GeoAust
Geotechnical Engineers

PROPOSED RESIDENTIAL DWELLING
14 VIEW POINT ROAD
McCRAE VIC

PREPARED FOR

FASHAM JOHNSON PTY LTD

JOB NO: 1624-9-R

14 SEPTEMBER 2011

DISTRIBUTION:

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1 INTRODUCTION

1.1 COMMISSION

The geotechnical investigation was commissioned by Mr Brian Stacey of Fasham Johnson Pty Ltd. The scope of works was in accordance with our fee proposal with reference 1624-1-Q, dated 24 March 2009.

1.2 PROPOSED DEVELOPMENT

Based on the plan extracts and information provided to us, it is understood that the proposed development at 14 View Point Road, McCrae, comprises the demolition of the existing dwelling at 16 View Point Road and construction of a new residential dwelling on the currently vacant portion of the site at 14 View Point Road.

The new dwelling at 14 View Point Road is proposed to be located at the top of an escarpment which has an approximate relief of 23 metres. Based on the plans of the proposed dwelling provided to us prepared Fasham Johnson, dated 8 February 2010 (Sheets 1 – 3) it is understand that the main portion of the dwelling will comprise a two level structure, which largely cantilevers out over the top edge of the escarpment. Two site cuts are proposed to accommodate the main portion of the dwelling. The details of the proposed site cuts are as follows:

- A 2.5 – 3.0 metre deep site cut is proposed to provide a benched level at RL 23, which will enable installation of bored piles at the outer (north) edge of the site cut. The piles will provide support to the proposed dwelling and retention of the upper section of the escarpment against landslide.
- A further 2 metre deep site cut is proposed to provide a benched level at RL 25, which will accommodate the lower level of the main structure.

To the south of the main section of the proposed dwelling a ground level garage with a single level of living space over is proposed. The garage will be constructed in a maximum 1.0 metre deep cut located behind the top edge of the escarpment. The proposed floor level of the garage is RL 26.65.

The precise structural details of the proposed structure were not known to us at the time of issue of this report. It is assumed that structural loads will be typical of residential construction and that no unusual performance criteria apply to the proposed structure.

1.3 PREVIOUS REPORTS

The following previous reports have been prepared by GeoAust for the proposed development at the subject site:

- Geotechnical Report with reference 1624-2-R dated 18 August 2009. The report assumed that the existing dwelling at 16 View Point Road will be retained and incorporated into the proposed development.

- Geotechnical Report with reference 1624-7-R dated 27 May 2011. The report was based on a revised design for the proposed development, including demolition of the existing dwelling at 16 View Point Road.

The comments and recommendations contained within the previous reports have been superseded by the comments and recommendations contained within this report. This report contains a number of amendments in response to a letter issued by Mornington Shire Council dated 15 August 2011.

1.4 GEOLOGY

Reference to the Geological Survey of Victoria, 1:63,360 series, Sorrento sheet indicates the site to be underlain by Devonian aged granodiorite. Weathering of the granodiorite has typically resulted in a deeply weathered profile comprising residual clay and sand grading to extremely weathered granodiorite.

The escarpment which intersects the property has a history of instability. The Mornington Peninsula Shire Council has identified the subject escarpment to be located within a zone of landslide risk.

The instability is as a result of the steepness of the escarpment, combined with uncontrolled flows of seepage water. Instability of the escarpment can typically range from long term creeping of the escarpment face, through to a large scale failure, which can occur almost instantaneously. Examples of both types of failure are evident in the immediate area.

2 INVESTIGATION METHODS

2.1 FIELD METHODS

Fieldwork was completed under the direct supervision of a qualified Geotechnical Engineer from GeoAust on 17 and 18 June 2009 and included the following.

2.1.1 Borehole Drilling

Three boreholes were drilled to depths ranging between 1.5 and 25 metres below the existing ground surface at the approximate locations indicated in Figure 1. Borehole 1, which was located adjacent to the top edge of the escarpment, was drilled using a track mounted Pioneer P160 rotary drilling rig equipped with 115 millimetre diameter solid, flighted augers. Boreholes 2 and 3 were drilled on the face of the escarpment. Due to restricted site access Boreholes 2 and 3 were drilled using portable hand auger equipment.

Bore logs were prepared in accordance with Australian Standard AS 1726-1993 'Geotechnical Site Investigations'. Definitions of the logging terms and symbols used are provided in Appendix A and the logs of the boreholes are provided in Appendix B.

2.1.2 In-situ Testing

Testing was carried out in accordance with the relevant test procedures in Australian Standard AS 1289, 'Methods of Testing Soil for Engineering Purposes' and included the following:

- Standard penetration testing (SPT).
- Vane shear strength testing of cohesive soils.

Test results are included on the logs of the bores.

2.1.3 Ground Water Monitoring Standpipe

A 50 millimetre diameter PVC ground water monitoring standpipe was installed in Borehole 1 to a depth of 20.5 metres below the existing ground surface. The standpipe was cased to 8.5 metres depth and screened below this depth. A bentonite seal was provided at the base of the casing. Results of groundwater monitoring are provided in Section 3.3.

3 RESULTS OF INVESTIGATION

3.1 SITE DESCRIPTION

The following site features were noted at the time of the field work:

- The subject site was situated along an escarpment, which sloped steeply down to the approximate north west. The total relief of the escarpment was approximately 23 metres.
- The escarpment was largely vegetated with a small to large shrubs and trees of varying sizes.
- There was an existing single level dwelling at 16 View Point Road, which is proposed to be demolished and removed from the site. The clad framed dwelling was supported on steel columns. Footings providing support to the steel columns appeared to comprise individual concrete pad footings. The details of the pad footings were not known. The section of escarpment beneath the dwelling comprised bare earth, which appeared, in part, to have been subject to erosion, possibly as a consequence of leaking pipes and/or uncontrolled stormwater runoff over the top edge of the escarpment.
- There was no obvious evidence of any recent appreciable slope instability at the site. However it was apparent that the surface soils had been subject to ongoing creep movements. The creep movements typically occur within the near surface colluvial soils on the face of the escarpment.
- There were no obvious signs of seepage water or springs on the face of the escarpment at the subject site.
- There was evidence of a significant landslide approximately 40 metres to the east of the subject site at 6 View Point Road, McCrae. The circular slip was estimated to have a depth of approximately 6 metres and a width of at least 25 metres. The back scarp was located several metres behind to former top edge of escarpment. The toe of the slide was not immediately apparent from the subject site, but appeared to be towards the base of the escarpment. The vegetation within the area of the slide indicated the presence of seepage water. No such vegetation was present adjacent to the failed section of the escarpment or at the subject site.

3.2 SUBSURFACE CONDITIONS

The logs of the boreholes are provided in Appendix B.

Bore 1 located adjacent to the top edge of the escarpment at 14 View Point Road intercepted some 3.1 metres of medium dense silty sand, underlain by silty and clayey sand, which was very dense.

The very dense silty and clayey sand contained trace quantities of fine grained granodiorite gravel. At a depth of 7.5 metres a 1.5 metre thick band of clay, which was of medium plasticity and hard consistency, was intercepted. The clay was underlain by fine to medium grained silty sand, which was very dense. The silty sand contained bands of high plasticity clay, which were of very stiff consistency, at depths of 12 and 15 metres below the existing ground surface. The clay layer at 12 metres was approximately 2.0 metres thick and the clay layer at 15 metres was approximately 1.0 metre thick. The silty sand at depths in excess of 16.5 metres was dense to very dense. The very dense to dense silty sand persisted to depths in excess of programmed termination depth of 25 metres below the existing ground surface.

Boreholes 2 and 3, which were drilled using portable hand auger equipment, intercepted approximately 1.0 metre of colluvium. The colluvium comprised fine to medium grained silty sand, which contained trace quantities of fine to coarse grained granodiorite gravel and was of medium relative density and to a lesser extent medium plasticity clay, which was of very stiff consistency. The colluvium was underlain by fine to coarse grained clayey and silty sand, which was dense. Effective hand auger refusal was encountered on the dense sand at depths of 1.5 and 3.4 metres in Boreholes 2 and 3 respectively.

The approximate 1.0 metre depth of colluvial soils intersected in Boreholes 2 and 3 are likely to be subject to creep movements on the face of the escarpment. Creep movements are extremely slow movement of the soil mass as a consequence of gravitational forces.

3.3 GROUND WATER

No ground water seepage was intercepted within Boreholes 1 - 3 during auger drilling of the boreholes. The introduction of water for rotary wash boring at depths in excess of 4.5 metres negated any further meaningful observation of water levels and inflow rates during drilling in Borehole 1.

A slotted 50 millimetre diameter PVC standpipe was installed in Borehole 1 to a depth of 20.5 metres upon completion of drilling to allow monitoring of the ground water level. The standpipe was cased to a depth of 8.5 metres and screened over the lower 12 metres. An annulus filter pack comprising coarse grained sand was provided for the screened length of the standpipe and a bentonite seal was provided at the base of the casing to prevent surface and near surface seepage water flows entering the standpipe.

The following standing water levels were measured within the standpipe:

- 18 June 2009 - 16.5 metres below the existing ground surface (Approximate RL 10.8)
- 24 May 2011: - The standpipe was dry.

The variation in the ground water level was attributed to the fact that the measurement taken on 18 June 2009 was taken only after 6 hours after drilling of the borehole was completed. The ground water level had obviously not stabilised as a consequence of the water being introduced into the borehole during rotary wash boring of the borehole.

On the basis of the ground water standpipe being dry in 24 May 2011 it can be concluded that the ground water table is present at depths below RL 6.8 metres. For the purpose of the assessment of stability of the subject site it has been conservatively assumed that the ground water table is present at RL 6.8 metres.

Whilst not observed at the time of drilling Boreholes 1 – 3, perched ground water seepage may develop within the surface silty sand overlying the less permeable clay and dense to very dense silty and clayey sand following periods of wet weather, particularly during the winter and spring months when rainfall levels are typically high and evaporation levels are low.

4 LANDSLIDE RISK ASSESSMENT

The Australian Geomechanics Society 'Practice Note Guidelines for Landslide Risk Management 2007' have been adopted for Landslide Risk Assessment at the subject site. Extracts from Australian Geomechanics Society (2007) regarding the terminology used in assessing risk are provided in Appendix D.

4.1 DEFINITIONS – DEVELOPED AND UNDEVELOPED CONDITIONS

Assessment of risk has been made based for the undeveloped condition and developed condition defined below.

The ***Undeveloped Condition*** is defined as the site conditions at the time of the field work for this geotechnical investigation.

The ***Developed Condition*** is defined as site conditions described in Section 1.2 in conjunction with all the foundation and slope stabilisation measures recommended in Section 5.

4.2 IDENTIFICATION OF HAZARDS

Hazard A: Collapse of the escarpment on which the dwelling at 14 View Point Road is proposed to be constructed. The occurrence of Hazard A will potentially adversely affect the proposed dwelling at the subject site and its occupants. A circular failure is most likely. The volume of the slide may be in the order of 5000 cubic metres. Failure is likely to be rapid. Saturated conditions are most likely to initiate a failure. Saturated conditions may be brought about by a change in ground water conditions, a leaking service pipe and/or poor site drainage. The landslide which took placed at 6 View Point Road is indicative of the failure which potentially could occur at the subject site. The travel distance of the failed mass of soil is estimated to be in the order of 25 – 40 metres. Rapid movement of the failed mass of soil is anticipated.

Hazard B: Collapse of the section of the escarpment below the proposed dwelling at 14 View point Road. The occurrence of Hazard B will potentially adversely affect the existing dwellings at the south end of the multi dwelling development at 613 Point Nepean Road, McCrae and its occupants. A circular failure is most likely. The volume of the slide may be in the order of 5000 cubic metres. Failure is likely to be rapid. Saturated conditions are most likely to initiate a failure. Saturated conditions may be brought about by a change in ground water conditions, a leaking service pipe and/or poor site drainage. The landslide which took placed at 6 View Point Road is indicative of the failure which potentially could occur at the subject site. The travel distance of the failed mass of soil is estimated to be in the order of 25 – 40 metres. Rapid movement of the failed mass of soil is anticipated.

Hazard C: Creep movements of the near surface colluvial soils on the face of the escarpment. The creep movements are common to the escarpment in the general area. The occurrence of Hazard C will potentially adversely affect the proposed dwelling at the subject site. Based on the soil profiles intercepted in Boreholes 2 and 3, which were drilled on the face of the escarpment, it is estimated that an approximate 1 metre depth of soil on the face of the escarpment is likely to be subject to creep movements. The travel rate of Hazard C is estimated to be extremely slow.

4.3 FREQUENCY OF HAZARDS

Hazard A: Hazard A is considered POSSIBLE (Approximate annual probability of 10^{-3}) as it is may occur within the design life of the proposed development.

Hazard B: Hazard B is considered POSSIBLE (Approximate annual probability of 10^{-3}) as it is may occur within the design life of the proposed development.

Hazard C: Hazard C is considered ALMOST CERTAIN (Approximate annual probability of 10^{-1}). There is evidence of creep movements having occurred on the face of the escarpment both at the subject site and in the general area.

4.4 CONSEQUENCES TO PROPERTY

A qualitative approach has been adopted for assessment of risk to property.

Hazard A: Assuming that no engineering measures are taken to safeguard the proposed dwelling at the subject site, the consequence of a rotational slide occurring on the face of the escarpment to the proposed dwelling is anticipated to be catastrophic. Complete destruction of the proposed dwelling is anticipated. Major engineering works will be required to stabilise the subject site and potentially the adjacent properties after the failure, before reconstruction of the dwelling at the subject site can be carried out. An appropriate descriptor for the consequence to property at the subject site is considered to be CATASTROPHIC.

Hazard B: Consequences to the adjacent property at 613 Point Nepean Road are anticipated to include moderate damage to the units at the south end of the site. An appropriate descriptor for the consequences to the adjacent property is considered to be MAJOR.

Hazard C: Assuming that no engineering measures are taken to safeguard the proposed dwelling at the subject site, the consequence of creep movements to the proposed dwelling at the subject site is anticipated to be moderate. An appropriate descriptor for the consequence to proposed dwelling at the subject site is considered to be MEDIUM.

4.5 SLOPE STABILITY ANALYSIS

Analysis of the stability of the subject site was performed using Galena version 5.02 slope stability analysis software. The analysis considered the stability of Section A-A shown in Appendix C, Figure C-1.

The stability analysis was conducted on a model based on the soil profile intersected in Borehole 1. Each layer of clay intersected in Borehole 1 was included in the stability model, assuming horizontal stratigraphy. The following clay layers were included in the model:

- 1.5 metre thick layer of clay at a depth of 7.5 metres below the existing ground surface.
- 2.0 metre thick layer of clay at a depth of 12 metres below the existing ground surface.
- 1.5 metre thick layer of clay at a depth of 15 metres below the existing ground surface.

Material properties adopted for stability analysis are given in Table 4.5.1:

Table 4.5.1: Material Properties Adopted for Cross Section A-A

Unit No	Material Type	Unit Weight (γ)	Effective Cohesion (C')	Effective Angle of Friction (Ø')
1	Medium Dense Sand	20 kN/m^3	0 kPa	29^0
2	Clay	18 kN/m^3	10 kPa	24^0
3	Dense Sand	21 kN/m^3	0 kPa	36^0
4	Very Dense Sand	22 kN/m^3	0 kPa	42^0

The material properties in Table 4.5.1 were based on the following.

