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consulting
geotechnical +
environmental
services

161059/1-B ku:PS
29 November 2016

Sunshine Coast Regional Council

Locked Bag 72

SUNSHINE COAST MAIL CENTRE QLD 4560

Attention: Mr Tim Mumford

RE: PROPOSED INTEGRATED TOURIST FACILITY - 24 & 25 BOX STREET BUDERIM

PEER REVIEW OF REPORT BY CORE CONSULTANTS PTY LTD

1. INTRODUCTION

At the request of Sunshine Coast Regional Council, Shaw Urquhart Pty Ltd has carried out a peer review of a geotechnical report prepared by Core Consultants Pty Ltd (Core), report number J000043-005-R-Rev2 (dated October 2016), regarding a proposed development at the above address.

It is understood that a development application has been submitted for an Integrated Tourist Facility to be constructed at the site. From the available drawings, it is understood that the development will cover much of the northern and central part of the site and will include the following elements:

- One hundred and twenty two accommodation units.
- Conference and function centre.
- Restaurants, bars and entertainment areas.
- Outdoor recreations areas.
- Entertainment area including bars and restaurants.
- Extensive landscaping.
- Administration and maintenance facilities.
- Parking for private and commercial vehicles.

A Layout Plan by Covey Associates Pty Ltd is attached for reference.

The preliminary drawings prepared by Williamson Architects and attached to the Core report indicate that the existing residence and guest house on the site will be demolished and that the proposed new development will be constructed largely on cut platforms benched into the hillside. The existing dam in the eastern area of the site will be retained.

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A Bulk Earthworks Plan by Covey Associates Pty Ltd shows proposed cuts of up to 6m to 7m and filling of up to 2m to 3m above the existing ground levels. A copy of the plan is attached for reference.

2. KEY GEOTECHNICAL ISSUES

In order to view the existing conditions, an Engineering Geologist from Shaw Urquhart Pty Ltd visited the site on 21 November 2016.

From a brief walk-over of the area, the key geotechnical hazards of relevance to the development application are considered to be as follows:

- The stability of the moderate to steep, boulder-covered slope along the northern boundary of the site and up-slope of the site on the adjacent property.
- The stability of temporary and permanent cut and fill batters for the proposed development.
- The stability of the existing farm dam and embankment. It is understood that this dam is to be retained as part of the future development.
- The stability of the natural steep to very steep slopes in the central and southern area of the site.

3. REPORT REVIEW

3.1 Regional and Site Specific Geology

The Core report refers to Figure 2 which is described as presenting an interpreted plan of the geological conditions on the site. Unfortunately this figure is missing from the report and it is therefore not possible to comment on its content.

From the text of the report the geology appears to be as follows:

- The northern and central parts of the site are underlain by a basalt cap and associated basaltic colluvium.
- The southern part of the site is underlain by residual soils and weathered rock of the Landsborough Sandstone Formation.
- The central part of the site is the contact areas between the basalt cap and the Landsborough Sandstone and includes areas of groundwater seepage (test pits TP5, TP6, TP8 and TP11) and fissured Tertiary Sediments. The presence of these fissured Tertiary Sediments is of particular concern as they have been the source of most of the instability which has occurred in the Buderim area.

3.2 General Report

The Core report can be broadly divided into five main parts.

- Sections 1 and 2 provide a description of the site conditions and a summary of the work carried out, including discussion of the nature of the development, scope of work and the methodologies used.
- Section 3 describes the results of surface and subsurface investigations (topography, geology and groundwater).

- Section 4 discusses the stability of cut and fill batters and the existing natural slopes on the site and includes numerical analysis of slope stability.
- Section 5 discusses design and construction issues relating to excavations, earthworks, footing design and batters.
- Sections 6 and 7 present a summary of the report findings and conclusions.

The appendices present borehole and test pit log sheets, interpreted geological cross-sections, slope stability analyses and architectural drawings of the proposed development.

The report provides a broad overview of the site and identifies areas where, for the existing site conditions, there is potential for slope instability. These are discussed further in Section 3.3 below.

It is our opinion that the greatest potential for major instability will occur during the construction phase of the proposed development and particularly where excavations are carried out in close proximity to the fissured Tertiary sediments.

3.3 Stability of Natural and Man-Made Slopes

Extensive excavations and earthworks are proposed for the site which will significantly alter the existing topography. Notwithstanding this, there are still some areas of the site where consideration of the stability of the existing natural slopes and existing cut and fill batters has relevance to the project as follows:

- The natural slopes immediately up-hill of the northern boundary of the site are locally steep and boulder-covered. Some boulders (or possible outcrops) were observed to be several metres in size. An assessment needs to be made of the stability of this area and the potential for boulders to be mobilised and roll onto the subject property.
- Section 3.2 of the report makes mention of a “basalt cap” and infers that this material is synonymous with basaltic colluvium. This term usually applies to a layer of in-situ weathered basalt bedrock at or near the crest of the escarpment. Such a layer was not observed on the site during our visit but may be present on the adjacent property to the north of the site. This requires clarification.
- Sections 3.2, 3.2.2 and 3.2.3 of the report make specific reference to the presence of Tertiary sediments (fissured clays), specifically between depths of 6m (BH8) and 9m (BH9) below ground level in the Bridge and Dam area and 1.5m (BH4) and 9m (BH2) below ground level in the Middle Area. It is noted that none of the test pit logs or borehole logs mention fissured clays. This requires explanation as the presence of fissured clays has a significant influence on the stability of the natural and proposed man-made slopes.
- The existing farm dam in the north eastern area of the site has very steep cut slopes of around 35° to locally 60° on the up-hill side. The cut batter is up to approximately 5m high. On the down-slope side, the fill embankment for the dam wall is up to around 6m to 7m high and has been formed at around 35° and locally steeper. Erosion of the surface of the soil embankment, likely due to over-topping, has resulted in localised erosion and subvertical scarring on the face of the batter.

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It is considered that, in its current condition, the dam embankment has a high likelihood of further erosion and possible failure of the dam embankment. The Core report notes that "*mitigation measures are required to reduce the landside risk to "Low"*.

The report recommends that a retaining structure (e.g. gabion wall) be constructed to support the existing dam embankment. The construction of this wall has the potential to cause instability if not carried out correctly (see 3.4 below).

There does not seem to have been any assessment of the capacity of the dam and/or the potential for the dam to overtop during storm periods or periods of long-term consistent rainfall. It is not clear how overflow from the dam will be managed given the proximity of Building B on the down-slope side.

