AGS VICTORIA 2016 SYMPOSIUM

Excavations and slope stability in Melbourne geology: experiences and recent developments

Wednesday, 16 November 2016, 12:00pm – 7:00pm
Engineers Australia, 600 Bourke Street, Melbourne

AUSTRALIAN GEOMECHANICS SOCIETY
VICTORIA CHAPTER

WELCOME

The Victorian chapter of the Australian Geomechanics Society (AGS) is pleased to welcome you to this half-day symposium titled “Excavations and slope stability in Melbourne geology: experiences and recent developments”.

Since the publication of the “Engineering Geology of Melbourne” in 1992, both the geotechnical profession and Melbourne has undergone significant change. Urban sprawl over the past few decades has seen increasing development in the hillside areas in the Dandenong and Mornington Peninsula regions. This coupled with changes to the regulatory environment and the introduction of the Landslide Risk Management Framework by the AGS in 2007 has changed the way in which local and state government as well as geotechnical practioners manage and assess slope stability.

In addition to development in hillside areas, significant development in the inner parts of Melbourne has posed many challenges for excavations not just in the soft soils of the Yarra Delta but also the weak rock of the Melbourne Formation.

This symposium seeks to bring together practioners from consulting, construction and academia to share and discuss their experiences on the separate, but related, topics of excavation and slope stability. Best practices, case histories and innovative solutions for dealing with these challenges will be presented and discussed, with a particular emphasis on local geotechnical issues.

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DESIGN “RIGHT” AT THE EDGE OF THE EARTH

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ABSTRACT

Since the publication “Innovative Slope Engineering” published in the Geomechanics Journal, March 2010, by the above author and colleague Shyamalee Herath, more people seem to want to build in more crazy places and this is a selected few examples in Melbourne and environs.

These case studies have been selected because of their unique solutions in looking, not only at the geological model and analytical solution, but also using the structure as part of the solution.

These solutions and case studies are at the edge of the earth at the following locations.

- Yarra River, Kew. A steep, inaccessible site on the Yarra where a “spider web” foundation arrangement was used.
- McCrae. A steep, inaccessible site where a large house was converted to one massive retention structure.
- Some exposed rock examples and risk assessment procedures, “the counting of rocks approach”.
- Kalorama, Mt Dandenong. The use of proprietary products in risk mitigation.

This paper demonstrates the approaches adopted to provide solutions with the application of AGS Guidelines for landslide risk management and mitigation strategies.

Examples are now built and performing well.

1 INTRODUCTION

This paper presents solutions both theoretical and practical to building in areas predominantly in the Melbourne environment. It links these solutions to the guidelines in AGS Guidelines “Landslide Risk” Management Concepts and Guidelines (2007), and how these are then linked to geotechnical input and parameters. They also are a demonstration of how structural engineers and geotechnical engineers can interact closely to obtain relatively simple solutions to what at first glance may be complex or difficult problems

2 HODGSON STREET, KEW

2.1 Geology & Background

This site is located on the Yarra River in terrain formed by erosion of Silurian Siltstone of the Melbourne Formation. Weathering of the siltstone would typically result in shallow surface silts underlain by clays grading to weathered siltstone. In this case however, given the slope of site and erosion over geological time, the profile now consists of unstable hillwash fill, silt and gravel overlying typically higher strength siltstone.

2.2 The Problem

The difficulties that exist for construction at the site can be seen by the following architectural diagrams (Figures 1 & 2). The slope ranges from 45°-60°, is largely inaccessible and had relatively dense vegetation. It did have a more stable zone that was able to be utilised as an “anchor point” for piles that were eventually adopted.