- Published correlations between standard penetration test results and internal angles of friction for granular soils.
- Previous experience in assessing soil properties in the general area.

Selected graphical results of critical stability analyses for the subject site are given in Appendix C, Figures C-2 and C-8.

In considering the results of the analyses it should be noted that a Factor of Safety (FoS) of 1.0 corresponds to the state at which forces driving failure are equal to those resisting failure. A FoS less than 1 indicates failure. A FoS greater than 1.0 indicates that restoring forces are greater than the forces driving failure and that failure has not occurred. Generally a FoS of 1.5 is considered acceptable for development.

The results of the stability analyses are summarised below:

Figure C-2 is the graphical result of the critical stability analysis for the following conditions:

- The existing site conditions at 14 View Street, that is, no earthworks or surcharge loading associated with a proposed dwelling at the top of the escarpment.
- The regional ground water table is present at RL 6.8 metres.
- No earthquake loading

The analysis returned a factor of safety against failure of 1.14, which indicates the escarpment in its existing condition to be marginally stable. This factor of safety is consistent with the anticipated factor of safety for the subject escarpment. Additionally, the shape of the critical failure surface approximately corresponds to the observed shape of the failure which took place at 6 View Point Road.

Figure C-3 is the graphical result of the critical stability analysis for the following conditions:

- The existing site conditions at 14 View Street, that is, no earthworks or surcharge loading associated with a proposed dwelling at the top of the escarpment.
- The regional ground water table is present at RL 6.8 metres.
- Earthquake loading is applied to the model.

The analysis returned a considerably lower factor of safety against failure of 0.98, which indicates the escarpment in its existing condition is likely to collapse in the event of an earthquake, assuming effective stress parameters for the soil profile.

Figure C-4 is the graphical result of the critical stability analysis for the following conditions:

- A tiered site cut, as detailed in the drawings of the proposed development prepared by Fasham Johnson, dated 8 February 2011, at the top of the escarpment.
- The proposed dwelling is supported on shallow footings at the top of the escarpment with a uniformly distributed load of 10 kPa applied to the plan area of the proposed dwelling.
- The regional ground water table is present at RL 6.8 metres.
- No earthquake loading

The analysis returned a factor of safety against failure of 1.17. This factor of safety is unacceptably low and clearly demonstrates the need for significant engineering measures to taken in the development of the site.

Figure C-5 is the graphical result of the critical stability analysis for the following conditions:

- A tiered site cut, as detailed in the drawings of the proposed development prepared by Fasham Johnson, dated 8 February 2011, at the top of the escarpment.
- The proposed dwelling is supported on shallow footings at the top of the escarpment with a uniformly distributed load of 10 kPa applied to the plan area of the proposed dwelling.
- The regional ground water table is present at RL 6.8 metres.
- Earthquake loading is applied to the model.

The analysis returned a considerably lower factor of safety against failure of 1.01, which indicates the escarpment and the proposed dwelling is likely to collapse in the event of an earthquake, assuming effective stress parameters for the soil profile. This again clearly demonstrates the need for significant engineering measures to taken in the development of the site.

Figure C-6 is the graphical result of the critical stability analysis for the following conditions:

- A tiered site cut, as detailed in the drawings of the proposed development prepared by Fasham Johnson, dated 8 February 2011, at the top of the escarpment.
- The proposed dwelling is supported on piled footings and a row of 15 metre deep reinforced piles is provided along the top edge of the escarpment at the north end of the lowest benched area.
- The regional ground water table is present at RL 6.8 metres.
- No earthquake loading.

The analysis, which examined the stability of the face of the escarpment to the north of the row of 15 metre deep reinforced piles, returned a factor of safety against failure of 1.17. This factor of safety is marginally greater than the factor of safety against failure given in Figure C-2 (1.17 c.f. 1.14) which represented the factor of safety against failure for the existing site condition. This is significant in that is confirms that the proposed development will not adversely affect the stability of the face of the escarpment below the proposed development.

Figure C-7 is the graphical result of the critical stability analysis for the following conditions:

- A tiered site cut, as detailed in the drawings of the proposed development prepared by Fasham Johnson, dated 8 February 2011, is carried out at the top of the escarpment.
- The proposed dwelling is supported on piled footings and a row of 15 metre deep reinforced piles is provided at the along the top edge of the escarpment at the north end of the lowest benched area.
- The regional ground water table is present at RL 6.8 metres.
- Earthquake loading is applied to the model.

By applying earthquake loading to the analysis, which examined the stability of the face of the escarpment to the north of the row of 15 metre deep reinforced piles, the factor of safety against failure was reduced to 1.02. This factor of safety is marginally greater than the factor of safety against failure given in Figure C-3 (1.02 c.f. 0.98), which represented the factor of safety against failure for the existing site condition with earthquake loading. This is significant in that is confirms that the proposed development will not adversely affect the stability of the face of the escarpment below the proposed development.

Figure C-8 is the graphical result of the critical stability analysis for the following conditions:

- A tiered site cut, as detailed in the drawings of the proposed development prepared by Fasham Johnson, dated 8 February 2011, at the top of the escarpment.
- The regional ground water table is present at RL 6.8 metres.
- No earthquake loading.

The analysis shows the failure rupture surface which provides a factor of safety against failure of approximately 1.5.

Based on the requirements outlined within the letter from Mornington Peninsula Shire Council dated 15 August 2011, all footings for the proposed dwelling must be founded below the rupture surface shown in Appendix C, Figure C-8.

Perched water flows within the near surface sands overlying the less permeable clay and dense to very dense silty and clayey sands have not been considered in the stability analyses, as the site cuts, which are proposed to be carried out to accommodate the proposed dwelling at the subject site, will intercept any potential perched seepage water flows. It has been assumed that drainage provisions for the proposed retention structures will allow any flows of perched seepage water to be effectively intercepted and discharged to a legal point of discharge clear of the escarpment.

4.6 RISK ASSESSMENT FOR PROPERTY

The above estimates of frequency and risk have been used in the qualitative risk matrix of AGS (2007) to derive the risk levels as summarised in Tables 4.6.1 - 4.6.3 below. A copy of the qualitative risk matrix of AGS (2007) is provided in Appendix D.

Table 4.6.1 provides an indication to the risk to property for the existing dwellings within the multi dwelling development at the base of the escarpment at 613 Point Nepean Road, McCrae, for the existing site conditions at 14 View Point Road.

Table 4.6.1 Summary of Assessment of Risk to Property at 613 Point Nepean Road, McCrae for the Existing Site Conditions at 14 View Point Road, McCrae.

Hazard	**B**
Property Likely to be Affected by Hazard	Existing Dwellings at 613 Point Nepean Road
Description	Rotational Slip of Escarpment
Likelihood	Possible
Indicative Annual Probability	10^{-3}
Consequence	Catastrophic
Risk	Very High Risk
Implication	**UNACCEPTABLE**

Table 4.6.2 provides an indication to the risk to property for a hypothetical scenario of the proposed dwelling being constructed with high level foundation systems, with no retention systems and no slope stabilisation measures.

Table 4.6.2 Summary of Assessment of Risk to Property for a Hypothetical Scenario of the Proposed Dwelling Constructed with High Level Foundation Systems, No Retention Systems and No Slope Stabilisation Measures.

Hazard	**A**	**B**	**C**
Property Likely to be Affected by Hazard	Proposed Dwelling at 14 View Point Road	Existing Dwellings at 613 Point Nepean Road	Proposed Dwelling at 14 View Point Road
Description	Rotational Slip of Escarpment	Rotational Slip of Escarpment	Creep Movement of Near Surface Silty Sand
Likelihood	Possible	Possible	Almost Certain
Indicative Annual Probability	10^{-3}	10^{-3}	10^{-1}
Consequence	Catastrophic	Catastrophic	Medium
Risk	Very High Risk	Very High Risk	Very High Risk
Implication	**UNACCEPTABLE**	**UNACCEPTABLE**	**UNACCEPTABLE**

Table 4.6.3 provides an indication to the risk to property for the proposed dwelling constructed with a piled footing and retention system.

Table 4.6.3 Summary of Assessment of Risk to Property for the Proposed Dwelling Constructed with a Piled Retention and Footing System.

Hazard	**A**	**B**	**C**
Property Likely to be Affected by Hazard	Proposed Dwelling at 14 View Point Road	Existing Dwellings at 613 Point Nepean Road	Proposed Dwelling at 14 View Point Road
Description	Rotational Slip of Escarpment	Rotational Slip of Escarpment	Creep Movement of Near Surface Silty Sand
Likelihood	Barely Credible	Possible	Barely Credible
Indicative Annual Probability	10^{-6}	10^{-3}	10^{-6}
Consequence	Catastrophic	Catastrophic	Minor
Risk	Low Risk	Very High Risk	Very Low Risk
Implication	**ACCEPTABLE**	**UNACCEPTABLE**	**ACCEPTABLE**

From Tables 4.6.1 - 4.6.3 the following must be noted:

- The risk to property for the proposed dwelling at 14 View Point Road necessitates that a piled footing and retention system be adopted for the proposed dwelling. Assuming that the retention and footing system for the proposed development is properly engineered and constructed the risk to property for the proposed dwelling at the subject site is low. This level of risk is normally considered acceptable by regulatory authorities.
- The proposed development does not alter the risk to the adjacent property at 613 Point Nepean Road, in the event of a rotational slip forming on the subject escarpment. It must be noted that, even if the proposed development at 14 View Point Road does not proceed, the risk to property for the dwellings at 613 Point Nepean Road, nearest to the base of the escarpment is unacceptable. This is a risk that is common to numerous properties along the toe of the escarpment in the immediate area. Extensive treatment of the subject escarpment would be required to reduce the risk to property along the tow of the escarpment to an acceptable level. Such treatment is likely to be extremely costly and may not be practicable to carry out.

4.7 RISK ASSESSMENT FOR LIFE

In the absence of any details for the proposed occupation of the proposed dwelling at the subject site and the two most vulnerable dwellings at the south end of the property at 613 Nepean Road, it has been assumed for the purpose of the risk assessment for life that the dwellings will be subject to full time occupation by a typical family of four. Whilst this level of occupation may not be proposed in the short term it is conceivable that it may be occur in the future.

A quantitative basis has been adopted for estimation of the risk to life. The risk assessments are summarised in Tables 4.7.1 - 4.7.3 below.

Table 4.7.1 provides an indication to the risk to life for the occupants of the existing dwellings at 613 Point Nepean Highway, nearest to the toe of the subject escarpment, for the existing site conditions at 14 View Point Road.

Table 4.7.1 Summary of Assessment for Risk to Life for the Occupants of the Dwellings at 613 Point Nepean Road, McCrae for the Existing Site Conditions at 14 View Point Road, McCrae.

Hazard	**B**
Description	Rotational slip of Escarpment
Likelihood	Possible
Indicative Annual Probability	10^{-3}
Probability of Spatial Impact	1
Occupancy (8 Occupants in 2 Dwellings)	8
Proportion of Time	0.5 (12 hours/day)
Probability of Not Evacuating	1.0 (Rapid Failure)
Vulnerability	0.5 (Not Buried)
Risk for Person Most at Risk	2.5×10^{-4}
Total Risk (8 Occupants in 2 Dwellings)	2.0×10^{-3}
Risk Evaluation	**INTOLERABLE**

Table 4.7.2 provides an indication to the risk to life for the occupants of the existing dwellings at 613 Point Nepean Highway, nearest to the toe of the subject escarpment and the occupants of the proposed dwelling at 14 View Point Road, assuming that the proposed dwelling is constructed with high level foundation systems, with no retention systems and no slope stabilisation measures..

Table 4.7.2 Summary of Assessment for Risk to Life for the Occupants of the Dwellings at 613 Point Nepean Road and 14 View Point Road, McCrae, for a Hypothetical Scenario of the Proposed Dwelling Constructed with High Level Foundation Systems, No Retention Systems and No Slope Stabilisation Measures.

Hazard	**A**	**B**	**C**
Description	Rotational slip of Escarpment	Rotational slip of Escarpment	Creep Movement of Near Surface Silty Sand
Likelihood	Possible	Possible	Almost Certain
Indicative Annual Probability	10^{-3}	10^{-3}	10^{-1}
Probability of Spatial Impact	1	1	1
Occupancy (4 Occupants in each Dwelling)	4	8 (2 Dwellings)	4
Proportion of Time	0.5 (12 hours/day)	0.5 (12 hours/day)	0.5 (12 hours/day)
Probability of Not Evacuating	1.0 (Rapid Failure)	1.0 (Rapid Failure)	0.01 (Extremely Slow)
Vulnerability	1	0.5 (Not Buried)	0.01
Risk for Person Most at Risk	5.0×10^{-4}	2.5×10^{-4}	5.0×10^{-6}
Total Risk	2.0×10^{-3}	2.0×10^{-3}	2.0×10^{-5}
Risk Evaluation	**INTOLERABLE**	**INTOLERABLE**	**ACCEPTABLE**

Table 4.7.3 provides an indication to the risk to life for the occupants of the existing dwellings at 613 Point Nepean Highway, nearest to the toe of the subject escarpment and the occupants of the proposed dwelling at 14 View Point Road, assuming that the proposed dwelling is constructed with a piled footing and retention system.

Table 4.7.3 Summary of Assessment for Risk to Life for the Occupants of the Dwellings at 613 Point Nepean Road and 14 View Point Road, McCrae, for the Proposed Dwelling Constructed with a Piled Footing and Retention System.

Hazard	**A**	**B**	**C**
Description	Rotational slip of Escarpment	Rotational slip of Escarpment	Creep Movement of Near Surface Silty Sand
Likelihood	Barely Credible	Possible	Barely Credible
Indicative Annual Probability	10^{-6}	10^{-3}	10^{-6}
Probability of Spatial Impact	1	1	1
Occupancy (4 Occupants in each Dwelling)	4	8 (2 Dwellings)	4
Proportion of Time	0.5 (12 hours/day)	0.5 (12 hours/day)	0.5 (12 hours/day)
Probability of Not Evacuating	1.0 (Rapid Failure)	1.0 (Rapid Failure)	0.01 (Extremely Slow)
Vulnerability	1	0.5 (Not Buried)	0.01
Risk for Person Most at Risk	5.0×10^{-7}	2.5×10^{-4}	5.0×10^{-11}
Total Risk	2.0×10^{-6}	2.0×10^{-3}	2.0×10^{-10}
Risk Evaluation	**ACCEPTABLE**	**INTOLERABLE**	**ACCEPTABLE**

The results of risk estimation have been compared to the acceptance criteria given in Table 4.7.4. It is noted that the Regulatory Authority (Mornington Peninsula Shire Council) should set the standard for risk criteria, which may differ from that assumed in this assessment. Generally acceptable risks are considered to be risks which everyone affected is prepared to accept. Action to further reduce risk is usually not required unless reasonably practical measures are available at low cost in terms of money, time and effort. Tolerable risks are typically considered to be risks within a range that society can live with so as to secure certain benefits. It is a range of risk regarded as non-negligible and needing to be kept under review and reduced further if applicable.

