NICK TASSIOS

VAST ARCHITECTS

PROPOSED RESIDENTIAL DEVELOPMENT

601 POINT NEPEAN ROAD,

MCCRAE

Report No: 116269

Date: 4 December 2014

GEOTECHNICAL INVESTIGATION

By

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1. INTRODUCTION

1.01 **Investigation Requested By:** The geotechnical investigation was commissioned by Nick Tassios by signed authorisation dated 22nd October 2014.

1.02 **Purpose of Investigation:** It is proposed to construct a new residential dwelling at 601 Point Nepean Road, McCrae. Herein a geotechnical investigation was required to provide advice and recommendations relating to the following geotechnical aspects of the proposed site development:

- Background Geology and Geomorphology
- Establish the subsurface profile, including ground water conditions of boreholes.
- A site classification in accordance with Australian Standard AS 2870 – 1996, "Residential Slabs and Footings, Constructions".
- Provide recommendations for appropriate footing arrangements for the proposed development, including a hazard factor for earthquake loading in accordance with Australian Standard 1170.4, 2007.
- Provide minimum founding depths and allowable bearing pressures for the recommended footing arrangements, together with predictions of short and long-term settlements.
- Provide advice in relation to the anticipated excavation conditions, including advice on site dewatering.
- Post construction risk assessment /any solutions to avoid /minimise the post construction risks for the proposed development including stability assessment.
- Discuss construction sequences.

In the absence of any precise structural details of the proposed structure it has been assumed for the purpose of this report that no unusual loading conditions or performance specifications apply. Furthermore, the geotechnical investigation was limited to the approximate building envelope indicated on Figure 2. If construction is to be contemplated in other areas of the site the recommendations provided in this report will need to be reviewed.

1.03 **Geology: Geology & Background:** The 1:63 360 series Geological Survey of Victoria, Sorrento Sheet, indicates the site to be underlain by sand deposits which are of the Quaternary age. At depth these sand deposits are underlain by Devonian Granite. Weathering of the granite has typically resulted in shallow residual sands and clays which grade to variably weathered granite at depths. The residual clays are generally moderately reactive and the depth to rock is often highly variable over short distances. However it appears only granitic residual soil has been encountered during bore hole investigations.

1.04 **Field Methods:** As part of the geotechnical investigation the following field methods were incorporated:

i) **Auger Drilling:** The boreholes were drilled using a Dingo K9-4 and Edson Versadrill 3000 drill rigs equipped with continuous flight 125 millimetre diameter augers fitted with tungsten carbide drill bits.

ii) **Standard Penetration Testing:** Standard Penetration testing was conducted at regular intervals within the boreholes in accordance with the test procedure outlined in Australian Standard 1289, "Methods of Testing Soils For Engineering Purposes," Test Method 6.3.1, June 1993 in accordance with Australian Standard AS 1726 - 1993, "Geotechnical Site Investigations."

iii) **Logging of Soil Profiles:** The soil profile encountered in the borehole was logged in accordance with Australian Standard AS 1726 - 1993, "Geotechnical Site Investigations."

1.05 **Site Supervision:** All site testing, including the drilling and logging of boreholes and in-situ testing performed within each of the boreholes was conducted under the supervision of a qualified engineer from A.S. James Pty Ltd.

1.06 **Laboratory Test Methods:** All soil samples were transferred to A.S. James' National Association of Testing Authorities registered Clayton South laboratory, where testing was undertaken by trained laboratory technicians. All laboratory testing was performed in strict accordance with the test methods outlined in Australian Standard AS 1289, "Method of Testing Soils for Engineering Purposes".

	AS Test Method
▪ Atterberg Limits	3.1.1, 3.2.1, 3.3.1.3.4.1
▪ Moisture content testing	2.1.1

1.07 **Test Program:** Upon receipt in the laboratory, the soil samples retrieved from boreholes were tested as follows:

Testing Depths	Atterberg Limits	Moisture Content
BH1	Sample (7.5) m	Sample (9.0) m Sample (10.5) m Sample (12.0) m
BH2	Sample (4.5) m Sample (6.0) m Sample (8.5) m Sample (11.5-12.0) m	Sample (1.0) m Sample (2.0) m Sample (2.5) m Sample (3.0) m Sample (3.5) m Sample (4.0) m Sample (5.0) m Sample (6.0) m Sample (7.0) m Sample (8.0) m Sample (9.0) m Sample (10.0) m Sample (11.0) m

Table 3: Soil Testing Program

2. RESULTS

2.1 FIELD TESTING

2.1.1 **Site Description:** The site of the residence is located on an escarpment, of which extends up from the Point Nepean Road. The slope was generally moderately to lightly vegetated and a retaining wall exists, with a height of 1.4 m above lower ground surface level. The retaining wall does not appear to be in any large state of distress nor undergone any significant deflections.

On average, the slope of the escarpment was measured to range between approximately 15-35° degrees on the upper section of the escarpment, up to approximately 50° on the steeper middle portion of the escarpment grading to essentially flat at the retaining wall. Drainage of the site is good.

At the time of the site inspection there were some slope instabilities such as creeping, slumping and deflection of small timber sleeper retaining walls (Up to 55° from the vertical). One tree was located on the escarpment and appeared to exhibit a downslope lean. The tree also exhibited a curved trunk indicative of hillside creep activity. The topography of the site and local area is indicative of past landslide activity with "hummocky" ground surfaces.

The steeper slopes in the general area are known to be potentially subject to slope instability, with two types of movements considered to be possible; deep seated movements and shallow translational slides associated with localised instability. The effects of shallow translational movements can be minimised in the construction and design, but there is little that can be done to minimise the effects of deep seated movements and this is a risk common to the majority of the buildings in this area.

