INNOVATIVE SLOPE ENGINEERING – VICTORIA – AUSTRALIA

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ABSTRACT

In Australia since mid 2002, the Australian Geomechanics Society has provided Geotechnical Engineers with landslide risk assessment and risk management strategies [1] and we have been involved in some significant examples of excellence in hillside engineering, applying bold investigative techniques, analytical techniques and solutions to a number of prominent residential developments within Victoria and Melbourne.

With increased affluence in our society, consumers are wishing to build their palaces in places not previously thought advisable and sometimes in relatively highly developed areas.

These developments present significant challenges to practising engineers in applying a combination of solutions and endeavouring to assign risks in accordance with the guidelines.

A number of our significant case studies are presented as a practical application of these guidelines and the way they are being interpreted by practising engineers.

Case studies and innovative solutions are presented for a site at Moreley Avenue, Wye River (South West of Melbourne - a slip area in the Otway Ranges) sites on the Mornington Peninsula South of Melbourne namely: Spindrift Avenue, Flinders (sites with a number of historical landslides and having an irregular slope of 15^0-45^0), Portsea (a 20 m high escarpment with a slope of 45^0 with seismic shocks originating from nearby fault in Port Philip Bay) and Nepean Highway, Frankston South (a site with a long history of slope instability with a slope of 40 degrees).

1 INTRODUCTION

With the desirability of land with a view people are tending to build their houses in places with unstable terrains. Some of these areas have undergone earth slips in the past and /or are subjected to ongoing slope movements. This has resulted in the need to find optimum solutions to overcome/minimize landslide risks.

A S James Pty Ltd, Geotechnical Engineers, have been involved in many slope stability projects providing investigation services and solutions to large number of slope problems for residential developments in Victoria, Australia.

This paper demonstrates the approach adopted to provide appropriate solutions with the application of Australian Geomechanics Society (AGS) guide lines [1] for landslide risk management and risk management strategies illustrating five case studies.

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Figure 1: Location of the study sites.

2 GEOLOGY GEOMORPHOLOGY AND BACKGROUND

2.1 MORELEY AVENUE, WYE RIVER

Wye River is one of a series of popular holiday destinations along the Great Ocean Road in the Otway Ranges, Victoria.

Significant uplift of the Otway ranges has occurred following the change in regional tectonic stress associated with the collision of the Australian plate and the Pacific Plate approximately 15 Million years ago. Equally significant in the evolution of the landscape within the region are the fluctuations in the sea level over the past million years that have changed the base level of the rivers and streams.

The Otway Group rocks were deposited in the Lower Cretaceous, about 100 Million years ago. They comprise siltstones and sandstones [2] with these exhibiting significant instabilities over recent geological time. These instabilities are as a result of alterations in sea level, the development of relatively steeper valleys by erosion and slides and more recently man made intervention from sub-division and road construction.

The deposits essentially comprise residual silts and clays overlying the siltstones and sandstones with the depth of clays generally being in the range of 1- 4 metres.

A significant amount of background material has been prepared predominantly on behalf of the Colac Otway Shire [3 & 4] and the following is an aerial photograph showing the slides or areas of instability known to the author or observed. The site under consideration at 35 Moreley Avenue is in a significant slide as indicated again on the aerial photograph.