Table 4.7.4 Acceptable and Tolerable Risk to Life Criteria

Situation	Tolerable Risk For Loss of Life	Acceptable Risk For Loss of Life
Existing Slopes	10^{-4} person most at risk	10^{-5} person most at risk
	10^{-5} average of persons at risk	10^{-6} average of persons at risk

In comparing the outcomes of the assessments for risk to life for the occupants of the existing dwellings at 613 Point Nepean Road and the proposed dwelling 14 View Point Road with the criteria given in Table 4.7.4 the following must be noted:

- The risk to life for the occupants of the proposed dwelling at 14 View Point Road necessitates that a piled footing and retention system be adopted for the proposed dwelling. Assuming that the retention and footing system for the proposed development is properly engineered and constructed, the risk to life to the occupants of the proposed dwelling at the subject site is **acceptable** in accordance with the criteria given in Table 4.7.4.
- The proposed development does not alter the risk to life for the occupants of the adjacent dwellings at 613 Point Nepean Road, in the event of a rotational slip forming on the subject escarpment. It must be noted that, even if the proposed development at 14 View Point Road does not proceed, the risk to life for the occupants of the adjacent dwellings at 613 Point Nepean Road, nearest to the base of the escarpment, is unacceptable. This is a risk that is common to numerous occupants of the existing dwellings along the toe of the escarpment in the immediate area. Extensive treatment of the subject escarpment would be required to reduce the risk to life for the occupants of the dwellings along the toe of the escarpment to an acceptable level. Such treatment is likely to be extremely costly and may not be practicable to carry out.

4.8 RISK MANAGEMENT

The level of risk to life for the proposed structure is intolerable and the risk to property is unacceptable, assuming that suitable precautions are not taken in the development of the subject site. To achieve an acceptable level of risk to life and a low risk to property it will be necessary to incorporate protective measures to prevent collapse of the proposed structure in the event of a landslide occurring on the face of the escarpment.

The proposed structure must be constructed in such a manner that it is either unaffected by a potential landslide at the subject site or the escarpment is stabilised such that an acceptable factor of safety against failure is maintained for the entire escarpment. The latter option is not likely to be viable. The height and steepness of the escarpment, combined with the size of the potential landslide would necessitate very substantial stabilisation works to be carried out both on the face and towards the base of the escarpment. Such remedial works will necessitate stripping substantial amounts of the existing vegetation, if not all of the vegetation from the face of the escarpment and significant earthworks to enable construction equipment to access the escarpment face. This process in itself is extremely undesirable in that it is likely to trigger instability. Recommendations for stabilisation of the proposed house site are given in Section 6.

5 COMMENTS AND RECOMMENDATIONS

5.1 SITE CLASSIFICATION

Classification of the site has taken into account the following:

- Identification of the sub soil profile.
- Field classification of soil type and plasticity.
- Established data on the performance of existing buildings on the soil profile.

Based on slope stability considerations the subject site has been classified as 'Class P' in accordance with Australian Standard Australian Standard AS 2870 – 2011, 'Residential Slabs and Footings'.

5.2 EARTHQUAKE SITE CLASSIFICATION

Australian Standard AS 1170.4 – 2007, 'Minimum Design Loads on Structures, Part 4: 'Site Sub-Soil Class' outlines the methods for assigning the sites Sub-soil Class. Based on the assumed stratigraphy and Table 4.1 "Maximum Depth Limits for Sub-soil Class C" and Figure 3.2(A) "Hazard Factor (Z) for Victoria" of the standard, we recommend the following Hazard Factor and Sub-Soil Class are adopted:

- Sub-soil Class: Class C_e – Shallow Soil Site
- Hazard Factor (Z): 0.09

5.3 NEW FOOTINGS

The following footing system would appear most suitable given the proposed development in conjunction with the prevailing conditions at the site.

- It is recommended that the proposed structure be fully suspended on a series of reinforced bored piles. Shallow pad and strip footings, and stiffened raft slabs are not considered appropriate for the support of the proposed structure given the potential instability of the escarpment.
- The row of piles along the north side of the proposed structure will need to be designed as retention piles to protect the proposed dwelling against a potential landslide which may occur on the face of the escarpment. The row of retention piles along the north end of the proposed dwelling, whilst protecting the proposed dwelling against slope instability, will not prevent the possibility of a landslide occurring on the face of the escarpment immediately to the north of the row of piles. It is therefore imperative that the no isolated pile footings be constructed downhill of the row of row of anchored retention piles.

- Based on the requirements outlined within the letter from Mornington Peninsula Shire Council dated 15 August 2011, all piles for the proposed dwelling must be founded below the rupture surface shown in Appendix C, Figure C-8.
- The proposed footing/retention system will not serve to stabilise the escarpment downhill from the proposed development. Stabilisation of the escarpment downhill of the proposed dwelling is anticipated to be extremely expensive. Additionally, in order to install piles, ground beams and ground anchors, which would be required to stabilise the section of escarpment extending downhill from the proposed dwelling, it will be necessary to strip substantial amounts of existing vegetation, if not all of vegetation from the face of the escarpment and carry out significant earthworks to enable construction equipment to access the escarpment face. Removal of vegetation and any earthworks on the face of the escarpment is highly undesirable in that it is likely to trigger instability.
- Assuming that the recommendations of this report are adhered to, it is emphasised that construction of the proposed dwelling will not adversely affect the stability of the section of escarpment downhill from the proposed dwelling. Provided that good hillside construction practices are adopted, the risk of instability on the section of escarpment downhill from the proposed dwelling may be marginally reduced when compared with the current uncontrolled site conditions.
- Retention along the south, east and west sides of the proposed bulk excavations for the two lower tiers of the proposed bulk excavation must comprise cantilevered soldier piles with reinforced shotcrete infill panels. Under no circumstances shall any unsupported excavation batters with a vertical height exceeding 1.0 metre and a batter exceeding 1 in 2 be carried out at the subject site.

5.4 RETENTION PILES ALONG THE NORTH END OF THE PROPOSED DWELLING

The row of piles along the north side of the proposed structure must be designed as retention piles to protect the proposed dwelling against a potential landslide, which may occur on the face of the escarpment. The row of retention piles, whilst protecting the proposed dwelling against slope instability, will not prevent the possibility of a landslip occurring on the face of the escarpment immediately to the north of the row of piles. It is therefore imperative that the no isolated pile footings be constructed downhill of the row of row of retention piles. Any portion of the proposed structure which extends to the north of the row of retention piles must be cantilevered.

The row of retention piles would best be located along the north edge of the proposed benched level at RL 23.0. The piles could be installed after the proposed site cut has been carried out, assuming that the south, east and west excavation batters are protected with a cantilevered or anchored solider pile retaining wall with reinforced shotcrete infill panels. This will ensure that a conventional piling rig is able to install the piles without any special requirements for site access.

The design and construction of the row of retention piles along the north edge of the proposed benched level at RL 23.0 must satisfy the following criteria:

- The piles must be founded into silty sand of very dense relative density, as intersected in Bore 1 at depths in excess of 16.5 metres.
- The piles must be founded at least four pile diameters below the failure rupture surface for a factor of safety of 1.5 indicated in Appendix C, Figure C-8. This equates to a minimum pile embedment of 12.5 metres + four pile diameters below the surface of the benched level at RL 23.0. As an example, a minimum founding depth of 18 metres will be required for a 0.75 metre diameter pile.
- In the event of the collapse of the escarpment adjacent to the row of retention piles, the uppermost 12.5 metre section of the piles must be designed to withstand the 'at rest' lateral earth pressure, assuming no support from the soil along the north side of the row of retention piles. The height of soil to be retained by the row of retention piles is based on the requirements outlined within the letter from Mornington Peninsula Shire Council dated 15 August 2011, that is, the complete loss of soil along the north of the row of retention piles for the failure rupture surface shown in Appendix C, Figure C-8.
- It is recommended that the centre to centre spacing of the row of retention piles not exceed 2.4 metres to ensure that soil arching occurs between the retention piles.
- Each of the retention piles must support a width of soil equivalent to the spacing of the piles. The recommendations given in Section 5.6 shall be used to determine the lateral earth pressure acting on the row of retention piles.
- Lateral support of the row of retention piles must be provided by a combination of head and toe restraint of the piles. Toe restraint shall be provided by providing a suitable depth of pile embedment below the failure rupture surface shown in Appendix C, Figure C-8. Head restraint of the retention piles shall be provided by structurally tying the retention piles to other piles situated to the south of the row of retention piles, via a suspended raft slab or a series of ground beams. The lateral capacity of any piles providing lateral restraint to the row of retention piles along the north side of the dwelling must be derived from the portion of the piles embedded below the failure rupture surface shown in Appendix C, Figure C-8. No lateral capacity shall be derived for the portion of the piles above the failure rupture surface shown in Appendix C, Figure 8.

In accordance with Australian Standard AS 2159 - 2009 'Piling Design and Installation' the geotechnical strength reduction factor is influenced by the scope of geotechnical investigation and means of determining/selecting geotechnical design parameters, the design methodology, construction controls and the method and extent of pile testing. Adopting a geotechnical strength reduction factor ($Ø_g$) of 0.45 for the design of bored piles and a load factor of 1.35 the following working pressure is estimated:

- Working base pressure on very dense silty sand: 1000 kPa

The above bearing pressure has been based on the assumption of a complete loss of soil to the north of the row of retention piles for the failure rupture surface shown in Appendix C, Figure C-8, that is, the effective overburden stress used in the estimation of the above bearing pressure has been based on a minimum pile embedment of four pile diameters below the failure rupture surface.

Assuming a load factor of 1.35, it is estimated that settlements at working loads will be approximately 1% of the pile diameter. Differential settlements are likely to be less than half the total settlement value. Settlements will significantly exceed these values if the bases of bored pile excavations are not clean and free of loose material.

The possible presence of flows of ground water seepage within the very dense silty sand below RL 6.8 will necessitate the use of either continuous flight auger piles or bored piles constructed under bentonite.

Pile excavations must be inspected by a qualified engineer, where appropriate, prior to the placement of concrete to ensure that the founding conditions are consistent with the above recommendations. If conditions are not consistent with the above recommendations it may be necessary to either increase the founding depth and/or diameter of the bored piles.

5.5 FOOTINGS PROVIDING SUPPORT TO THE PROPOSED DWELLING

5.5.1 Bored Pile Footings

It is recommended that the proposed structure be fully suspended on a series of reinforced bored pile footings. It is recommended that all bored piles be structurally tied together with either a series of suspended ground beams or a suspended raft slab. The spacing, reinforcing and diameter of the piles need to take into account both structural requirements for the support of the proposed dwelling and the provision of lateral restraint for the row of retention piles along the north side of the dwelling.

Based on the requirements outlined within the letter from Mornington Peninsula Shire Council dated 15 August 2011, all piles for the proposed dwelling must be founded below the failure rupture surface shown in Appendix C, Figure C-8.

The design and construction of the general piles providing support to the proposed structure must satisfy the following criteria:

- The piles must be founded on either very dense sand or very stiff clay at a minimum founding depth of 8 metres below the various proposed bulk excavation levels, and:
- The piles must be founded at least four pile diameters below the failure rupture surface for a factor of safety of 1.5 indicated in Appendix C, Figure C-8.
- Significantly greater piles embedments below the failure rupture surface for a factor of safety of 1.5 indicated in Appendix C, Figure C-8 will be required for piles which provide lateral support for retaining wall structures and the row of retention piles along the north side of the dwelling.

The presence of bands of clay within the soil profile will restrict the maximum base bearing capacity that can be utilised for the piles.

In accordance with Australian Standard AS 2159 - 2009 'Piling Design and Installation' the geotechnical strength reduction factor is influenced by the scope of geotechnical investigation and means of determining/selecting geotechnical design parameters, the design methodology, construction controls and the method and extent of pile testing. Adopting a geotechnical strength reduction factor ($Ø_g$) of 0.45 for the design of bored piles and a load factor of 1.35 the following working pressure is estimated:

- Working base pressure on very stiff clay or very dense clayey and silty sand: 450 kPa

Assuming a load factor of 1.35, it is estimated that settlements at working loads will be less than approximately 1% of the pile diameter. Differential settlements are likely to be less than half the total settlement value. Settlements will significantly exceed these values if the bases of bored pile excavations are not clean and free of loose material.

A cleaning bucket or plate must be used to clean the base of each pile excavation prior to the placement of concrete. The use of a toothed auger is unacceptable for cleaning pile bases.

Concrete should be poured as soon as the pile excavations have been cleaned and approved to prevent accumulation of seepage water within the pile excavations. Any seepage water must be removed prior to concrete placement. Temporary liners may need to be provided where seepage water flows destabilise the pile excavations.

All bored pile excavations must be inspected by a qualified engineer prior to the placement of concrete to ensure that the founding conditions are consistent with the above recommendations. If conditions are not consistent with the above recommendations it may be necessary to either increase the founding depth and/or diameter of the bored piles.

5.6 RETENTION OF PROPOSED SITE CUTS

Uncontrolled earthworks involving cutting and filling must not be carried out at the site. Such earthworks have the potential to trigger slope instability at the site.

5.6.1 Soldier Pile Retention System

Where the depth of site cut exceeds approximately 1.0 metre we recommend the installation of soldier piles prior to excavation.

Based on the requirements outlined within the letter from Mornington Peninsula Shire Council dated 15 August 2011, all piles for the proposed dwelling must be founded below the failure rupture surface shown in Appendix C, Figure C-8. Additionally, lateral restraint for soldier pile walls can only be derived from the portion of the piles embedded below the failure rupture surface shown in Appendix C, Figure C-8.

The soldier piles may also be designed as load bearing in accordance with Section 5.5.1. Reinforced shotcrete panels must be provided between the soldier piles.

Soldier pile spacing should not exceed 1.5 metres where adjacent structures are within the zone of influence of the excavation. The zone of influence may be taken to extend a horizontal distance of 1.5 times the excavation depth out from the excavation perimeter. Additionally piles should be positioned such that any adjacent high level footings are continuous between piles. Elsewhere spacing should not exceed 2.4 metres.

At locations where the depth of site cut exceeds approximately 3.0 metres consideration should be given to the use of anchored soldier piles. Where required, temporary ground anchors must be installed incrementally as excavation proceeds. Ground anchors must be installed immediately once the anchoring points have been exposed. The bond length of temporary ground anchors must be located behind/below the failure rupture surface shown in Appendix C, Figure C-8.

5.6.2 Lateral Earth Pressures

The design lateral earth pressure distribution for a retaining wall should be chosen so as to suitably limit deformation outside of the excavation. The magnitude of deformation is also time dependent and influenced by construction methods and quality. We recommend the following for the design of temporary and permanent retention systems for the proposed basement level assuming a horizontal backfill surface and that the walls are designed as permanently drained.

- For *permanently* cantilevered retaining walls, which allow lateral yield of the retained soil, adopt an ‘active’ lateral earth pressure distribution increasing linearly with depth expressed as $q_a = K_a\gamma'z$ (kPa) where K_a is the coefficient of active earth pressure, γ' is the effective unit weight of the retained materials (kN/m^3) and z is the depth in metres. Relevant parameters are provided in Section 5.6.3.

- For *permanently* cantilevered retaining walls, where lateral yield of the retained soil is to be limited, adopt an 'at rest' lateral earth pressure distribution increasing linearly with depth expressed as $q_a = K_o\gamma'z$ (kPa) where K_o is the coefficient of at rest earth pressure, γ' is the effective unit weight of the retained materials (kN/m^3) and z is the depth in metres. Relevant parameters are provided in Section 5.6.3.

- For propped or anchored walls or where the completed structure will provide lateral restraint adopt a uniform earth pressure distribution. Where minor movements can be tolerated adopt a uniform pressure of 4H kPa where H is the total retained height in metres. An average earth pressure coefficient (K) of 0.42 should be used to calculate lateral earth pressures generated by surcharge loads.

