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15th September 2009

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Mornington Peninsula Shire
Anthony Matthews- Team Leader, Statutory Planning
Private Bag 1000
Rosebud 3939

MORNINGTON PENINSULA SHIRE
RECEIVED
1118
Anthony Matthews
P08/1163

Dear Anthony

PLANNING APPLICATION P08/1163
14-16 VIEW POINT ROAD, MCCRAE

I refer to the above planning application and Council most recent correspondence dated 19th August 2009.

Please find attached 2 copies of a Geotechnical Report prepared by GeoAust Geotechnical Engineers Pty Ltd that outlines how appropriate construction methods can be undertaken to avoid adverse affects of land slippage. The attached information also includes schematic details of these footings consistent with the recommendations outlined by the Geotechnical Engineer.

The construction methods to be employed do not alter the appearance, height, siting or design of the dwelling currently before Council.

As such, we believe Council now has adequate information to progress the application to a decision.

Should you require any further information I can be contacted on 5986 1323.

Kind regards

Irrelevant / Sensitive

Jackie Prossor
Town Planning Consultant

GeoAust
Geotechnical Engineers

Mr Brian Stacey
Fasham Johnson Pty Ltd
PO Box 8242
ARMADALE VIC 3143

11 September 2009

REF: 1624-4-L

Dear Brian,

REVIEW OF SCHEMATIC FOOTING DESIGN

Proposed Residential Dwelling: 14-16 View Point Road McCRAE VIC.

We confirm receipt of schematic section (SK1) and plan (SK2) from Eckhaus Story and Partners Pty Ltd relating to the conceptual footing system for the proposed dwelling at the above site. Copies of the sketches are attached.

Geotechnical investigation for the proposed development was completed by GeoAust and presented in report with reference 1624-2-R, dated 18th August 2009.

We understand the footing design will be further refined once the design of the proposed structure is progressed. However, in concept the proposed schematic footing system is consistent with the recommendations contained within our geotechnical report. On this basis the risk to property for the proposed dwelling is low and the risk to life for the occupants of the proposed dwelling is tolerable, based on a landslide risk assessment in accordance with The Australian Geomechanics Society 'Practice Note Guidelines for Landslide Risk Management', 2007.

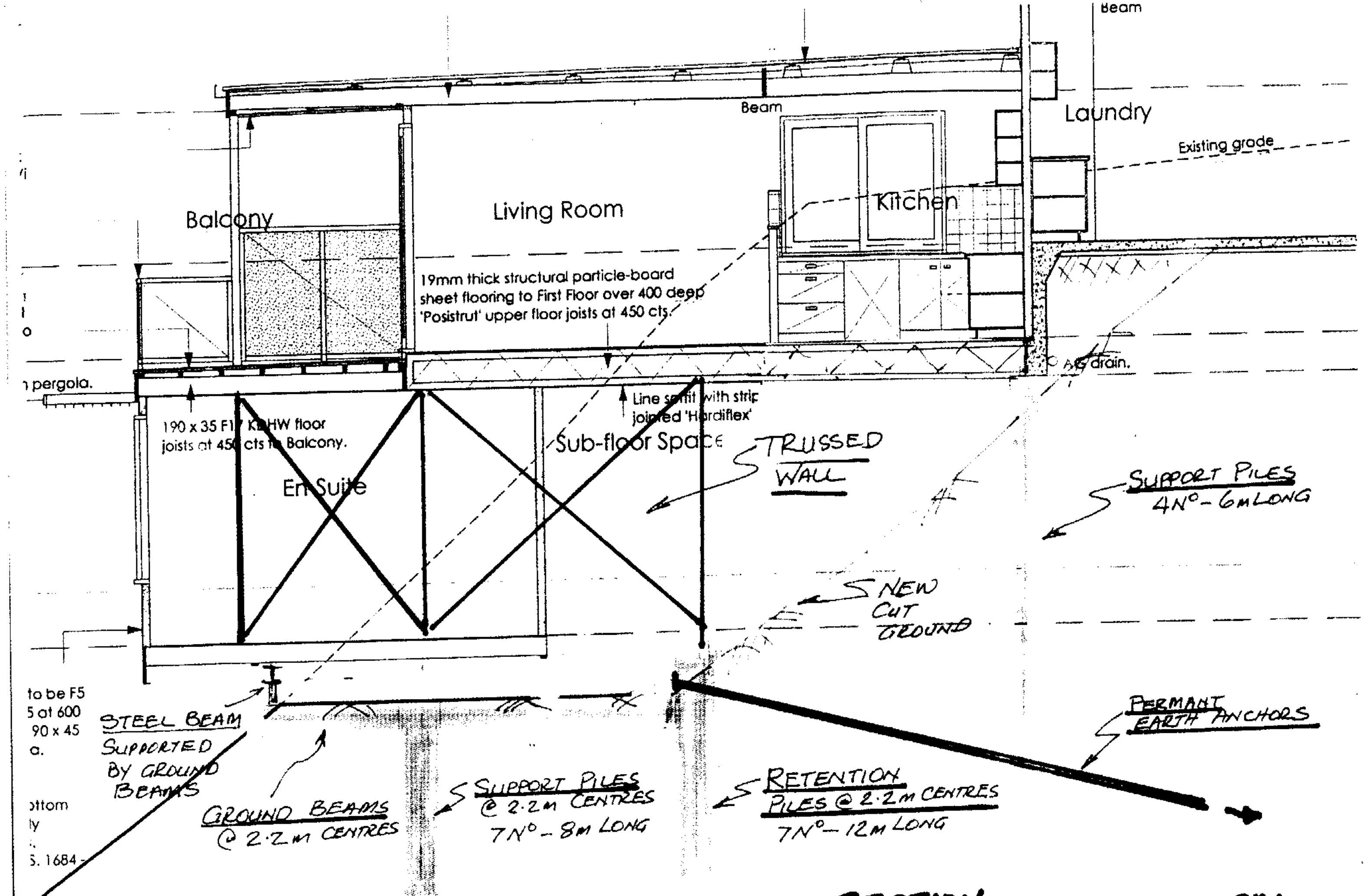
The final design of the footing and retention systems for the proposed development must be approved by this office prior to issue of the building permit.

Should you require any further information or clarification of any part of this letter please contact the undersigned.