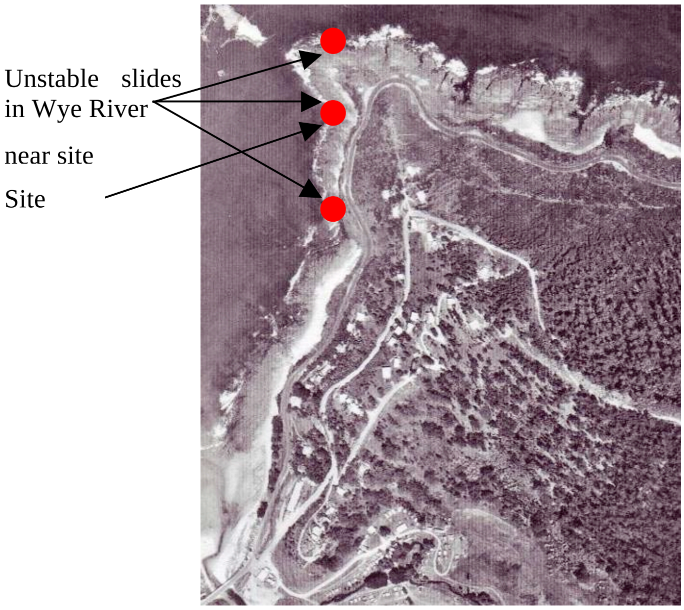

Fig. 2. Aerial Photograph – Wye River

2.1.1 Subsurface profile and parameters used in Slope Stability analysis

For the site under consideration the depth of residual silts and clays was generally of the order of 1.5 metres with a zone then existing to 2.5 – 3.0 metres of extremely weathered siltstone tending to clayey sands. Beyond this depth of 2.5 metres the weathering in the siltstones and sandstones decreased significantly with these being a relatively stable rock mass. The top 2.5 – 3.0 metres on this particular site were assessed as unstable and probably associated with the previous slide.

Rainfall in the Otway Ranges is extremely high typically of the order of 1000mm per year with relatively high variations in temperature. The likely saturation following a prolonged rain event of the surface silts and clays needs to be taken into account.

The following design parameters were used in the slope stability analysis:

In selecting soil strength parameters for the slope stability analysis, the results of previous laboratory tests in the adjacent sites and experience in assessing the soil properties in the general area were considered.

- Silty/sandy Clay : C' = 5 kPa, **Ǿ'** = 25°
- Extremely Weathered Sandstone : C' = 5 kPa, **Ǿ'** = 40°

2.2 SPINDRIFT AVENUE, FLINDERS

Flinders is a coastal town on Westernport on the Mornington Peninsula South of Melbourne. Many of the slopes prone to instability within the Southern Mornington Peninsula are underlain by tertiary or older volcanics [5]. Weathering of the basalt has typically resulted in shallow, surface residual silts underlain by firm to very stiff residual clays, which grade to variably weathered basalt at depths. The residual clays are generally highly reactive and the depth to rock is often highly variable over short distances.

A number of historical landslides subject to ongoing creep movement are known to exist in the local area along Spindrift Avenue. The topography of the site, and local area, is indicative of past landslide activity with hummocky ground surfaces and the remnants of relic back scarps evident. Many trees exhibit curved trunks indicative of hillside creep slide activity.

Spindrift Avenue in Flinders is a little known secluded road with panoramic views over Westernport demanding extremely high prices and prone to significant instabilities.

Some years ago recognizing creep and instability were occurring in 2001 inclinometers were installed along Spindrift Avenue (refer figure 3) and these are now showing cumulative displacement of approximately 4 mm at the surface per year, grading to displacements some 50% to 60% of this at a depth of 6 metres [6].

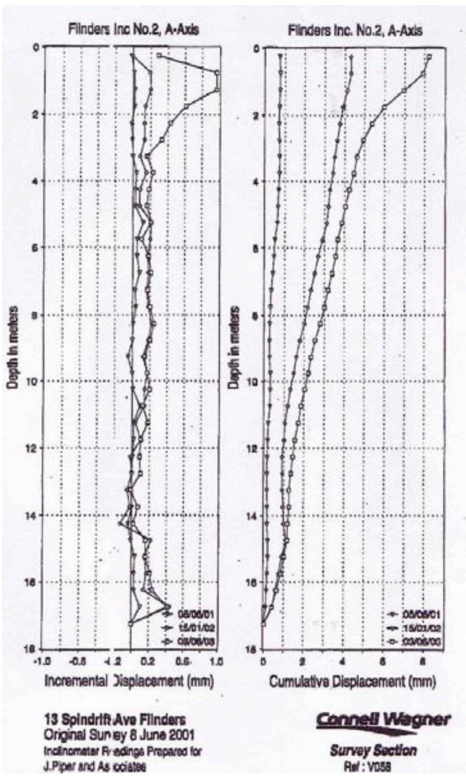

Fig. 3: Inclinometers – Spindrift Avenue, Flinders

Vegetation while largely removed was an indicator of the movement (refer Figure 4, taken in 2005) showing in the background pines less susceptible to movement and in the foreground pines clearly moving with the instability.

