Pacific Geotech
Consulting Geotechnical Engineers

GEOTECHNICAL REPORT: Proposed Residence

1 Brownell Drive

Byron Bay

Scott Didier

November, 2021

PG-1162

P: (07) 5636 4680 | **F:** (07) 5636 0286
E: info@pacgeo.com.au
A: 47 Brisbane Rd, Biggera Waters QLD 4216
Postal: PO Box 499, Paradise Point QLD 4216

www.pacgeo.com.au ABN: 62 615 248 952

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Author: Peter Elkington

5th November, 2021

Scott Didier
1 Williamsons Road
Doncaster VIC 3108
c/- dan@beacharchitects.com.au

ATTENTION: **DAN CONNOLLY**

Dear Sir,

GEOTECHNICAL INVESTIGATION – PROPOSED RESIDENCE
1 BROWNELL DRIVE, BYRON BAY

1.0 INTRODUCTION

This report contains the results of the geotechnical investigation and provides advice and recommendations relating to the following:

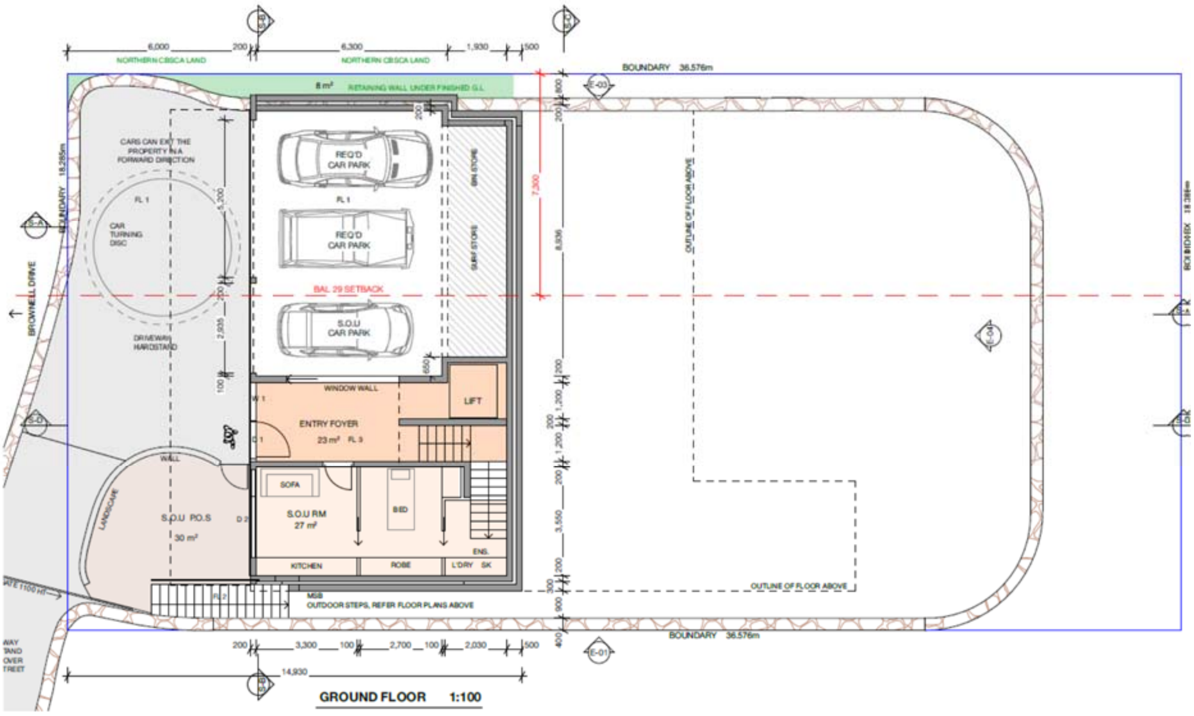
- Subsurface conditions in accordance with AS 1726
- Foundation recommendations
- Characteristic ground surface movements
- Earthworks considerations
- Retaining wall design parameters
- Construction considerations

Proposed Development

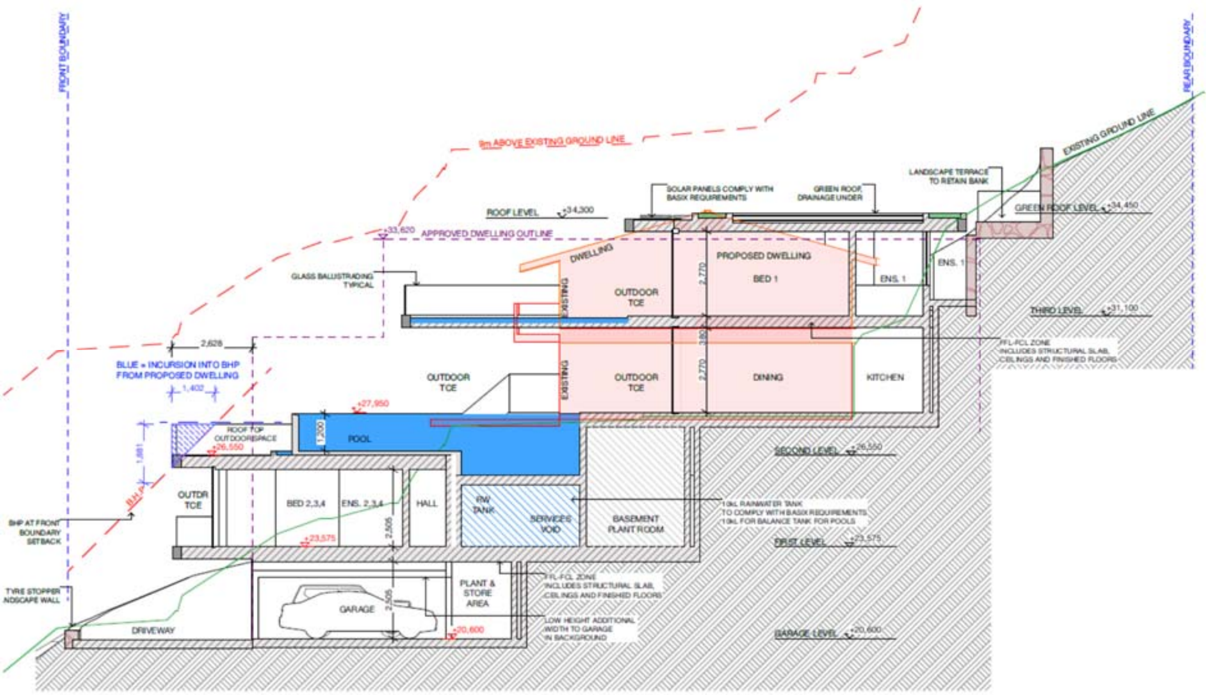
It is understood that the proposed development is to comprise the construction of a new four storey residence with swimming pool at the above site. Earthworks are envisaged to consist of significant cuts of up to 4m to create the building platform.