2.1.2 **Borehole Drilling:** Two (2) boreholes were drilled at the approximate locations indicated on Figure 1. Both boreholes were carried out at the toe of the escarpment due to access restrictions to our drilling machinery. Of these, borehole 2 was drilled at a sub-horizontal angle (Measured to be 10.5°) to a depth of 12.0 m, whereas borehole 1 was drilled down vertically to 12 m depth. Logs of the boreholes, together with the results of the standard penetration tests and moisture content tests in each of the boreholes are given on Figures 5-8.

2.1.3 **Sub-surface Soil Profile:** **(BH1) Vertical Bore:** Shallow sand fill was encountered before natural in-situ medium to coarse grained sand was encountered to 1.5 m of medium density. The sand continuing from 1.5 m appeared to tend fine to medium grained, with some coarse grains and silty. The clay content appeared to increase with depth and some grinding was experienced which might indicate some lightly cemented sand lenses. At a depth of 7.0 m, the material exhibited characteristics that off a clayey sand and / or gravel / clay / sand mix. With connection to the geology of the area, the gravels appeared granitic consistent with granitic residual soil.

(BH2) Sub-Horizontal Bore: Proceeding into the escarpment, 1.1 m of sandy clay / clayey sand fill was encountered before a natural in-situ very sandy clay up to 2.7 m. Beyond this, a clayey sand / sandy clay was encountered, predominantly fine to medium grained with some coarse grains and small gravels extending to 7.5 m. Beyond 7.5 m the observed material exhibited characteristics more of a clay and some granitic gravels were observed beyond 8.0 m. The clay appeared very sandy, silty, in moist condition and is anticipated to be of firm to stiff consistency.

A distinct change in the material characteristics was observed at 11.0 m appearing as a clayey sand and or clay / gravel / mix, distinctly characteristic of granitic residual soil.

Notes: From observing the samples, it is evident that that the clay content or 'cohesiveness' of the material retrieved from the sub-horizontal bore into the escarpment is noticeably increased than that of the vertical bore within 1.5 m to 7.0 m.

2.1.4 **Ground Water:** The elevation of the site above the adjacent sea level ensures that natural surface drainage is for most part good. Groundwater was encountered however at relatively shallow depths at the toe of the escarpment. The following is a summary of the groundwater level recordings at various stages of the investigation.

Table 1: BH1 Groundwater level observations

BH1	Groundwater depth below ground surface level
Observed during Drilling	1.2 m
Observed Immediately after drilling	2.1 m
Observed > 48 Hrs after drilling	1.05 m

The permanent ground water table at the subject site would be anticipated to approximately correspond with the sea level and has been recorded at approximately 1.0m from the base of the escarpment. Permanent ground water was not encountered in the horizontal BH2

2.1.5 **Moisture Content of Subsurface Materials Comments:** Within BH2 drilled sub-horizontally into the escarpment. The material beyond 2.0 m appeared to be very moist, which would likely indicate a degree of higher saturation within this unit of the escarpment. Therefore, seemingly indicative of a shallow seepage path for water down the escarpment. The moisture content did however appear to decrease from 2.7 to 4.5 m. It is considered the moisture content of the materials at deeper depths into the escarpment are 'normal'.

Note: Moisture contents of the profile will vary with seasonal changes.

2.2 SLOPE STABILITY ANALYSIS

To ascertain the overall stability of the proposed structure, including the stability of the escarpment located immediately to the north, a slope stability analysis was performed using Geostudio 2004 (version 6.10, Build 1320), slope stability analysis program. The following assumptions were made and applied to the analysis:

- The geometry of the site will not be altered to any significant extent from the currently prevailing site conditions except the proposed cuts and small degree of filling.
- The uniform dead load imposed by the construction of the proposed dwelling will not exceed an average value of 35 kPa. However, in our analysis, split level construction and the nett pressure resulted from bulk excavation was considered.
- An earthquake acceleration coefficient of 0.09 applies in the horizontal and vertical directions.
- The characteristic subsurface soil profile encountered within the depths investigated continues to the full depth of the model.

The following effective stress parameters were adopted for the analysis.

- Fill : **Region (1, 2, 4)** — C' = 2 kPa, **Ó'** = 27°, γ = 16 kN/m^3
- Shallow Sand : **Region (5)** — C' = 0 kPa, **Ó'** = 28°, γ = 17 kN/m^3
- Rock Gabion Wall : **Region (3)** — C' = 10 kPa, **Ó'** = 40°, γ = 24 kN/m^3
- Medium Dense to Dense Sand : **Region (7)** — C' = 3 kPa, **Ó'** =36°, γ = 18 kN/m^3
- Firm to Stiff Sandy Clay / Medium Dense Clayey Sand: **Region (8)** — C' = 7 kPa, **Ó'** =34°, γ = 17 kN/m^3
- Residual Granitic Soil (Gravelly / Sandy Clay): **Region (6)** — C' = 10 kPa, **Ó'** =32°, γ = 18 kN/m^3

In order to analyse the stability of the site the critical cross section, indicated on Figure 1 as Section A-A', was analysed. The cross section is created from the proposed altered slope indicated on the VAST ARCHITECTS New Residence Plan, *'Side Elevation – North/East'*. The 'Side Elevation – South/West' New Residence Plan was also provided, however the Side Elevation – North/East was chosen due to the degree of cutting from the natural ground level shown, opposed to the South/West.

The slope to the north was extended beyond the site boundary at the indicated gradient to aid the stability modelling. The anticipated site cuts have been integrated into the modelling and therefore the subsurface anticipated ground profile has been adjusted accordingly.

A conservative pore water pressure gradient has been adopted due to the increased moisture content results in samples retrieved from the horizontal bore.

Section A-A' (Overall Section with Seismic Loading) – The factor of safety for the critical failure across the section A-A' with seismic loading is **1.16** (Figure 12).

Section A-A' (Shallow Failure with Seismic Loading) – The factor of safety for the critical failure across the section A-A' with seismic loading is **1.12** (Figure 14).

Based on the Slope Stability Analysis, the factor of safety for shallow translational failure of the escarpment is 1.1 suggesting the slope as it currently exists in a marginal state at the point of failure, and is at risk of instability of the surface soils (See Figure 14). Minor slumping could be anticipated as observed in the field. This type of instability is common in the general area.