- Drawing No. Section B indicates that a building housing a spa and gym is proposed immediately downslope of the dam embankment, with vertical cuts of around 3m. This excavation, if not properly managed, will considerably increase the likelihood of failure of the dam embankment (see 3.4 below).
- There is an area of very steep slopes (30° to 35°) with groundwater seepage in the central/southern area of the site (area of test pits TP11 and TP12). One shallow landslide and one possible shallow landslide were observed in this area. Test pits TP11 and TP12 encountered basalt colluvium overlying residual clay soils and weathered sandstone and it is considered that this area has a high likelihood of further ground movement. It is recommended that further comment be provided on the stability and treatment of the very steep slopes and groundwater seepage in this area.
- Appendix D of the report presents the results of computer slope stability analyses using various batter geometries and subsurface conditions. All of the analyses presented in the report use circular failure plane geometries. It is considered that, where narrow layers and distinct interfaces are present, such as between fill and natural soils and residual soils overlying weathered rock, non-circular failure geometries are possible and may yield lower factors of safety than circular failure geometries.
- Section 3.4 presents the results of laboratory testing carried out on soil samples. The results of Direct Shear tests indicate a broad spread of values for peak effective cohesion (C') and internal angle of friction (ϕ'). The report does not use the actual test results and adopts lower, assumed values for the stability analyses. This measure of engineering judgement is acceptable provided the assumed values are truly representative of the strength of the fissured clays on this site.

3.4 Excavations

The overall approach adopted by Core is that "*it will be important to develop a comprehensive construction methodology during the design stage of the proposed development and for all parties involved to liaise closely with Core during the detailed design stage. Core must carry out a review of the design drawings and proposed methodology prior to construction."*

Due to the presence of the fissured Tertiary sediments, it is our opinion that there is potential for major instability to occur during the construction phase of the project and particularly during excavations. The impact of groundwater on stability is also a key consideration. The report does not appear to address this specific risk in the recommendations and rather provides generic advice in relation to temporary and permanent excavation slopes.

We agree with Core that there must be close liaison between the geotechnical consultant and all other parties during the design phase. It is not clear how this can be conditioned in an approval as, at this stage, neither the types of retaining walls nor the construction sequence and methodologies to be used are known. Council is not in a position to require the continued involvement of Core Consultants.

The geotechnical consultant must also be closely involved during the construction works. This has been alluded to in Section 5.1.8 where the report states "*Cut batters should be assessed by a geotechnical engineer from Core during excavation. Such assessments should be undertaken progressively, with excavated face heights prior to inspection being limited to 1m.*"

There is potentially a conflict between the statement in Section 5.1.8, *"excavated face heights prior to inspection be limited to 1m",* and Section 5.2 where it states that, "*hit and miss construction panels should not exceed 2.5m vertical height; higher cuts will require the upper section of the slope to be battered to a stable angle such that vertical excavation height does not exceed 2.5m."* This requires further review and comment by Core. If the contractor was to excavate a vertical face in the fissured Tertiary sediments there is likely to be a failure.

Section 5.1.8 presents maximum batter gradients for cut batters in different materials. These are generic and are acceptable for most soils but would not apply to fissured clays. Further comment is required regarding the maximum batter gradients for fissured clays.

Table 8 in Section 5.4 presents recommendations on design lateral earth pressure coefficients for different materials which are acceptable for most soils but would not apply to fissured soils. The report also nominates a triangular pressure distribution for the soil pressures which may not be appropriate for propped basement walls where the wall forms part of the final structure. The pressure distribution also depends to some extent on the construction sequence and the reactivity of the retained soils which is why the geotechnical engineer needs to be involved throughout the entire design and construction process.

3.5 Footing Design

Section 5.5.1 presents comments and recommendation regarding site Classifications in accordance with AS2870-2011. Whilst it is recognised that these are preliminary only, consideration needs to be given to the effects of trees and landscaping as well as abnormal conditions due to the removal of existing buildings.

3.6 Tunnel

Section 5.1.7 described likely subsurface conditions which may be encountered in the area of the proposed tunnel and notes that excavations will encounter a range of ground conditions including basalt colluvium, basalt rock, Tertiary sediments and sandstone and that "*groundwater and/or groundwater springs are likely to be encountered*". This aspect of the proposed development is likely to be technically challenging and requires extended geotechnical comments and recommendations beyond stating that "*additional geotechnical investigations will be required*".

At this stage insufficient information has been provided to assess the feasibility of the tunnel and/or identify the likely geotechnical constraints on such a structure being constructed.

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4. SUMMARY

The Core report provides a comprehensive review of the site conditions and includes preliminary recommendations for the geotechnical aspects of the design and construction of the proposed development.

On the basis of the information provided it is our opinion that the site can be developed as proposed. The greatest potential for slope instability will occur during the construction phase of the project.

We are in general agreement with the recommendations, many of which are generic and represent good design and construction practice. These however may not be appropriate in areas where fissured clays are present.

The presence of fissured Tertiary sediments and groundwater has the potential to cause major instability during the construction phase and particularly during excavations. The report does not appear to set out an approach or method by which this risk can be adequately managed and there is some inconsistency in the recommendations in relation to excavations.

The report appears to manage the risk by requiring the development of a comprehensive construction methodology during the design phase where all parties involved liaise closely with Core who also must carry out a review of the design drawings and proposed methodology prior to construction. It is not clear how this can be conditioned in an approval as, at this stage, neither the types of retaining walls nor the construction sequence and methodologies to be used are known. Council is also not in a position to require the continued involvement of Core Consultants.

It is recommended that Core prepare a plan for Council showing the areas of the site where fissured clays are likely to be encountered in the proposed bulk earthworks. A step by step approach should then be documented as to how bulk earthworks in such areas are to be carried out to mitigate risk. This approach can then be incorporated in conditions.

The main areas where the stability of the existing natural slopes and existing cut and fill batters will be of concern include the adjacent up-slope property, in the area of the existing dam and causeway and in the "remnant vegetation" area in the centre of the site. Further comment is required from Core on these areas as presented in Section 3.3 above.

There is an area of very steep slopes (30° to 35°) with groundwater seepage in the central/southern area of the site (area of test pits TP11 and TP12). One shallow landslide and one possible shallow landslide were observed in this area. Further site specific comment is required on the stability and treatment of the very steep slopes and groundwater seepage in this area.

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If you have any questions or if you wish to discuss or clarify any of the issues raised in this report, please contact Philip Shaw at our Brisbane office.