The proposed development is indicated below.

PROPOSED DEVELOPMENT



PROPOSED SECTION



2.0 METHODOLOGY

The geotechnical investigation comprised the drilling and sampling of 2 boreholes to depths of 3.3m and 3.6m (drill rig refusal), using an EVH1750P drilling rig mounted on a tracked mini loader and 100mm solid flight augers, and 1 hand auger hole to a depth of 0.2m (hand auger refusal), using hand held equipment. Dynamic Cone Penetrometer (DCP) testing was conducted adjacent to the boreholes.

The soil classification descriptions and field tests were carried out in general accordance with Australian Standards.

AS 1726	Geotechnical Site Investigations
AS 1289	Methods of Testing Soils for Engineering Purposes

Borehole records, Dynamic Cone Penetrometer test results and a site plan showing the test locations are appended to the report.

3.0 SITE DESCRIPTION

The site of the Proposed Residence is located at 1 Brownell Drive, Byron Bay. The site had been historically excavated to create the building platform for the existing on-site residence, with vegetation surrounding the existing structure and regrowth across the batters. The cut faces expose a highly weathered rock.

Refer following aerial and site photographs for typical site conditions.

AERIAL IMAGE

SITE PHOTOGRAPHS

4.0 GEOTECHNICAL MODEL

The subsurface profile encountered in the boreholes consisted of up to 250mm of very loose silty/gravelly sand overlying weathered siltstone to the borehole termination depths. The rock was initially highly weathered with an increase in rock strength noted at rig refusal.

Table 1 presents a summary of the encountered subsurface profile. Detailed borehole record sheets are appended to this report.

TABLE 1 SUBSURFACE PROFILE SUMMARY

BH No.	Gravelly/Silty SAND	SILTSTONE		BH TD
	Very Loose	HW	MW	
BH 01	0.0-0.25	0.25-3.55	3.55-TD	3.6[3)]
BH 02	0.0-0.2	0.2-3.2	3.2-TD	3.3[3)]
BH 03	0.0-0.15	0.15-TD	NE	0.2[4)]

Notes:
1. All depths in metres below ground level at time of investigation.
2. TD - Termination Depth.
3. Maximum TC bit; drill rig refusal.
4. Hand auger refusal.

Groundwater or subsurface seepage was not encountered in the boreholes at the time of drilling. Seepage could be expected through the surficial soils and along the soil/rock interface and possibly within the foliations in the rock, following periods of rainfall.

5.0 POTENTIAL GROUND SURFACE MOVEMENTS

The investigation and laboratory test results indicate that the development site would be classified Class S, in accordance with AS 2870-2011 'Residential Slabs and Footings' from a reactivity perspective, given the high level rock across the site.

6.0 BUILDING FOUNDATIONS

It is recommended that a high level footing system founding through the existing fill and surficial soils and into the underlying rock, be adopted for the support of the proposed building.

An allowable bearing capacity of 400kPa in the extremely weathered siltstone and would be available, subject to inspection at the time of excavation. A higher bearing capacity will be available in the rock as the weathering decreases. Subject to confirmation on-site at the time of excavation, an allowable bearing capacity of 600kPa will be available in the moderately weathered rock.

Where uncontrolled fill is encountered, footings should be deepened to penetrate the uncontrolled fill material. The use of short bored piers or mass concrete pedestals may be required where footings are located adjacent to cuts/batters, etc.

Where footings are located adjacent to excavations such as underground service trenches, basement walls, etc., it is recommended that the footings be deepened to found at least 200mm below a line drawn up at 45 degrees from the base of the trench.

It is recommended that footing inspections be undertaken by Pacific Geotech, following excavation, to confirm the specified founding strata has been reached.

7.0 RETAINING WALLS

Retaining walls should be specifically engineer designed in accordance with AS 4678-2002.

The design of flexible retaining walls could be undertaken using a triangular pressure distribution and the earth pressure parameters given in Table 2. Flexible walls are those which are free to rotate or tilt (such as cantilevered walls) and should be designed using active (Ka) earth pressure coefficient. Rigid walls are those which are restrained against rotation or tilt (i.e. single anchored/propped walls) and should be designed using the at-rest earth pressure (Ko).

Passive resistance (Kp) at the toe of the wall should be ignored in the zone where future disturbance (e.g. services trenches) could occur.

The effects of surcharge in the retained zone should be included by multiplying the vertical pressure developed by the surcharge by the appropriate lateral earth pressure coefficient. Allowance should also be made for the surcharge due to sloping crests if applicable.

TABLE 2 EARTH PRESSURE COEFFICIENTS (NON-SLOPING CREST BACKFILL)

Material	Unit Weight (kN/m^3)	Friction Angle (degrees)	Active Ka	At Rest Ko	Passive Kp
Existing Fill & Surficial Soils	18	24	0.45	0.60	2.40
Siltstone – HW - MW	20 21	35 40	0.27 0.22	0.43 0.36	4.50 4.60

Preference should be given to adopting thin soil layers and using small hand-controlled compaction equipment during backfilling against retaining walls. This is in order to limit the stress applied to the walls during construction. Should heavy compaction be required, then wall stresses will be well in excess of Ko and temporary propping should be used.