Yours faithfully

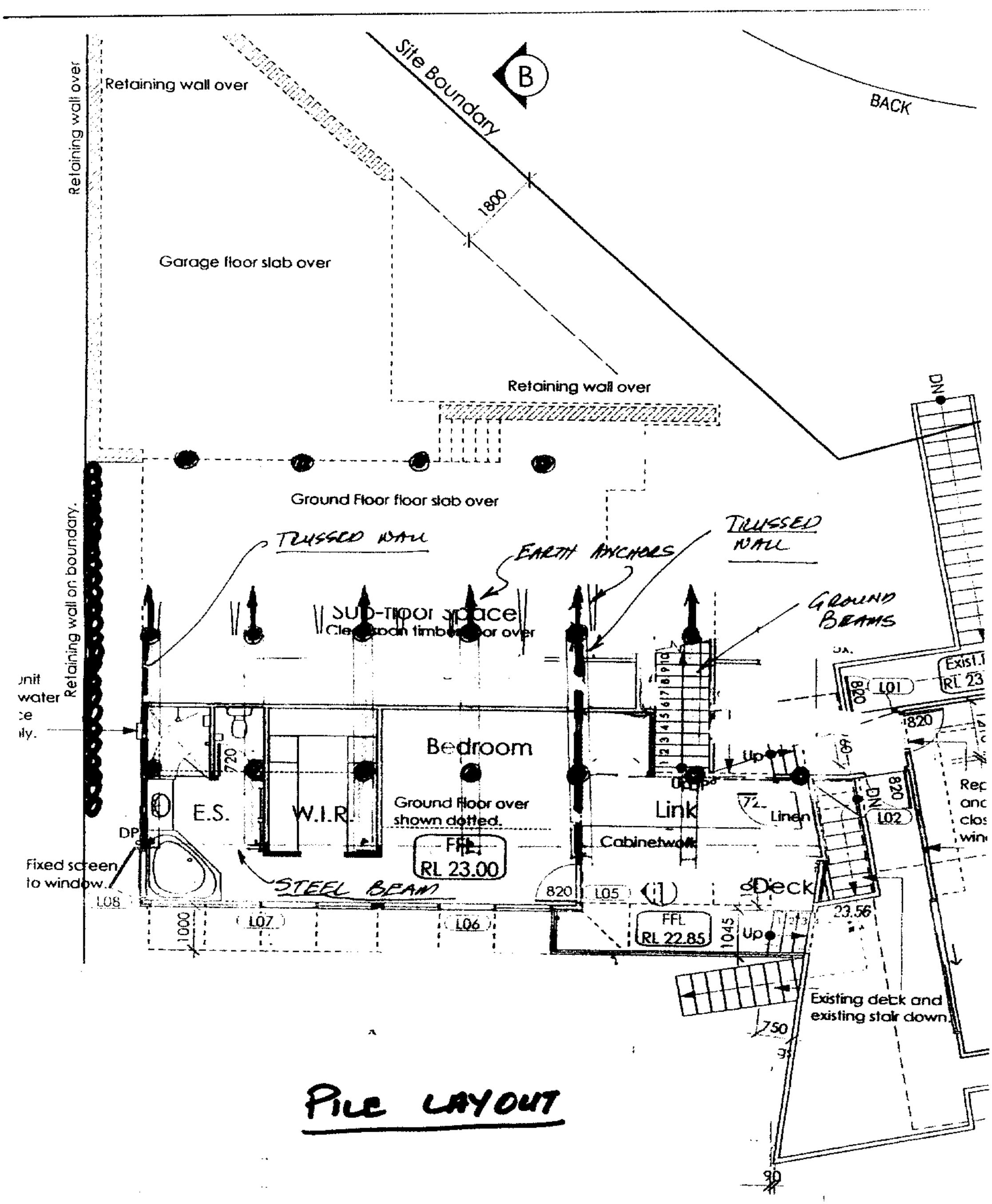
Irrelevant / Sensitive

Stephen Mayer
BEng MIEAust CPEng EC-2262



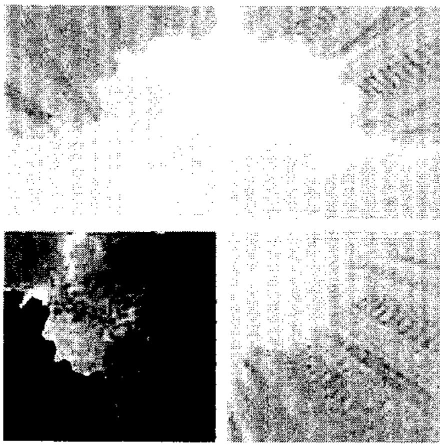
SCHEMATIC SECTION

SK1



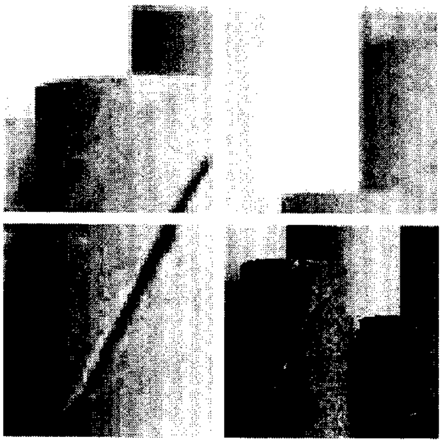
PILE LAYOUT

SK2



GeoAust

Geotechnical Engineers



PROPOSED RESIDENTIAL DWELLING
14-16 VIEW POINT ROAD
McCRAE VIC

PREPARED FOR
FASHAM JOHNSON PTY LTD

JOB NO: 1624-2-R
18 AUGUST 2009

DISTRIBUTION:
FASHAM JOHNSON PTY LTD
ECKHAUS STOREY PARTNERS PTY LTD

GeoAust Geotechnical Engineers Pty Ltd
ACN: 114 447 371 ABN: 14 030 388 760

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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 is to comprise construction of a new residential dwelling to the north east of the existing dwelling at 14 – 16 View Point Road, McCrae. The dwelling is proposed to be located at the top of an escarpment which has an approximate height of approximately 22 metres. Based on the plan extracts provided to us we understand the proposed dwelling will comprise three levels. The upper level will approximately coincide with the ground level at the top of the escarpment. The two lower levels will extend out over the upper edge of the escarpment and will be supported on a series of steel columns. It is understood bulk excavation to a reduced level of approximately 23.0 metres is proposed to accommodate the lower ground floor level of the proposed dwelling.

The precise 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 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 form 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 documented in the immediate area.