- For minimal deflection where there are movement sensitive structures or buried services within the zone of influence of the excavation, adopt a uniform pressure of 5H kPa where H is the total retained height in metres. An average earth pressure coefficient (K) of 0.5 should be used to calculate lateral earth pressures generated by surcharge loads. The zone of influence of the excavation should be taken to extend a horizontal distance of 1.5 times the excavation depth out from the excavation perimeter.

- The relevant coefficients of lateral earth pressure may be used in conjunction with elastic theory (e.g. a stress distribution derived using Boussinesq's solutions or equivalent) to determine the lateral earth pressure distributions due to surcharge loads.

- Sloping backfill should be incorporated as surcharge loading. Any temporary or permanent surcharge loads such as nearby high level footings, traffic loading and compaction stresses, should also be included in design.

- If the retaining wall backfill is compacted it is possible that stresses induced on the wall may exceed the recommended design lateral earth pressure distributions. The magnitude of the additional stresses will be dependent on the mechanical properties of the backfill material and the compactive effort applied.

5.6.3 Design Parameters for Retention Structures

The soil parameters given in Table 5.6.3.1 may be adopted for the design of retaining walls. It must be noted however that the parameters given in Table 5.6.3.1 are unfactored. Appropriate strength reduction factors must be applied in accordance with Australian Standard AS 4678 - 2002 'Earth Retaining Structures'.

TABLE 5.6.3.1: Design Parameters for Retention Structures

	Fill and Near Surface Silty Sand	Medium Dense Silty Sand	Very Dense Clayey and Silty Sand	Very Stiff Clay	Dense Silty Sand
Depth Interval in Bore 1 (metre)	0.0 – 0.9	0.9 – 3.1	3.1 – 7.5, 9.0 – 12.0, 14.0 – 15.0 and 16.5 – 21.5	7.5 – 9.0, 12.0 – 14.0, and 15.0 – 16.5	21.5 – 25.0
(γ) Soil Unit Weight (kN/m^3)	20	20	21	18	21
($Ø_u$) Undrained Angle of Internal Friction	29^o	32^o	42^o	0^o	36^o
(Ø') Effective Angle of Internal Friction	29^o	32^o	42^o	24^o	36^o
(C_u) Undrained Cohesion (kPa)	0	0	0	150	0
(C') Effective Cohesion (kPa)	0	0	0	10	0
(E) Elastic Modulus (MPa)	20	35	80	40	60
(v) Poisson's ratio	0.3	0.3	0.3	0.5	0.3
(K_a) Coefficient of Active Earth Pressure	0.35	0.31	0.20	0.42	0.26
(K_0) Coefficient of At Rest Earth Pressure	0.52	0.47	0.33	0.59	0.41
(K_p) Coefficient of Passive Earth Pressure	N/A	3.26	5.0	2.37	3.85

5.6.4 Retaining Wall Backfill and Drainage

Retention structures must be designed such that the soil behind the wall is completely and permanently drained. It is recommended that subsurface drains incorporate a non woven geotextile filter fabric to minimise silting of drains and erosion of backfill.

All drains must discharge to a legal point of discharge clear of the site. Under no circumstances shall seepage water be allowed to discharge onto the face of the escarpment.

5.6.5 Ground Anchors

It has been assumed that permanent lateral support of retaining walls will be provided by the completed structure and that any anchors will be designed as temporary. Design of any permanent anchors must make allowance for corrosion and long term durability.

Ground anchors drilled using auger methods may be designed using an allowable bond strength of 60 kPa within very dense sand or very stiff clay. Anchors should be installed approximately 15°-20° below the horizontal and bond length should not exceed 10 metres. All anchors must be proof tested to 1.5 times the working load under the supervision of an experienced engineer. The testing may allow an upgrade of the above allowable bond stresses.

To guard against a sliding wedge failure behind the retaining wall, the free length of anchors should extend approximately 1.5 metres beyond the 45° line extending up from the toe of the retaining wall. Additionally, the bond length of temporary ground anchors must be located behind/below the failure rupture surface shown in Appendix C, Figure C-8. Local and global stability of the proposed retaining wall should be analysed once retaining wall geometry and anchor locations have been determined.

5.6.6 Ground Movements Related to Excavation

Adjacent to any excavations there will be some movement of the ground within the zone of influence of the excavation. The magnitude of ground and wall movement is highly dependent on the wall design, construction sequence, quality of installation and elapsed time.

As a guide, precedence suggests that for similar conditions to those anticipated at the subject site, lateral deflection of a relatively stiff cantilevered wall of good workmanship is likely to be in the order of 1.0% of the excavation depth. On a similar basis propped or anchored walls designed for a uniform lateral earth pressure distribution of 5H kPa, and constructed with good workmanship, may experience lateral deflection in the order of 0.2% of the excavation depth. Consistent with the above horizontal deflections, vertical settlements of less than 1.0% of the excavation depth could be expected for cantilevered walls and less than 0.2% for propped or anchored walls.

The distribution of vertical ground settlement adjacent to the excavation is highly dependent on the deflected shape of the retention system. However settlement can be expected to diminish to negligible magnitude at the outer extent of the zone of influence of the excavation. The zone of influence of the excavation should be taken to extend a horizontal distance of 1.5 times the excavation depth out from the excavation perimeter.

In addition to the inherent deformations which will take place within the proposed excavation, there may be some minor delays between excavation and the establishment of a suitable or anchoring arrangement, during which time additional minor lateral deflections may take place.

5.7 GENERAL GUIDELINES FOR HILLSIDE CONSTRUCTION

The local geology is susceptible to instability where development does not observe good hill side construction practice. Extracts from the Australian Geomechanics Society Volume 42, No. 1, March 2007 are provided in Appendix E as a further guide to good hillside construction practices.

5.7.1 Water Bearing Services

The integrity of all water supply and drainage pipes (pressurised and non pressurised) on the site must be maintained at all times to ensure that no leaks occur.

No pipes shall be installed on the face of the escarpment.

Water supply and drainage pipes (pressurised and non pressurised) located beneath the proposed structure must be suspended from the underside of the raft slab or grid of strip footings. In the event of potential creep movements or slope instability adjacent to the proposed dwelling, flexible (sleeved) couplings must be provided at all locations where pipes connect to the proposed structure. The couplings must allow for potential horizontal and vertical movements.

5.7.2 Earthworks

Uncontrolled earthworks involving cutting and filling must not be carried out at the site. Such earthworks have the potential to trigger slope instability at the site. Under no circumstances shall any fill be placed on the face of the escarpment or adjacent to the top edge of the escarpment. All soil excavated from any site excavations must be removed from the site. Under no circumstances shall any soil be placed on the face of the escarpment or adjacent to the top edge of the escarpment,

If a site cut is to be considered at the site to accommodate the proposed dwelling, the site cut should be restricted to the very top of the escarpment. Removal of soil from the top edge of the escarpment will assist to marginally reduce the potential for a landslide to occur at the subject site. However the site cut must be fully retained at all times during and after construction.

5.7.3 Site Drainage

All surface water runoff from both the site and the adjacent properties uphill of the site, and any collected stormwater from the development, must be drained to a legal point of discharge well clear of the escarpment. Treated sewage must not be discharged onto the site by way of soakage pits or irrigation. All sewage must be discharged to a legal point of discharge offsite.

5.7.4 Removal of Vegetation

Removal of existing vegetation from the site should be avoided, in particular from the face of the escarpment. Additional vegetation ranging from dense ground cover through to shrubs and trees with extensive root systems should be established on the more steeply sloping portions of the site as soon as possible to improve long term stability of the site.

5.8 CONSTRUCTION REQUIREMENTS

5.8.1 Inspection of Footing Excavations

All footing excavations should be inspected by a qualified geotechnical engineer to ensure that the required founding stratum has been achieved. The presence of any unusual features or conditions should be brought to the attention of this office before construction proceeds.

5.8.2 Articulation of Structure

Adequate articulation should be provided in accordance with 'The Cement and Concrete Association of Australia' – Technical Note TN61. In addition to the requirements of TN61 a full height articulation joint should be provided at the following locations:

- At the junction where two different footing types intersect.
- Where founding depths vary.
- At all locations where appreciable stress concentrations are anticipated.

5.9 REPORT LIMITATIONS

This report is for the use of the party to whom it is addressed only and has been produced for the proposed development as described and for no other purpose. It has been assumed that the conditions encountered by the limited number of boreholes are representative of the site in general. Some variation from the conditions encountered by the boreholes is expected over the site. It is beyond the scope of this report to comment on any possible contamination of the site.

This report should only be reproduced in full.

If you require any further information please do not hesitate to contact the undersigned.

For and on behalf of
GEOAUST GEOTECHNICAL ENGINEERS PTY LTD

Irrelevant & Sensitive

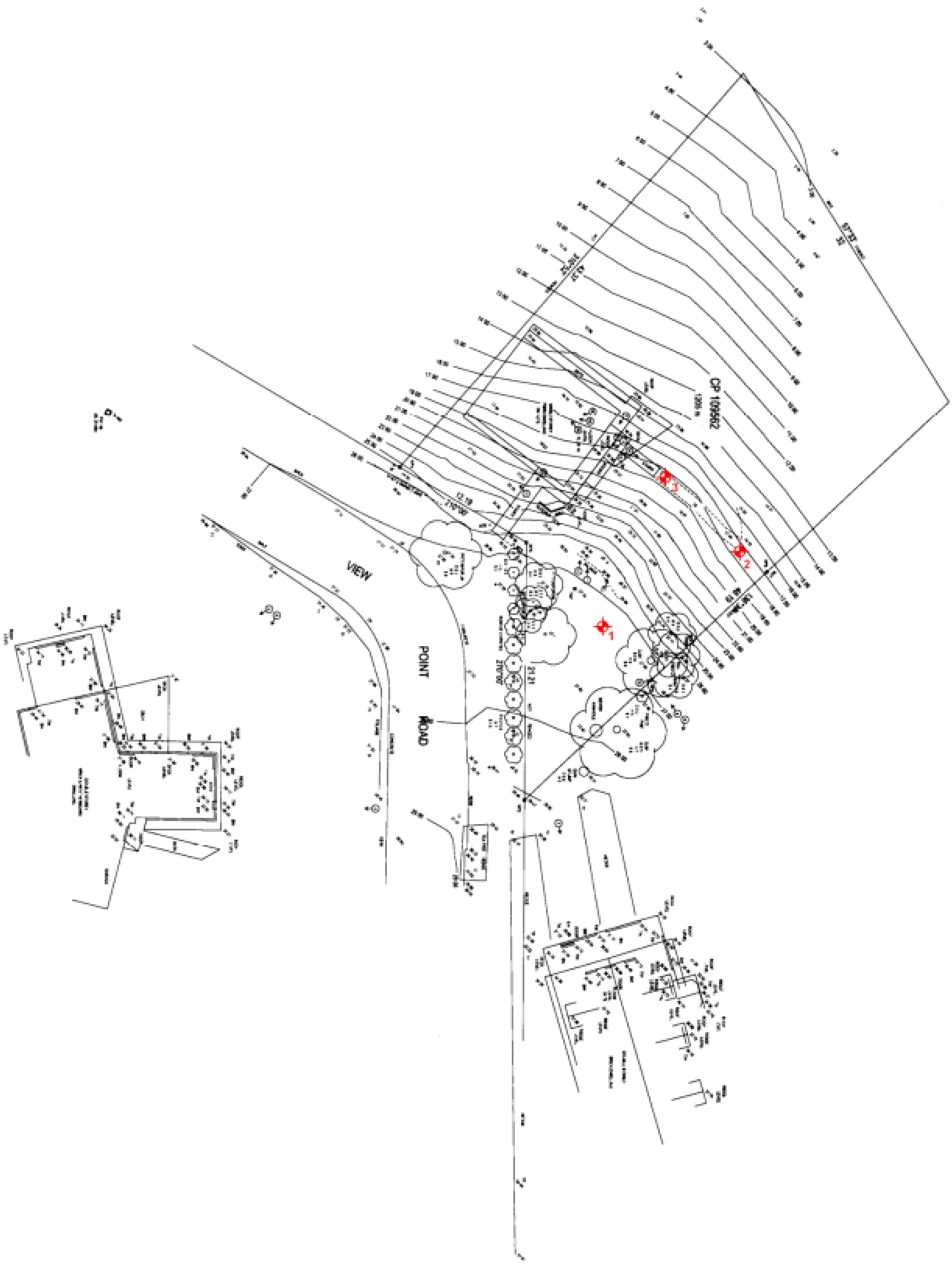
Stephen Mayer

BEng(Hons) MIEAust CPEng EC-2262

GeoAust
Geotechnical Engineers

TEST LOCATION PLAN

JOB No: 1624
CLIENT: Fasham Johnson Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 14-16 ViewPoint Road, MCCRAE



NOT TO SCALE

LEGEND

Denotes approximate borehole location

N

Figure 1

GeoAust
Geotechnical Engineers

APPENDIX A

Definitions of Logging Terms and Symbols

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APPENDIX A

EXPLANATION NOTES FOR BOREHOLE AND TEST PIT LOGS

SOIL CLASSIFICATION AND LOG SYMBOLS

SOIL CLASSIFICATION CHART

	MAJOR DIVISIONS		SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	**GRAVEL AND GRAVELLY SOILS**	**CLEAN GRAVELS** (LITTLE OR NO FINES)		**GW**	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
	MORE THAN 50% OF COARSE FRACTION IS LARGER THAN 2.0MM	**GRAVELS WITH FINES** (APPRECIABLE AMOUNT OF FINES)		**GM**	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% OF MATERIAL SMALLER THAN 63MM IS LARGER THAN 0.075MM	**SAND AND SANDY SOILS**	**CLEAN SANDS** (LITTLE OR NO FINES)		**SW**	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION IS SMALLER THAN 2.0MM	**SANDS WITH FINES** (APPRECIABLE AMOUNT OF FINES)		**SM**	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	**SILTS AND CLAYS**	LIQUID LIMIT LESS THAN 50		**ML**	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL SMALLER THAN 63MM IS SMALLER THAN 0.075MM	**SILTS AND CLAYS**	LIQUID LIMIT GREATER THAN 50		**MH**	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				**PT**	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

GROUND WATER

	Inflow
	Outflow
	Standing level on completion
1/2	Standing level 1/2 hour after completion
	Collapse of borehole annulus
S	Slight seepage rate
M	Moderate seepage rate
H	High seepage rate
NOT OBSERVED	Ground water observation not possible. Ground water may or may not be present
NOT ENCOUNTERED	Ground water was not evident during excavation or a short time after completion

SAMPLING AND TESTING

DS	Disturbed sample
U60	Thin walled tube sample. Number indicates nominal sample diameter in mm
ES	Environmental sample
SPT	Standard penetration test
3/6/9 N=15	3,6 and 9 refer to blows per 150mm penetration. N=15 is the sum of blows after the initial 150mm penetration
3/6/9 blows for 20mm penetration: N>15.	3 and 6 refer to blows per 150mm penetration. 9 blows resulted in 20mm penetration at which point practical refusal of penetration occurred
S=47kPa	In-situ vane shear test. Result expressed as peak undrained shear strength in kPa
PP=145kPa	Pocket penetrometer test. Result expressed as dial reading in kPa
DCP	Dynamic Cone Penetrometer Test
EX	Excavation. Test starts at base of excavation
S	DCP sank under own weight or last blow of previous 100mm increment
E	End of DCP test
R	End of DCP test due to effective refusal of penetration