Clay backfill should not be placed dry of optimum moisture content, as this can lead to increased future swelling with changes to moisture content or inundation from water creating additional load on the back of the wall.

It is recommended that all retaining walls be drained for full height in order to minimise hydrostatic pressure build-up behind the wall. Additional guidelines on wall drainage are provided in Appendix G of AS 4689-2002.

7.1 Temporary Batter Slopes

7.1.1 Soils

It is likely that a battered, temporary, partial bulk excavation will be undertaken cross the site during bulk excavation. It is envisaged that a maximum temporary batter to heights in the order of 6m may be required. A temporary batter slope angle of 45^0 is recommended for the surficial soils.

7.1.2 Rock

Localised unsurcharged excavations in medium strength (or stronger) rock to depths of up to 4m, and located outside the zone of influence of excavation faces from the site boundaries, should be feasible if cut at a maximum temporary batter slope of 75^0, subject to inspection during construction and the possible need to use some limited extent of shotcrete (supported by rock dowels) or bottled mesh, primarily as a personal protection measure.

It will be necessary to design an appropriate temporary support system for each proposed local excavation, based on the actual conditions encountered.

7.2 Temporary Excavation Support Methods

7.2.1 Soldier Pile and Shotcrete Facing

Surface filling, soils and the low to medium strong rock will require continuous boundary support. It is considered that an appropriate system for the site comprises anchored, soldier piles (bored piers), in turn, retaining shotcrete facing between the piles.

An indicative layout of this option is indicated in Figure 1.

Soldier Pile Excavation Methodology

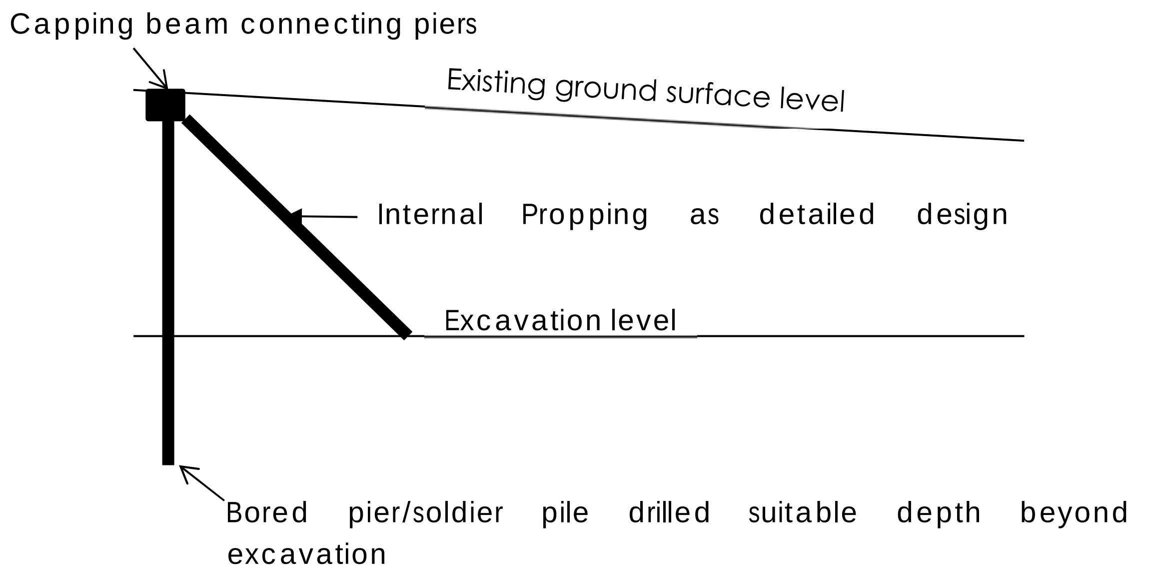


FIGURE 1

Some difficulties in achieving the penetration of the bored piers into the high strength rock may be encountered. Whist it is expected that full penetration of the piers below the excavation depths may be possible, if sufficient penetration is not able to be achieved, a panel system may be required beyond the toe of the soldier piles.

It will be necessary to fully restrain the toe of soldier piles using stressed rock anchors and shotcrete/cast insitu panels, prior to excavating beyond the required minimum toe embedment depths unless piles are supported in competent rock at a sufficient distance below adjacent excavation level.

7.2.2 Sprayed Panels and Panel System

The use of an anchored panel system beyond the soldier piles could be considered. This can typically take the form of panels approximately 2m x 2m (possibly increasing to 3m deep, subject to inspection by Pacific Geotech).

The construction is achieved by excavating a panel leaving a rock buttress either side. The panels are usually excavated on a hit-one miss-two basis. The anchors are installed in the panel face and the panel steeled and sprayed. The anchor is then stressed.

Following completion of the panel, the adjacent panel could be excavated and the process repeated around the excavation, before the excavation can continue to the next level. Overall excavation support could be provided by use of a regular grid of prestressed rock anchors. The exact location of anchors would need to be determined by inspection during excavation, however an anchor spacing of 2m to 3m is suggested for initial costing.

Hit and Miss Construction

Subject to ground conditions actually encountered, it will be necessary to adopt a 'hit and miss' excavation approach along parts of the site boundaries. On-site supervision during excavation may enable the hit and miss approach to be deleted; however this cannot be determined in advance of excavation.

7.3 Temporary Excavation Support

7.3.1 Soil Pressures

For short-term support (i.e. during the construction period), the trapezoidal lateral earth pressure distribution described below would be suitable for the preliminary design of anchors/props for fully drained walls unsurcharged by building loads; however, it is suggested that a 'better' approach to the design of temporary and permanent shoring would be to utilise detailed computer analysis program.