It should be noted, however, that the stability analysis completed is not able to take into account the stabilising effects of the vegetation currently growing on the face of the escarpment. In reality it is therefore considered that the existing surface vegetation is contributing significantly to the stability of the slope. In effect it would seem preventing failure. The variation of pore water pressure within the escarpment is also expected to vary to a reasonable extent, which can have a large effect, however as mentioned earlier, a conservative shallow pore water pressure level was adopted. Adequate cut off drainage would help negate pore water fluctuations within the escarpment.

Figure 13, represents a failure plane for deeper seated movement without an earthquake loading on the existing overall escarpment. A factor of safety of 1.5 indicates the depths where factors of safety are adequate. It is considered provided footings extend to the granitic residual soil (Gravelly / Sandy Clay), at least below these depths a 1.5 factor of safety with seismic loading is applicable.

A deep foundation arrangement such as piles will be required and these will need to be extended to the minimum depths, as discussed later in this report, to negate the potential for near surface failures. They will also need to accommodate the lateral forces generated by shallow instabilities.

It is considered that a deeper foundation arrangement in accordance with this report, together with good hillside practice, mentioned attached, would negate any risk of failures.

Maintaining and planting additional vegetation on the steep slope where possible and control of drainage will assist in reducing the risk of instability to the escarpment.

3. RECOMMENDATIONS

3.1 PROPOSED RESIDENCE

3.1.1 **Site Classification:** In selecting an appropriate site classification for the currently proposed building envelope, in accordance with Australian Standard 2870 - 1996, "Residential Slabs and Footings - Construction", the following factors have been taken into account:

- Sub-surface soil profiles and results of in-situ testing
- Results of classification testing
- Local geological conditions
- Results of slope stability analyses
- Knowledge of similar conditions in the area

On the basis of the potential of a slope instability developing on the escarpment, the site has been classified as "Class P" in accordance with Australian Standard AS 2870 - 1996, "Residential Slabs and Footings - Construction".

In spite of the "P" classification, overall it is considered that the construction of the proposed structure is feasible and safe, provided that an appropriate footing arrangement is adopted and that certain basic precautions are taken in the development of the overall site. The proposed construction sequence must also be strictly adhered to.

3.1.2 **Footing Construction:** In view of the potential instability of the escarpment at the base and firm to stiff sandy clay / clayey sand within the escarpment of the subject site, it is recommended the proposed structure be fully supported on a pile foundation arrangement. Bored piles should be preferred to driven piles as the latter causes vibration during pile driving operations and can have adverse impacts on nearby structures. Considering the sand fraction in the underlying soils are possibly of a collapsible nature and the ground water table, C.F.A piles should be considered. Herein, details of the logs should be given to specialist piling companies and firm proposals requested in the light of their particular proprietary pile and the eventual column loads.

Such piles should be designed and constructed in accordance with the following criteria:

- All piles should be founded on granitic residual soil or medium dense to dense sand at the base of any fill or firm to stiff clay/ medium dense sand or disturbed zones. The minimum piling depth of 8.0 m should be adopted below the proposed site surface level (Incorporating proposed site cuts modification). **These minimum depths relate to stability considerations and retention design may override these minimum depths.**

- An end bearing capacity of 500 kPa can be assumed to be available within the Residual Granitic Soil (Gravelly / Sandy Clay) commencing estimated below 8.0m.

- Extreme care will be required for each row of piles on the escarpment to prevent disturbance of the soils on the face of the escarpment (refer also to construction comments).

- For the row of piles at the base of the escarpment (Grid 1) lateral forces need not be considered, except that the spacing of piles should be such that a deeper seated failure surface cannot develop and this is thought to be 8.0 m, i.e. this row will act as a 'shear key' at the base of the escarpment.

  Spacing of these piles should not exceed 2.5 diameters.

  Following construction of Grid Line 1 the construction of Grid Line 2 should be in maximum 3.0 m drives. The retaining walls and piles should be constructed to resist the lateral forces associated with the sandy clay / clayey sand to the excavated depth, using a uniform pressure distribution of 6H. In addition, the surcharge loads associated with the structure and overburden should also be taken into account. Anchors should be considered in relation to these piles.

  These anchors could be designed as temporary until such time as the lateral forces are transferred through the structure or as permanent anchors.

  Given the recommendations in Section 3.1.4, it will be more effective to transfer loads into the structure to, in effect, create retention, rather than using permanent anchors.

The construction at Grid Line 1, 3 & 4 is expected to extend a minimum 8.0m in depth. Grid 3 will, however, be required to be designed for the construction loads during construction of Grid Line 4.

Note: Piles for the cable car footings should be completed in conjunction with the staged approach to the proposed residence and should also extend to the granitic residual soil anticipated to be in the order of 8.0 m down.

3.1.3 **Ground Anchors:** As it appears on both side elevation plans, a relatively significant Level 2 excavation is proposed as indicated by the plans to be in the range of 1.6 – 2.2 m. (If required) Ground anchors used in connection with the temporary support of any retention structures should extend into the Residual Granitic Soil (Gravelly / Sandy Clay).

For anchors installed into Residual Granitic Soil (Gravelly / Sandy Clay) it may be necessary to grout the anchor in a two stage process. First, low-pressure grout is applied and allowed to stiffen before high grout is used to hydro fracture the surrounding ground mass. Alternatively, the use of multiple under-reamed anchors may need to be considered.

Details of the logs should be given to proprietary companies and firm proposals requested.
The free length of the ground anchors should be sufficient to ensure that failure cannot occur on a sliding wedge behind the retention wall structures.

Generally, ground anchors should be installed at an angle of approximately 15^0 to 20^0 below the horizontal and where possible the ground anchor bond length should not exceed 12 metres to ensure adequate load transfer characteristics.

Anchors should be de stressed on completion.