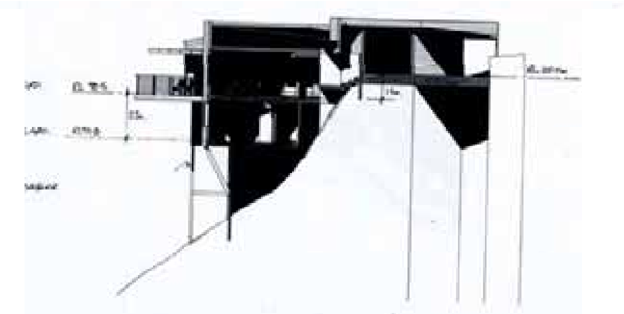

Figure 1. Typical Elevation indicating slope and structure layout

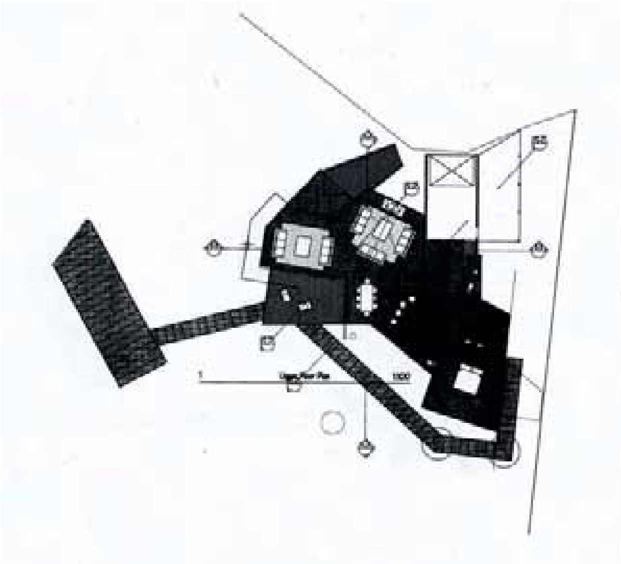

Figure 2. Typical Plan View (A complex layout)

- The site has an inherently unstable surface profile (approx. 1.0-1.6m) slumping and progressively slipping.
- An architecturally "interesting" shape not suited to conventional strip or grid footing layouts.

2.3 Investigated Profiles & Geology

The site is underlain on the high elevation by 1.5m of residual or hillwash deposits overlying distinctly weathered siltstone of low to moderate rock strength that extended to at least 8.0m. This shallow profile was also able to be confirmed down the escarpment, although somewhat less reliably with a hand auger.

Moisture contents in the siltstone ranged between 4.9% and 8.3%.

2.4 The Solution

The foundation arrangement chosen was a "spider web" arrangement where bored piles were utilised to anchor the residence on the accessible area. These were designed as laterally loaded bored piers with the lateral load being the forces at ground level induced by the tie rods. Pad and strip footings down the slope were effectively suspended on tie rods running down the escarpment and picking up each individual footing and joining it to the web and then to the piles. This is indicated on Figure 3.

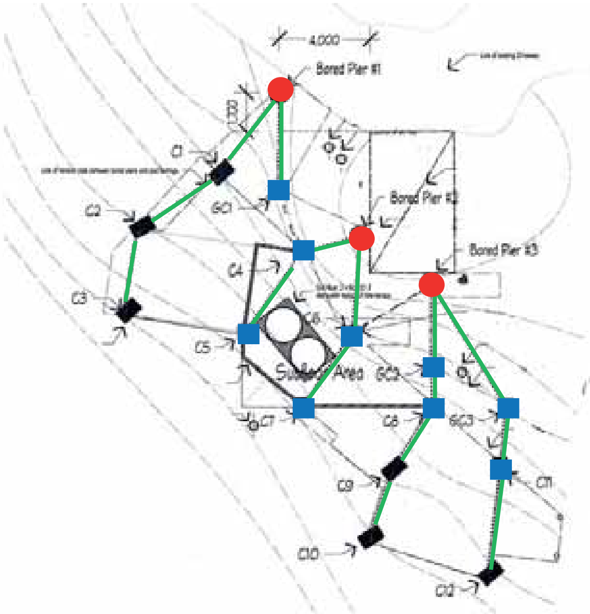

Figure 3. Footing Layout (Key elements labelled)

1. (Red) The piles are anchor piles (laterally loaded vertical piles). These were proportioned using the following parameters for a distinctly weathered siltstone.
   Short term – C_u = 200 kPa below 1.5m
   Serviceability – E = 200 MPa

2. (Green) The green lines are tension rods running down the slope and connecting pad footings. The design of the lateral forces transferred into the tie rods and then back to the anchor piles was advised to be taken as the sum of the "At Rest" lateral force of 1.5m height of soil (active zone) acting against each individual pad footing on the grid.