Suggested Design Pressures

- a linear increase in pressure from 0 kPa at the top of the vertical cut to a maximum of 6H kPa at a depth equal to 25% of the vertically cut slope height;
- uniform pressure of 6H kPa from this depth to a depth equal to 75% total vertically cut slope height; and
- linear decrease in pressure to 0 kPa at the base of the vertical cut slope.

  (where H = total vertical cut slope height)

If required, Precise Geotechnical are able to undertake a WALLAP analysis of the excavation profile.

7.4 Temporary Anchor Design

For the purpose of estimating temporary anchor bond lengths, the maximum working stresses give in Table 3 may be used for sizing. Insitu proof testing of each anchor should be undertaken to confirm the design capacity has been achieved.

TABLE 3 ROCK ANCHOR WORKING BOND STRESS

Rock Strength	Working Bond Stress MPa
Low	0.15
Medium	0.25
High	0.75

The design of the rock anchors should take into consideration the following:

- The anchors be designed as temporary

- Anchors should be checked for rock/grout failure, grout/tendon failure and tendon failure.
- The free length of the anchors should be a distance equal to a distance of 0.15 x excavation height, behind a line drawn up at 60^0 from the base of the excavation.
- The anchor bond strength will be dependent upon numerous factors including borehole hole size, material properties, required capacity, etc. Confirmation during pull out tests during the excavation operation should be undertaken.

It is considered essential that anchor lift-off tests be conducted on selected anchors to confirm the creep is not occurring.

7.5 Groundwater Control

Temporary groundwater control will be required to enable satisfactory completion of construction and to enable economic sizing of temporary support works.

It will be essential to ensure that all services connecting into the site that are to be terminated are properly sealed off (including trench backfill) to prevent them becoming water conduits to the site and causing water pressure surcharging of excavation support works.

If a deep, temporarily battered excavation is to be adopted (refer Section 4.2) it may be necessary (subject to the results of detailed stability assessment) to undertake temporary dewatering of soils using a pumped well point system.

7.5.1 Wall Drainage

In order to control the groundwater level behind the walls, it will be necessary to install drainage behind shotcrete facing, typically comprising full height strips of ‘Core Drain’ (or equivalent) of 20mm minimum thickness, spaced horizontally at not more than approximately 1.8m centres.

7.5.2 Under Slab Drainage

Groundwater seepage will occur through the floor of the excavation that could adversely affect floor slab performance and building amenity, unless a fully tanked basement is adopted. In order to control seepage flows for a ‘non-tanked’ floor slab, the slab should be cast over a free draining layer (incorporating agricultural drains) graded to sumps so that groundwater is removed and pressure does not build-up under the slab; ‘normal’ slab joint spacing could be adopted with this option. The drainage system would need to be properly analysed and designed, however, for initial costing a 150mm thick layer of single sized aggregate should be allowed for, together with 100mm diameter geotextile wrapped agricultural drains laid one way at 6m to 8m centres and connected to sumps. It may be necessary to place the drainage layer over a geotextile to prevent 'contamination' by the underlying subgrade.

If a non-tanked basement is to be adopted, assessment of the seepage inflow and groundwater levels should be made to allow the drainage/pumping system to be appropriately sized. Alternatively, a tanked basement, held down with an appropriate ground anchor system should be considered. It is expected that hold down would be achievable through the use of rock bolts or anchors.

7.6 Construction Monitoring

7.6.1 Rock Strength and Structure

It is suggested that all excavation and support works at the site be conducted under the supervision of Precise Geotechnical so that ground conditions as exposed can be compared to the ground conditions assumed for excavation support analysis, etc., enabling design modifications to be (if required).

7.6.2 Vibration

Vibration will be caused by excavation (and demolition) work at the site. Vibration restrictions should be set with a realistic appreciation for the normal operational environment of the site and the type of plant/methods to be used for excavation.

Unless very heavy and sustained hydraulic rock breaker excavation is undertaken in close proximity to site boundaries, it is considered unlikely that excavation induced vibration will pose any significant threat to properly constructed, adjacent buildings. However, a dilapidation survey of adjacent buildings (and services) is strongly recommended prior to commencement of site work.

7.6.3 Boundary Movement

Boundary movements will occur as a consequence of the excavation process; such movements are not possible to prevent or to predict with great accuracy, however, estimates can be calculated as part of the detailed design analysis. The potential movements are not expected to be excessive (ie. <5mm) with appropriate design. Vertical movement could also result. It is considered essential than an accurate survey monitoring program of the excavation boundary be put in place for the duration of the excavation works. The monitoring should be to not less than 1mm (horizontal and vertical) accuracy, so that any movement trends can be readily identified.

8.0 SLOPE STABILITY ASSESSMENT

There is no evidence of existing or recent past slope instability involving small scale or large scale movements of significant quantities of soil or rock in a short duration event such as a slip or landslide within the Lot. There is, however evidence of minor instability within the excavation face around the site. Uncontrolled fill material is also evident at the surface within the Lot.

Several retaining walls are evident around the site ranging up to approximately 1m in height with some walls showing signs of degradation and damage. As part of the proposed new development, these walls will be removed and the existing fill material either removed or adequately retained.

The likelihood of slope movement in the form of deep seated failures through the rockmass is considered to be low, due to the relatively shallow depths to bedrock and the estimated friction angle of the rock being significantly higher than the angle of the slope.

Due to the presence of uncontrolled fill, damaged retaining walls and sloping terrain, precautions must be taken during development in relation to earthworks, foundations and drainage to reduce the potential for instability and erosion. This includes implementing and maintaining properly designed and constructed drainage structures upslope and around the proposed development areas and extending the foundations into the bedrock. Earthworks to remove or retain the existing fill would also need to be carried out as part of the proposed new development.

8.1 <u>Landslide Susceptibility Rating)</u>

The slope stability assessment methodology adopted for this assessment is in accordance with the City of Gold Coast (Council) "Geotechnical Stability Assessment Guidelines," March 2016 and the AGS. These guidelines have been applied to the Lot in its existing condition.