Fig. 4. Spindrift Avenue, Flinders

Two sites have been considered and the solutions to these two sites have differed significantly due to the risk assessments. One site located on the beach, but significantly elevated with a low risk to life or property within the foreshore precinct in front of the residence, was indicated in the risk assessment as having no need for significant protection works for shallow failures in this zone. The second approach had another property between the one under consideration and the beach and hence the risk assessment beyond the property line needed to also be considered.

2.2.1 Subsurface profile and parameters used in Slope Stability analysis

Subsurface profile of the subject site comprised residual basaltic clays overlying a fill. The clay was of high plasticity and of very stiff consistency extending approximately down to a depth of 14.5m and underlain by extremely weathered basalt of low rock strength.

Laboratory moisture content and Atterberg Limit tests were carried out on silty clay. Results indicate that the liquid limit is varying in the range of 80 to 170% and the plasticity Index in the range of 45 to 115%. This reveals that the clays are highly to extremely reactive with respect to shrink and swell behaviour.

The following parameters shear strength parameters were used in the slope stability analysis.

- High Plasticity Clay : C' = 10 kPa, **Ǿ'** = 21°
- Extremely Weathered Basalt : C' = 15 kPa, **Ǿ'** = 21°
- (Highly weathered basalt) Distinctly weathered Basalt : C’ = 15kPa, **Ǿ'** = 35°

It is seen that the movement of the residual clays and extremely weathered basalt over the more competent basalt rock at depth is the most prominent failure criterion. The mechanism of movement in this area is a creep movement rather than a “Slip Circle”.

2.3 PORTSEA

Portsea is a popular holiday spot located at the tip of the Mornington Peninsula between Port Philip Bay and Bass Strait, South of Melbourne.

The Point Nepean landscape, where the site under consideration is located, has developed from the dynamic movement of sea levels and erosion forces. The Bridgewater Formation was formed during the ice age about 18,000 years ago and the sea level during that time was about 100 m lower than it is today. Due to the repeated changes in sea levels and climate the Bridgewater Formation comprises a mix of marine sediments (mostly sand) and windblown sands. These materials have been variably cemented by local solution and re- precipitation of the calcium carbonate.

The surface geology of the Portsea area is known as Bridgewater Formation, which is Pleistocene sand. This formation comprises well bedded, variably weathered limestones, weakly cemented sandstones to loose sand. The Bridgewater Formation is overlain by windblown sands.

Point Nepean Road where the proposed site is located travels virtually along the foreshore of Port Philip Bay. The proposed site is gently sloping but the 20m high escarpment to the northern end of the property which is extending to the beach level has a slope of 30^0 on the lower part and 40^0-45^0 on the upper part of the escarpment.

Portions of the escarpment were of marginal stability with evidence of ongoing erosion and fretting at a number of locations on the face of the escarpment. Though there was no evidence of ground movements remote from the escarpment, erosion and minor slumping was expected, especially following prolonged rains and the surface material will become wet and unstable. The area is also subject to limited seismic activity [2].

2.3.1 Subsurface profile and parameters used in Slope Stability analysis

Boreholes indicated the anticipated subsurface soil profile with medium dense silty sand being underlain by dense to very dense sand of a fine grading. Limestone bands were also present within the sand layers.

The following effective stress parameters were adopted for the analysis.

- Medium dense sand: C' = 0 kPa, Ǿ' = 30°,
- Dense silty sand : C' = 0 kPa, Ǿ' = 32°
- Very dnse sand : C' = 0 kPa, Ǿ' = 35°

Based on the Slope Stability Analysis, it is understood that critical failures are obvious near the surface of the escarpment.

2.4 NEPEAN HIGHWAY, FRANKSTON SOUTH

"Olivers Hill" - Frankston South has magnificent bay views and is situated in the Mornington Peninsula, Melbourne.

The surface geology of Olivers Hill consists of tertiary sedimentary deposits and form part of the Baxter sandstone formation [2] and has a long history of slope instability with major instabilities.

Typically the tertiary sedimentary deposits consist of clayey sand, sandy clay, ferrugnised sandstone and occasional gravel. At highly variable depths the Baxter Sandstone is known to be underlain by Balcombe Clay, which is also of the Tertiary age and forms part of Fyansford formation. The occurrence of the Balcombe Clay is variable, due to both heaving along the beach front, associated with previous landslides and significant undulations within the pre-existing Miocene topography, which underlies the Balcombe Clay. Balcombe clay is highly plastic and "slippery" material and saturation of this material caused many landslides in the general area.