3. The solid lines are "bracing" infill walls where compression forces resulted.

4. (Blue) Pads are conventional pads hand excavated to the siltstone and to resist smaller lateral (sideway) forces. Grout bars were used into the siltstone as a precaution.

5. Thus, this "spider web" foundation arrangement was an early example of

numerous similar anchor and tension arrangements that have been adopted. Such an approach has been used regularly on the inaccessible escarpments of Portsea when supporting inclinator structures.

3 POINT NEPEAN ROAD, MCCRAE

3.1 Geology & Background

The construction of this dwelling is at the base of the escarpment and extending up from the base. The site is underlain on the escarpment by residual clays and sands formed by the weathering of Devonian Granite with hillwash sands and clays. On the upper elevations of the escarpment Tertiary Deposits exists. On the low level elevations Quaternary Aged Sands exist overlying Tertiary deposits.

At the time of our investigation slope instabilities existed such as creeping, slumping and past landslip was evident, with hummocky ground surfaces.

Surrounding vegetation exhibited curved trunks and/or basal flexure.

3.2 The Problem

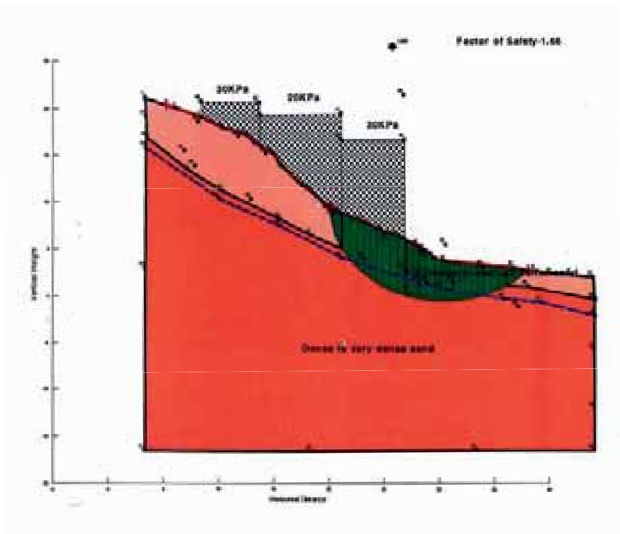

Figure 4. Sample Slope Stability Output (Deep Seated Failures, Factor of Safety 1.66)

3.2.1 Setting at Point Nepean road

- The proposed house is situated at the base of the escarpment with the escarpment being steep (approximately 50°) and inaccessible.
- The base of the escarpment is characterised by low-medium strength sands and clays.
- As a consequence of the above two settings, a probability existed of "global" failure of the escarpment particularly given the proposal to excavate into the escarpment in a "tiered" type of construction and lower strength sands and clays.

The global stability issue at this site is best highlighted by reference to Figure 4.

3.3 Investigated Profiles & Geology

The profiles of this escarpment were investigated at this site by vertical and horizontal boring.

Some typical parameters and those used in our models are as follows:

- Surface Fill and Sand (Hillwash Deposits) - C´ = 0 kPa, Ø´ = 28°, γ = 17 kN/m^3
- Firm-Stiff Silty Sandy Clay (Tertiary) – C´ = 10 kPa, Ø´ = 22°, γ_w = 18 kN/m^3
- Medium Dense Silty Clayey Sand - C´ = 3 kPa, Ø´ = 34°, γ_w = 18 kN/m^3
- Medium Dense to Dense Silty Clayey Sand (Extremely Weathered Granite) - C´ = 5 kPa, Ø´ = 36°, γ_w = 18.5 kN/m^3

Note: Given no deeper seated failure has occurred, residual strength parameters were not considered.

3.4 The Solution

Due to the significant lateral (retaining) forces to be resisted and poorer quality founding until dense sand or weathered granite was reached, the house structure itself became the retention with this structure also transferring the lateral forces to raking piles at the lower level. Of course piles were also utilised over the remainder of the structure to extend through the potential failure plane, but these were not capable of resisting lateral load.

This can be seen in the following photograph (Figure 5) taken during construction where the lateral loads are being transferred to the base by blade columns.