- For the row of piles and retention on Grid Line 2, the proportioning, spacing and minimum founding depths of the piles will need to be based on the piles being able to withstand lateral earth pressure forces above.

Lateral active earth pressure forces, which will need to be restrained by the proposed footing arrangement, should be calculated using the following parameters or forces provided:

- Coefficient of Active Earth Pressure (K_A) - 0.33
- Coefficient of Passive Earth Pressure (Kp) - 3.0
- Bulk Density - 1.7 tonne/metre^3
- Coefficient of 'at rest' Earth Pressure - 0.5

These assume a drained condition and any surcharge is superimposed.

These lateral forces should be applied over the top 6.0 m of the site and these would obviously best be transferred into the structure and to the base of the escarpment by raking beams and racking piles. We enclose a photo of the construction of an adjoining residence in close proximity as an indication of such strutting. Clearly the generated lateral forces are significant.

- The structural continuity of the footing arrangement spanning between the pile at the ground surface level should be maximised to ensure that the building acts as a whole, perpendicular to the slope.

3.1.4 **Construction Issues and Construction Sequence:** Considering the anticipated stability risks, the most appropriate construction sequence is suggested as follows for the residence. This should be read in conjunction with Figure 4 'Construction Sequence'. However, this may require review, depending upon plant availability and final design.

1. Construct the piles on Grid Line 1 and raking piles.

   Design of Grid Line 1 should be based on the minimum depth, as per the stability requirement and the loads expected from the construction machinery. The piling rig constructing Grid Line 2 may use these piles for a construction platform. Further to this, once the details of the rig are known, a further check on construction stability should be carried out.

2. Construct working platform for Grid Line 2

3. Construct the piles on Grid Line 2. This will act as retention pile/retaining wall for the height indicated, but should be checked for construction loads.

4. Install anchors to ensure the stability of the slope in Grid Line 2, with drives not exceeding 3.0m.

5. Move the construction machinery on the slope to Grid Line 3 & 4 and excavate. It may be necessary to use a long reach auger which was used on the adjoining site.

6. Construct ground beams perpendicular to the slope prior to further construction at any level. Under no circumstances should the slope be excavated first as collapse is possible.

3.1.5 Inherent in the success of any foundation arrangement on this site, is to maximise strength perpendicular to the slope and use the piles and ground beams discussed to resist the large lateral forces generated.

Precautions or piles should also be installed on the boundaries as required.

3.2 PROPOSED GARAGE

3.2.1 **Stiffened Raft Slab:** Edge beams and internal load bearing ribs for a stiffened raft slab should be founded on medium dense sand natural 'shallow sand', subject to a minimum depth of 0.3 metres below the finished ground surface level. A maximum bearing pressure of 100 kPa should be adopted in the design of all edge beams and internal load bearing ribs. Internal non load bearing stiffening ribs, subject to a maximum bearing pressure of 50 kPa may be founded on medium dense sandy fill or medium dense sand. BH1 recorded 0.4 m of fill before the commencement of the natural sand, indicating deepening of the edge beams would likely be necessary below the minimum depth.

3.2.2 **Minimum Dimensions and Reinforcement for Raft Slab:** The minimum basic dimensions and reinforcement for the stiffened raft slab should correspond to the details given for a Class "M" stiffened raft slab detailed in current Australian Standard 2870 – 2011, "Residential Slabs and Footings".

3.2.3 **Maximum Depth of Fill beneath Stiffened Raft Slab Panels:** A total depth of up to approximately 600 millimetres of granular fill, or 400 millimetres of cohesive fill, is permissible beneath the panels of a stiffened raft slab. In areas of the raft slab where the maximum allowable depth of fill is exceeded, it will be necessary to deepen all edge and internal beams to the underlying medium dense sand throughout, and design the slab panels as fully suspended elements.

3.2.4 **Raft Slab Subgrade Preparation:** Preparation for raft slab construction should consist of stripping to grade and proof rolling the subgrade ensuring that any localised soft areas are removed and made good with clean granular filling compacted to a dry density not less than that of the surrounding acceptable material. If work is carried out following prolonged rain periods it is quite possible that the subgrade may exist in a condition significantly wet of optimum moisture content. Under these conditions it is not possible to proof roll the subgrade and it will be necessary to review the situation.

3.3 SITE EARTHWORKS AND RETENTION SYSTEMS

3.3.1 **Cut and Fill Earthworks:** Earthworks on or adjacent to the escarpment involving cutting and/or filling should be minimised, thereby reducing the risks of producing localised instabilities. Any major cut or fill operations will require to be assessed in detail by a qualified engineer and most certainly any permanent site cuts with batter grades steeper than 1 in 2 and a vertical height exceeding 0.6 metres will required an engineer designed retaining wall structure to provide support.

3.3.2 **Retaining Walls:** By observing site elevations, it is evident some site cuts exist in addition to the Level 2 excavation. Retaining walls less than 1.5 metres in height could simply be designed on the basis of an equivalent fluid of 0.56 $t/_{m}^{3}$. This assumes a drained situation and any surcharge loading is superimposed.

Alternatively, retaining walls could also be designed using parameters in 3.1.3.

3.3.3 **Drainage Behind Retention Structures:** The sands on this site should be self draining, but drainage should be provided to prevent temporary build-up.

3.3.4 **Proposed Driveway:** Although it was not requested nor any details provided regarding the driveway.

The following is recommended:

Preparation of pavement subgrades should consist of stripping to grade (Natural sand at the base of any fill) compacting the existing sand with appropriate compactive equipment to a dry density not less than 98% of the maximum dry density value determined by the Standard compaction test in accordance with current Australian Standard 1289.

The moisture content of the subgrade should be within 85-115% of the Standard optimum moisture content at the time of compaction.

Upon completion of compaction the subgrade should be thoroughly proof rolled with an appropriate roller, ensuring that any localised soft or spongy areas are removed and made good with clean granular filling, which should be compacted to a minimum dry density ratio of 98% Standard.