For and on behalf of

SHAW URQUHART PTY LTD

Philip Shaw

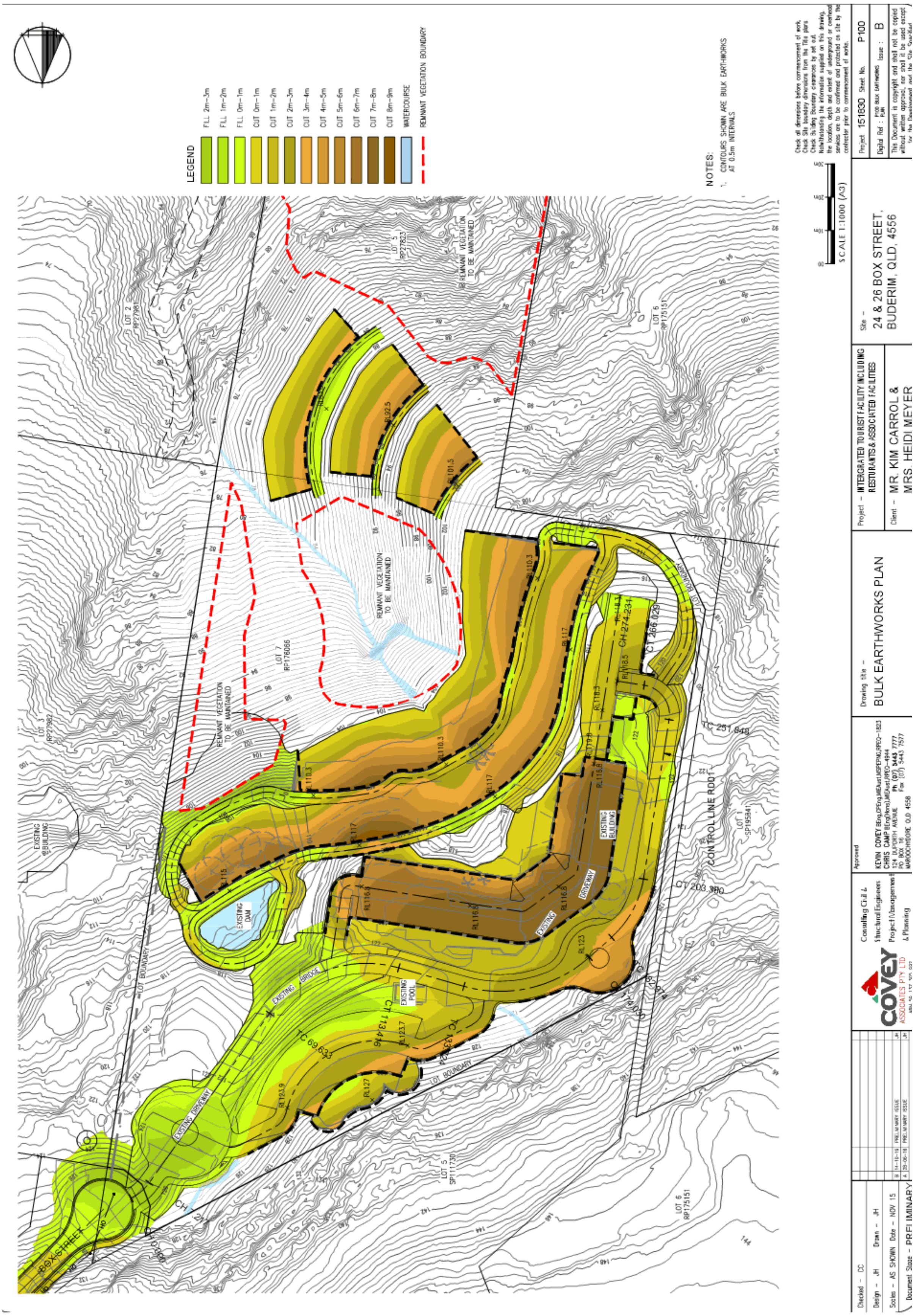
PHILIP SHAW

Principal Geotechnical Engineer

Att Layout Plan by Covey Associates Pty Ltd

Bulk Earthworks Plan by Covey Associates Pty Ltd


IPA ISSUE
Eco Luxe Resort & Spa
LEGEND/SUMMARY
REMNANT VEGETATION (retained)
SKYRAIL ACCESS TO "F" SUITES
A WELLNESS SPA (GFA 1,015m²)
B SERVICES BUILDING (GFA 544m²)
C BIRDWING BUILDING (GFA 3,815m²)
D GARDEN SUITES (38 rooms)
E LOFT SUITES (27 rooms)
F VINE FOREST SUITES (60 rooms)
VIEWS TO OCEAN & CALOUNDRA
VIEWS TO GLASSHOUSE MOUNTAINS
LOT BOUNDARY
REMNANT VEGETATION TO BE MAINTAINED
LOT 5 RP27823
LOT 7 RP176065
BOX STREET
DRIVEWAY 1
CARPARK ENTRY BASEMENT 2
DAM
POOL
REFLECTIVE POOL
LOWER TERRACE
4 STOREY SUITES
3 STOREY SUITES
2 STOREY SUITES
1 STOREY SUITES
ROAD
ELECTRIC GOLF BUGGY PATH
PEDESTRIAN PATH
APPROVAL FOR UNITS ON ADJOINING SITE
SITE MASTER PLAN 1:1000
WilliamsonArchitects
Purple Cow ARCHITECTURAL Imagery
HEIDI MEYER CREATIVE DIRECTOR
Job # 267-01
Drawing SP_SMP_01
Date 2016.09.30
Scale in metres
0 10 20 30 40 50


LEGEND
NOTES:
BULK EARTHWORKS PLAN
24 & 26 BOX STREET, BUDERIM, QLD, 4556
MR. KIM CARROL & MRS. HEIDI MEYER
BOX STREET

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161059/1-C ps:ps
16 May 2017

Sunshine Coast Council

Locked Bag 72

SUNSHINE COAST MAIL CENTRE QLD 4560

Attention: Mr Tim Mumford

RE: 24 & 26 BOX STREET, BUDERIM (MCU15/0270)

REVIEW OF RESPONSE TO RFI DATED 4 JANUARY 2017

1. INTRODUCTION

Shaw Urquhart Pty Ltd was requested by Sunshine Coast Council to review and provide comments on the response by Core Consultants to the RFI from Council dated 4 January 2017. The response by Core Consultants is presented in their letter dated 18 April 2017, ref. J000043-010-L-Rev.1.

2. REVIEW AND COMMENTS

The item numbers in the following comments match those in the response by Core Consultants.

Item 14: A copy of Figure 2 has been provided as requested.

This response is satisfactory.

Item 15: The presence of a basalt cap has been confirmed with the edge of the basalt cap inferred to be approximately near the existing driveway.

This response is satisfactory.

Item 16: No explanation has been provided as to why the presence of fissured soils was not identified on the test pit and borehole logs. If the test pit or borehole logs are separated from the main text of the report, there is nothing on the logs to alert the reader to the presence of fissures.

Core Consultants have taken the conservative approach in assuming that all of the Tertiary Sediments are fissured.

This response is conservative but satisfactory.

Item 17: Figure 2 shows the inferred extent of the Tertiary Sediments which, for the purpose of this assessment, are all assumed to be fissured.

This response is satisfactory.