Figure 5. Foundation Arrangement (Piles & Lateral Transfer)

Highlights in regards to the structural solution:

The retention/support piles, spacing 2.5m or less.
A "bound" foundation arrangement links piles with substantial ground beams. "Bound" in this context means tied together.
"Buttress" (Blade Columns) type lower structure that transferred lateral load to raking piles at the base (not seen, but below ground level).

This residence was a forerunner to several similar houses on Point Nepean Road, McCrae, at the base of the escarpment.

4 COUNTING ROCKS (AN ANALYSIS OF PROBABILITY AND CONSEQUENCE)

4.1 Geology & Background

This example is in Tasmania, and is included not because of its location but the principles can be used in any exposed rock face.

Design Issues: The following three photographs are taken around the third of approximately five inspections to this site. It was early in the construction of this exposed rock feature wall constructed in Sandstone, it became apparent as can be seen in numerous road cuttings coming from the airport that once exposed the sandstone weathers quickly accelerated by what appears to be an immediate increase in moisture content, causing perhaps slight volume increase and joint weakness.

Figure 6. Example of Face Cut with Diamond Saw Fitted to Excavator, Rock Bolts in Place

Figure 7. Example of Potential Rockfalls

Figure 8. Example of Potential Rock Falls

Rock Bolting had been carried out of the medium and large block wedge potential rock falls. A study of discontinuities joints and bedding had previously been carried out by others and despite this bolting falls were being experienced. Failures were defined by the following Table 1.

Table 1: Characterisation of Rockfalls

Name	Size/Shape	Characterisation Failure Mechanism
Scat	<4cm wide, usually tabular in shape	Small chips of rock within joints on rock face that become dislodged due to weathering
Small Wedge	<40cm in size, usually tabular[3] to equant[3] in proportion	Usually the lowermost portion (within a thinly-bedded layer) of a larger wedge formed by two intersecting joints. These fail by sliding along one or both of the joint sets following release along the terminating upper bedding plane.
Small Block	<40cm in size, usually equant to prismatic[3] in proportion	Usually the uppermost portion of a medium wedge, which are sufficiently tall and narrow that they may fail by toppling, the intersecting vertical joints providing release surfaces.
Medium Wedge	<2m in size, usually prismatic in proportion	Usually the entire thickness of a more competent layer which fails by sliding along two joint surfaces.
Medium Block	<2m in size, usually prismatic to equant in proportion	Usually the entire thickness of a more competent layer which fails by toppling.
Large Wedge	<8m in size, usually prismatic in proportion	Usually more than one layer which fails by sliding along two joint surfaces.
Large Block	<8m in size, usually prismatic to equant in proportion	Usually more than one layer which fails by toppling as a single block.
Entire Face - Corner	Whole height of face of corner edge – prismatic in shape	Intersecting joints allow stress to build up on weak layer which is forced outwards, allowing rest of corner to slide down along the joints.
Entire Face - Wall	Whole height of face along wall – slip mass will have horizontal wedge or semi-circular form	Entire slope fails by planar if circular failure.

The builder was concerned as to continued fall of scat and small and medium wedge and block failures. It was determined early in the construction to begin "counting" and classifying all falls and determining a probability of rock fall by reference to Table C7 (Barneich et al 1996) as provided in the AGS Guidelines 2007. By the physical counting and recording, the size and number of falls, a risk assessment was performed taking into account the probability and consequence of the rock fall.

The following probabilities were assigned, based on the counting and recording.

Scat 1.0 Approx 10/year
Small block/wedge $5x10^{-1}$ Approx 2/year
Medium block/wedge $1x10^{-3}$ Approx 1/50 years
Large block/wedge $5x10^{-4}$ Approx 1/50 years
Entire section of face Approx 1/100 years

A risk assessment was done for both during construction and in use. I will focus on the during construction only.

Using the AGS Guidelines an analysis of probability and severity consequence was carried out using Temporal Spatial Probability methodology.

$$R\,(_{L}\text{ of }_{L}) = P_H \times P(_{S:H}) \times P(_{T:S}) \times V(_{D:T}) \qquad (1)$$

Where;

$R_{(L\ of\ L)}$ is the risk of loss of life (annual probability of death of an individual).

P_H - Annual probability of a rock slide. The size of a rock fall can be classified into five (5) main sizes; scat, small block/wedge, medium block/wedge, large block/wedge, entire face.