3.3.5 **Removal of Vegetation:** It is advised that the minimal possible amount of additional vegetation be removed from the face of the escarpment. In order to improve the long term stability of the escarpment, it is essential that additional planting of vegetation, ranging from dense ground cover through to large trees with extensive root systems be carried out. A landscape specialist should be consulted for appropriate plantings.

3.3.6 **Site Drainage:** An effective drainage system must be installed and maintained, such that all surface run-off, seepage water contained in the surface soils, and stormwater drainage from the proposed structure is drained to a point of discharge well clear of the escapement. Under no circumstances should any water be allowed to discharge onto the escarpment.

3.3.7 **General:** All footing excavations must be carefully examined to ensure that the required founding soil has been exposed throughout. Any unusual features must be reported to this office immediately to ensure that the recommendations outlined in this report remain relevant.

The above recommendations are based on the borehole and test results, together with experience of similar conditions and are expected to be typical of the area or areas being considered. Nevertheless, all excavations should be examined carefully and any unusual feature reported to us in order to determine whether any changes might be advisable.

4. LANDSLIDE RISK ASSESSMENT

4.1 HAZARD IDENTIFICATION

The potential hazards as they relate to this proposed development are seen as follows. It is emphasised this risk assessment is carried out following construction and assuming all the recommendations of this report are adopted.

Shallow Translational Slides – Hazard A

This mechanism is seen as the movement of the surface materials that is the fill and firm to stiff sandy clay / medium dense clayey sand. As there was evidence of ongoing minor erosion and fretting, movement of surface residual materials such as surface fill, is "possible". Especially following prolonged rains, the surface material will become unstable. The recommended footing system however will negate this influence on the structure, but it remains elsewhere.

Deep Seated Failure – Hazard B

There is a tendency for deep seated slips to occur in this area due the steep escarpment. Therefore, the hazard potential would be "possible".

Our foundation approach recognises a "possible" potential hazard.

The approach adopted is to ensure there is no impact from this hazard and to ensure the integrity of the residence in the event of the hazard by extending construction well beyond these potential failure zones.

4.2 POST CONSTRUCTION RISK ASSESMENT

Shallow Translational Slides – Hazard A.

Shallow translational failures will remain uphill of the proposed structure (structure constructed on piers) and can only be addressed by vegetation and slope maintenance.

Deep Seated Failure – Hazard B

There is a risk common to this area in considering the deep seated movements which cannot be avoided. However, as we are ensuring the stability of the proposed house by avoiding the failure slip surface, this hazard can be considered as "unlikely".

RISK TO PROPERTY (QUALITIVE)
POST CONSTRUCTION
(FOOTING CONSTRUCTION TO DENSE RESIDUAL GRANITIC SOIL (Gravelly / Sandy Clay)

HAZARDS		LIKELIHOOD	CONSEQUENCE	RISK
A	Shallow Translational Slide or Debris Creep	Possible	Minor Will not affect the structure	Adopted Low to Medium
B	Deep Seated Failure	Rare	Major	Low

POST CONSTRUCTION
(FOOTING CONSTRUCTION TO DENSE TO VERY DENSE CLAYEY SAND

HAZARD	A	B
INDICATIVE ANNUAL PROBABILITY	$0.5\ 10^{-3}$	0.5×10^{-5}
PROBABILITY OF SPATIAL IMPACT P_{SH}	0.1	1.0
OCCUPANCY NO. OF PEOPLE	0.8	0.8
VULNERABILITY	0.1 Unlikely to be buried due to footing arrangement	1.0
INDIVIDUAL RISK	0.4×10^{-5}	0.4×10^{-5}
RISK EVALUATION	Acceptable	Acceptable

4.3 TOLERABLE RISK CRITERIA

The table below provides AGS recommendations in relation to tolerable risk for loss of life. It is important to note that "Existing Slope" and "Existing Development" situations incur a tolerable loss of life risk for the person most at risk, one (1) order of magnitude lower than for "Newly Constructed Slopes," "New Developments," and/or "Existing Landslides."

Situation	Suggested Tolerable Loss of Life Risk for the person most at risk
Existing Slope / Existing Development	10^{-4} / annum
New Constructed Slope / New Development / Existing Landslide	10^{-5} / annum

As outlined in the above risk assessment, the required tolerable risk for property and loss of life is within the acceptable risk criteria in accordance with AGS Guidelines 2007. Given our foundation arrangement.

Based on the geotechnical investigation and risk assessment carried out as part of our investigation, and based on the "Tolerable Risk Criteria" outlined above, our risk evaluation for the proposed works at 601 Point Nepean Road, McCrae indicates construction is feasible and safe with risk to life, and risk to property within the acceptable tolerable limits in accordance with AGS guidelines 2007. This is subject to the recommendations contained in this report.