The weathered granite, which underlies the Balcombe Clay, is of the Devonian age and, typically, the granite is very highly weathered to significant depths underlying the subject site.

The proposed site located at Nepean Highway is almost flat and is located on a plateau that extends to the west from Nepean Highway. The site has a mild slope of 7 degrees towards the escarpment located a few meters to the west of the property boundary and the escarpment drops steeply down at angle of approximately 40 degrees.

Ground movements have occurred to the west of the escarpment with major landslides having occurred prior to 1900 and in 1970 and again in 1973. Since then the land at the base of the escarpment has been creeping at significant rates. The instabilities were almost certainly caused by saturation of the Balcombe Clay, which underlies this area at variable depths.

During the period from 1995 -1999, a series of deep cut-off drains had been cut to an approximate depth of 4.0 - 6.0 metres below the existing ground surface level at this site. After the installation of the drains, it is understood that significant flows of seepage water were being intercepted by the drains and flowed from the drains at the points where they discharge onto the beach. Also it is indicated that the creep movements have been reduced since the drains were installed. The potential for movement is also thought to be diminished due to the installation of a sewerage system 20 years ago minimizing the discharge in this area.

2.4.1 Subsurface profile and parameters used in Slope Stability analysis

Subsurface profile consists of moderately reactive clays underlain by dense clayey sands. These layers then graded to Baxter Sandstone clay which comprises silty clay and clayey sands with ferruginised sandstone gravel. Anticipated Balcombe clay was not found within the investigated depth. However, slope stability analysis was modeled incorporating the Balcombe clay and underlying granite.

Clay samples collected at different depths were tested for Atterberg Limits. Results indicated that the liquid limits are varying in the range of 30 - 60% and the Plasticity Index in the range of 18 - 43%.

The following effective stress parameters were adopted for the analysis based on drained testing in the immediate area.

- Sandy Clay : C' = 35 kPa, Ǿ' = 20°
- Clayey Sand : C' = 10 kPa, Ǿ' = 36°
- Silty Clay (Baxter Sandstone Clay) : C' = 20 kPa, Ǿ' = 24°

- Extremely weathered granite : C' = 0 kPa, Ǿ' = 42°

3 SOLUTIONS

It has been the experience of the authors with many existing sites in the regions shown that inadequate treatment is given to the potential lateral forces generated by potential slides. Deep foundation arrangements are simply being adopted that extend the foundations to stable horizons rather than improving overall factors of safety by contributing a lateral capacity.

With the adoption of the Australian Geomechanics Society landslide risk assessment and risk management strategies [1] there is increasing emphasis on improving overall site factors of safety and in many cases overall area factors of safety in slope prone zones.

On this basis it becomes inevitable to improve factors of safety and for all of the sites under consideration a zone or block of soil has been reinforced, strengthened or anchored to increase factor of safety to approximately 1.5.

A brief description of the solutions of each of the sites is outlined below.

3.1 MORELEY AVENUE, WYE RIVER

For this site a block of soil was reinforced by a series of closely spaced piers designed for the lateral at rest pressures that would be mobilized in the longer term against the block. This site had the advantage of a relatively shallow depth to competent rock and the lateral forces could be transferred by frame action and transferred to the rock. Decisions as to soil arching between piers/piles governed the spacing of the piles. The overall site stability was assessed using GEO-SLOPE - SLOPE/W software with the overall site stability increased as consequence of the block of soil to acceptable levels in accordance with the AGS guidelines [1].

The following photographs are indicative of the solutions at this site which is the simplest of the solutions in this paper.