2 INVESTIGATION METHODS

2.1 FIELD METHODS

Field work 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 115mm 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 Australian Standard test procedure 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.

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 22 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 the south west corner of the site, which is proposed to be retained. 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.
- 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.

3.2 SUBSURFACE CONDITIONS

The logs of the boreholes are provided in Appendix B.

Borehole 1 located adjacent to the top edge of the escarpment 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 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.

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 upon completion of drilling to allow monitoring of the ground water level. The standpipe was screened over the lower 12 metres and the annulus was backfilled with sand. A bentonite seal was provided near the surface. With six hours of installation of the ground water monitoring standpipe the water level was measured to be present at a depth of 16.5 metres below the existing ground surface.

4 STABILITY ANALYSIS

Analysis of the stability of the escarpment was performed using Galena version 4.02 slope stability analysis software. The analysis considered the stability of Section A-A shown in Figure 1.

The stability analysis was conducted on a model based on the soil profile intersected in Borehole 1. The following soil profile was used in the stability analysis:

- Medium Dense Sand: $Ø' = 30^0$, $c' = 0$ kPa, $\gamma = 20$ kN/m^3
- Clay: $Ø' = 24^0$, $c' = 10$ kPa, $\gamma = 18$ kN/m^3
- Dense Sand: $Ø' = 36^0$, $c' = 0$ kPa, $\gamma = 21$ kN/m^3
- Very Dense Sand $Ø' = 42^0$, $c' = 0$ kPa, $\gamma = 22$ kN/m^3

The above soil strength parameters were selected based on the following.

- Published correlations between soil classification and soil parameters.
- Results of field classification testing and in situ testing completed within the borehole.
- Previous experience in assessing soil properties in the general area.

A graphical summary of the critical stability analysis is given in Appendix C, Figures C1 and C2.

Figure C1 represents the site with the proposed bulk excavation at the top of the escarpment, with no earthquake loading. The critical failure surface returned a factor of safety (FoS) of 1.24. The shape of the critical failure surface approximately corresponds to the observed shape of the failure which took place at 6 View Point Road. When an earthquake load is introduced (Figure C2) the critical failure surface returned a FoS of 1.05.

A FoS of 1.0 corresponds to the state at which forces driving failure are equal to those resisting failure. A FoS less than 1.0 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.

5 LANDSLIDE RISK ASSESSMENT

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

Assessment of risk has been made based on the currently prevailing site conditions, assuming that not measures are taken to stabilise the escarpment prior to development. Section 5.6 provides a discussion of measures to reduce risk.

5.1 IDENTIFICATION OF HAZARDS

Hazard A: Collapse of the escarpment on which the proposed dwelling is proposed to be constructed. 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 be 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.

5.2 FREQUENCY ANALYSIS

Hazard A: Hazard A is considered POSSIBLE as it is may occur within the design life of the proposed development.

5.3 CONSEQUENCES TO PROPERTY

A qualitative approach has been adopted for assessment of risk to property. The assessment is made on the basis that no effort is made to reduce the risk of landslide risk at the subject site.

Hazard A: The consequences to property are considered CATASTROPHIC. Complete destruction of the proposed structure is anticipated in the event of a landslide occurring at the site.

5.4 RISK ASSESSMENT FOR PROPERTY

The above estimates of frequency and consequence have been used in the qualitative risk matrix of AGS (2007) to derive the risk levels summarised in Table 5.4.1 below. A copy of the qualitative risk matrix of AGS (2007) is provided in Appendix D.

Table 5.4.1 Summary of Risk to Property for the Existing Conditions

HAZARD		LIKELIHOOD	CONSEQUENCE	RISK
A	Rotational slip failure of escarpment	Possible	Catastrophic	VERY HIGH

These results show that the risk to property is VERY HIGH. This level of risk is considered unacceptable for a new structure. Risk treatment is required to reduce the level of risk to at least LOW, if not VERY LOW levels. Sections 5.6 and 6 provide discussion of measures to reduce risk.

5.5 RISK ASSESSMENT FOR LIFE

A quantitative basis has been adopted for estimation of the risk to life. Table 5.5.1 summarises the estimation of risk to life.

The following factors have been considered in the analysis.

- The proposed structure will be occupied on average by up to 4 people for 16 hours per day.
- There are unlikely to be any obvious warning signs of a large failure of the escarpment. Failure is anticipated to be rapid.

Table 5.5.1 Summary of Risk to Life for the Existing Condition.

HAZARD	A
DESCRIPTION	Rotational slip failure of the escarpment
LIKELIHOOD	Possible
INDICATIVE ANNUAL PROBABILITY	10^{-3}
PROBABILITY OF SPATIAL IMPACT	1.0
OCCUPANCY (number of people)	4
PROPORTION OF TIME	16hr/day = 0.667
PROBABILITY OF NOT EVACUATING	1.0
VULNERABILITY	1.0
RISK FOR PERSON MOST AT RISK	6.7×10^{-4}
TOTAL RISK	2.7×10^{-3}
RISK EVALUATION	**INTOLERABLE**

The results of the above risk estimations have been compared to the acceptance criteria given in AGS (2007). Tolerable Risk criterion of 10^{-5} applies. Acceptable risk would be an order of magnitude smaller. Compared to these criteria the level of risk to life is considered INTOLERABLE for a structure constructed on the subject site without taking into account the potential hazards at the site.

Tolerable risks are 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.

Acceptable risks are risks which everyone affected is prepared to accept. Action to further reduce acceptable risk is usually not required unless reasonably practical measures are available at low cost in terms of money, time and effort.

5.6 RISK MANAGEMENT

The level of risk to life for the proposed structure is intolerable and the risk to property is very high, 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.

Assuming that the proposed development is designed and constructed in accordance with the recommendations of this report, the levels of risk to life and property (proposed dwelling) are considered to be acceptable and no further risk reduction measures are considered a necessity.

6 COMMENTS AND RECOMMENDATIONS

6.1 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 site has been classified as 'Class P' in accordance with Australian Standard Australian Standard AS 2870 – 1996, 'Residential Slabs and Footings'.

6.2 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 D_e – Deep Soil Site
• Hazard Factor (Z):	0.09

6.3 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 permanently anchored retention piles to protect the proposed dwelling against a potential landslide which may occur on the face of the escarpment. The row of anchored piles, 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.