On this basis, it can be expected that some areas of the Lot may contain higher Levels of Risk to Property than the overall assigned rating for the Lot. Depths to bedrock and soil profiles have been obtained from borehole drilling at the site as well as observations of outcropping rock at the site.

The Lot has been assigned a Landslide Susceptibility Rating based on:-

- Ground surface slope angle and shape
- Geology
- Typical depth of soil cover
- Presence of uncontrolled fill, erosion features, drainage features, slips or surface irregularities
- Seepage and drainage conditions as assessed during the walkover survey carried out for this study

The results of the slope stability assessment are presented in Table 2 below.

TABLE 2 – LANDSLIDE SUSCEPTIBILITY RATING

Site	Relative Frequency	Assessed Landslide Susceptibility Rating	Required Works to Maintain Susceptibility Rating to Low or Better
1 Brownell Drive	0.4953	Low	Sections 7.0 to 12.0 as reported herein

From results of the desk study and walkover survey, we assess the Lot as having a Landslide Susceptibility Rating of "Low" in its existing condition. However, due to the presence of sloping terrain, uncontrolled fill and degraded retaining walls, precautions must be followed to maintain a Landslide Susceptibility Rating to a level of "Low" or better.

If all foundations for the proposed development are founded in the stable bedrock and the recommendations outlined in Sections 7.0 to 12.0 below are followed, the Lot is expected to maintain a "Low" or better Landslide Susceptibility Rating for the long term.

9.0 GEOTECHNICAL ISSUES OR CONSTRAINTS FOR DEVELOPMENT

The Lot in its existing condition is considered to be suitable for residential development as the site contains no major geotechnical issues or constraints which would impact on the proposed residential development with appropriate construction techniques. There are however several minor geotechnical issues which must be addressed prior to or as part of the development or earthworks on the site which includes:

- Sloping terrain.
- The presence of uncontrolled fill ranging up to 1.5m.
- Degraded and damaged retaining walls ranging up to approximately 1.0m.
- Drainage issues.

Recommendations to overcome or manage these geotechnical issues and to maintain the Landslide Susceptibility Rating to a level of "Low" or better are described in the following Sections.

10.0 RECOMMENDATIONS TO MAINTAIN LANDSLIDE SUSCEPTIBILITY TO LOW

The Landslide Susceptibility Rating for the Lot is assessed to be “Low” for the long term with appropriate construction methodology. However, due to the presence of several minor development constraints listed in Section 6.0 above, risk maintenance and reduction strategies are required to maintain the Landslide Susceptibility Rating to level of “Low” or better.

- All uncontrolled fill material should be removed or alternatively, the uncontrolled fill can be retained by structurally engineered and designed retaining walls.
- All foundations must extend into the weathered rock. All structures must be adequately designed to restrain any upper level soil creep and withstand lateral forces from the movement of soil.
- No additional filling should be carried out at the site unless the proposed design is approved by Pacific Geotech as satisfactory. Minor filling may be carried out behind structurally designed retaining walls subject to the detailed design constraints of the site and structure.
- In areas where any filling is to be undertaken, the stripped natural surface should be benched to key the new fill into the sloping surface.
- All cuts and any new or existing structural filling at the site at the site should be battered to the safe batter angles described in this report or suitably retained by structurally engineered and designed retaining walls, as proposed in the engineering drawings.
- The recommendations described in Sections 7.0 to 12.0 of this report must be followed which includes recommendations for foundations, drainage, site preparation prior to filling, filling, safe cut and fill batters slope angles and general recommendations for residential development.

11.0 DRAINAGE RECOMMENDATIONS

Properly designed, constructed and maintained surface and subsurface drainage is critical to the long term performance of the site in terms of slope stability, soil creep and erosion. The ingress of surface water into cleared sloping terrain or fill platforms is assessed to be a potential trigger for instability at the site. Therefore, minimising the ingress of surface and subsurface water into the sloping terrain or fill platforms is of significant importance as saturation of these materials can result in creep movements and slips.

The following recommendations with regards to drainage at the site should also be implemented:-

- Upslope surficial and subsurface water flows should be intercepted and directed away from the proposed development area, future batters and retaining walls limiting the ingress of water into the sloping terrain and cut/fill platform.

- All building pad runoff and roof water should be discharged into stormwater systems via a system of pipe conduits clear of the site to minimise water infiltration into the slopes and fill platforms. Alternatively, roof water can be discharged into roof water tanks for storage.
- Storage tanks must be located to the side or downslope of the dwelling.

12.0 SITE MANAGEMENT

To maintain the long term performance of the structure, good management of the soil conditions and the development is vital throughout the life of the development.

The following are some specific comments with respect to site management.

- The ground surface around the perimeter of the building should slope away from the structure and fall to the stormwater system. Water should not be allowed to pond adjacent to the buildings.
- Founding soils should not be allowed to become saturated.
- Service trenches under the building should be kept to a minimum. Saturation of the on-site material will result in an increase in potential ground surface movements.
- Footings should be poured immediately after excavation. If footings cannot be poured on the same day as excavation, a blinding layer of 50mm thickness is recommended.

13.0 LIMITATIONS

We have prepared this report for the Proposed Residence at 1 Brownell Drive, Byron Bay. The report is provided for the exclusive use of Scott Didier, for this project only and for the purposes outlined in the report. It should not be used by, or relied upon, for other projects on the same or different sites or by a third party. In preparing this report, we have relied upon information provided by the client or their agents.

The results are indicative of the subsurface conditions on site only at the specific testing locations. Subsurface conditions can change between test locations and the design and construction should take the spacing of the testing and testing methods adopted and the potential for variation between the test locations.

It is recommended that Pacific Geotech be engaged to provide advice and ensure the development is undertaken in accordance with the assumptions made in writing this report.

This is not to reduce the level of responsibility accepted by Pacific Geotech, but rather to ensure that the parties who may rely on the information contained in this report are aware of the responsibilities they assume in doing so.