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Item 18: The response is satisfactory in relation to dealing with boulders encountered in excavations but does not address the question that was asked in the RFI. The question that was asked related to the stability of the locally steep, boulder covered, natural slopes immediately up-hill of the northern boundary of the site.

This response is not satisfactory but can be addressed by including a condition that requires the stability of the locally steep, boulder covered, natural slopes immediately up-hill of the northern boundary of the site to be assessed along with an assessment of the potential for boulders to be mobilised and roll onto the subject property.

Item 19: The steep slopes are located down-slope of the proposed buildings and the RFI requested site specific comments on the stability of the area and how the steep slopes and groundwater seepage would be managed to ensure the stability of the buildings which are located above the steep slopes.

The response did not address the stability of these steep slopes and their impact on the proposed development.

This response is not satisfactory.

Item 20: Core Consultants state that they carried out non-circular failure surfaces but advise that the factors of safety were higher and the probability of failure lower than that of circular failure surface and were therefore not considered to be critical.

This response is satisfactory.

Item 21: The dam will be lined and an overflow pipe and spillway designed to prevent overtopping of the dam.

This response is satisfactory.

Item 22: This response is satisfactory.

Item 23: This response is satisfactory and clarifies that all cuts will need to be progressively inspected at 1m vertical intervals.

Item 24: The implication of the response is that large open cut faces will be avoided in fissured clays. Hit and miss panel excavation (say 3m width) or construction in sections will be utilised.

Taken in conjunction with the response to Item 23, where cuts will be progressively inspected at 1m vertical intervals, this response is satisfactory.

Item 25: This response is satisfactory.

Item 26: We do not agree with the statement "*the strength of fissured clay is defined by the peak phi values and residual cohesion*". The residual values of both phi and the cohesion are less than the peak values.

On the basis of a laboratory test result, it is proposed that a cohesion of 1kPa and a phi of 18° be adopted for Tertiary sediments. Core Consultants will need to satisfy themselves that these are truly "residual parameters" and not just "fully softened "values.

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Notwithstanding the response from Core Consultants, Table 8 in Section 5.4 of their report does not make any reference to fissured clays nor does it provide any advice on lateral earth pressure coefficients for fissured clays.

This response is satisfactory in that it nominates values for use in design involving fissured clays. Whether the values nominated are correct is the responsibility of Core Consultants. Design which includes fissured clays involves much more than just assigning "residual" parameters. Assessments need to made of the geometry, dimensions and orientation of the fissures to allow the determination a design value for shear strength. In reality this is likely to be somewhere between the peak or fully softened strength of the un-fissured soils and the residual strength along the fissures.

Item 27: This response is satisfactory.

3. GENERAL COMMENTS

The responses generally address most of the issues raised. The nature of the responses to the RFI assume that Core Consultants will continue to be involved through the final detailed design and construction stages of the project.

It was always our concern as independent reviewers that the report involved certain underlying assumptions as to the competence of the final designer and another consultant taking up the task may not be fully aware of the issues involved particularly with respect to the potential impact of the fissured clays on the development.

The site is large and the proposed development will involve significant construction works. The greatest risk on this site is likely to be during the construction phase of the works and any failures which may occur are likely to be confined to the subject site with the exception of the up-hill property. Any conditions of approval should include assurance that the proposed works will not adversely impact the stability of the up-hill property.

This independent review has been provided to assist Council in their assessment of the proposed development. Shaw Urquhart does not take any liability for the work carried out by Core Consultants.

If you have any questions or if you wish to discuss or clarify any of the matters raised in this correspondence, please contact Philip Shaw at our Brisbane office.

For and on behalf of

SHAW URQUHART PTY LTD

Philip Shaw

PHILIP SHAW

Principal Geotechnical Engineer

Douglas Partners
Geotechnics / Environment / Groundwater

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Project 93277.00
23 January 2018
ARM

Sunshine Coast Regional Council
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Sunshine Coast Mail Centre Qld 4560

Attention: Tim Mumford

Email: tim.mumford@sunshinecoast.qld.gov.au

Dear Tim

Peer Review
Proposed Development
24 & 26 Box Street, Buderim

1. Introduction

This report presents the results of a desktop peer review by Douglas Partners Pty Ltd (DP) of geotechnical assessment provided for a proposed tourist resort development at 24 and 26 Box Street, Buderim. The work was carried out for Sunshine Coast Regional Council (SCRC) in accordance with a DP proposal BNE171297 dated 4 December 2017 and acceptance received from SCRC.

It is understood that the site is approximately 4 ha in size and located within an area of high and very high landslip hazard and slope exceeding 20%. The development proposes extensive earthworks to accommodate the buildings including cut and associated retaining walls up to approximately 9 m in height.

The work was carried out to provide assessment of:

- identify significant geotechnical risks associated with the development including unresolved issues (and whether they could be resolved) and any key information that may be lacking:
- whether risk based approach to instability is appropriate in this case;
- further outline the likely implications of 'low' instability risk particularly under adverse climatic conditions, including whether this level of risk should be acceptable in engineering terms given past instability of development works in the local area;
- whether the proposed recommendations are reasonably practical to be implemented by a competent contractor with sufficient confidence of outcome;
- implications for instability risk if the project is halted before completion of key works;
- whether the inherent geotechnical risks are sufficiently high even after implementing reasonable engineering measures to prevent the development; and
- the residual risks arising from the limitations identified in the applicant's geotechnical reports, and whether these are unusual limitations.

The assessment comprised review of previous reports prepared by Core Consultants Pty Ltd for the project applicant and Shaw Urquhart Pty Ltd for SCRC, and other available information including aerial imagery followed by desktop assessment.

The details of the assessment are provided in this report along with comments on the issues indicated above.

2. Site Description

The site is located on the southern side of the Buderim mountain plateau and slopes down towards the south east from Box Street at around RL 125 down to a cut and fill platform around RL 120, and then further down in several south-south-easterly spur ridges to the lowest part of the site to be developed at around RL 75.

Figure 1 – Site Extent

The ground at the northern boundary slopes steeply up at around 30^0 up to the plateau around RL 145 and is densely vegetated. The cut and fill platform area is occupied by an existing residence and other buildings and is mostly grassed with a small dam. Below the platform the ground slopes down typically at around 24^0 to 30^0, but locally up to 40^0 adjoining gully lines. This area is grassed or densely vegetated.

3. Proposed Development

The proposed development comprises the construction of a number of buildings of up to 4 storeys in height for resort and conference facilities and accommodation with associated driveways, carparking and associated landscaping. Extensive earthworks are proposed to accommodate the buildings including cut and associated bored pile or soil nail and shotcrete retaining walls mostly up to 6 m in height but locally as much as 9 m in height.