- 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 cost prohibitive. 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 will be marginally reduced when compared with the current uncontrolled site conditions.
- Retention along the south, east and west sides of the proposed bulk excavation for the lower ground level should comprise either cantilevered or anchored soldier piles with reinforced shotcrete infill panels.

6.4 RETENTION PILES ALONG THE NORTH END OF THE PROPOSED DWELLING

The row of piles along the north side of the proposed structure will need to be designed as permanently anchored 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 anchored 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 site cut for the lower ground level. The piles could be installed after the proposed site cut has been carried out. This will ensure that a conventional piling rig is able to install the piles without any special requirements for site access.

The piles must be founded on either very dense sand or very stiff clay at a minimum founding depth of 15 metres below the level of the proposed site cut. It is recommended that the centre to centre spacing of the piles not exceed 2.0 metres. The uppermost 8 metre section of the piles must be designed to withstand a uniform lateral earth pressure of 60 kPa, assuming no support from the soil on the north side of the piles. This allows for the possible development of an 8 metre deep tension crack forming immediately adjacent to the north side of the row of retention piles. Assuming that full soil arching occurs between piles each pile must support a 2.0 metre width of soil.

The ultimate geotechnical strength (R_{ug}) of piles with spacings of three pile diameters must be determined in accordance with Section 4 of Australian Standard AS2159 - 1995, 'Piling – Design and Installation' on the basis of the following pressure.

Ultimate base pressure on very stiff clay or very dense sand (f_b):	1350 kPa
Ultimate average skin friction in very dense sand (f_s): (z is the depth from the top of the pile)	$6z$ kPa

The design geotechnical strength of a pile (R_g*) must be calculated by multiplying the ultimate geotechnical strength by a geotechnical strength reduction factor ($\emptyset_g$) of 0.45.

If design is not in accordance with Section 4 of Australian Standard AS2159 - 1995, 'Piling – Design and Installation', and a working stress methodology is adopted then bored piles should be designed on the basis of the following maximum allowable pressures.

Allowable base pressure on very stiff clay or very dense sand:	450 kPa
Allowable average skin friction in very dense sand (f_s): (z is the depth from the top of the pile)	$4z$ kPa

Skin friction may be applied only to that portion of bored piles founded within very dense sand at depths in excess of 8 metres below the bulk excavation level. Furthermore due to the susceptibility of the walls of the pile excavation to smearing, skin friction can only be adopted if the sides of the pile excavations have been roughened using a suitable grooving tool.

Assuming that the bases of pile excavations are free of loose or softened material, the likely total elastic and consolidation settlements under the above pressures are estimated to be less than 1% of the pile diameter. Differential settlements are expected to be approximately half of the total settlement value. These values will be exceeded where the base of the pile excavations are not suitably clean.

Bored pile excavations, which intercept seepage water, may require temporary liners to maintain stability of the excavation during construction. All seepage water must be pumped from the pile excavations prior to pouring concrete.

A cleaning bucket or plate must be used to clean the base of each pile excavation prior to the placement of concrete. All bored pile excavations must be inspected by a qualified geotechnical 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.

If ground anchors are used to provide lateral restraint of the row of retention piles, the design of the 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 75 kPa. 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.

The free length of the ground anchors should extend at least 1.5 metres beyond the 45° line extending up from a point on the piles located 8 metres below the excavated ground surface level.

6.5 FOOTINGS PROVIDING SUPPORT TO LOWER GROUND FLOOR LEVEL

6.5.1 Bored Pile Footings

It is recommended that the lower ground floor be fully suspended on a series of reinforced bored pile footings. It is recommended 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 only take into account structural requirements. Bored piles must be founded on either very dense sand or very stiff clay at a recommended minimum founding depth of 8 metres below bulk excavation level.

The ultimate geotechnical strength (R_{ug}) of piles with spacings exceeding three pile diameters must be determined in accordance with Section 4 of Australian Standard AS2159 - 1995, 'Piling – Design and Installation' on the basis of the following pressures.

Ultimate base pressure on very stiff clay or very dense sand (f_b):	1350 kPa
Ultimate average skin friction in very dense sand (f_s): (z is the depth from the top of the pile)	$6z$ kPa

The design geotechnical strength of a pile (R_g*) must be calculated by multiplying the ultimate geotechnical strength by a geotechnical strength reduction factor ($\emptyset_g$) of 0.45.

If design is not in accordance with Section 4 of Australian Standard AS2159 - 1995, 'Piling – Design and Installation', and a working stress methodology is adopted then bored piles should be designed on the basis of the following maximum allowable pressures.

- Allowable base pressure on very stiff clay or very dense sand: 450 kPa
- Allowable average skin friction in very dense sand (f_s): 4*z* kPa
  (*z* is the depth from the top of the pile)

Skin friction may be applied only to that portion of bored piles founded within very dense sand below the bulk excavation level. Furthermore due to the susceptibility of the walls of the pile excavation to smearing, skin friction can only be adopted if the sides of the pile excavations have been roughened using a suitable grooving tool.

Assuming that the bases of pile excavations are free of loose or softened material, the likely total elastic and consolidation settlements under the above pressures are estimated to be less than 1% of the pile diameter. Differential settlements are expected to be approximately half of the total settlement value. These values will be exceeded where the base of the pile excavations are not suitably clean.

Bored pile excavations, which intercept seepage water, may require temporary liners to maintain stability of the excavation during construction. All seepage water must be pumped from the pile excavations prior to pouring concrete.

A cleaning bucket or plate must be used to clean the base of each pile excavation prior to the placement of concrete. 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.

6.6 RETENTION OF BULK EXCAVATION FOR LOWER GROUND FLOOR LEVEL

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.

6.6.1 Soldier Pile Retention System

Where the depth of site cut exceeds approximately 1.0 – 1.5 metres we recommend the installation of soldier piles prior to excavation. Lateral restraint of the toe of piles may be achieved by suitably socketing the piles into the very dense sand as noted to present at depths in excess of approximately 3.1 metres in Borehole 1. The piles may be designed to provide permanent lateral support with bracing from the completed structure. The piles may also be designed as load bearing in accordance with Section 6.5.1. Reinforced shotcrete panels are recommended 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, anchors or internal props must be installed incrementally as excavation proceeds. Props or anchors must be installed immediately once the propping/anchoring points have been exposed.

6.6.2 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 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 75 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 1.5m beyond the 45° line extending up from the toe of the retaining wall. Local and global stability of the proposed retaining wall should be analysed once retaining wall geometry and anchor locations have been determined.