P. ELKINGTON (RPEQ 7226)

For and on behalf of
PACIFIC GEOTECH PTY LTD

Attached	Notes Relating to this Report
	Borehole Record Sheets
	Site Plan

Notes Relating to this Report

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis.

Every care has been taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical conditions and contains recommendations or suggestions for design and construction. However, unexpected variations in ground conditions will occur. The potential for this will depend partly on testing, spacing and sampling frequency.

If variations are identified, Pacific Geotech would be pleased to assist with additional investigations or advice to resolve the matter.

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Borehole and Test Pit Logs

The borehole and test pit logs presented in this report 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 or excavation.

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

Description and Classification Methods

The description and classification of soils and rocks used in this report are based on AS 1726.

Soil types are described according to the predominating particle size and behaviour as set out in the attached Unified Soil Classification Table qualified by the percent of other particles present (e.g. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silty	0.002 to 0.06mm
Sand	0.06 to 2mm
Gravel	2 to 60mm

Non-cohesive soils are classified on the basis of relative density which can be correlated from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very Loose	less than 4
Loose	4 – 10
Medium Dense	10 – 30
Dense	30 – 50
Very Dense	greater than 50

Cohesive soils are classified on the basis of strength (consistency) and can be quantified by the Pocket Penetrometer test, Vane Shear test, laboratory testing or engineering examination. The strength terms are defined as follows:

Classification	Unconfined Compressive Strength kPa
Very Soft	less than 25
Soft	25 - 50
Firm	50 – 100
Stiff	100 – 200
Very Stiff	200 - 400
Hard	greater than 400
Friable	strength not attainable – soil crumbles

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc.

Sampling

Sampling is undertaken during the fieldwork to allow examination of the soil or rock and to allow laboratory testing to be undertaken.

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content and minor constituents. Bulk samples are similar but of greater volume required for some test procedures such as CBR testing.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and collecting a sample of the soil contained 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.

Details of the type and method of sampling used are given on the attached logs.

Investigation Methods

Test Pits: These are typically undertaken with a backhoe or a tracked excavator, allowing examination of the insitu soils. Limitations of test pits are the problems associated with collapse of the pits, disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of typical diameter of between 50mm to 75mm advance manually operated equipment. Premature refusal of the hand augers can occur on a variety of materials such as fill, gravel, hard clays and collapse of the borehole (typically in non-cohesive soil).

Continuous Spiral flight Augers: The borehole is advanced using 65mm to 100mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and insitu testing. Augers of up to 300mm in diameter are used to recover larger volumes of sample. Samples are returned to the surface by the flights or may be collected after withdrawal of the auger flights. Samples can be disturbed and layers may become mixed. Augering below the groundwater table can be less reliable than augering above the water table.

A Tungsten Carbide (TC) bit for auger drilling into rock can be used to indicate rock strength and continuity by variation in drilling resistance and from examination of recovered rock fragments but provides only an indication of the likely rock strength. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is advanced by a bit attached to the end of a hollow rod string, with water being pumped down the drill rods and returned up the annulus of the borehole, carrying the drill cuttings. Changes in stratification can be determined from the return, together with information from “feel” and rate of penetration.

The borehole can be stabilised through the use of drilling mud as a circulating fluid. The term ‘mud’ encompasses a range of products ranging from bentonite to polymers such as Revert or Biogel.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. This technique provides a reliable (but relatively expensive) method of investigation. In rocks, an NMLC triple tube core barrel is used, which gives a core of about 50mm diameter. The length of core recovered is compared to the length drilled and any length not recovered is shown as CORE LOSS.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils as a means of indicating density or strength and also of obtaining a disturbed sample. The test procedure is described in Australian Standard 1289, “Methods of Testing Soils for Engineering Purposed”, Test 6.3.1.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63kg hammer, with a free fall of 760mm. The sample is driven in three successive 150mm increments and the ‘N’ value is taken as the number of blows for the last 300mm. In dense soils, 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 blows, as
  N = 13
  4, 6, 7
- In a 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, as
  N > 30
  15, 30/40mm

Cone Penetrometer Testing (CPT): Cone Penetrometer Testing with or without pore pressure measurement (CPTu) is carried out using a Cone Penetrometer in general accordance with AS 1289 6.5.1, 1999.

In the tests, a 36mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the fractional resistance on a separate 135mm long sleeve, immediately behind the cone. Pore Pressure is recovered through a pore ring located either within, or more usually immediately behind the cone/tip.

As penetration occurs (at a rate of approximately 20mm per second) and data is recorded every 20mm of penetration, the results are presented graphically.

The information provided on the plot comprises:

- Cone resistance – expressed in mPa
- Sleeve friction – expressed in kPa
- Friction ratio – the ratio of sleeve friction to cone resistance expressed as a percentage.
- Pore pressure in kPa
- Tilt of probe (in degrees).

The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and rising to 2% to as high as 8%, and higher in organic soils. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes, etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive.

Dynamic Cone Penetrometers:

Dynamic Cone Penetrometer (DCP) tests are carried out by driving a 16mm diameter rod into the ground with a 9kg sliding hammer dropping 510mm and counting the blows for successive 100mm increments of penetration.

Logs

The borehole or test pit logs are an interpretation of the subsurface conditions, and their reliability will depend to some extent on the frequency of sampling and the method of drilling or excavation.

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of the boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than “straight line” variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

Groundwater

Where groundwater levels are measured in boreholes, there are several potential problems:

- Although groundwater may be present, in low permeability soils it 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 and may not be the same at the time of construction.

- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be flushed from the hole and drilling mud must be washed out of the hole or 'reverted' chemically if water observations are to be made.

More reliable measurements can be made by installing standpipes from which ongoing monitoring can be undertaken.

Fill

The present of fill materials can often be determined only by the inclusion of foreign objects (e.g. bricks, steel ,etc.) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult to reliably determine the extent of the fill.