$P_{(S:H)}$ - The probability of spatial impact of the rock fall impacting on an individual.

$$P(_{T:S})\ V(_{D:T}) \qquad (2)$$

Taking into account the travel distance and travel direction, it was assumed that a rock fall of the scat and small block/wedge size will occur within a distance of 1.5m from the base of the wall. When this distance was considered in combination with the locations and extent of walls that are faced by pre-cast panels, this area was calculated to be approximately 0.05 of the total work area.

Hence for scat and small block wedge falls,

$P_{(S:H)} = 0.05$

Taking into account the travel distance and travel direction, it is assumed that a rock fall of the medium block/wedge size and large block/wedge size will occur within an area of 5.0m from the base of the wall. When this distance is considered in combination with the locations and extent of walls that are faced by pre-cast panels, this area is calculated to be approximately 0.17 of the total work area.

Hence for medium block/wedge size, large block/wedge size falls,

$P_{(S:H)} = 0.17$

Taking into account the height of the wall and travel direction, it is assumed that a rock fall which includes the entire face falling, will occur within an area of 10.0m from the base of the wall. When this distance is considered in combination with the locations and extent of walls that are not faced by pre-cast panels, this area is calculated to be approximately 0.33 of the total work area.

Hence for the entire face size falls,

$P_{(S:H)} = 0.33$

$P_{(T:S)}$ - The temporal spatial probability of the area being occupied by an individual.

$V_{(D:T)}$ The vulnerability of the individual (the probability of loss of life of the individual, given the impact).

Based on the assumption that works will be carried out during the day for a period of eight (8) hours per day, five days per week, and based on the assumption that at any one time 2% of the area at risk actually contains workers, the temporal spatial probability is

$$P_{(T:S)} = (8/24) \times (5/7) \times (2/100) = 0.005 \qquad (3)$$

The vulnerability of the individual is based on the size of the rock fall (The values assigned for the probability of loss of life).

4.2 Risk Estimation

In adopting the Temporal Spatial Probability methodology in accordance with the AGS 2007 guidelines, the following Risk Matrix is considered appropriate, as represented in Table 2.

Table 2: Risk Matrix for Site Personnel Wearing the Correct Personal Protective Equipment During the Construction Phase

Assigned Probability	**Size of Rock Fall**				
	Scat	**Small Block/Wedge**	**Medium Block/Wedge**	**Large Block/Wedge**	**Entire Face**
P_H	1.0	5.0×10^{-1}	1.0×10^{-3}	5.0×10^{-4}	1.0×10^{-4}
$P_{(S:H)}$	0.05	0.05	0.17	0.17	0.33
$P_{(T:S)}$	0.005	0.005	0.005	0.005	0.005
$V_{(D:T)}$	0.01	0.5	1.0	1.0	1.0
$R_{(L\ of\ L)}$	2.4×10^{-6}	6.0×10^{-5}	7.9×10^{-7}	3.96×10^{-7}	1.58×10^{-7}

It could be said the risk under the worst case is the sum of the above risks.

$$R_{(L\ of\ L)} = R_{(L\ of\ L\ Scat)} + R_{(L\ of\ L\ Small\ Block/Wedge)} + R_{(L\ of\ L\ Medium\ Block/Wedge)} + R_{(L\ of\ L\ Large\ Medium\ Block/Wedge)} + R_{(L\ of\ L\ Entire\ Face)} \quad (4)$$

$$= 2.4 \times 10^{-6} + 6.0 \times 10^{-5} + 7.9 \times 10^{-7} + 3.96 \times 10^{-7} + 1.58 \times 10^{-7}$$

$$= 6.37 \times 10^{-5}$$

Therefore from a simple rock counting exercise and using the AGS Guidelines, a risk to construction workers and, in the future, the general public could be determined. I understand this monitoring still continues by others for the purpose of occupancy.

5 ERNEST ROAD, KALORAMA (DANDENONG RANGES)

5.1 Geology & Background

This site within the Dandenong Ranges is underlain by Devonian Ferny Creek Ryodacite of the Mount Dandenong Volcanics Group. Weathering has typically resulted in shallow surface residual silty clays and sandy silts overlying weathered rock at depth. There are numerous reported smaller landslides identified in this valley area.