6.6.3 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 assuming a horizontal backfill surface and that the walls are designed as permanently drained.

- Permanently cantilevered retaining walls may be considered where deformation and movement behind the walls can be tolerated, such as for garden or grassed areas. A triangular lateral earth pressure distribution and an active earth pressure coefficient (Ka) of 0.33 should be adopted. The active earth pressure coefficient should be used to calculate lateral earth pressures generated by surcharge loads.
- For retaining walls which will be cantilevered during the construction period, but fully restrained by the completed structure, adopt an earth pressure distribution increasing linearly from zero kPa at the ground surface to $K\gamma H$ kPa at the base of the retained excavation. Take H as the full retained height in metres. Adopt a lateral earth pressure coefficient (K) of 0.50 where there are any movement sensitive structures or services within the zone of influence of the excavation. Adopt a lateral earth pressure coefficient (K) of 0.42 elsewhere. 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.
- For progressively anchored or propped walls where minor movements can be tolerated, adopt a uniform earth pressure distribution of 4H kPa where H is the total retained height in metres. A lateral earth pressure coefficient (K) of 0.42 should be used to calculate lateral earth pressures generated by surcharge loads.
- For minimal deflection of progressively propped walls where there are movement sensitive structures or buried services within the zone of influence of the excavation, adopt a uniform earth pressure distribution of 5H kPa where H is the total retained height in metres. A lateral earth pressure coefficient (K) of 0.50 should be used to calculate lateral earth pressures generated by surcharge loads.
- A soil unit weight (γ) of 20 kN/m^3 should be adopted for medium dense sand and 22 kN/m^3 for very dense sand.
- 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.
- A passive earth pressure coefficient (Kp) of 4.6 may be used to estimate lateral toe resistance for the portion of the retaining wall founded in very dense sand as encountered in Borehole 1 at depths in excess of 3.1 metres below the existing ground surface. A reduced passive earth pressure coefficient (Kp) of 2.5 may be used to estimate lateral toe resistance for the portion of the retaining wall founded in clay as encountered in Borehole 1 between the depths approximately 7.5 and 9.0 metres below the existing ground surface. Resistance should be ignored to a depth of 1.5 pile diameters to allow for disturbance effects. This assumes a horizontal ground surface at the toe of the wall and that a factor of safety of 2.0 is applied to limit deformations.
- It is noted that design of any cantilevered retention piles may be governed by lateral deflection at the top of the pile rather than ultimate lateral resistance provided by the soils. Deflections of piles can be modelled using the following parameters:
    - Medium Dense Sand: Elastic Modulus = 35 kPa
      Poisson's Ratio = 0.3
    - Very Dense Sand: Elastic Modulus = 80 kPa
      Poisson's Ratio = 0.3
    - Very Stiff to Hard Clay: Elastic Modulus = 40 kPa
      Poisson's Ratio = 0.5

6.6.4 Retaining Wall Backfill and Drainage

All retention structures should be designed such that the soil behind the wall is completely and permanently drained. If this cannot be ensured then hydrostatic pressure must be included in design. Backfill to retaining walls should comprise selected free draining granular material. It is recommended that subsurface drains incorporate a non woven geotextile filter fabric to minimise silting of drains and erosion of backfill.

6.6.5 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 0.5% of the excavation depth. On a similar basis propped or anchored walls designed for a uniform lateral earth pressure distribution of 8H kPa, and constructed with good workmanship, may experience lateral deflection in the order of 0.1% of the excavation depth. Consistent with the above horizontal deflections, vertical settlements of less than 0.5% of the excavation depth could be expected for cantilevered walls and less than 0.1% 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.

6.7 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 D as a further guide to good hillside construction practices.

6.7.1 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.

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.

6.7.2 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.

6.7.3 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.

6.8 CONSTRUCTION REQUIREMENTS

6.8.1 Construction Adjacent to Excavations, and Service Pipe Trenches

Buried services should not be located adjacent to footings. Where this cannot be avoided the trench should be backfilled in such a way as to prevent moisture ingress. Any footings located adjacent to excavations or backfilled service trenches should be founded below a line drawn up at 30° to the horizontal from the base of the excavation.

6.8.2 Site Drainage and Maintenance of Footings

Effective drainage of the site should be maintained at all times. Water run-off should be collected and diverted away from the structure during construction. Water should not be allowed to pond against footings during or after construction. The ground adjacent to footings should be graded to provide a permanent fall of 1(V):50(H) away from the footings over at least the first 2.0m. Water supply and drainage infrastructure should be maintained so that no leakage occurs. Construction of garden beds and the planting of trees and large shrubs, adjacent to footings should be avoided. Excessive watering adjacent to footings should be avoided and the installation of an irrigation or sprinkler system adjacent to the structure is not recommended.

6.8.3 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.

6.8.4 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.