Fig. 5. Solution Concept – Moreley Avenue, Wye River

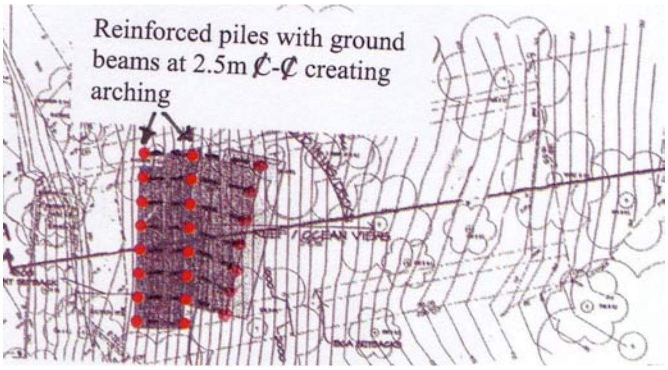

Figure 6: Pier Reinforcement – Moreley Avenue, Wye River

3.2 FLINDERS, SPINDRIFT AVENUE

3.2.1 Site 1 - Flinders

On this site where significant improvement was required to the potential failure plane a frame action was developed within the foundation arrangement capable of withstanding the "active" and "at rest" pressure distribution indicated on Figure 7.

This pressure distribution takes into account some frame yield and therefore active state pressures but due to the significant movements over the surface residual profile "at rest" pressures were assumed on the upper portion of the frame.

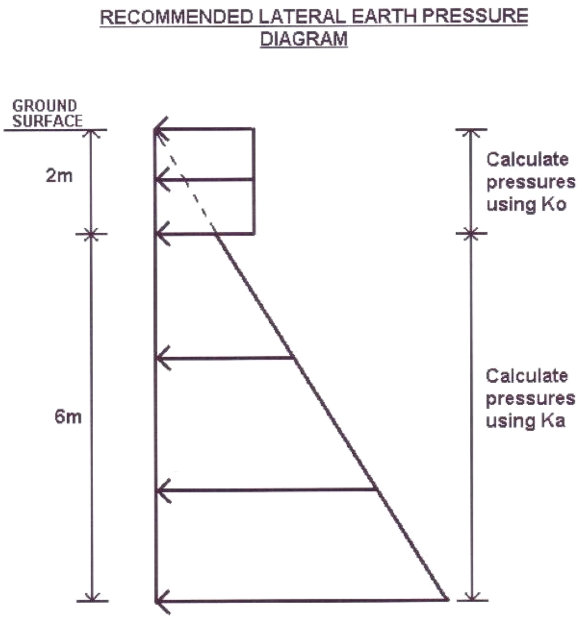

Fig. 7. Lateral Earth Pressure Distribution

It was seen that the lateral forces on the foundation arrangement, which was bored piles socketed into the underlying weathered basalt, were large. A framework of bored piles and ground beams to withstand these lateral forces was installed to agreed arrangement. This in effect created a buttress with the bored piers placed sufficiently close to ensure soil arching between soldiers. These piers were required to some 18m in a concept similar to that for Wye River.

3.2.2 Site 2 - Flinders

For the second residence in Spindrift Avenue where the risk assessment was influenced by the proposed residence further down the slope a retaining wall and ground anchors solution was adopted to increase the overall factor of safety of the escarpment.

Anchors were used with modeling software resulting in the overall factor of safety of the escarpment being increased from marginal factors of safety of between 1 and 1.1 to factors of safety approaching 1.5. With a factor of safety of 1.5 the escarpment was considered by the risk assessment as acceptable or tolerable.

The following analytical sections are illustrative of the approach adopted with anchors extending beyond the failure plane by a suitable development length. Anchors were spaced at 1.0m of 500kN S. W. L. and some 20-25m long.