4. Regional Geology

The Queensland Government Department of Natural Resources and Mines digital geological mapping indicates the site is underlain by:

- Tertiary age Basalt along the northern boundary; and
- Triassic-Jurassic Age Landsborough Sandstone comprising "Lithofeldspathic labile and quartzose sandstone, siltstone, shale, minor coal, ferruginous oolite marker".

A detailed summary of the geology and geomorphology of Buderim taken from Geological Survey of Queensland (GSQ) Record 1977/13 "*Buderim Sewerage, Preliminary Geological Report*" is provided in the following report:

- "*Landslip Occurrence at Buderim Mountain*", Report No. B10031/1F dated September 1981 prepared by Coffey & Partners Pty Ltd.

In broad summary, this report indicates the geology and geomorphology of the Buderim area comprises:

i. Deposition and formation of the sandstones of the Marburg formation (includes the Landsborough Sandstone) in the Jurassic period (201 million years ago "mya" to 145 mya). This Sandstone reportedly generally dips gently towards the north-east at around 7^{O};

ii. Deposition of Tertiary age (65 mya to 2.6 mya) Sediments over the Jurassic sandstones which comprise poorly consolidated sandstone, siltstone, shale and clay which "probably occur extensively beneath basalt cap" and provide a discontinuous cover of variable thickness. The

GSQ report notes that the Sediments "appear to fill depressions and valleys in the Tertiary land surface prior to the extrusion of the basalt". The reports indicates the Sediments are similar to the completely weathered rocks of the underlying Jurassic sandstones, and weather to form plastic clays and are "permeable in part";

iii. Tertiary age olivine Basalt capping with thickness 15 m to 25 m with the base elevation ranging from about 75 m to 140 m above sea level. "The basalt occurs as several flows, sometimes separated by a thin layer of sediment"

5. Previous Reports

The previous reports provided by SCRC for review were:

- Core Consultants report "*Report on Geotechnical Investigation & Slope Stability Assessment, Integrated Tourist Facility including Function Facility, Restaurants & Short Term Accommodation, 24 & 26 Box Street, Buderim Q 4556*", Report No. J000043-005-R-Rev2 dated October 2016;
- Core Consultants correspondence "*Response to Information Request, Application No: MCU15/0270, 24 & 26 Box Street, Buderim*" dated 18 April 2017;
- Shaw Urquhart correspondence "*Proposed Integrated Tourist Facility – 24 & 25 Box Street Buderim, Peer Review of Report by Core Consultants Pty Ltd*" dated 29 November 2016; and
- Shaw Urquhart correspondence "*RE 24 & 26 Box Street, Buderim (MCU15/0270), Review of Response to RFI dated 4 January 2017*" dated 16 May 2017.

A further report was also obtained from SCRC library that provides further useful background:

- "*Detailed Landslip Study in the Coolum View Terrace to Pertaka Street Area, Buderim Mountain*", Report No. B10031/2-C dated September 1982 prepared by Coffey & Partners Pty Ltd report.

The Core Consultants investigation has completed a significant amount of investigation in April 2016 and September 2016 to assess the geotechnical conditions for the development; including twelve test pits, twelve bores, geophysical testing along with laboratory testing to assess plasticity and limited testing to assess strength characteristics. The geotechnical model derived describes four geotechnical units of natural materials:

- Basalt Cap and Basaltic Colluvium encountered in the northern and middle parts of the site and comprising high plasticity silty clays with sand, gravel, basalt cobbles and boulders. One borehole (BH 10) encountered basalt from 7.5 m depth to the end of the hole at 12 m depth; the geophysical testing indicated seismic velocities of around 800 m/sec where the bore ended.

- Tertiary Sediments encountered within the northern and middle parts of the site and comprising mostly pale grey high plasticity silty clay with some clayey sand zones. Fissuring was noted in this layer.

- Landsborough Sandstone encountered in all boreholes except BH10, typically of extremely low to low strength with overlying medium plasticity residual sandy clay soils; and

- Slopewash to shallow depth overlying the Landsborough Sandstone in the southern part of the site.

Filling was also encountered in a number of bores and pits in the investigation, typically less than 2m in depth but up to 4.3m at BH11 and possibly locally up to 8 m (with Basaltic Colluvium) in an old watercourse between boreholes BH8 and BH9.

Groundwater seepage was observed in four of the test pits in the middle and upper part of the slope, and groundwater levels were measured in September 2016 in standpipes in the bores at depths between 3.1 m and 9.3 m below ground level. These groundwater levels mostly corresponded to the Tertiary Sediments or underlying Residual Soils.

Laboratory testing indicates that the silty clays (Tertiary Sediments) are of high plasticity with liquid limit values of 52 to 89. The sandy clay materials (Residual Soils) were of intermediate or high plasticity with Liquid Limit values of 37 to 57.

Shear box testing was carried out on only two samples of the Tertiary Sediments to provide an indication of cohesion (c) and angle of friction (Φ) values; the testing results ranged quite significantly and interpretation was required to select "residual" strength parameters of $c'=1kPa$ and $\Phi'=18^{O}$.

6. Comments

6.1 Significant Geotechnical Risks

The previous investigation indicates the geotechnical conditions are likely to comprise a range of materials, with Basaltic Colluvium and Tertiary Sediments in the upper and middle parts of the site, and shallow slopewash over residual soils and weak rocks of Landsborough Sandstone in the lower part of the site.

Given the geotechnical conditions, the natural slopes and the cut earthworks proposed, it is evident that the major geotechnical risk of the development will be instability, both during temporary conditions induced during construction and permanent. The other geotechnical risks such as earthworks, foundations, retaining walls, pavements etc are not considered likely to be unusual for development works of this type.

Core Consultants have addressed the instability risks with both qualitative landslide instability assessment as well some representative stability analysis. In terms of this analysis it is noted that some further consideration may be warranted at this stage in respect of the following issues:

- the shear strength parameters adopted (c'=0kPa and Φ'=18^{O}) for the stability model appear to be based on limited testing with quite variable results, and will need to be verified with detailed testing. However, these values are significantly higher than those in previous investigations; the Coffey & Partners 1982 report for the northern side of the mountain indicates shear strength values of c'=0kPa and Φ'=9^{O}. The Coffey & Partners 1981 report indicates angle of friction parameters of Φ' of 8.3^{O} at peak shear stress and 4.2^{O} at residual stress when tested along a prior shear surface, but 28.6^{O} at peak stress when tested across the existing shear surface. The lower strength parameters reported by Coffey & Partners are able to provide a plausible explanation for the flatter slopes that are present in the Tertiary clays at the base of the Basalt cap.

  The nature of the fissured soils and the extensiveness of the prior shear surfaces and the strength parameters that should be adopted is of paramount importance in being satisfied about the stability of slopes and the minimum engineering retaining works required to ensure stability.