6.9 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

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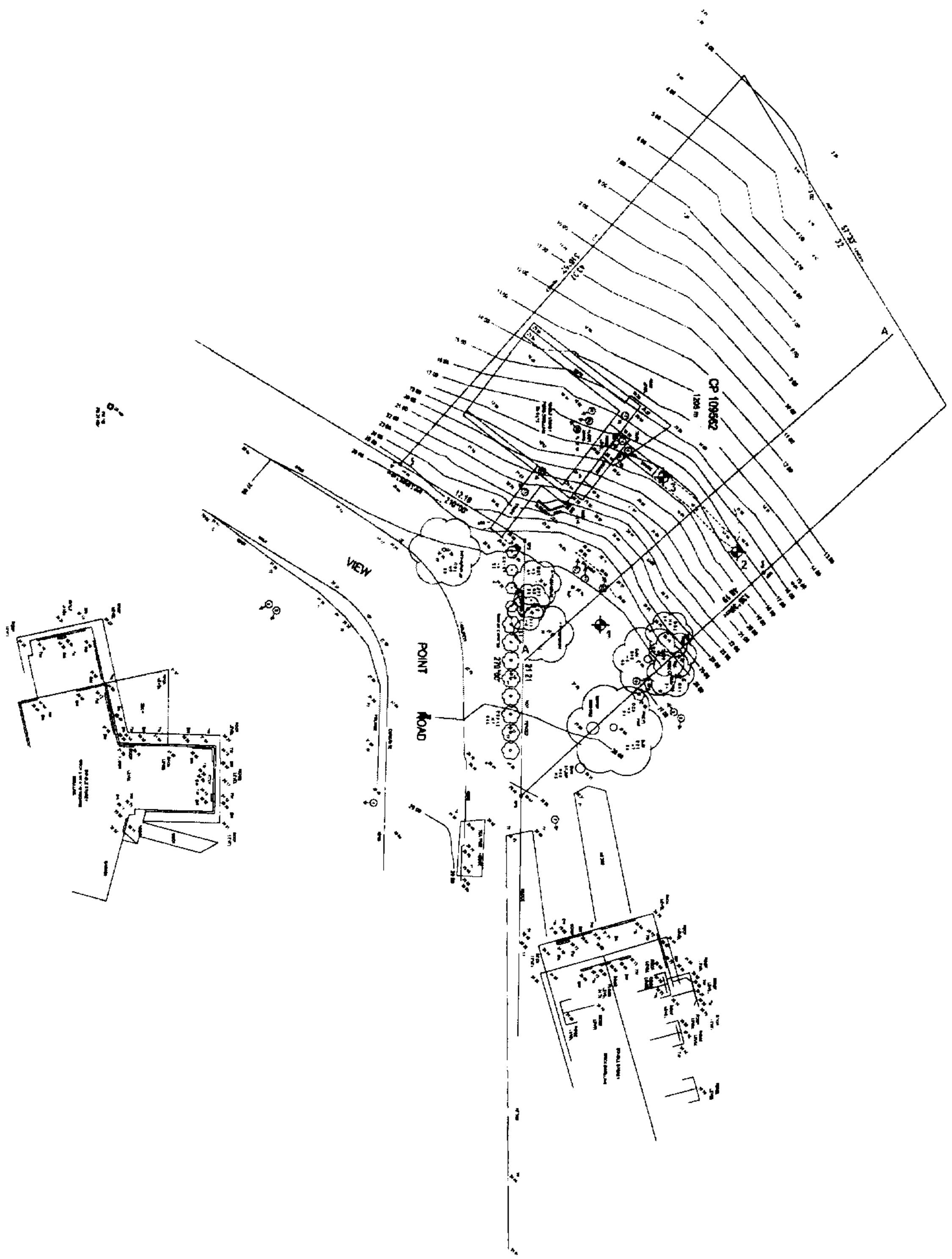
TEST LOCATION PLAN

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

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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		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS	▨	GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		(LITTLE OR NO FINES)	▨	GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
	MORE THAN 50% OF COARSE FRACTION IS LARGER THAN 2.0MM	GRAVELS WITH FINES	▨	GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)	▨	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	▨	SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		(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	▨	SM	SILTY SANDS, SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)	▨	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

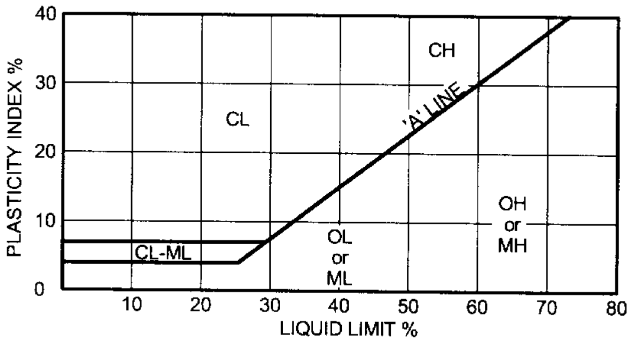
SOIL DESCRIPTION

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

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APPENDIX B

Bore Logs

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E-mail: enquiries@geoaust.com.au

BOREHOLE LOG

TEST LOCATION
1
SHEET 1 of 3

JOB No: 1624
CLIENT: Fasham Johnson Pty Ltd
PROJECT: Proposed Residential Development
14-16 ViewPoint Road, MCCRAE
LOCATION: Refer to Test Location Plan (Figure 1)

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	Sample U60	Sample ES	Depth	DCP Test	Test	Comments and Test Results
		0.3		-	**FILL: Silty Sand,** fine to medium grained, with tree roots, dark grey	Moist	APC							
		0.9		SM	**SAND:** fine to medium grained, silty, grey tending pale grey with depth	Moist	MD				0.5			
		3.1		SM	**SAND:** fine to medium grained, silty, with fine grained gravel, pale brown-grey tending pale brown with depth	Moist	MD				1.0 1.5 2.0 2.5 3.0		☒	7/8/8 N = 16.
		5.1		SM	**SAND:** fine to coarse grained, silty, with clay fines, with fine to coarse grained granite gravel, brown with grey	Moist	VD				3.5 4.0 4.5 5.0		☒ ☒	10/33/- N > 33. Hammer double bouncing 18/25 blows for 50mm penetration: N > 25. Hammer double bouncing
		7.5		SC	**SAND:** fine to medium grained, very clayey, trace coarse grained sand with silt fines, grey (completely weathered granodiorite)	Moist	VD				5.5 6.0 6.5 7.0 7.5		☒	18/25/- N > 25.
		9		CL	**CLAY:** medium plasticity, silty, with sand, grey mottled yellow-brown	MC<PL	H				8.0 8.5 9.0		☒	8/14/17 N = 31.
				SM	**SAND:** fine to medium grained, trace coarse grained sand, silty, trace clay fines, grey	Moist	VD				9.5 10.0		☒	21/33/- N > 33.

Refer Appendix A for definition of logging terms and symbols

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BOREHOLE LOG

TEST LOCATION
1
SHEET 2 of 3

JOB No: 1624
CLIENT: Fasham Johnson Pty Ltd
PROJECT: Proposed Residential Development
14-16 ViewPoint Road, MCCRAE
LOCATION: Refer to Test Location Plan (Figure 1)

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	Sample U60	Sample ES	Depth	DCP Test	Test	Comments and Test Results
					SAND: fine to medium grained, trace coarse grained sand, silty, trace clay fines, greyContinued next page	Moist	VD							
		10.5		SM	**SAND:** fine to medium grained, silty, trace clay fines, grey, trace yellow-brown	Moist	D to VD				10.5 11.0 11.5			10/18/34 N = 52.
		12		CH	**CLAY:** high plasticity, silty, trace sand, grey, trace yellow-brown	MC>PL	VSt				12.0 12.5 13.0 13.5			5/7/11 N = 18.
		14		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		CL	**CLAY:** medium plasticity, silty and sandy, trace coarse grained sand, grey with orange-brown, trace red-brown	MC>PL	VSt				15.0 15.5 16.0			5/9/8 N = 17.
	3.0hr	16.5		SM	**SAND:** fine to medium grained, silty, trace coarse grained sand, trace clay fines, grey, mottled yellow-brown	Wet	D to VD				16.5 17.0 17.5 18.0 18.5 19.0 19.5 20.0			13/25/26 N = 51. 21/23/27 N = 50.