Laboratory Testing

Laboratory testing is carried out in general accordance with Australian Standard 1289 'Methods of Testing Soil for Engineering Purposes'.

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.

Review of Design

Where major civil or structural developments are proposed or where only a limited investigation has been completed or where the geotechnical conditions/constraints are quite complex, it is prudent to have a design review.

Site Inspection

Pacific Geotech would be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related:

Requirements could range from:

i. a site visit to confirm that conditions exposed are no worse than those interpreted, to
ii. a visit to assist the contractor or other site personnel in identifying various soil/rock types such as appropriate footing or pier founding depths, or
iii. full time engineering present on site.

Pacific Geotech
Consulting Geotechnical Engineers

Borehole No.
BH 01

Project No.: PG-1162

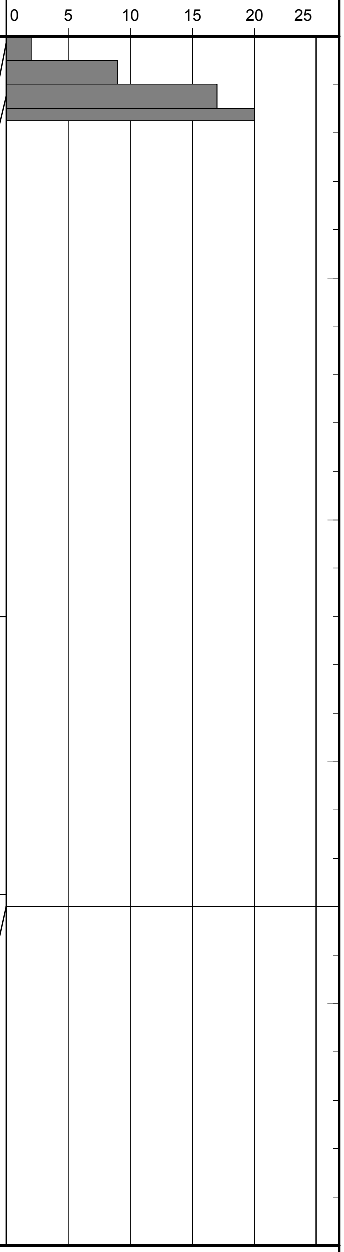
Client:	S Didier	Commenced:	08/01/2018
Project Name:	Proposed Residence	Logged By:	TTE
Hole Location:	1 Brownell Drive, Byron Bay	Checked By:	
Hole Position:			

Drill Model and Mounting:	EVH1750P	RL Surface:	No survey		
Hole Diameter:		Datum:	AHD	Operator:	TTE

Method	Casing	Water	Samples Tests Remarks	Recovery	RL (m)	Depth (m)	Classification Symbol	Material Description Fraction, Colour, Structure, Bedding, Plasticity, Sensitivity, Additional
AD/T						0.03	SM	NATURAL Silty SAND (SM) Very loose, fine to medium grained, dark grey, low plasticity fines, organics throughout, moist. (top soil)
						0.25	SM	Silty SAND (SM) Very loose, fine to medium grained, dark grey, low plasticity fines, moist.
			D 1.50-2.00 m			2.40		SILTSTONE (HW) Highly weathered, very low strength, light grey and yellow brown, moist.
						3.55		SILTSTONE (HW) Highly weathered, low strength, light grey and dark grey, moist.
						3.60		SILTSTONE (MW) Moderately weathered, low to medium strength, light grey and dark grey, moist.
								Hole Terminated at 3.60 m

DCP

DCP TEST (AS 1289.6.3.2-1997) Blows per 100 mm



Method
AS - Auger
RR - Rock Roller
WB - Washbore

Support
C - Casing

Water
Level (Date)
Inflow

Samples and Tests
U - Undisturbed Sample
D - Disturbed Sample
SPT - Standard Penetration Test
B - Bulk Sample

Classification Symbols and Soil Descriptions
Based on Unified Soil Classification System

Remarks
1. Groundwater not encountered.
2. DCP refusal met at 0.35m.
3. Maximum 'TC' bit refusal met at 3.6m.

Pacific Geotech
Consulting Geotechnical Engineers

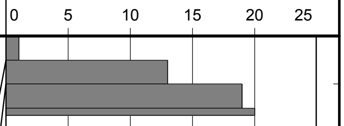
Borehole No.
BH 02

Project No.:	PG-1162
Client:	S Didier
Project Name:	Proposed Residence
Hole Location:	1 Brownell Drive, Byron Bay
Hole Position:	
Commenced:	08/01/2018
Logged By:	TTE
Checked By:	
Drill Model and Mounting:	EVH1750P
Hole Diameter:	
RL Surface:	No survey
Datum:	AHD
Operator:	TTE

Method	Casing	Water	Samples Tests Remarks	Recovery	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description Fraction, Colour, Structure, Bedding, Plasticity, Sensitivity, Additional
AD/T						0.10		SP	NATURAL Gravelly SAND (SP) Very loose, fine to medium grained, brown, fine to medium grained gravel, organics throughout, moist. (top soil)
						0.20		SP	Gravelly SAND (SP) Very loose, fine to medium grained, brown, fine to medium grained gravel, moist.
									SILTSTONE (HW) Highly weathered, very low strength, light grey and yellow brown, moist.
			D 2.50-2.80 m			2.80			SILTSTONE (HW) Highly weathered, low strength, light grey and yellow brown, moist.
						3.20			SILTSTONE (MW) Moderately weathered, low to medium strength, very light grey, moist.
						3.30			Hole Terminated at 3.30 m

DCP

DCP TEST
(AS 1289.6.3.2-1997)
Blows per 100 mm



Method
AS - Auger
RR - Rock Roller
WB - Washbore

Support
C - Casing

Water
Level (Date)
Inflow

Samples and Tests
U - Undisturbed Sample
D - Disturbed Sample
SPT - Standard Penetration Test
B - Bulk Sample

Classification Symbols and Soil Descriptions
Based on Unified Soil Classification System

Remarks
1. Groundwater not encountered.
2. DCP refusal met at 0.33m.
3. Maximum 'TC' bit refusal met at 3.3m.