- The assumed groundwater conditions appear to be based largely on groundwater monitoring in September 2015, which Core Consultants indicate (p13) is after 80% of the annual rainfall (at Sunshine Coast Airport). However, referring to rainfall records over a much longer period from Palmwoods (refer Figure 2 below, plotted with rainfall as July-June so that summer rainfall is reported in the same year), the investigation and groundwater monitoring has occurred in a series of years of average or well below average rainfalls. As highlighted in the previous Coffey & Partners reports, because of the fissuring in the Tertiary clays the groundwater levels can rise rapidly in response to rainfall. It is likely that the groundwater levels are highly responsive to rainfall, and undertaking one or two discrete readings is prone to underestimate the likely reasonable extreme case condition that might be experienced.

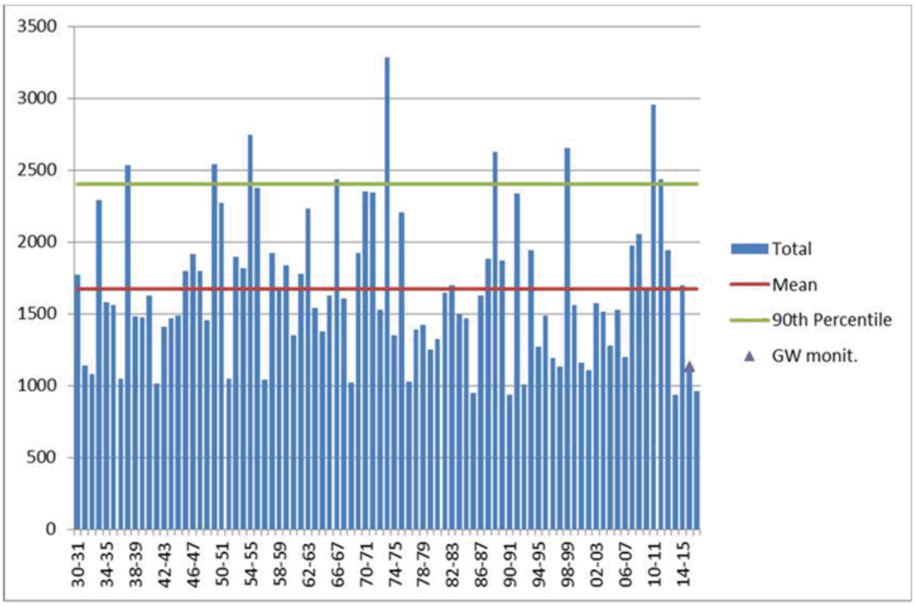

Figure 2 – Annual Rainfall (Palmwoods)

With this in mind, whilst it would be preferable to obtain groundwater levels over a much longer period including at least one summer of above average rainfall, there should preferably at least be continuous groundwater level monitoring (using in borehole data loggers) in selected key locations with close examination following significant rainfall events.

At the current preliminary phase, it would be at least prudent to adopt more elevated groundwater conditions to be more representative of the 90th percentile case rainfall events (which last occurred in 2010/2011 and 2011/2012 years) to understand the magnitude (and economic viability) of works required to deal with likely extreme rainfall conditions. It is expected that it will be necessary to give detailed consideration to a groundwater control system (and proposed discharge options/approaches) for the project which may be required to cope with significant flows during and after significant rainfall events.

- It is also noted that there is significant risk of instability in the basaltic colluvium, in particular for boulders to become dislodged and roll downslope from within the site due to construction or erosion, and also from within the vegetated area upslope of the site. Whilst Core have indicated appropriate care is required for works within the site to manage this issue, some consideration of protecting the site occupants and development from the upslope boulders that may become mobile in the lifetime of the development is considered warranted.

6.2 Risk Based Approach for Quantitative Stability Analysis

Core Consultants assessment of the instability risk posed by the proposed works has included both a qualitative assessment (Section 4.3.2) and a "semi-quantitative" assessment (Section 4.3.1). The use of a risk based approach for the qualitative assessment is appropriate to provide a high level understanding of the risks and measures (including engineering works) that may be required to reduce the risks to an appropriate level. It is noted that the method requires considerable judgment and interpretation in its application, particularly in terms of assessing the likelihood (refer Appendix E of Core Consultants reports).

For making assessment of the engineering works required to ensure stability even under the likely reasonable worst case, it is usual to undertake analysis using an estimate of reasonable lower range soil strength parameters (in the opinion of the geotechnical engineer), and design the works to achieve a safety factor of 1.5. Where there is concern about uncertainly of strength, it is a simple matter to run the analysis with several other parameters and observe the effect. It is then the decision of the designer to choose an appropriate parameter. It is common that the geotechnical designer is required to provide a form of certification that the slopes and retaining works attain a safety factor of 1.5 for the permanent works.

It is considered that given the difficulty of the conditions and the sensitivity of the outcomes, "semi-quantitative" analysis is not particularly clear in its outcomes and risks transferring the key engineering decisions to parties who do not have the engineering expertise. It is considered that an unequivocal quantitative analysis (with appropriate qualifications pertaining to the site conditions) sufficient that the geotechnical engineer can confirm that the proposed works can attain a safety factor of 1.5 for the permanent works would be warranted in this case.

6.3 Likely Implications of 'Low' Instability Risk

The terminologies for risk in terms of landslides are defined in Appendix C of the AGS Practice Note Guidelines for Landslide Risk Management (copy appended). These guidelines therefore indicate that 'Low' Risk requires acceptance of the following likelihood of certain damaging events:

Table 1 – 'Low' Risk Landslide Damage and Likelihood

Damage to Property Event	Approximate Cost of Damage(1)	Highest Likelihood
Catastrophic consequence damage: structure(s) completely destroyed and/or large scale damage requiring major engineering works for stabilisation. Could cause at least one adjacent property(2) Major consequence damage.	200%	The event is inconceivable or fanciful over the design life (1 in 1,000,000 year recurrence interval).
Major consequence damage: 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.	60%	The event is conceivable but only under exceptional circumstances over the design life (1 in 100,000 year recurrence interval).
Medium consequence damage: 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.	20%	The event might occur under very adverse circumstances over the design life (1 in 10,000 year recurrence interval).
Minor consequence damage: limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works.	5%	The event might occur under very adverse circumstances over the design life (1 in 10,000 year recurrence interval).
Insignificant consequence damage: little damage.	0.5%	The event will probably occur under adverse conditions over the design life (1 in 100 year recurrence interval).

Notes:
1. 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.
2. Limited adjoining buildings at current time but assumption of possible future development within life of the development.

As indicated previously, using risk based approach requires considerable judgment and interpretation in its application, particularly in terms of assessing the likelihood, and it is therefore difficult to be confident that the assessed risk is 'Low'.