Refer Appendix A for definition of logging terms and symbols

Figure B-2

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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
14-16 ViewPoint Road, MCCRAE
LOCATION: Refer to Test Location Plan (Figure 1)

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	Sample U60	Sample ES	Depth	DCP Test	Test	Comments and Test Results
		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	Wet	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	Wet	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	Wet	D				23.5 24.0 24.5		☒	12/16/24 N = 40.
		25									25.0			
					END OF BOREHOLE LOG AT 25M									

Refer Appendix A for definition of logging terms and symbols

Figure B-3

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E-mail: enquiries@geoaust.com.au

BOREHOLE LOG

TEST LOCATION
2
SHEET 1 of 1

JOB No: 1624
CLIENT: Fasham Johnson Pty Ltd
PROJECT: Proposed Residential Development
14-16 ViewPoint Road, MCCRAE
LOCATION: Refer to Test Location Plan (Figure 1)
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			Depth	DCP Test	Test	Comments and Test Results
								DS	U60	ES				
	NOT ENCOUNTERED	0.6	▨	SM	**SILTY SAND:** fine to medium grained, trace fine to coarse grained granodiorite gravel, grey-brown	Dry	MD				0.5			
		0.9	▨	SM	**SILTY SAND:** fine to medium grained, trace fine to medium grained granodiorite gravel, yellow-brown	Dry	D							
		1.5	▨	SC	**SAND:** fine to coarse grained, clayey, trace mica, dark grey and pale grey	Moist	D				1.0 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

Figure B-4

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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
14-16 ViewPoint Road, MCCRAE
LOCATION: Refer to Test Location Plan (Figure 1)
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			Depth	DCP Test	Test	Comments and Test Results
								DS	U60	ES				
	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

Figure B-5

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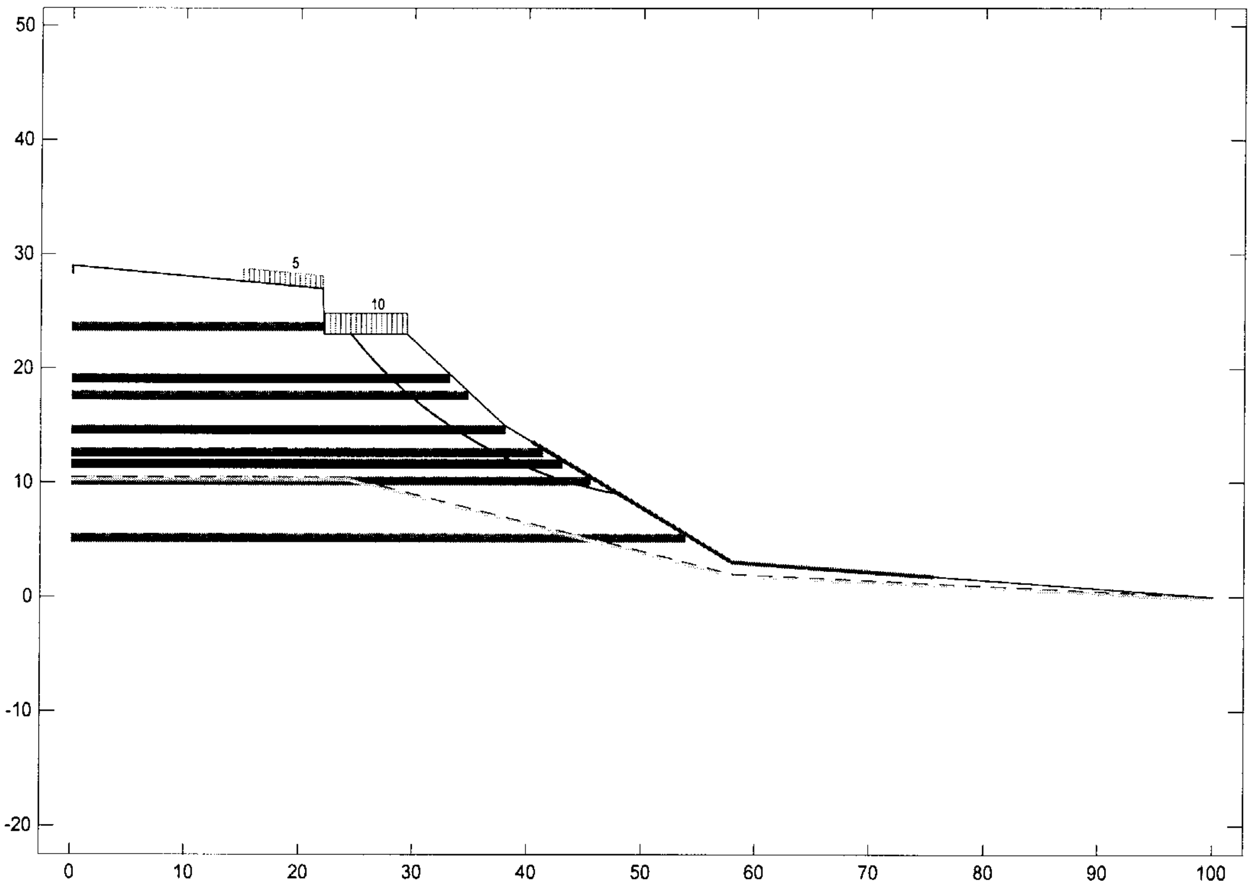
APPENDIX C

Slope Stability Analysis

(Graphical Summaries of
Critical Stability Analyses)

GALENA Version 4.02



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

Results
Critical (minimum)
Factor of Safety: 1.24

Project: 14 - 16 View Point Road, McCrae
Stability of Existing Escarpment Incorporating Proposed Site Cut (No Earthquake Load)
File: X:\1624MC~1\1624GA~1\1624-1.gmf