- Barbers Road (1934)
- Olinda Creek Road (date unknown)
- Rosemount Crescent (1934-1982)
- Tourist Road (date unknown)
- Woodhurst Grove, North and South (Late 1800s)

The site ranges in slope between 15-20° and was relatively heavily vegetated.

5.2 The Problem

The issues at this site were twofold to firstly provide a foundation arrangement for the residence, but most importantly to construct a debris flow barrier as indicated on Figures 9 and 10 in the uphill zone was subject to localised shallow instabilities and debris flow.

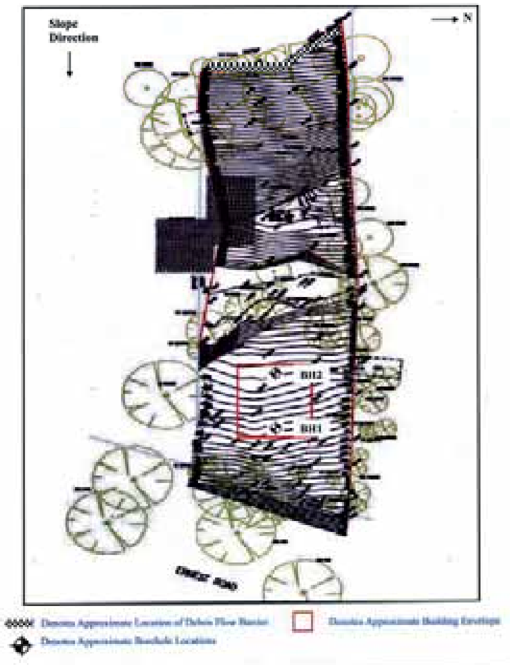

Figure 9. Site Layout

Figure 10. Debris Flow Barrier (Geobrugg)

5.3 The Solution

Geobrugg were contracted to design and construct a debris flow barrier based on a flow volume of 400m^3, being the predicted volume of shallow instability “uphill”, three (3) surges and material density of their solutions is summarised by the following Figures 11 and 12.

The design philosophy was simply based on the velocity, volume and number of surges and a fence of sufficient strength provided.

This example is included in that the “uphill” instabilities are often ignored.