The guidelines also state that acceptance of low risk requires that where treatment has been required to attain this level, that ongoing maintenance of the measures is required.

The ground with the highest risk of instability is that where the Tertiary soils are at or close to the surface, in particular in the middle part of the site where Buildings C and D and located. The proposed scheme (refer proposed building excavations superimposed onto the Core Consultants geotechnical cross sections below in Figures to 5) indicates basement excavation to RL117 for Building C, which current geotechnical information suggest will provide a subgrade in basaltic colluvium, with Tertiary

Sediments around RL 116 and Sandstone around RL 114. It is reasonably likely that Building C will require a grid of bored pile foundations into the Sandstone, and these measure will provide a significant stabilising influence for this part of the slope. The Building C basement wall will provide a significant engineering structure to support the slope in that location. The Building D retaining walls will likely require quite extensive engineering measures with continuous positive support (eg soldier pile bored pile wall) because of the presence of the Tertiary Sediments in that location. Given the above, it seems it should be reasonably practical in this part of the site to provide measures that will provide a safety factor of 1.5 for stability.

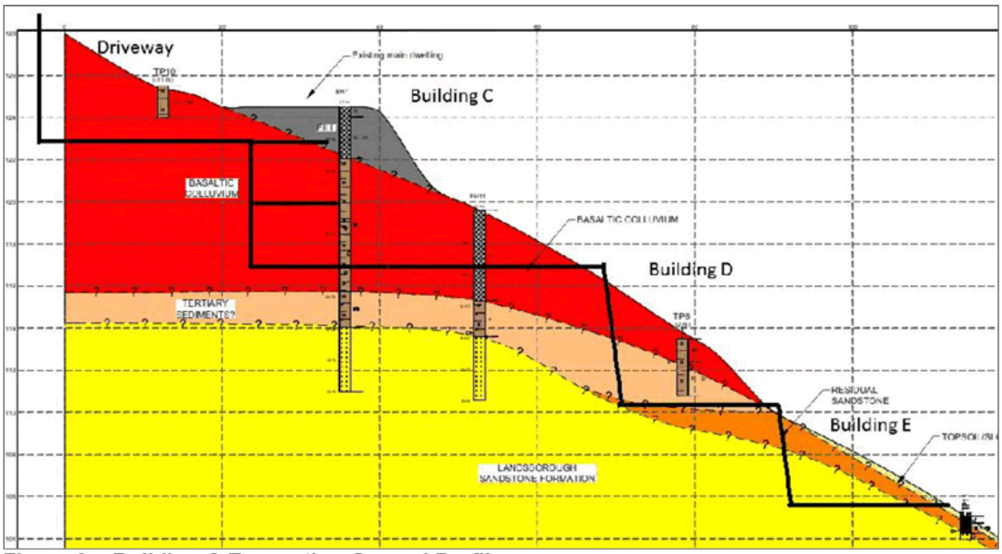

Figure 3 – Building C Excavation Ground Profile

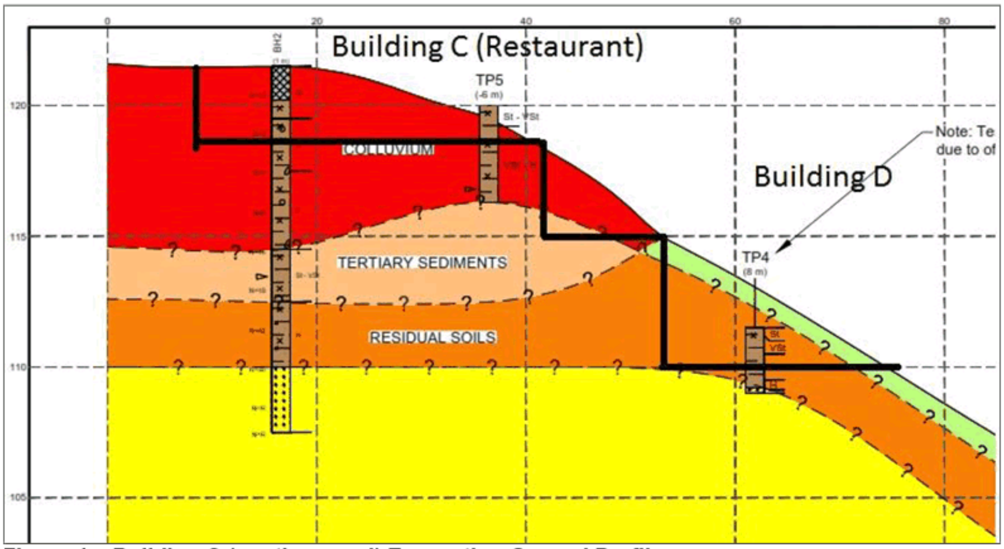

Figure 4 – Building C (southern end) Excavation Ground Profile

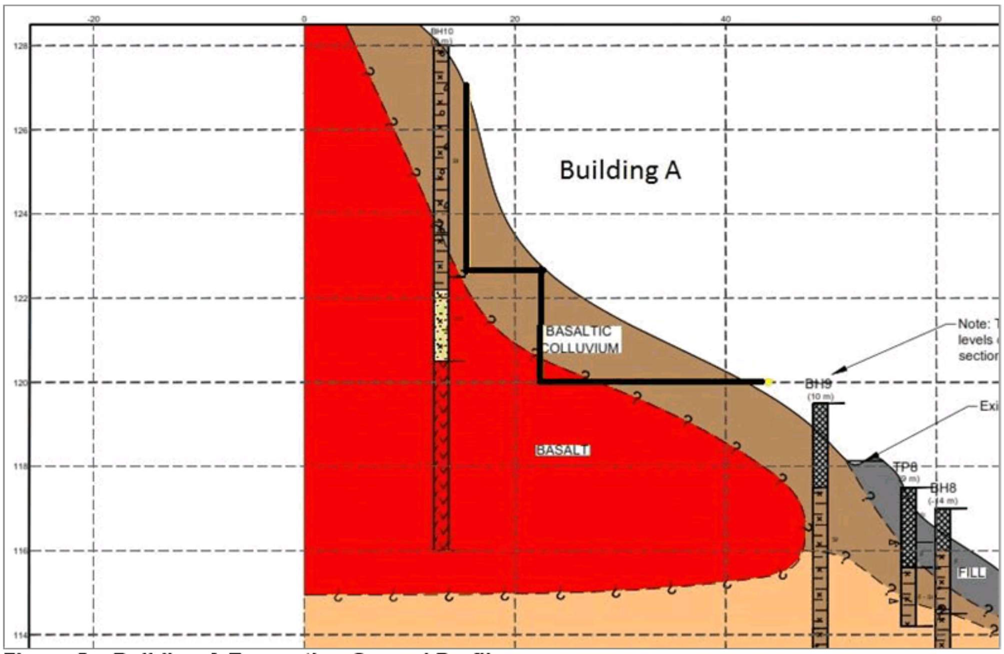

Figure 5 – Building A Excavation Ground Profile

Figures 3 and 5 also indicate there are two areas where retaining structures will be required to prevent instability in the Basaltic colluvium with a significant slope upwards behind the wall crest. These structures also do not benefit from direct propping from structure (eg basement slabs), and are located close to the site boundary, so permanent anchoring will not be feasible. Mass gravity walls may be a solution but could be very large structures. Given the steep upslope it is considered that it will be highly desirable to maintain positive support at all times with measures such as temporary anchoring (if the neighbour permits) whilst walls are constructed. Further, these walls would be subject to the risk of boulders from upslope areas as indicated in Section 6.1 previously. Given the significant constraints, these walls may be quite significant features and it is recommended that further consideration of the proposed solutions for the walls be undertaken.