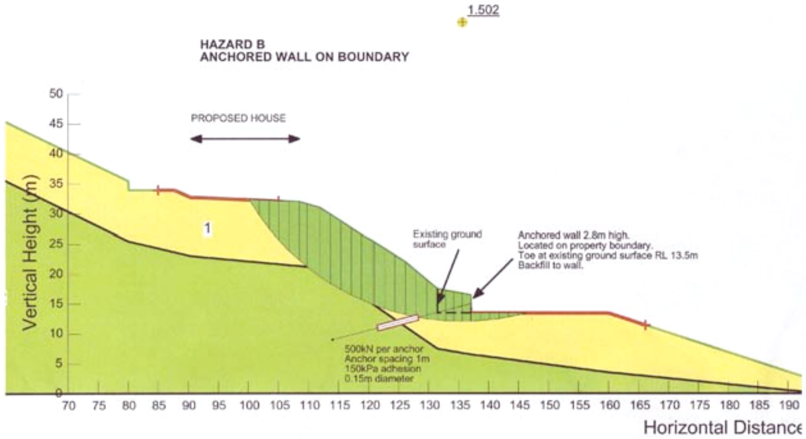

Figure 8: Concept of the Approach Adopted

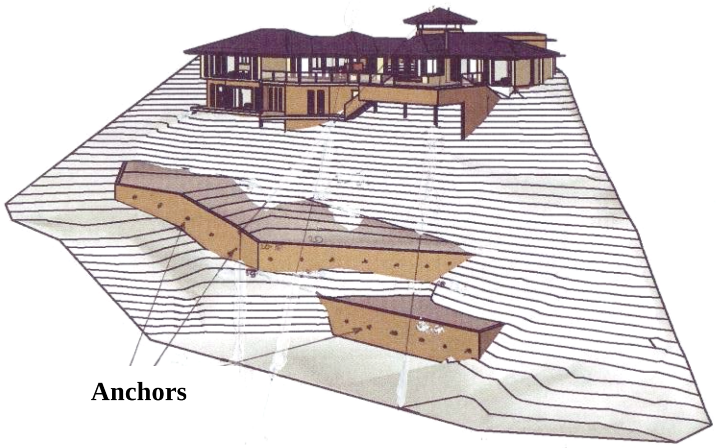

Figure 9: Typical Analytical Section

In both of these sites a risk assessment prior to development would indicate unacceptable outcomes. The adoption of stiffened foundation arrangements enables a revised risk assessment to life and property to be carried out that has an acceptable outcome both for the development of the sites and for overall regional stability. The adoption of similar arrangements along this secluded road should progressively significantly minimize risk to all properties along this valuable and picturesque peninsula area.

3.3 PORTSEA

Slope Stability Analysis was carried out for the shallow translational and deep seated failures for the slope before any construction and after the excavation for basement. Subsequent construction of the building indicated unstable to marginally stable conditions (refer Figure 10).

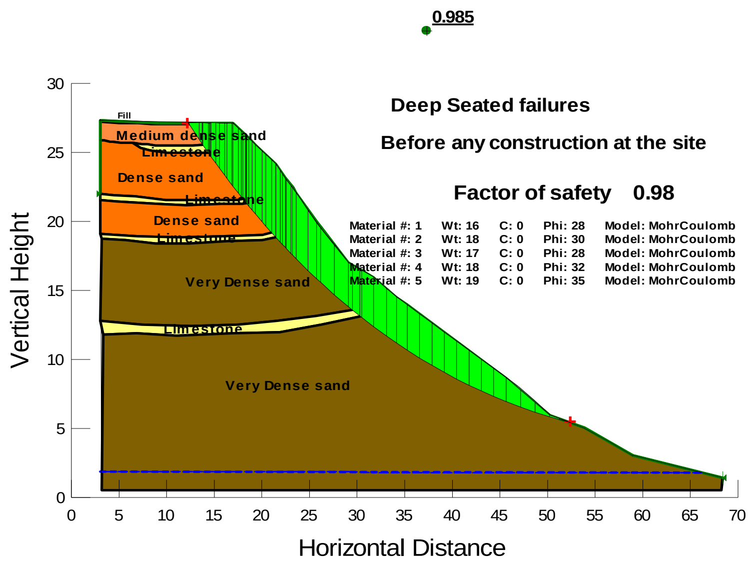

Fig. 10. Slope Stability Analysis

In view of the potential instability of the escarpment and the depth of the medium dense sand layers, bored - Continuous Flight Auger (C.F.A) piles down to the dense sand layer were adopted. Minimum founding depths were based on the row of piles at the top of the escarpment being able to withstand the lateral earth pressure forces associated with the "active" failure condition. These piles were structurally tied back into the portion of the proposed building located well remote from the escarpment (refer Figure 11).

