maximum allowable bearing capacity of 600kPa, whilst low strength bedrock is suitable for 1000kPa and medium strength rock (if encountered) may be designed for a maximum allowable bearing capacity of 2000kPa. If higher footing pressures are required then additional testing of the bedrock below footing level will be required.

All new footings must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity and the insitu nature of the founding strata. This is mandatory to allow them to be ‘certified’ at the end of the project.

4.3.2. Excavation:

It is understood that excavation of up to approximately 2.50m depth is required as part of the proposed works. The excavation will extend to within 1.0m of the northern boundary and approximately 1.50m of the southern boundary which will be within approximately 3.20m of the northern neighbouring house and 3.50m from the southern neighbouring house. The investigation results suggest the majority of the excavation will intersect fill, residual clay soils and extremely weathered bedrock with the base of the excavation potentially intersecting low strength rock with higher strength ironstone bands.

The excavation of soil and any extremely low strength, extremely weathered bedrock may be readily achieved using conventional earth moving equipment or hydraulic excavators with the assistance of ripping for the very low strength bedrock and thin ironstone bands. This method of excavation through soils and weathered bedrock will not create excessive vibrations provided it is undertaken with medium scale (<20 tonne excavator) excavation equipment in a sensible manner.

The excavation of low to medium strength sandstone bedrock as well as any high to very high strength ironstone bands will require the use of rock excavation equipment (i.e. rock hammer / breaker / saw / grinder). The selection of excavation machinery must take into account the following information: Vibration levels from rock breakers can be excessive (Peak Particle Velocities (PPV) greater than 50mm per second) and cause damage to adjacent structures, particularly if high to very high strength iron cemented sandstone bands or major south-east to north-east sub-vertical joints are encountered.

It is recommended that a small rock hammer (<350kg) and rock saw be utilised for the excavation of hard bedrock on this site. Based on the identified buffer distances this should be suitable to ensure that a vibration limit of 5mm/s PPV is maintained at the footings of adjacent house structures. It is not expected that significant excavation of bedrock greater than low strength will be required for the proposed development based on the geotechnical investigation. Should larger scale equipment be proposed then Crozier Geotechnical Consultants should be consulted prior to its use. This may invoke the need for vibration calibration testing of the equipment and site characteristics and potentially for full time vibration monitoring.

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

Upper horizons in the bedrock may be detached along bedding and joint defects whilst boulders may be located above/within the residual soils. Where these sections are impacted via rock hammering the opposite end, potentially located below adjacent building footings, will deflect more than expected creating damage to the structure. The rock sawing of the hard rock perimeter or boulders prior to rock hammering will significantly reduce the risk of this hazard.

4.3.3. Excavation Support:

Recommended maximum batter slopes for excavation through fill and natural soils/rock on this site are presented below in Table: 1. Where these batters cannot be implemented then the excavation will require temporary support until permanent retaining walls can be completed. If suitable measures are not implemented then the stability of this excavation until permanent retaining walls are completed cannot be guaranteed.

Table 1 - Safe Batter Slopes

Material	**Safe Batter Slope (H:V)**	
	Short Term/ Temporary	**Long Term/ Permanent**
Fill and natural soils	1:1	2:1
Extremely Low to Very Low strength bedrock	1:1	1.25:1
Medium strength, defect free bedrock	vertical	0.25:1

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. 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.

4.3.4. Retaining Structures:

New retaining wall systems will need to be designed in accordance with Australian Standard AS 4678-2002 Earth Retaining Structures. Lateral support to the retaining structures can be provided by temporary propping or anchoring however it is recommended that the new building slabs provide permanent restraint once completed.

Earth pressure distribution on retaining structures depends upon the type of retaining support. For retaining structures that are supported by multiple rows of anchors or props for simplicity sake a rectangular distribution can be used with lateral earth pressures estimated as 6H kPa for soil (H is the soil depth in metres) and 4H kPa for extremely low to low strength rock.

For cantilevered walls or where one row of anchors is proposed, it is recommended that the design be based on a triangular distribution with the lateral earth pressure determined as a proportion of the vertical stress as below:

$\sigma_z = Kz\gamma$ where σ_z = Horizontal pressure at depth z
K = Earth pressure coefficient
Z = Depth (m)
γ = Unit weight of soil or rock

Backfilled retaining walls within the site, away from site boundaries or existing structures, that have the potential for deflection, may utilise active earth pressure coefficients (Ka). Retaining support close to existing structures should utilise the at rest (K_0) factors to minimise the potential for movement within retaining structures and surrounding areas.

Table 2 - Retaining Structures Design Parameters

Material	Unit Weight (kN/m^3)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
			Active (Ka)	At Rest (Ko)	
Natural clay soils and filling	20	ϕ' = 25°	0.40	0.57	N/A
Extremely low strength rock	22	ϕ' = 38°	0.24	0.38	N/A
Very low strength rock	22	ϕ' = 38°	0.22	0.36	4.60
Low strength jointed rock	24	ϕ' = 40°	0.21	0.34	4.80

* Ultimate design values

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

4.3.5. Acid Sulfate Soils

The site is situated within Class 5 and Class 2 Acid Sulfate Soils hazard zone under Pittwater Councils LEP 2014. The soils encountered during the investigation showed no obvious signs of actual or potential acid sulfate characteristics and the works will not lower any water table that may be present below the site. The proposed excavation for the pool and upper levels will be located upslope to the west within a residual soil and weathered bedrock setting. Therefore Acid Sulfate Soils are not expected to be impacted by the proposed works. Where deeper pier footings are proposed at the front of the site then supervision of the pier drilling, on site assessment or laboratory confirmation of the excavated material for acid sulfate characteristics may be required with neutralization of excavated spoil where acid sulfate soils are identified.

4.3.6. Drainage and Hydrogeology:

The site is situated within moderate to steeply east sloping topography. A groundwater table was not identified within the boreholes and is not expected within the depth of the proposed excavation or works. Therefore the proposed works should have minimal to no impact on local hydrogeology and encounter only minor groundwater seepage. An excavation trench should be installed at the base of excavation cuts to below floor slab levels to reduce the risk of resulting dampness issues. All new building gutters, down pipes and stormwater intercept trenches should be connected to a new stormwater system and discharged to the Council's stormwater system off site.

4.4. Conditions Relating to Design and Construction Monitoring:

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

1. Review and approve the structural design drawings, including the retaining structure design and construction methodology, for compliance with the recommendations of this report prior to construction,
2. Inspect all excavations and installed support measures along with new footings and earthworks to confirm compliance to design assumptions with respect to anchor capacity, allowable bearing pressure, basal cleanness and stability prior to the placement of steel or concrete.
3. Inspect completed works to ensure no new landslip hazards have been created by site works and that all required stabilisation and drainage measures are in place.

The client and builder should make themselves familiar with the Councils Geotechnical Policy and the requirements spelled out in this report for inspections during the construction phase. Crozier Geotechnical Consultants can <u>not</u> provide certification for the Occupation Certificate if it has not been called to site to undertake the required inspections.

4.5. Design Life of Structure:

We have interpreted the design life requirements specified within Councils Risk Management Policy to refer to structural elements designed to support the house etc, the adjacent slope, 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 soil slope erosion 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 (50 years)). In order to attain a design life of 100 years as required by the Councils Risk Management Policy, 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. It should be noted that timber log/sleeper retaining walls do not satisfy a 100 year design life.

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