6.4 Practicality of Recommended Works

It is considered that whilst the current analysis requires some further consideration to address groundwater conditions and possible lower strengths in the Tertiary Sediments, it is likely that the proposed works should be reasonably practical and within the competency of a mid-sized reputable building contractor based in south east Queensland with suitably qualified and experienced professional advisors. As with any project there is a risk of low cost / high risk builder contractor seeking to reduce the construction requirements which needs to be managed.

6.5 Implications of Halting Key Works

There is a risk that if works on bulk excavation and retaining systems is halted for a significant time period that the risk of instability is elevated. Typically, the factor of safety for temporary works is adopted at 1.2 or 1.3 as compared to 1.5 for permanent works.

If continuous positive support measures (eg soldier pile walls, temporary anchors etc) are not used for the key risk excavations, then there is a significantly elevated risk of inducing slope failure particularly during or soon after a major rainfall event. Given the site slopes and geological conditions, such a failure could be quite large in size, impact and rectification requirements including impacting property including protected vegetation areas onsite and offsite.

Any temporary anchors (and other temporary steel elements) are not typically corrosion protected, so these would be subject to corrosion over time, although the use of galvanised products would somewhat mitigate this risk.

If groundwater and/or surface water required active control during temporary works, then allowing these to become uncontrolled could present some quite significant risks of instability.

6.5 Do the Inherent Geotechnical Risks Prevent the Development

It is considered that with the information available, there is not sufficient evidence to confirm that the inherent geotechnical risks cannot be reduced to acceptably low levels ordinarily accepted with reasonably common and economic engineering measures.

6.6 Residual Geotechnical Risks arising from Report Limitations

The Core Consultants Limitations (ref FRM-065 Issue 1.01 dated 01/10/2015) includes clauses that in terms of geotechnical aspects (and excluding any matters of law) in summary indicate:

- the work is specific to the scope requested and the intended development works;
- ground conditions can vary between test locations and further investigation may be required;
- the ground conditions may change over time; and
- the supplied data is assumed to be correct unless otherwise stated.

These are issues that are all commonly advised in geotechnical reports by reputable consultants.

The risks associated with the intended works changing should be able to be readily identified. The minimum scope of investigation required to provide a sufficient degree of confidence that variations in ground conditions of significance will be detected in investigation or construction with competent geotechnical supervision, and to provide design advice that accommodates reasonable changes in conditions over time, will somewhat be a matter of opinion. However, only preliminary investigation has been undertaken and further detailed investigation is obviously required at the appropriate time(s). An obligation to provide Form 16 Certification of key geotechnical matters including stability by an experienced RPEQ Geotechnical Engineer will certainly be an essential requirement in this case. Independent engineering review / verification of any engineering design is recommended.

In this case the supplied data was limited in extent and it appears very little to no reliance was placed upon it, and this does not seem to be of major significance.

7. Limitations

Douglas Partners Pty Ltd (DP) has prepared this report for this project at Box Street, Buderim in accordance with DP's proposal BNE171297 dated 4 December 2017 and acceptance received from Sunshine Coast Regional Council dated 5 December 2017. This report is provided for the exclusive use of Sunshine Coast Regional Council for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

DP's advice is based upon the information provided for the assessment and any other information above to be obtained and the accuracy of the advice may therefore be affected by the accuracy of that information as well as any undetected variations in ground conditions across the site between, beyond or below the sampling and/or testing locations. Sub-surface conditions can change abruptly due to variable natural processes and also as a result of human influences; such changes may occur after the field testing has been completed.

This report must be read in conjunction with all of the attached and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by DP. This is because this report has been written as advice and opinion rather than instructions for construction.

The contents of this report do not constitute formal design components such as are required, by the Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during construction and the controls required to mitigate risk.

Yours faithfully
Douglas Partners Pty Ltd

Andrew Middleton
Principal

Reviewed by:

Bruce Stewart
Principal

Attachments: Notes About this Report
Appendix C of the AGS Practice Note Guidelines for Landslide Risk Management

About this Report

Douglas Partners

Introduction

These notes have been provided to amplify DP's report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

DP's reports are based on information gained from limited subsurface excavations 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.

Copyright

This report is the property of Douglas Partners Pty Ltd. The report may only be used for the purpose for which it was commissioned and in accordance with the Conditions of Engagement for the commission supplied at the time of proposal. Unauthorised use of this report in any form whatsoever is prohibited.

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. 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 and test pits 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 or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

Groundwater

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

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole 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; and
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water measurements 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 advisable in low permeability soils or where there may be interference from a perched water table.

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. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP 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 conditions, discussion of geotechnical and environmental aspects, and recommendations or suggestions for design and construction. However, DP cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

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

July 2010

About this Report

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, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

Information for Contractual Purposes
Where information obtained from this report 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 specially edited document. DP 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 and environmental 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.

July 2010

PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT 2007

APPENDIX C: LANDSLIDE RISK ASSESSMENT

QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

QUALITATIVE MEASURES OF LIKELIHOOD

Approximate Annual Probability		Implied Indicative Landslide Recurrence Interval		Description	Descriptor	Level
Indicative Value	Notional Boundary					
10^{-1}	$5x10^{-2}$	10 years	20 years	The event is expected to occur over the design life.	ALMOST CERTAIN	A
10^{-2}		100 years		The event will probably occur under adverse conditions over the design life.	LIKELY	B
	$5x10^{-3}$		200 years			
10^{-3}		1000 years		The event could occur under adverse conditions over the design life.	POSSIBLE	C
	$5x10^{-4}$		2000 years			
10^{-4}		10,000 years		The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
	$5x10^{-5}$		20,000 years			
10^{-5}		100,000 years		The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
	$5x10^{-6}$		200,000 years			
10^{-6}		1,000,000 years		The event is inconceivable or fanciful over the design life.	BARELY CREDIBLE	F

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

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

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

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

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

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

PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT 2007

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.