Figure A-1

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APPENDIX A

EXPLANATION NOTES FOR BOREHOLE AND TEST PIT LOGS

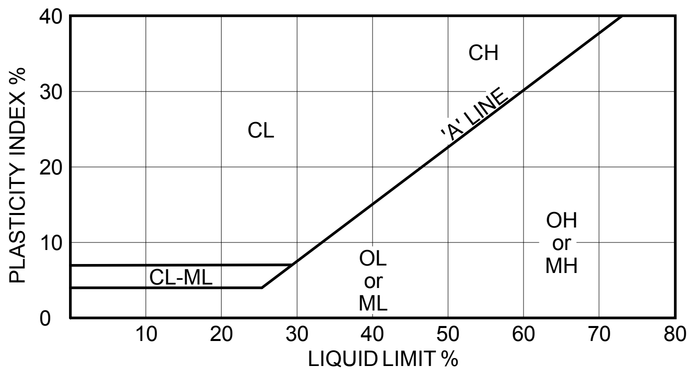
SOIL DESCRIPTION

PARTICLE SIZE

MAJOR DIVISION	SUB-DIVISION	SIZE (mm)
Boulders		>200mm
Cobbles		63 to 200mm
Gravel	Coarse	20 to 63mm
	Medium	6 to 20mm
	Fine	2.36 to 6mm
Sand	Coarse	0.6 to 2.36mm
	Medium	0.2 to 0.6mm
	Fine	0.075 to 0.2mm

0.075mm is the approximate minimum particle size discernible by eye

PLASTICITY CHART



MATERIAL PROPORTIONS

COARSE GRAINED SOILS		FINE GRAINED SOILS		IDENTIFICATION
% Fines	Modifier	% Coarse	Modifier	Field Assessment
⩽ 5	Omit or use 'trace'	⩽ 15	Omit or use 'trace'	Presence just detectable by feel or eye. Properties little or no different to those of primary soil
> 5 ⩽ 12	Describe as 'with clay/silt' as applicable	> 15 ⩽ 30	Describe as 'with sand/gravel' as applicable	Presence easily detected by feel or eye. Properties little or no different to those of primary soil
> 12	Prefix soil as 'silty/clayey' as applicable	> 30	Prefix soil as 'sandy/gravelly'	Presence obvious by feel or eye. Properties of soil are altered from those of the primary soil

COHESIVE SOILS - CONSISTENCY TERMS

LOG SYMBOL	TERM	UNDRAINED STRENGTH	FIELD ASSESSMENT
VS	Very Soft	<12kPa	Exudes between fingers when squeezed
S	Soft	12 - 25kPa	Can be moulded by light finger pressure
F	Firm	25 - 50kPa	Can be moulded by strong finger pressure
St	Stiff	50 -100kPa	Cannot be moulded by fingers. Can be indented by thumb
VSt	Very Stiff	100 - 200kPa	Can be indented by thumb nail
H	Hard	> 200kPa	Can be indented by thumb nail with difficulty

GRANULAR SOILS - DENSITY

LOG SYMBOL	TERM	DENSITY INDEX (%)
VL	Very Loose	< 15
L	Loose	15 - 35
MD	Medium Dense	35 - 65
D	Dense	65 - 85
VD	Very Dense	> 85

MOISTURE CONDITION

LOG SYMBOL	TERM	FIELD ASSESSMENT
D	Dry	Clay and silt are hard, friable, powdery, well dry of plastic limit. Sands and gravels are cohesionless, free running
M	Moist	Feels cool, darkened colour. Cohesive soils can be moulded. Granular soils tend to cohere
W	Wet	Feels cool, darkened in colour. Cohesive soils weakened, free water forms on hands when handling. Granular soils cohere

FIELD ASSESSMENT OF FILL COMPACTION

LOG SYMBOL	TERM
APC	Appears poorly compacted
AMC	Appears moderately compacted
AWC	Appears well compacted

Figure A-2

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APPENDIX A

EXPLANATION NOTES FOR BOREHOLE AND TEST PIT LOGS

ROCK DESCRIPTION

STRENGTH OF INTACT ROCK MATERIAL

LOG SYMBOL	TERM	POINT LOAD INDEX (MPa) Is50	FIELD ASSESSMENT
EL	Extremely Low	Is50 <0.03	Easily remoulded by hand to a material with soil properties
VL	Very Low	0.03 ⩽ Is50 < 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with knife; pieces up to 30mm thick can be broken by finger pressure
L	Low	0.1 ⩽ Is50 < 0.3	Easily scored with knife; indentations 1mm to 3mm after firm blows with pick point; core 150mm long and 50mm diameter can be broken by hand; sharp edges of core friable
M	Medium	0.3 ⩽ Is50 < 1.0	Readily scored with knife; core 150mm long and 50mm diameter can be broken by hand with difficulty
H	High	1 ⩽ Is50 < 3	Core 150mm long and 50mm diameter cannot be broken by hand but can be broken by single firm blow of pick; rock rings under hammer
VH	Very High	3 ⩽ Is50 < 10	Hand held specimen breaks with pick after more than one blow; rock rings under hammer
EH	Extremely High	10 ⩽ Is50	Specimen requires many pick blows to break intact rock, rock rings under hammer

ROCK WEATHERING CLASSIFICATION

LOG SYMBOL		TERM	DEFINITION
EW		Extremely Weathered	Rock is weathered to such an extent that it has soil properties, i.e. it either disintegrates or can be remoulded in water
DW	HW	Distinctly Weathered	Rock strength usually changed by weathering. May be discoloured. Porosity may be increased by leaching, or may be decreased by deposition of weathering products in pores. Subdivided into HW and MW with alteration less for MW
	MW		
SW		Slightly Weathered	Rock is slightly discoloured but shows little or no change of strength from fresh rock
FR		Fresh	Rock shows no sign of decomposition or staining

ROCK MASS PROPERTIES

TERM	SEPARATION OF STRATIFICATION PLANES
Thinly laminated	< 6mm
Laminated	6mm to 20mm
Very thinly bedded	20mm to 60mm
Thinly bedded	60mm to 200mm
Medium bedded	0.2m to 0.6m
Thickly bedded	0.6m to 2.0m
Massive	> 2m

TERM	DESCRIPTION
Fragmented	Primarily fragments < 20mm length and mostly of width < core diameter
Highly fractured	Core lengths generally less than 20mm to 40mm with occasional fragments
Fractured	Core lengths mainly 30mm to 100mm with occasional shorter and longer pieces
Slightly fractured	Core lengths generally 0.3m to 1.0m with occasional longer and shorter sections
Unbroken	Core has no fractures

ROCK QUALITY DESIGNATION (RQD). RQD is calculated for each core run. The RQD is the sum of the length of all pieces of rock core longer than 100mm expressed as a percentage of the total length of rock core recovered.

CORE RECOVERY. Core recovery is calculated for each core run. Core recovery is the total length of core, rock or soil, recovered expressed as a percentage of the total length of the core run.

ROCK DEFECT DESCRIPTION - Description order: type, orientation in degrees, infill, infill thickness, surface shape, roughness

DEFECT TYPE		INFILL		INFILL THICKNESS		SURFACE SHAPE		ROUGHNESS	
LOG SYMBOL	TERM	LOG SYMBOL	TERM	LOG SYMBOL	TERM	LOG SYMBOL	TERM	LOG SYMBOL	TERM
BP	Bedding parting	KL	Clean	V	Veneer	PL	Planar	SL	Slickensided
JT	Joint	CL	Clay		<1mm thick	CV	Curved	PO	Polished
FT	Fault	CA	Carbonate	SN	Stain	IR	Irregular	SO	Smooth
SM	Seam	RF	Rock fragments		<1mm thick	UN	Undulose	RO	Rough
SH	Sheared zone	RC	Rock fragments and clay	5	5mm thick	ST	Stepped	VR	Very Rough
CR	Crushed seam								
IF	Infilled zone								
FR	Fractured zone								

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APPENDIX B

Bore Logs

BOREHOLE LOG

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1/63 Industrial Drive BRAESIDE VIC 3195
T: (03) 9587 1811 F: (03) 9587 9411
E-mail: enquiries@geoaust.com.au

TEST LOCATION
1
SHEET 1 of 3

JOB No: 1624
CLIENT: Fasham Johnson Pty Ltd
PROJECT: Proposed Residential Development
14 ViewPoint Road, MCCRAE
LOCATION: Refer to Test Location Plan (Figure 1)
RL:
DATUM:
DRILLED BY: C.C
LOGGED BY: S.M
DATE: 17/06/2009

Method	Ground Water	Depth	Classification Symbol	Material description	Moisture / Weathering	Density / Consistency	Test depth	Comments and Test Results
100mm Diameter Auger		0.3	-	**FILL: Silty Sand,** fine to medium grained, with tree roots, dark grey	Moist	APC		
100mm Diameter Auger		0.9	SM	**SAND:** fine to medium grained, silty, grey tending pale grey with depth	Moist	MD		
100mm Diameter Auger		3.1	SM	**SAND:** fine to medium grained, silty, with fine grained gravel, pale brown-grey tending pale brown with depth	Moist	MD	1.5	7/8/8 N = 16.
							3.0	10/33/- N > 33. Hammer double bouncing
100mm Diameter Auger / Wash Boring with Drag Bit		5.1	SM	**SAND:** fine to coarse grained, silty, with clay fines, with fine to coarse grained granite gravel, brown with grey	Moist	VD	4.5	18/25 blows for 50mm penetration: N > 25. Hammer double bouncing
Wash Boring with Drag Bit		7.5	SC	**SAND:** fine to medium grained, very clayey, trace coarse grained sand with silt fines, grey (completely weathered granodiorite)	Moist	VD	6.5	18/25/- N > 25.
Wash Boring with Drag Bit		9	CL	**CLAY:** medium plasticity, silty, with sand, grey mottled yellow-brown	MC<PL	H	8.0	8/14/17 N = 31.
Wash Boring with Drag Bit			SM	**SAND:** fine to medium grained, trace coarse grained sand, silty, trace clay fines, grey	Moist	VD	9.5	21/33/- N > 33.

Refer Appendix A for definition of logging terms and symbols

BOREHOLE LOG

GeoAust
Geotechnical Engineers
1/63 Industrial Drive BRAESIDE VIC 3195
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TEST LOCATION 1
SHEET 2 of 3

JOB No: 1624
CLIENT: Fasham Johnson Pty Ltd
PROJECT: Proposed Residential Development
14 ViewPoint Road, MCCRAE
LOCATION: Refer to Test Location Plan (Figure 1)
DRILLED BY: C.C
LOGGED BY: S.M
RL:
DATUM:
DATE: 17/06/2009

Method	Ground Water	Depth	Classification Symbol	Material description	Moisture / Weathering	Density / Consistency	Test depth	Comments and Test Results
Wash Boring with Drag Bit		10.0–10.5			Moist	VD		
		10.5–12	SM	**SAND:** fine to medium grained, silty, trace clay fines, grey, trace yellow-brown	Moist	D to VD	11.0–11.5	10/18/34 N = 52.
		12–14	CH	**CLAY:** high plasticity, silty, trace sand, grey, trace yellow-brown	MC>PL	VSt	12.5–13.0	5/7/11 N = 18.
		14–15	SM	**SAND:** fine to coarse grained, silty, trace clay fines, occasional seams and bands of cemented sand, grey mottled yellow-brown	Moist	VD	14.0–14.5	17/30/30 blows for 50mm penetration: N > 60. Hammer double bouncing
		15–16.5	CL	**CLAY:** medium plasticity, silty and sandy, trace coarse grained sand, grey with orange-brown, trace red-brown	MC>PL	VSt	15.5–16.0	5/9/8 N = 17.
	6.0hr ▼	16.5–20.0	SM	**SAND:** fine to medium grained, silty, trace coarse grained sand, trace clay fines, grey, mottled yellow-brown	Moist	VD	17.0–17.5; 18.5–19.0	13/25/26 N = 51.; 21/23/27 N = 50.

Refer Appendix A for definition of logging terms and symbols

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BOREHOLE LOG

TEST LOCATION 1
SHEET 3 of 3

JOB No: 1624
CLIENT: Fasham Johnson Pty Ltd
PROJECT: Proposed Residential Development
14 ViewPoint Road, MCCRAE
LOCATION: Refer to Test Location Plan (Figure 1)
RL:
DATUM:
DRILLED BY: C.C
LOGGED BY: S.M
DATE: 17/06/2009

Method	Ground Water	Depth	Graphic Log	Classification Symbol	Material description	Moisture / Weathering	Density / Consistency	Sample (DS / U60 / ES)	Depth	DCP Test	Test	Comments and Test Results
Wash Boring with Drag Bit		20.2										25 blows for 105mm penetration: SPT. Hammer double bouncing
				SM	**SAND:** fine to medium grained, silty, trace coarse grained sand, with grandiorite gravel and cobbles, grey and yellow-brown	Moist	VD		20.5 21.0			
		21.5		SC	**SAND:** fine to coarse grained, clayey, with silt fines, with mica, trace fine grained granodiorite gravel, grey and orange-brown	-	D		21.5 22.0 22.5 23.0			10/15/19 N = 34. 13/17/22 N = 39.
		23.5		SM	**SAND:** fine to medium grained, silty, with seams and bands of clayey sand, grey with yellow-brown	-	D		23.5 24.0 24.5 25.0			12/16/24 N = 40.
		25										
					END OF BOREHOLE LOG AT 25M							

Refer Appendix A for definition of logging terms and symbols

GeoAust
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BOREHOLE LOG

TEST LOCATION 2
SHEET 1 of 1

JOB No: 1624
CLIENT: Fasham Johnson Pty Ltd
PROJECT: Proposed Residential Development
14 ViewPoint Road, MCCRAE
LOCATION: Refer to Test Location Plan (Figure 1)
DRILLED BY: C.C
LOGGED BY: S.M
RL:
DATUM:
DATE: 17/06/2009

Method	Ground Water	Depth	Graphic Log	Classification Symbol	Material description	Moisture / Weathering	Density / Consistency	Sample DS	Sample U60	Sample ES	Depth	DCP Test	Test	Comments and Test Results
100mm Diameter Hand Auger	NOT ENCOUNTERED			SM	**SILTY SAND:** fine to medium grained, trace fine to coarse grained granodiorite gravel, grey-brown	Dry	MD				0.5			
		0.6		SM	**SILTY SAND:** fine to medium grained, trace fine to medium grained granodiorite gravel, yellow-brown	Dry	D							
		0.9		SC	**SAND:** fine to coarse grained, clayey, trace mica, dark grey and pale grey	Moist	D				1.0			
		1.5									1.5			
					END OF BOREHOLE LOG AT 1.5M									EFFECTIVE HAND AUGER REFUSAL ON DENSE CLAYEY SAND

Refer Appendix A for definition of logging terms and symbols

GeoAust
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BOREHOLE LOG

TEST LOCATION 3
SHEET 1 of 1

JOB No: 1624
CLIENT: Fasham Johnson Pty Ltd
PROJECT: Proposed Residential Development
14 ViewPoint Road, MCCRAE
LOCATION: Refer to Test Location Plan (Figure 1)
RL:
DATUM:
DRILLED BY: C.C
LOGGED BY: S.M
DATE: 17/06/2009