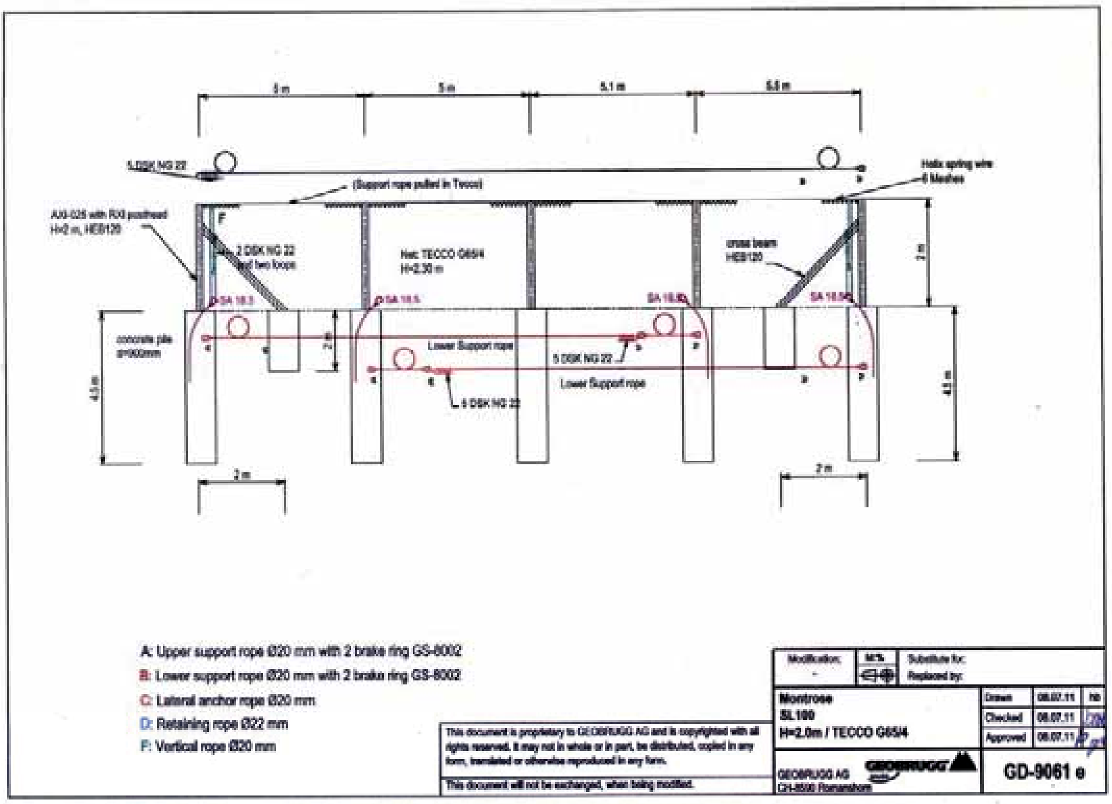

Figure 11. Typical Details Debris Flow Barrier

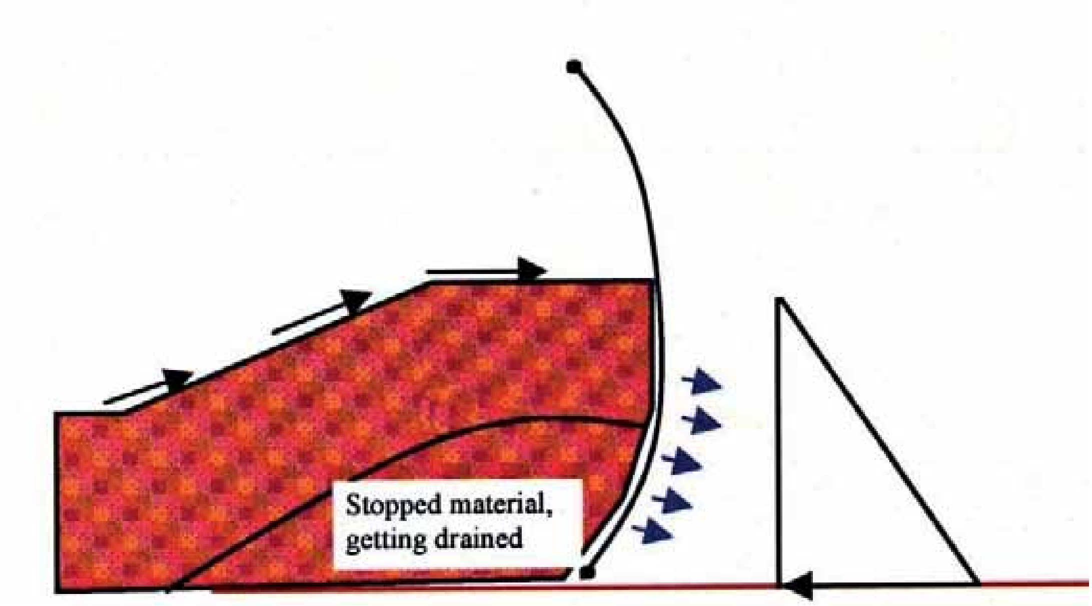

Figure 12. Design Concept

6 CONCLUSIONS

Slope instability and associated hazards cannot be fully avoided. However, impact from the hazard to the proposed developments can be significantly minimised by recognising the possible potential hazard and adopting appropriate footing arrangements and precautions, together with suitable earthworks/retention systems to ensure the integrity of the structure and lives.

The use of the risk assessment is a useful tool to show some practical applications.

7 ACKNOWLEDGEMENTS

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I also acknowledge the structural engineers associated with the above projects, in particular John Fitzgerald, Matt Anderson, Peter Felicetti and Geobrugg.

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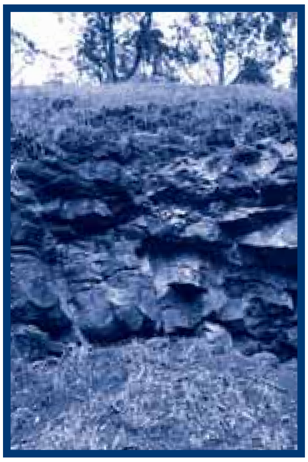

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