Pacific Geotech
Consulting Geotechnical Engineers

Borehole No.
BH 03
Page 1 of 1

Project No.: PG-1162

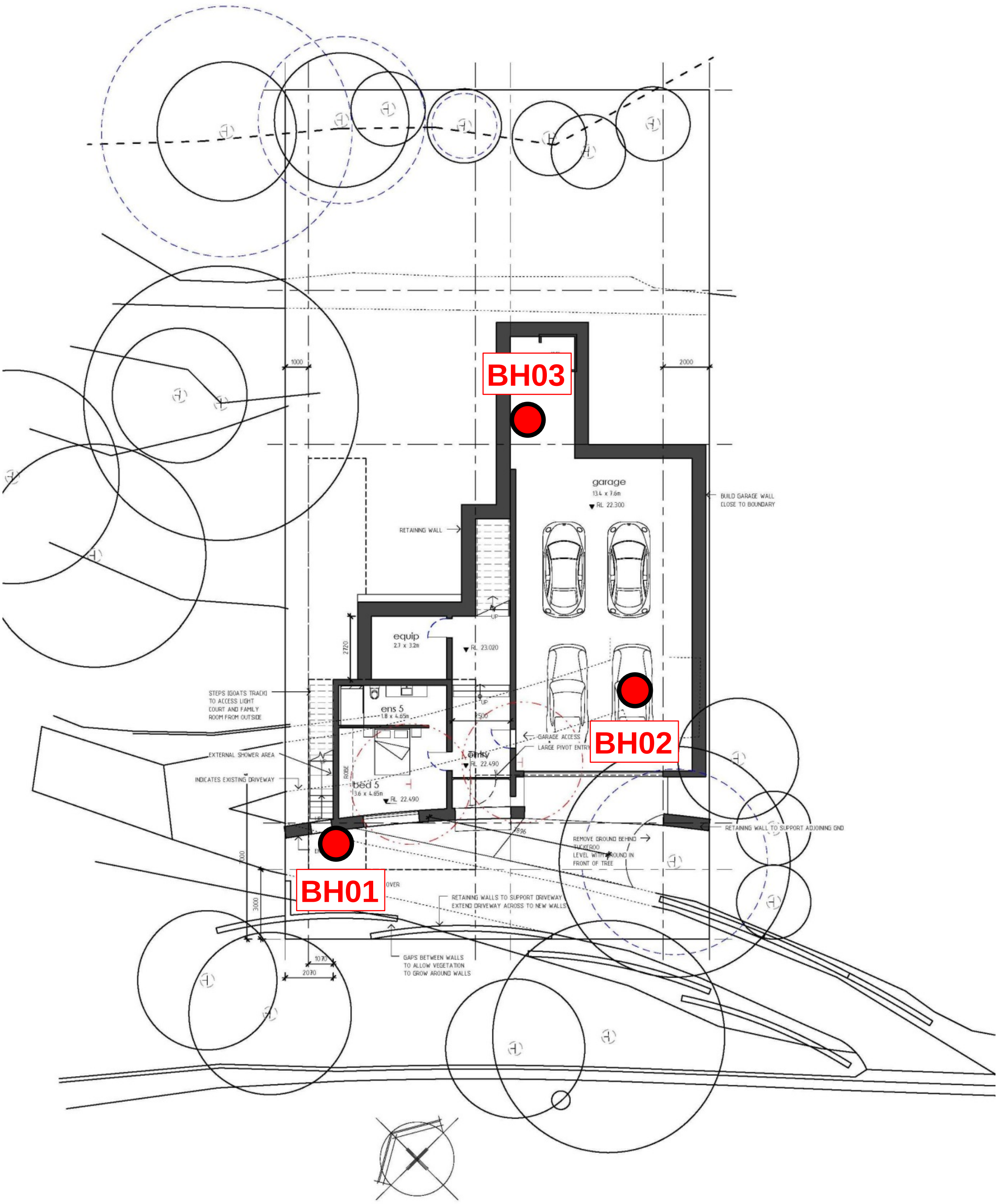
Client:	S Didier	Commenced:	08/01/2018
Project Name:	Proposed Residence	Logged By:	TTE
Hole Location:	1 Brownell Drive, Byron Bay	Checked By:	
Hole Position:			

Drill Model and Mounting:	Hand Auger	RL Surface:	No survey		
Hole Diameter:		Datum:	AHD	Operator:	TTE

Drilling Information							***Soil Description***			***DCP***
Method	Casing	Water	Samples Tests Remarks	Recovery	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description Fraction, Colour, Structure, Bedding, Plasticity, Sensitivity, Additional	DCP TEST (AS 1289.6.3.2-1997) Blows per 100 mm (0 5 10 15 20 25)
HA						0.15		SP	NATURAL Gravelly SAND (SP) Loose, fine to medium grained, dark grey, fine to medium grained gravel, organics throughout, moist. (top soil)	4
						0.20			SILTSTONE (HW) Highly weathered, low strength, light grey and yellow brown, moist.	16
									Hole Terminated at 0.20 m	

Depth scale: 1, 2, 3, 4

Method	***Water***	***Samples and Tests***	***Remarks***
AS - Auger RR - Rock Roller WB - Washbore	Level (Date) Inflow	U - Undisturbed Sample D - Disturbed Sample SPT - Standard Penetration Test B - Bulk Sample	1. Groundwater not encountered. 2. DCP refusal met at 0.21m. 3. Hand Auger refusal met at 0.2m.
Support C - Casing		***Classification Symbols and Soil Descriptions*** Based on Unified Soil Classification System	



Pacific Geotech
Consulting Geotechnical Engineers

Drawn HI	Project:	Proposed Residence	Drawing No. PG-1162-01	A4
Date Jan 2018	Location:	1 Brownell Drive, Byron Bay		
Checked	Client:	S Didier		