Method	Ground Water	Depth	Graphic Log	Classification Symbol	Material description	Moisture / Weathering	Density / Consistency	Sample (DS / U60 / ES)	Depth	DCP Test	Test	Comments and Test Results
100mm Diameter Hand Auger	NOT ENCOUNTERED	0.6		SM	**SAND:** fine to medium grained, silty, grey	Dry	MD		0.5			
		0.8		SM	**SAND:** fine to medium grained, silty, trace clay fines, yellow-brown and grey	Dry	MD					
		1		CL	**CLAY:** medium plasticity, silty, with sand, yellow-brown and grey	MC>PL	VSt		1.0			S > 120kPa
		3.4		SM	**SAND:** fine to medium grained, silty, trace clay fines, pale grey and yellow-brown	Moist	D to VD		1.5, 2.0, 2.5, 3.0			
					END OF BOREHOLE LOG AT 3.4M							EFFECTIVE HAND AUGER REFUSAL ON VERY DENSE SAND

Refer Appendix A for definition of logging terms and symbols

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APPENDIX C

Slope Stability Analysis

(Graphical Summaries of Critical Stability Analyses)

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Site Plan Showing Location of Section AA used in Galena Stability Analysis

JOB No: 1624
CLIENT: Fasham Johnson Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 14-16 ViewPoint Road, MCCRAE

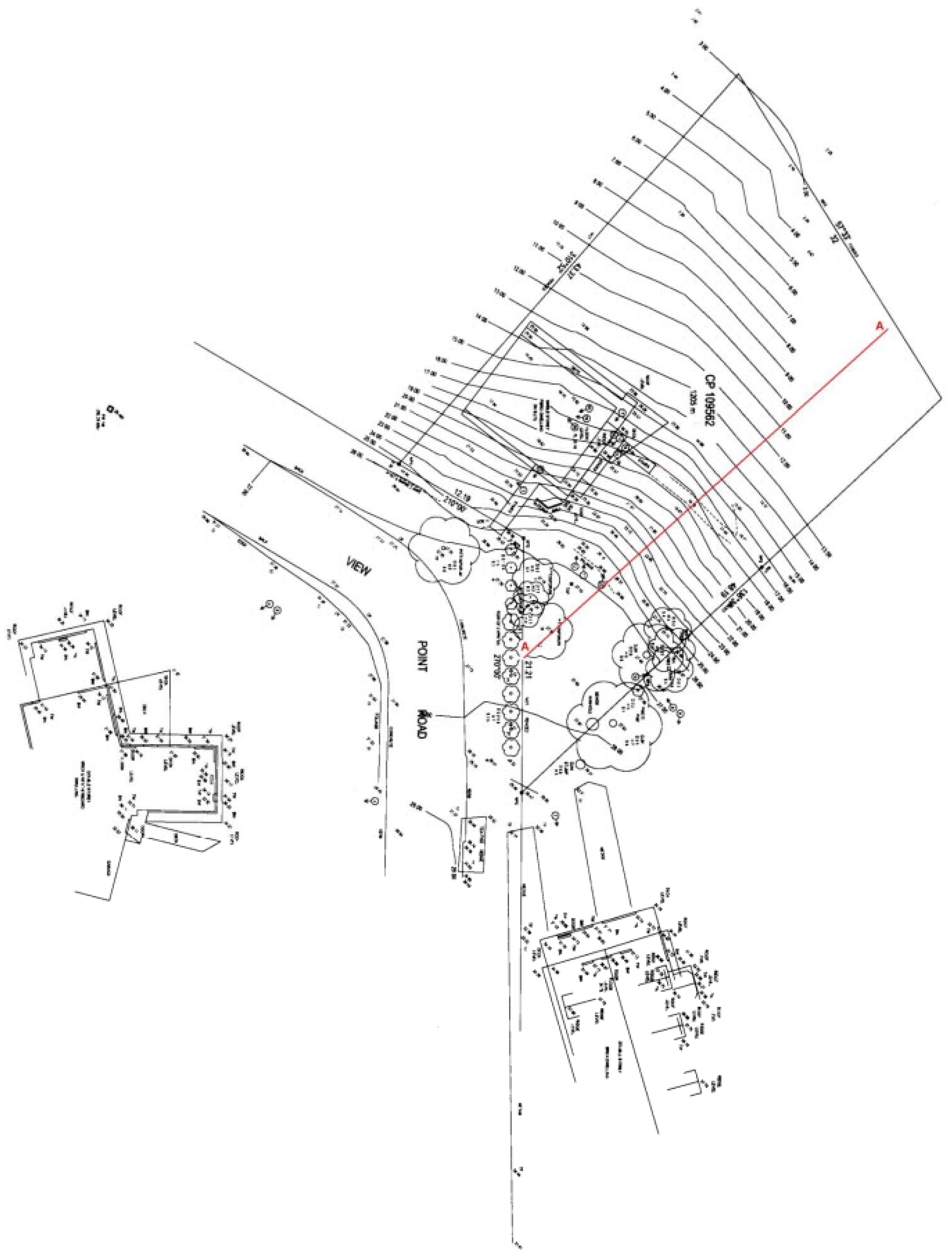
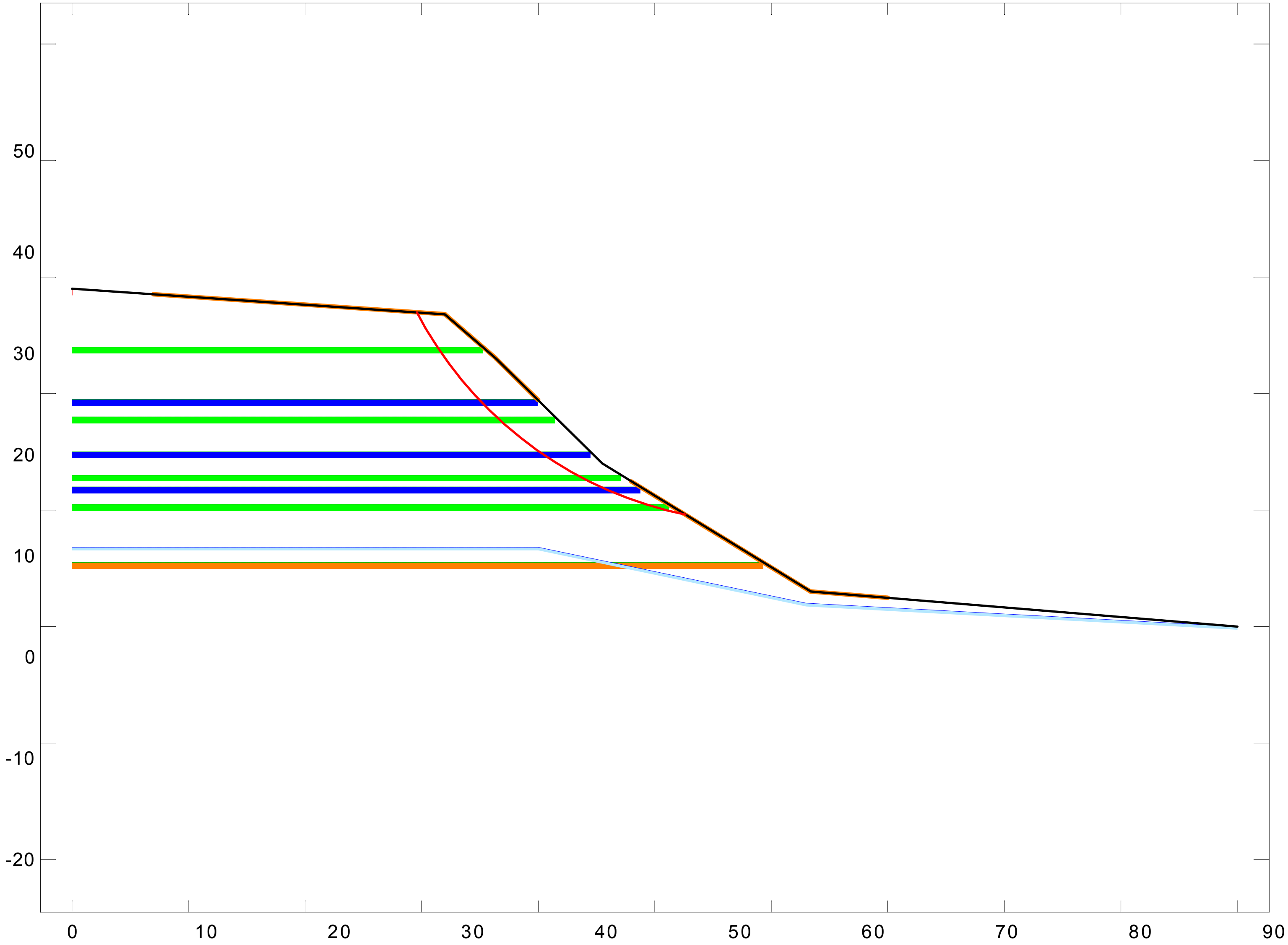


Figure C1

GALENA Version 5.02



Analysis: 1
Multiple Stability Analysis
Method: Bishop Simplified
Surface: Circular

Results
Critical (minimum) Factor of Safety: 1.14

100

Edited: 27 May 2011 *Processed:* 27 May 2011

Project: 14 - 16 View Point Road, McCrae
Stability of Existing Escarpment with No Site Cut and Dwelling - No Earthquake Load

File: C:\Program Files (x86)\Galena\SGM Galena Files\Model Files\1624 McCrae\1624-May 11 Existing Not Cut or Dwelling.gmf

Figure C2

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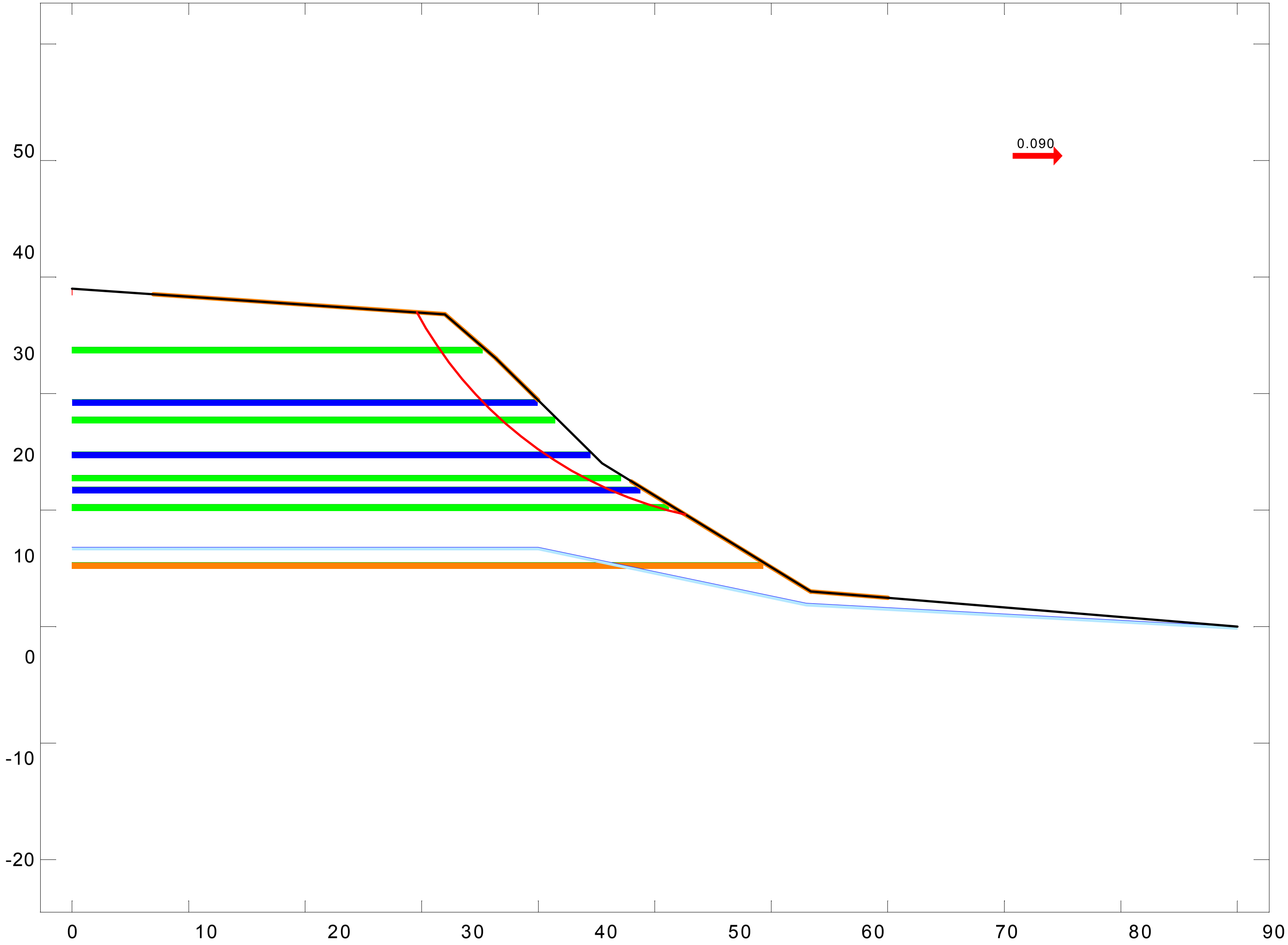
GALENA Version 5.02



Analysis: 1
Multiple Stability Analysis
Method: Bishop Simplified
Surface: Circular

Results
Critical (minimum) Factor of Safety: 0.98

100

Edited: 27 May 2011 *Processed:* 27 May 2011

Project: 14 - 16 View Point Road, McCrae
Stability of Existing Escarpment with No Site Cut and Dwelling - With Earthqauke Load

File: C:\Program Files (x86)\Galena\SGM Galena Files\Model Files\1624 McCrae\1624-May 11 Existing Not Cut or Dwelling.gmf