Figure C1

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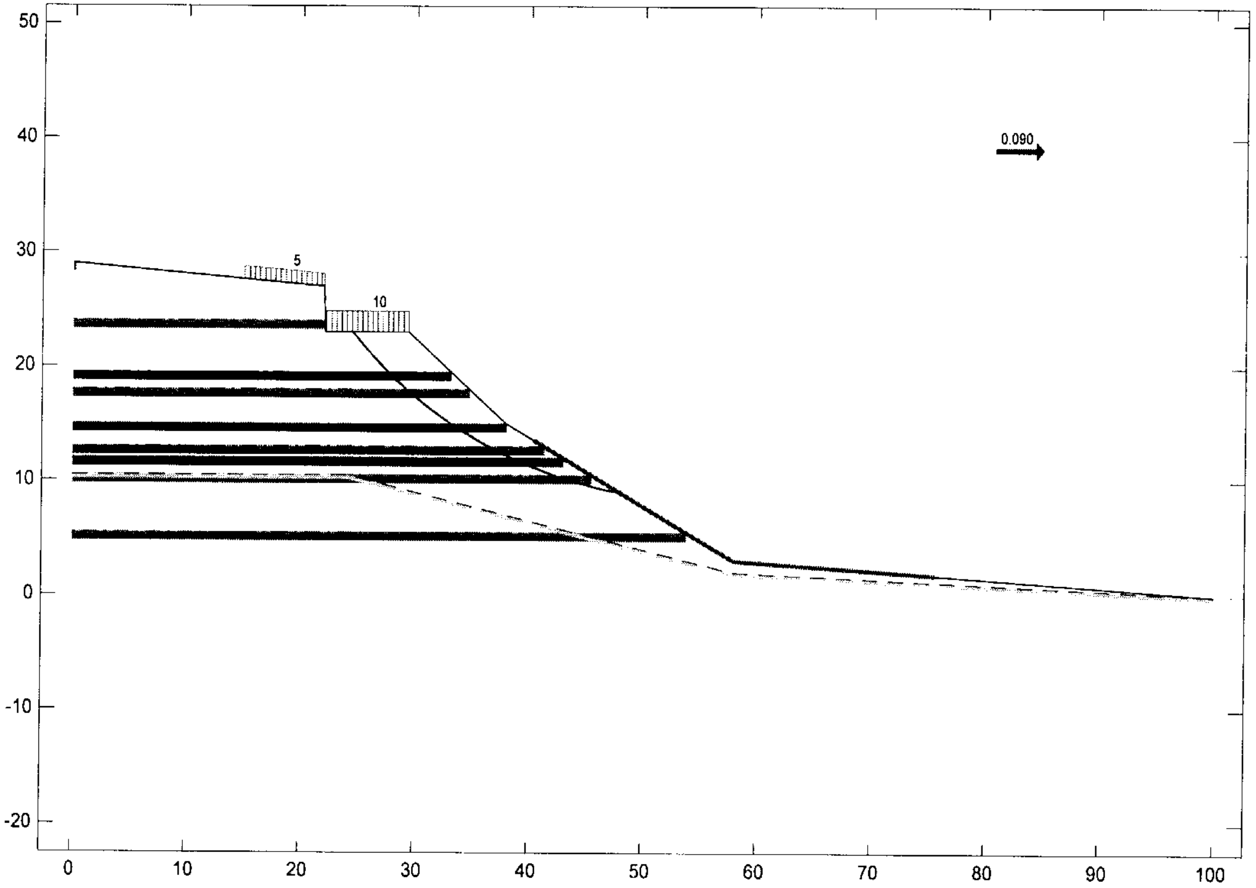
GALENA Version 4.02



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

Results
Critical (minimum)
Factor of Safety: 1.05

Project: 14 - 16 View Point Road, McCrae
Stability of Existing Escarpment Incorporating Site Cut and Earthquake Loading
File: X:\1624MC~1\1624GA~1\1624-1.gmf

Figure C2

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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}	[illegible]	[illegible]	[illegible]	H	M or L (5)
B - LIKELY	10^{-2}	[illegible]	[illegible]	H	M	L
C - POSSIBLE	10^{-3}	[illegible]	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)
[illegible]	[illegible]	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.

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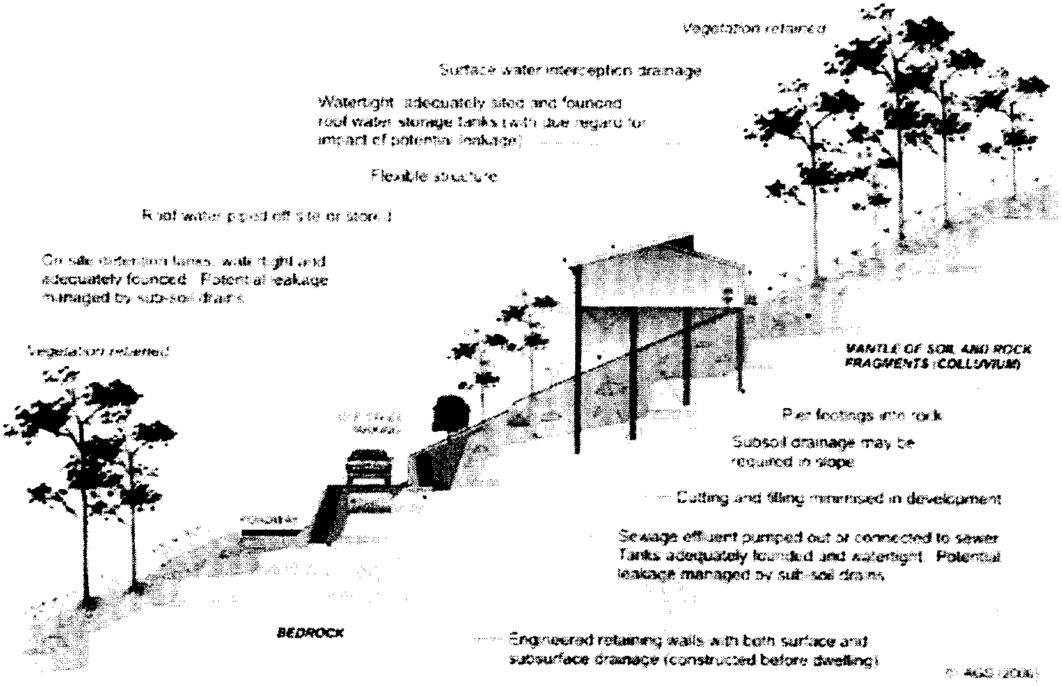
APPENDIX E

Guidelines for Hillside Construction

APPENDIX G - 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



EXAMPLES OF **POOR** HILLSIDE PRACTICE