Irrelevant / Sensitive

T.J. HOLT MIEAust CPEng EC-1022
A.S. JAMES PTY LTD

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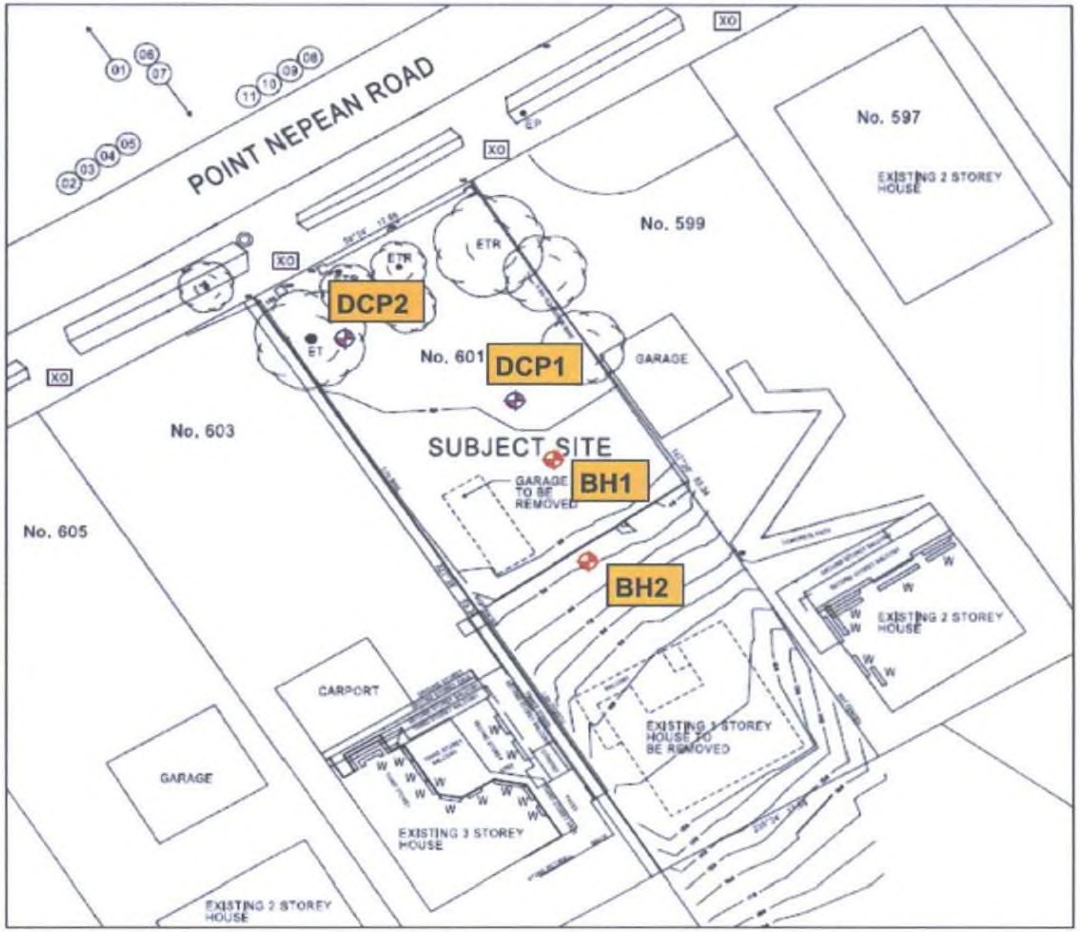
EXISTING SITE CONDITION AT THE TIME OF INVESTIGATION.



A.S. JAMES PTY LTD
Geotechnical Engineers

JOB: 601 POINT NEPEAN ROAD
MC CRAE, VIC

JOB No: 116268 DATE: Nov '14

Denotes Approximate Borehole Location. (BH)