Figure C3

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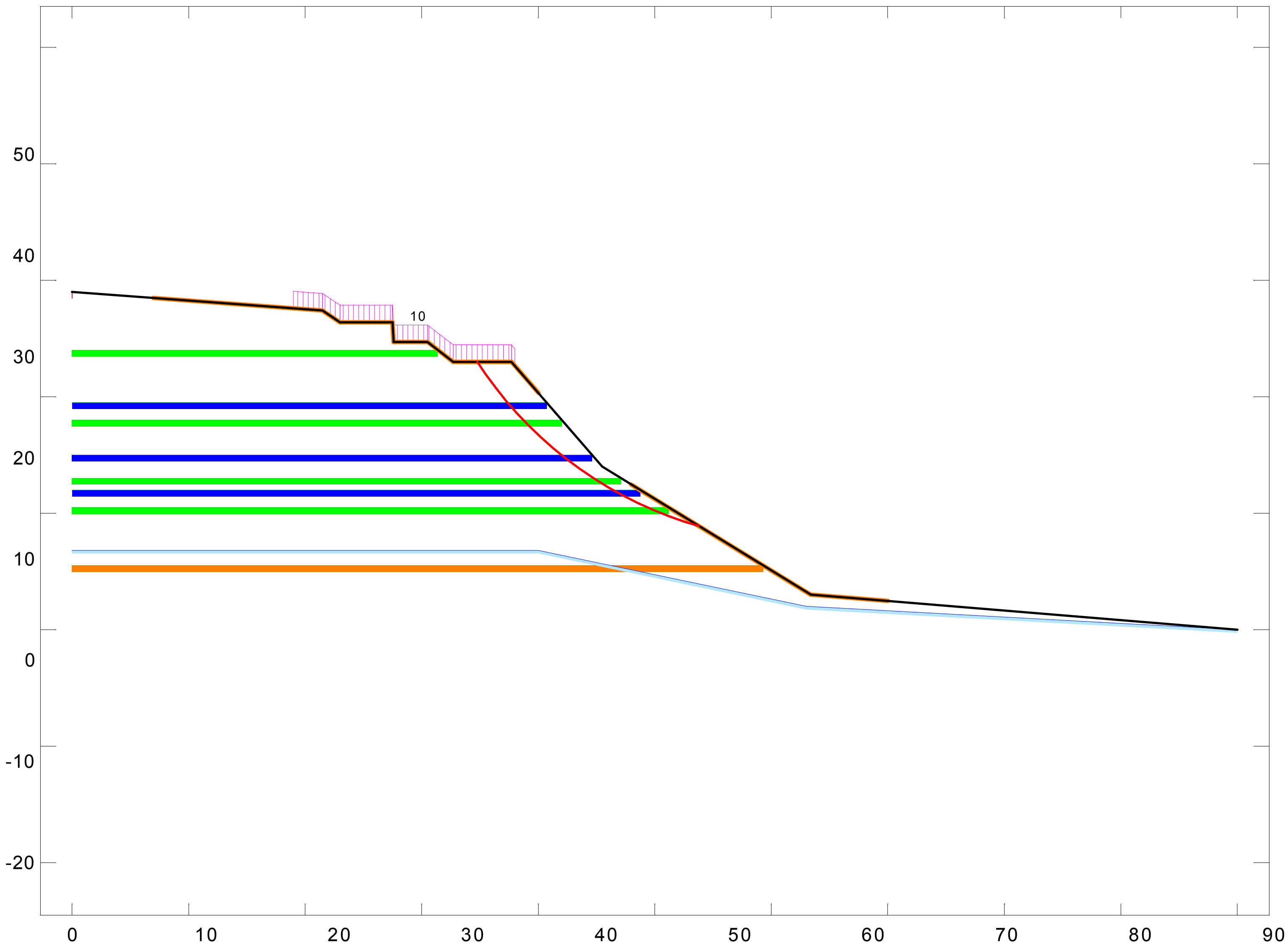
GALENA Version 5.02



Analysis: 1
Multiple Stability Analysis
Method: Bishop Simplified
Surface: Circular

Results
Critical (minimum)
Factor of Safety: 1.17

100

Edited: 27 May 2011 *Processed:* 27 May 2011

Project: 14 - 16 View Point Road, McCrae
Stability of Escarpment with Site Cut and Proposed Dwelling on Shallow Footings - No Earthquake Load
File: C:\Program Files (x86)\Galena\SGM Galena Files\Model Files\1624 McCrae\1624-May 11 No Piles No EQ Low GW.gmf

Figure C 4

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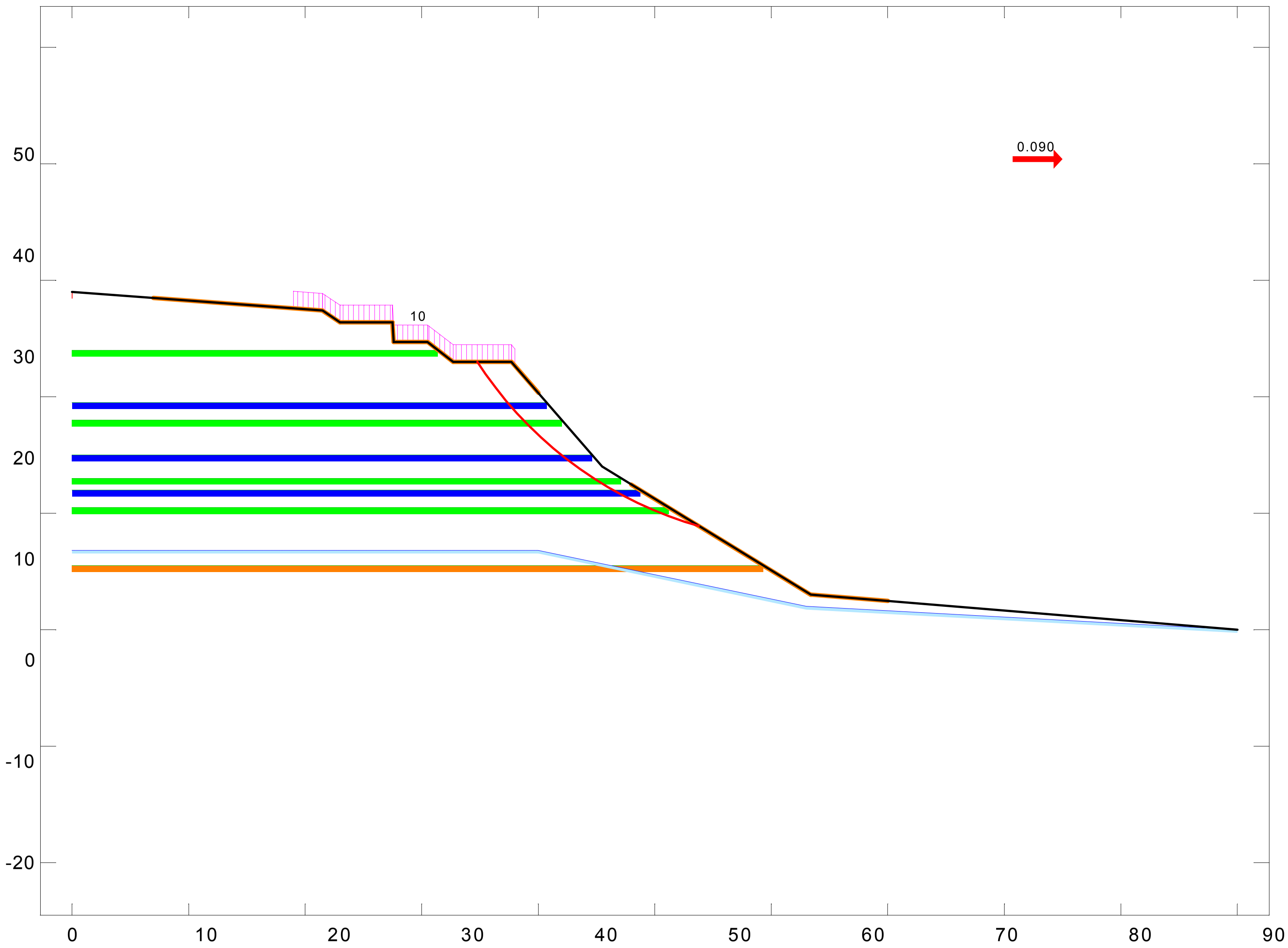
GALENA Version 5.02



Analysis: 1
Multiple Stability Analysis
Method: Bishop Simplified
Surface: Circular

Results
Critical (minimum) Factor of Safety: 1.01

100

Edited: 27 May 2011 *Processed:* 27 May 2011

Project: 14 - 16 View Point Road, McCrae
Stability of Escarpment with Site Cut and Proposed Dwelling on Shallow Footings - With Earthquake Load

File: C:\Program Files (x86)\Galena\SGM Galena Files\Model Files\1624 McCrae\1624-May 11 No Piles No EQ Low GW.gmf

Figure C 5

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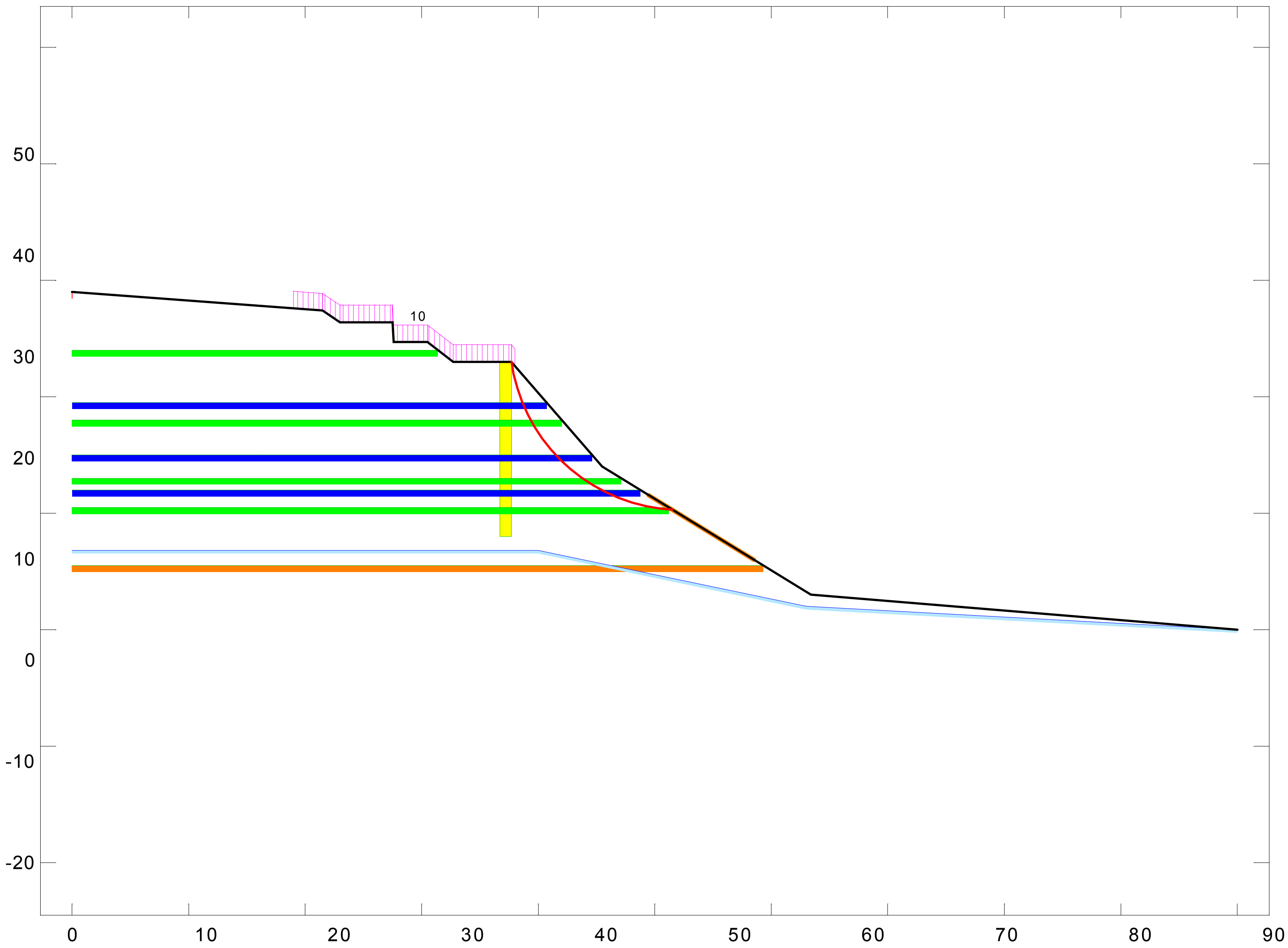

GALENA Version 5.02

Concrete

Analysis: 1

Multiple Stability Analysis

Method: Bishop Simplified

Surface: Circular

Results

Critical (minimum)
Factor of Safety: 1.17

100

Edited: 27 May 2011 *Processed:* 27 May 2011

Project: 14 - 16 View Point Road, McCrae
Stability of Face of Escarpment Incorporating Site Cut and Deep Piles - No Earthquake Load

File: C:\Program Files (x86)\Galena\SGM Galena Files\Model Files\1624 McCrae\1624-May 11 15m Piles No EQ Low GW.gmf

Figure C6

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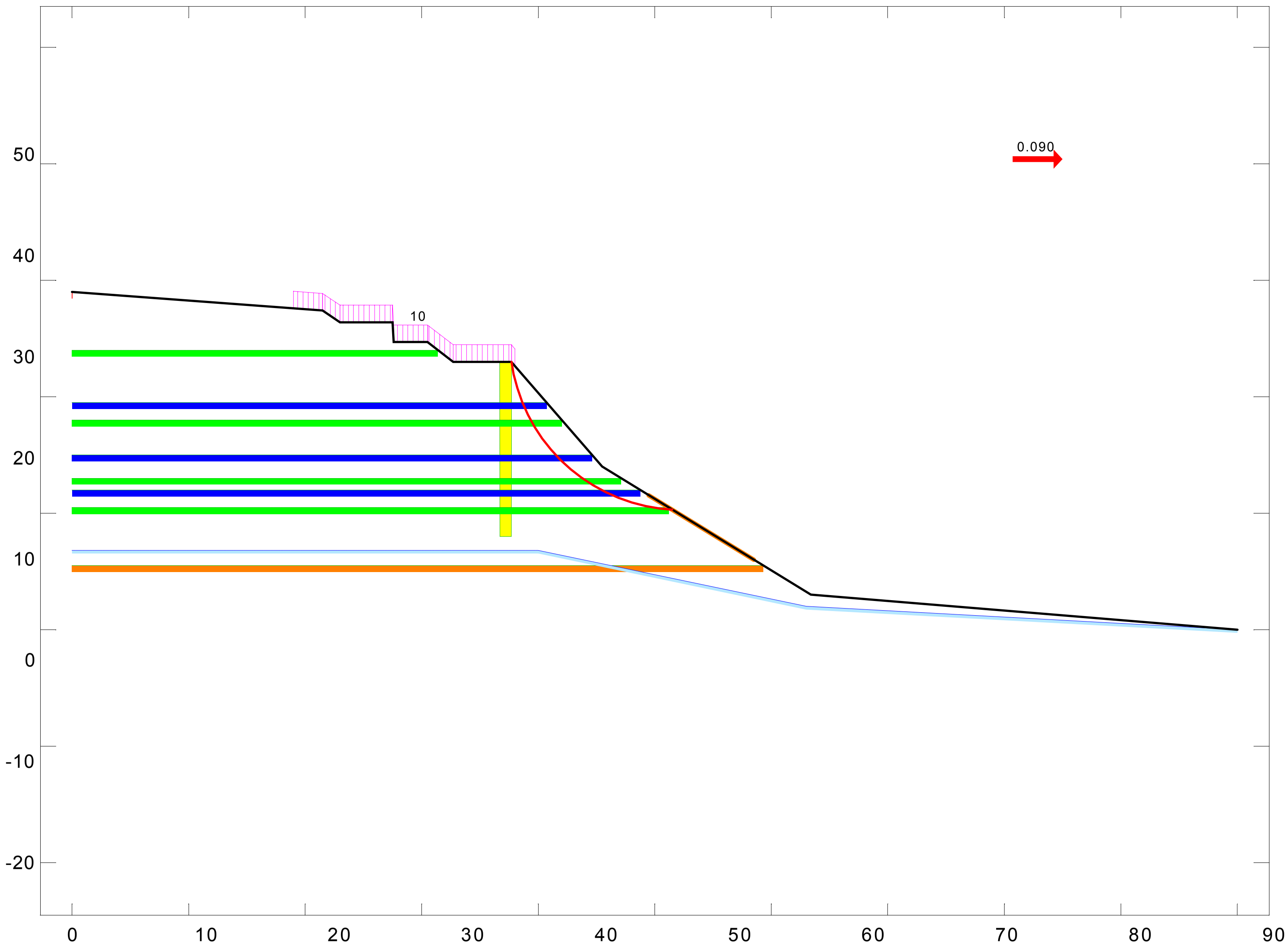
MSC.5065.0001.0052