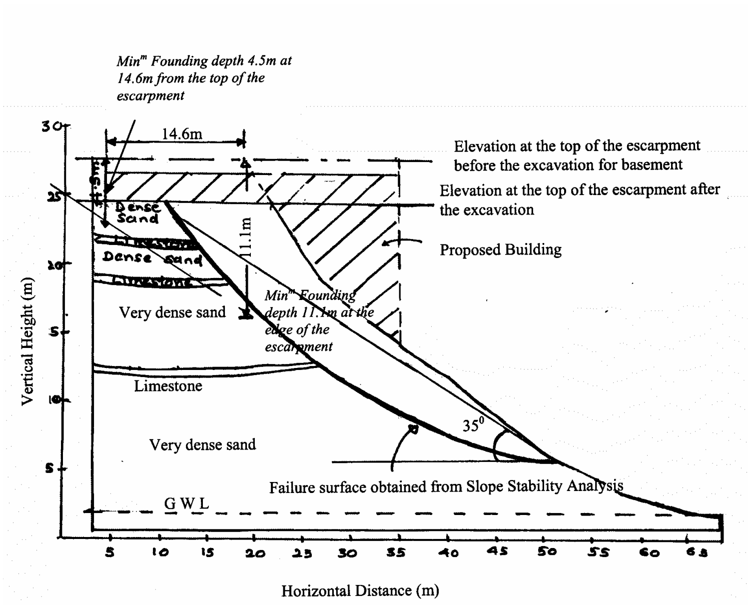

Figure 11: Minimum Founding Depth

Construction beyond the escarpment needed extreme care to prevent disturbance of the soils on the face of the embankment and thus safe lateral distance requirements for temporary piles were to be considered. The outermost piles were designed as propped cantilevers with sufficient embedment at the base of the escarpment.

3.4 NEPEAN HIGHWAY, FRANKSTON SOUTH

Shallow translational and deep seated failures were analyzed and indicated marginal factor of safety of 1.1-1.2 for both shallow translational and deep seated failures (refer Figure 12). It was understood that there is a tendency for deep seated slips due to tertiary clays and anticipated seismic activity, but more risk exists for shallow movements due to moisture build up.

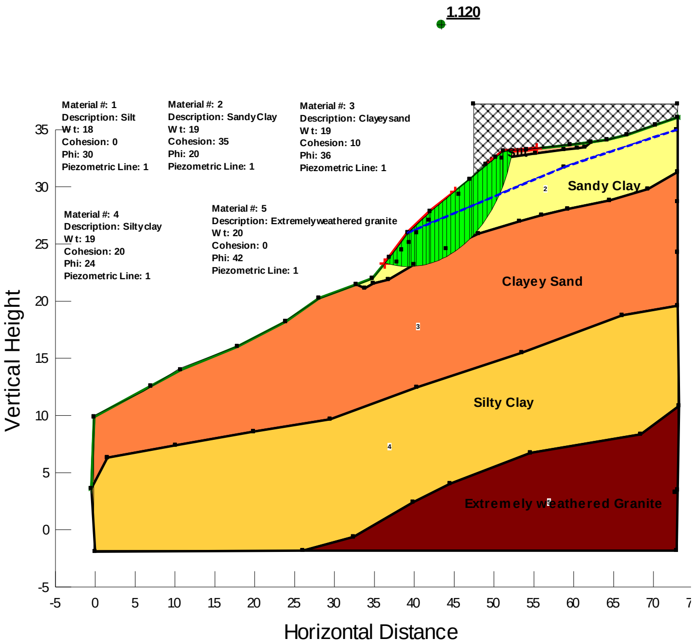

Figure 12: Slope Stability Analysis

Footings were needed to be constructed to a dense sand layer and to a depth obviously below the potential deeper seated failure planes to ensure the soil creep and instability did not influence considerations. Bored pier arrangements with a series of piers were adopted and the design was based on the bored piers being able to withstand a lateral load against the active failure condition. For the proposed house location it was 8 m (refer Figure 13).