Denotes Approximate DCP test Location

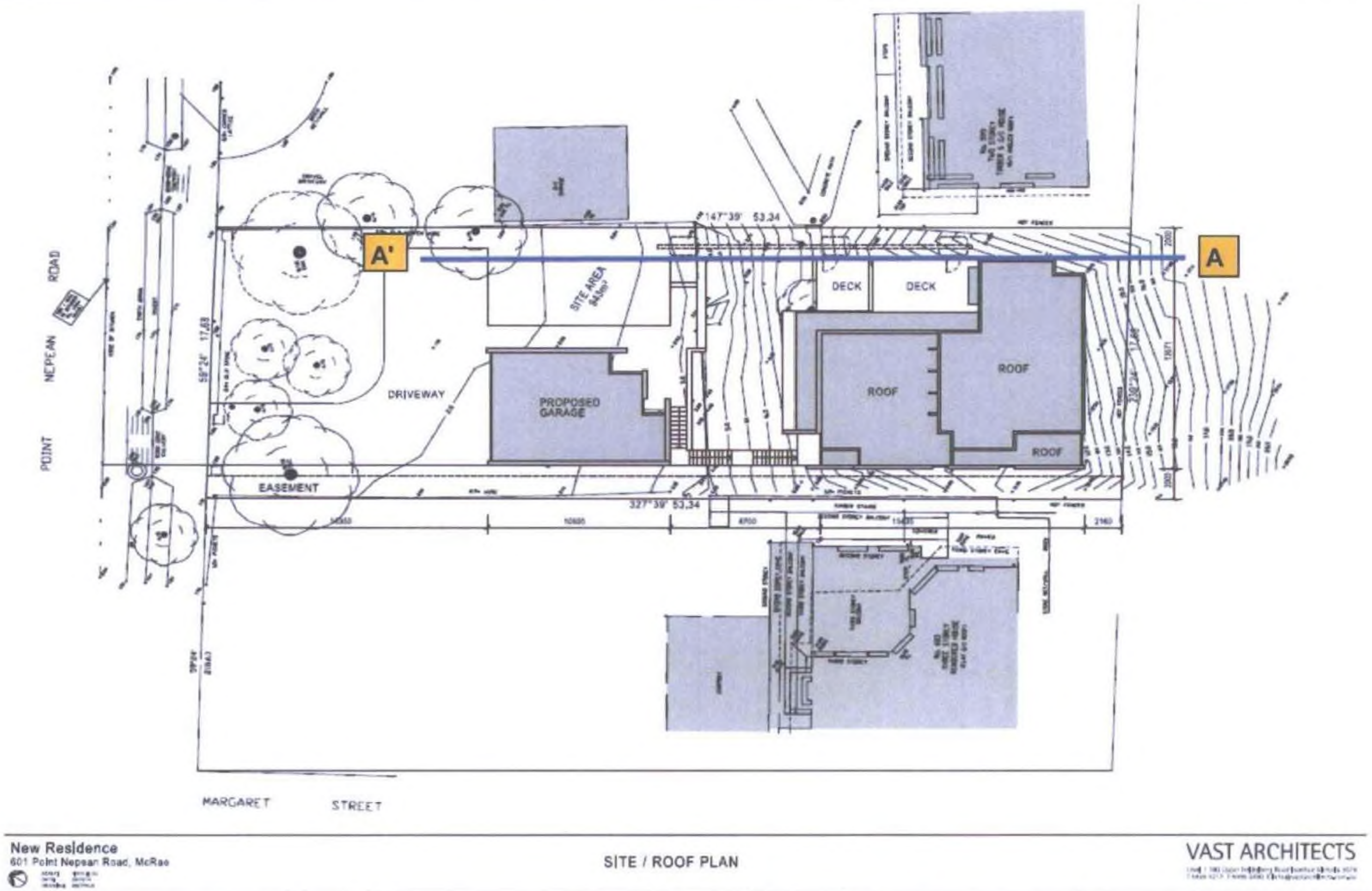
CHECKED: T. HOLT
DRAWN: J. HOLT

BORE AND DCP MAP

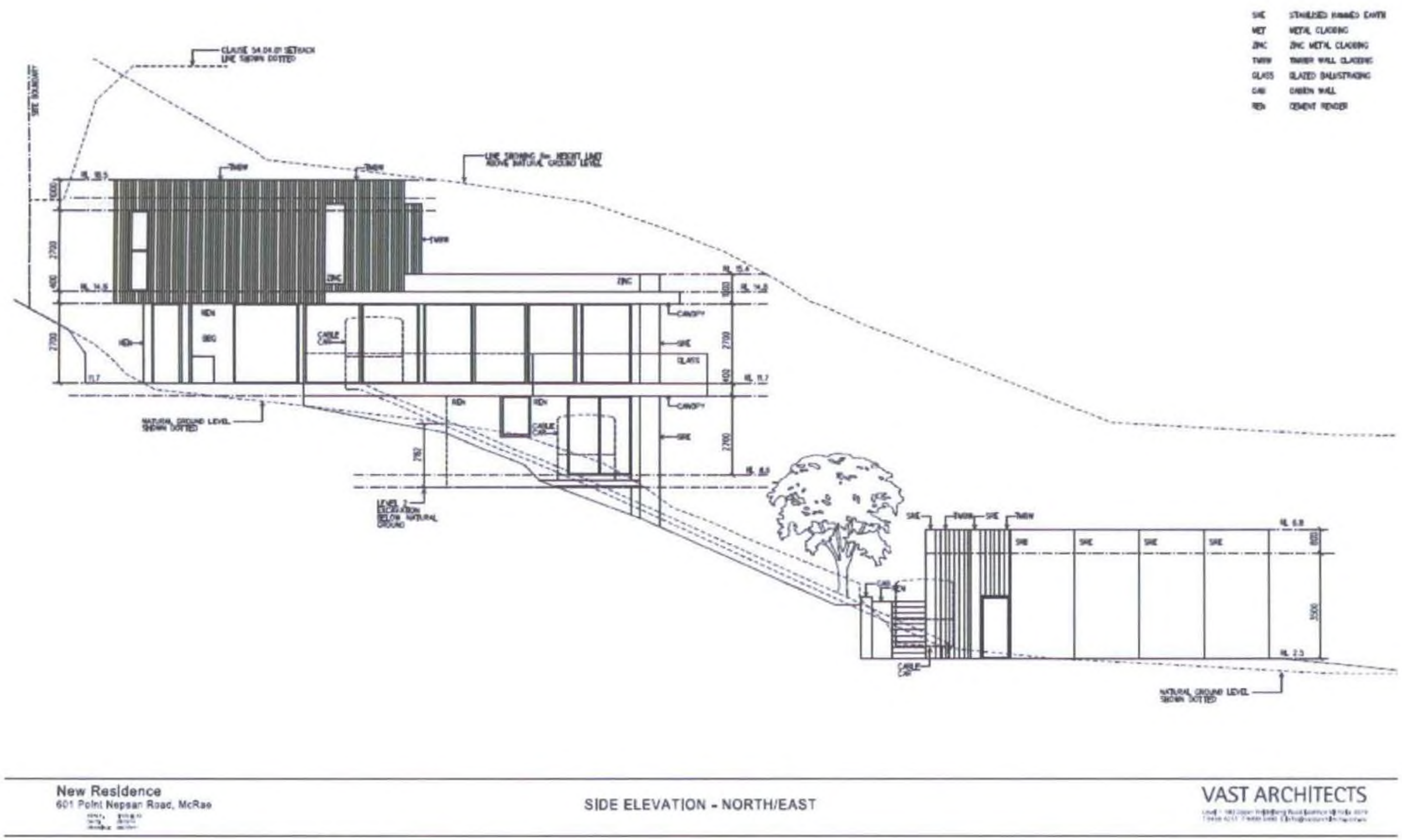
N

SOURCE: Google Earth Image
VAST Architects

FIGURE 1



A.S. JAMES PTY LTD Geotechnical Engineers		CHECKED: T. HOLT DRAWN: J. HOLT
JOB: 601 POINT NEPEAN ROAD MC CRAE, VIC		**PROPOSED BUILDING PLANS WITH MODELLED SECTION INDICATED.**
JOB No: 116268 DATE: Nov '14		SOURCE: VAST ARCHITECTS — FIGURE 2