GALENA Version 5.02

Concrete

Analysis: 1

Multiple Stability Analysis

Method: Bishop Simplified

Surface: Circular

Results

Critical (minimum) Factor of Safety: 1.02

100

Edited: 27 May 2011 *Processed:* 27 May 2011

Project: 14 - 16 View Point Road, McCrae

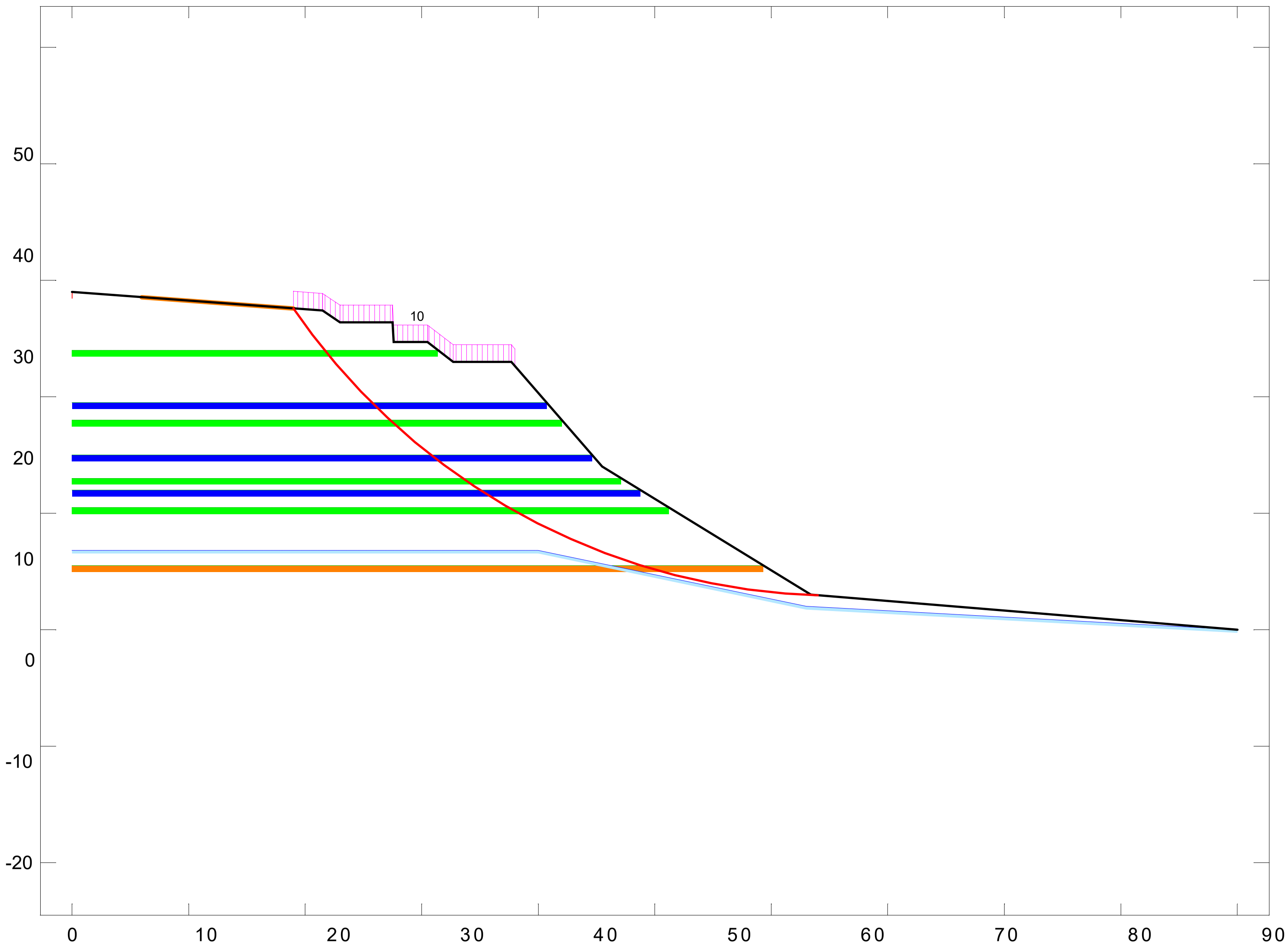
Stability of Face of Escarpment Incorporating Site Cut and Deep Piles - With Earthquake Load

File: C:\Program Files (x86)\Galena\SGM Galena Files\Model Files\1624 McCrae\1624-May 11 15m Piles No EQ Low GW.gmf

Figure C7

GeoAust Geotechnical Engineers

GALENA Version 5.02



Analysis: 1
Multiple Stability Analysis
Method: Bishop Simplified
Surface: Circular

Results
Critical (minimum)
Factor of Safety: 1.51

100

Edited: 14 Sep 2011 *Processed:* 14 Sep 2011

Project: 14 - 16 View Point Road, McCrae
Failure Rupture Surface for a Factor of Safety of 1.51

File: C:\Program Files (x86)\Galena\SGM Galena Files\Model Files\1624 McCrae\1624-May 11 No Piles No EQ Low GW.gmf

Figure C 8

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APPENDIX D

Terminology used in Landslide Risk Assessment

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}	$5x10^{-3}$	100 years	200 years	The event will probably occur under adverse conditions over the design life.	LIKELY	B
10^{-3}	$5x10^{-4}$	1000 years	2000 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10^{-4}	$5x10^{-5}$	10,000 years	20,000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10^{-5}	$5x10^{-6}$	100,000 years	200,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
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%	100%	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
60%	40%	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
20%	10%	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
5%	1%	Limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works.	MINOR	4
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

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.

GeoAust
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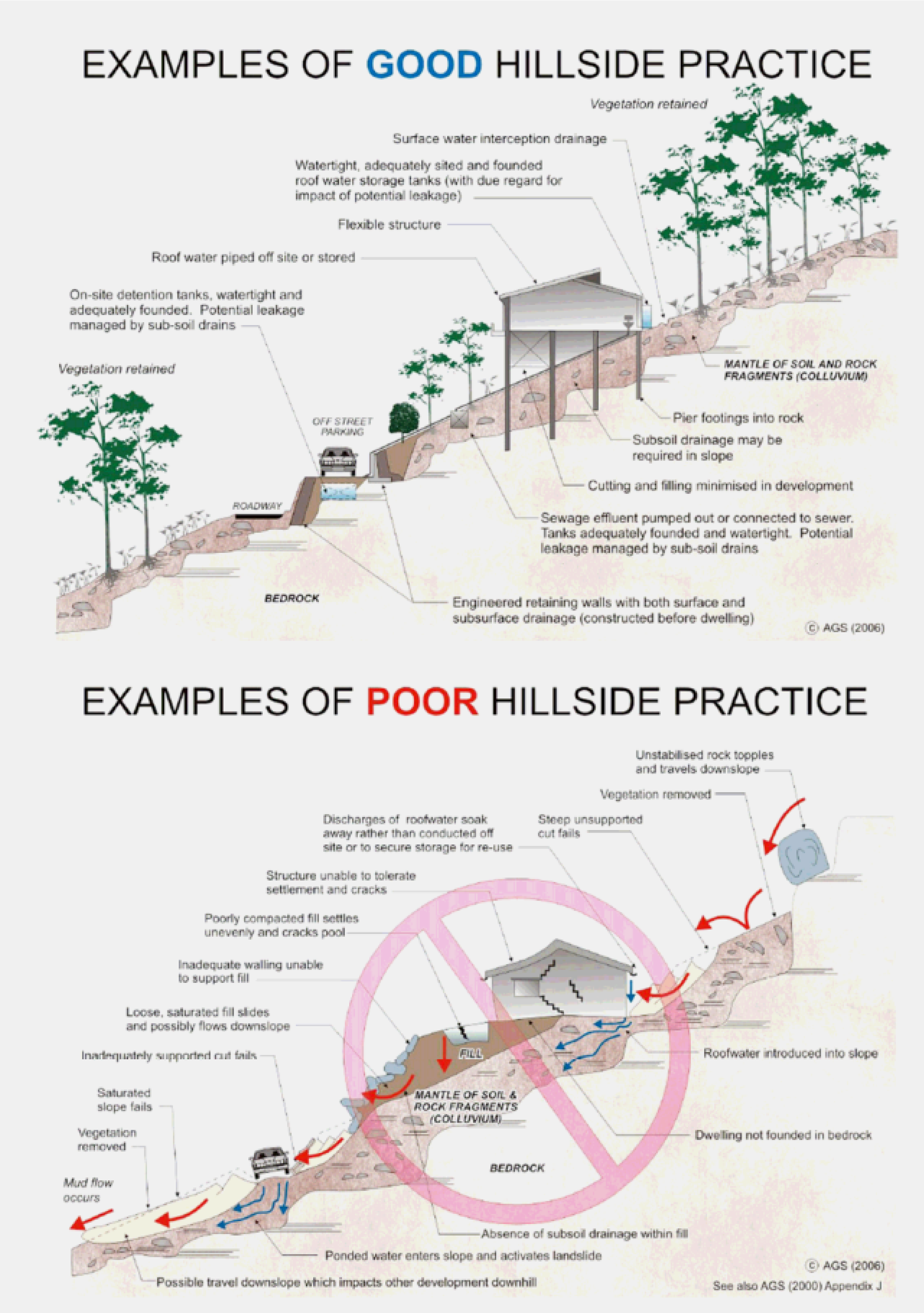
APPENDIX E

Guidelines for Hillside Construction

PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT 2007

SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

	GOOD ENGINEERING PRACTICE	*POOR ENGINEERING PRACTICE*
ADVICE		
GEOTECHNICAL ASSESSMENT	Obtain advice from a qualified, experienced geotechnical practitioner at early stage of planning and before site works.	Prepare detailed plan and start site works before geotechnical advice.
PLANNING		
SITE PLANNING	Having obtained geotechnical advice, plan the development with the risk arising from the identified hazards and consequences in mind.	Plan development without regard for the Risk.
DESIGN AND CONSTRUCTION		
HOUSE DESIGN	Use flexible structures which incorporate properly designed brickwork, timber or steel frames, timber or panel cladding. Consider use of split levels. Use decks for recreational areas where appropriate.	Floor plans which require extensive cutting and filling. Movement intolerant structures.
SITE CLEARING	Retain natural vegetation wherever practicable.	Indiscriminately clear the site.
ACCESS & DRIVEWAYS	Satisfy requirements below for cuts, fills, retaining walls and drainage. Council specifications for grades may need to be modified. Driveways and parking areas may need to be fully supported on piers.	Excavate and fill for site access before geotechnical advice.
EARTHWORKS	Retain natural contours wherever possible.	Indiscriminatory bulk earthworks.
CUTS	Minimise depth. Support with engineered retaining walls or batter to appropriate slope. Provide drainage measures and erosion control.	Large scale cuts and benching. Unsupported cuts. Ignore drainage requirements
FILLS	Minimise height. Strip vegetation and topsoil and key into natural slopes prior to filling. Use clean fill materials and compact to engineering standards. Batter to appropriate slope or support with engineered retaining wall. Provide surface drainage and appropriate subsurface drainage.	Loose or poorly compacted fill, which if it fails, may flow a considerable distance including onto property below. Block natural drainage lines. Fill over existing vegetation and topsoil. Include stumps, trees, vegetation, topsoil, boulders, building rubble etc in fill.
ROCK OUTCROPS & BOULDERS	Remove or stabilise boulders which may have unacceptable risk. Support rock faces where necessary.	Disturb or undercut detached blocks or boulders.
RETAINING WALLS	Engineer design to resist applied soil and water forces. Found on rock where practicable. Provide subsurface drainage within wall backfill and surface drainage on slope above. Construct wall as soon as possible after cut/fill operation.	Construct a structurally inadequate wall such as sandstone flagging, brick or unreinforced blockwork. Lack of subsurface drains and weepholes.
FOOTINGS	Found within rock where practicable. Use rows of piers or strip footings oriented up and down slope. Design for lateral creep pressures if necessary. Backfill footing excavations to exclude ingress of surface water.	Found on topsoil, loose fill, detached boulders or undercut cliffs.
SWIMMING POOLS	Engineer designed. Support on piers to rock where practicable. Provide with under-drainage and gravity drain outlet where practicable. Design for high soil pressures which may develop on uphill side whilst there may be little or no lateral support on downhill side.	
DRAINAGE		
SURFACE	Provide at tops of cut and fill slopes. Discharge to street drainage or natural water courses. Provide general falls to prevent blockage by siltation and incorporate silt traps. Line to minimise infiltration and make flexible where possible. Special structures to dissipate energy at changes of slope and/or direction.	Discharge at top of fills and cuts. Allow water to pond on bench areas.
SUBSURFACE	Provide filter around subsurface drain. Provide drain behind retaining walls. Use flexible pipelines with access for maintenance. Prevent inflow of surface water.	Discharge roof runoff into absorption trenches.
SEPTIC & SULLAGE	Usually requires pump-out or mains sewer systems; absorption trenches may be possible in some areas if risk is acceptable. Storage tanks should be water-tight and adequately founded.	Discharge sullage directly onto and into slopes. Use absorption trenches without consideration of landslide risk.
EROSION CONTROL & LANDSCAPING	Control erosion as this may lead to instability. Revegetate cleared area.	Failure to observe earthworks and drainage recommendations when landscaping.
DRAWINGS AND SITE VISITS DURING CONSTRUCTION		
DRAWINGS	Building Application drawings should be viewed by geotechnical consultant	
SITE VISITS	Site Visits by consultant may be appropriate during construction/	
INSPECTION AND MAINTENANCE BY OWNER		
OWNER'S RESPONSIBILITY	Clean drainage systems; repair broken joints in drains and leaks in supply pipes. Where structural distress is evident see advice. If seepage observed, determine causes or seek advice on consequences.	


EXAMPLES OF GOOD HILLSIDE PRACTICE
Vegetation retained
Surface water interception drainage
Watertight, adequately sited and founded roof water storage tanks (with due regard for impact of potential leakage)
Flexible structure
Roof water piped off site or stored
On-site detention tanks, watertight and adequately founded. Potential leakage managed by sub-soil drains
Vegetation retained
OFF STREET PARKING
ROADWAY
MANTLE OF SOIL AND ROCK FRAGMENTS (COLLUVIUM)
Pier footings into rock
Subsoil drainage may be required in slope
Cutting and filling minimised in development
Sewage effluent pumped out or connected to sewer. Tanks adequately founded and watertight. Potential leakage managed by sub-soil drains
BEDROCK
Engineered retaining walls with both surface and subsurface drainage (constructed before dwelling)
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EXAMPLES OF POOR HILLSIDE PRACTICE
Unstabilised rock topples and travels downslope
Vegetation removed
Discharges of roofwater soak away rather than conducted off site or to secure storage for re-use
Steep unsupported cut fails
Structure unable to tolerate settlement and cracks
Poorly compacted fill settles unevenly and cracks pool
Inadequate walling unable to support fill
Loose, saturated fill slides and possibly flows downslope
Inadequately supported cut fails
FILL
Roofwater introduced into slope
Saturated slope fails
MANTLE OF SOIL & ROCK FRAGMENTS (COLLUVIUM)
Vegetation removed
Dwelling not founded in bedrock
Mud flow occurs
BEDROCK
Absence of subsoil drainage within fill
Ponded water enters slope and activates landslide
Possible travel downslope which impacts other development downhill
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See also AGS (2000) Appendix J