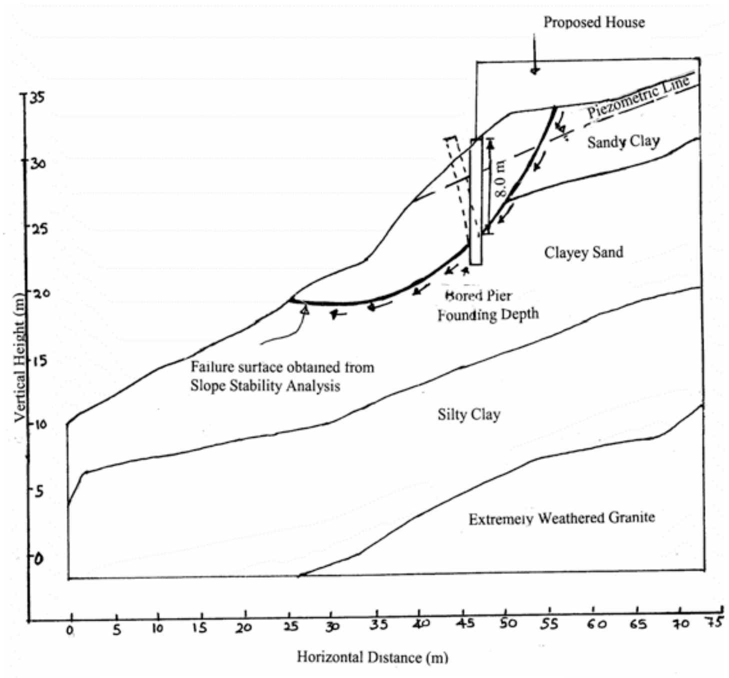

Fig. 13. Bored Pier Founding Depth

Pre-construction risk assessment indicates unacceptable risk level for both shallow translational and deep seated failures. The adoption of deeper footings and providing proper drainage to reduce the shallow water impacts from the above mentioned hazards is reduced and acceptable risk level is achieved for the proposed development.

4 RISK ASSESSMENT

Slope instability impacts on housing, roads, railways and other developments. This has been recognized by many local government authorities and others and has led to preparation of guidelines for landslide risk management. Australian Geomechanics Society (AGS) provided Geotechnical Engineers with landslide risk assessment and risk management strategies and we have adopted these guidelines [1] to assess the risk levels and to produce solutions to a number of prominent residential developments within Melbourne and Victoria.

As per the AGS Guidelines [1] for the sites under consideration, landslide risk assessments were made for pre construction and post construction and the risk levels were determined both qualitatively and quantitatively. In most of the sites demonstrated in this paper, deep seated and shallow translational failures are the most common hazards identified.

Shallow translational failures anticipated were mostly due to occurrence of wet and unstable surface residual material following prolonged rains. Past landslide activities with ongoing creep movements of the displaced materials were also considered to be a triggering factor for shallow failures.

Deep seated movements are common to most of the subject sites and are due to the existence of thick layers of tertiary clays, variably weathered Tertiary and Quaternary deposits or cretaceous deposits together with the effects of seismic shocks originating from nearby faults.

Likelihood of the failure mechanisms was assessed qualitatively for pre construction. Assessment of post construction risk levels were performed as per AGS guidelines based on results of Slope Stability Analysis incorporating the appropriate foundation arrangements together with the proper earthworks and retention systems.

A sample of risk assessment performed for one of the sites considered is given below.

Table 1. Risk to property (qualitative) – Post Construction
(Footing Construction to dense sand and below potential deep seated failures)

Hazard		Likelihood	Consequence	Risk
A	Shallow Translational slide or debris creep	Possible	Minor will not affect the structure	Moderate to low
B	Deep seated failure	Unlikely/rare	Major	Low to Moderate

Table 2. Risk to Life (quantitative) – Post Construction
(Footing Construction to dense sand and below potential deep seated failures)

Hazard	A	B
Indicative Annual Probability	10^{-3}	$0.5x10^{-5}$
Probability of spatial Impact	0.1	1.0
Occupancy No. People	0.8	0.8
Vulnerability	0.1 unlikely to be buried due to footing arrangement	1.0
Individual Risk	$0.8x10^{-5}$	$0.4x10^{-5}$
Risk Evaluation	Tolerable/ Acceptable	Tolerable and usually Acceptable

5 CONCLUSIONS

Slope instability and associated landslide hazards which are common to the general area cannot be fully avoided. However, impact from the hazard to the proposed developments can be significantly minimized by recognizing the possible potential hazard and adopting appropriate footing arrangements together with suitable earthworks/retention systems to ensure the integrity of the structure.

The introduction of the risk assessment in Australia is a useful tool however, it is hoped this paper reveals that it will never replace Innovative Engineering.

6 ACKNOWLEDGEMENTS

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