A.S. JAMES PTY LTD
Geotechnical Engineers

JOB: 601 POINT NEPEAN ROAD
MC CRAE, VIC

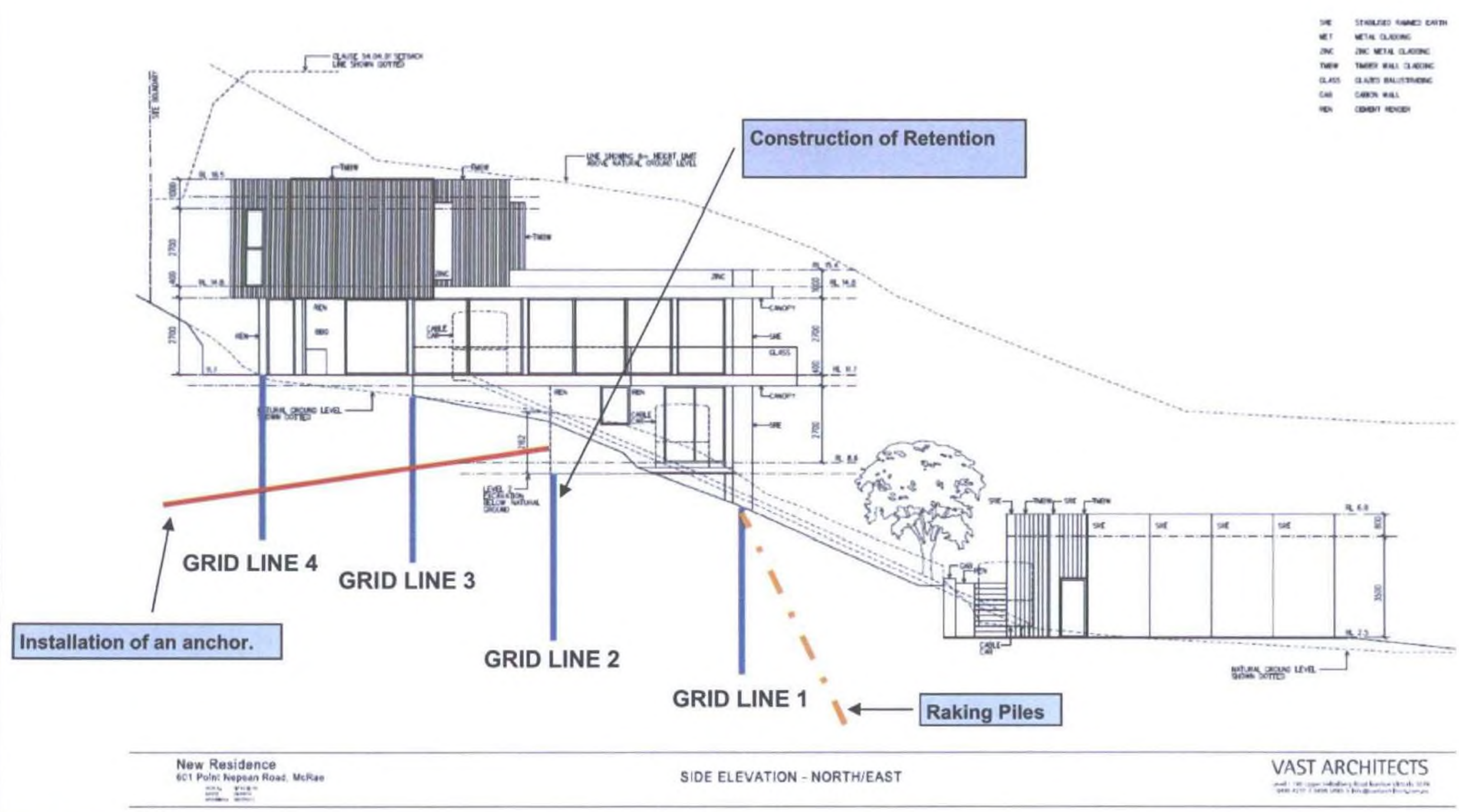
JOB No: 116268 DATE: Nov '14

CHECKED: T. HOLT
DRAWN: J. HOLT

MODELLED SECTION

SOURCE: VAST ARCHITECTS

FIGURE 3



A.S. JAMES PTY LTD
Geotechnical Engineers

JOB: 601 POINT NEPEAN ROAD
MC CRAE, VIC

JOB No: 116268 **DATE:** Nov '14

CHECKED: T. HOLT
DRAWN: J. HOLT

CONSTRUCTION SEQUENCE

SOURCE: VAST ARCHITECTS

FIGURE 4

A.S.JAMES PTY. LTD.
Geotechnical Engineers

Location: 601 Point Nepean Rd, MCCRAE VIC
Borehole: 1
SHEET 1
Job No. 116269
Date: Nov '14
Ground Water: 1.05 m

Soil Type	Description	Depth	Tests	Results
	VERTICAL BORE			
FILL (0.0-0.4 m)	Medium to Coarse Grained Dark Grey Mottled Brown **Sand** Accessory Materials: Clayey, Trace Brick / Concrete Fragments, Shallow Organics Moist, Medium Dense	0		
		0.4		
SAND (SP) (0.4-1.2 m)	Medium to Coarse Grained Brown / Grey With Silt Moist, Medium Dense	1.2		
		1.5	+	(1.5 m) N = 14 / 17 / 9 for 70 mm
SAND (SP) (1.2-1.5 m)	Medium to Coarse Grained Grey With Silt Moist, Medium Dense			
			+	(3.0 m) N = 11 / 15 / 19 for 80 mm
SAND (SM / SC) (1.5-7.0 m)	Fine to Medium Grained, Some Coarse Grains Orange Brown / Grey /Intermittent Red Brown from 6.0 m Silty, Some Clay, Possibly trace Cemented Sand Lenses Moist, Medium Dense to Dense		+	(4.5 m) N = 11 / 16 / 13 for 80 mm
			+	(6.0 m) N = 11 / 26 / -
		7.0		
SAND (SC / GC) (7.0-10.0 m) / GRANITE (RS)	Granitic Fine to Medium Grained, Some Coarse Grains Low to Medium Plasticity Grey / Orange Brown With Silt, Very Clayey, Coarse Sand Sized Quartz Particles Moist, Dense / Stiff		+	(7.5 m) N = 12 / 20 / -
			+	(9.0 m) N = 12 / 16 / - , w = 17.4 %
	Continue over to Sheet 2	10.0		

+ Standard Penetration Test - N blows/150mm. incr.	c Apparent Cohesion	L.L. Liquid Limit
I Undisturbed Sample - Diameter Stated	Ø Friction Angle	P.L. Plastic Limit
s Vane Shear Strength	P Wet Density	P.I. Plasticity Index
p Pocket Penetrometer Resistance	w Moisture Content	L.S. Linear Shrinkage

Figure 5

A.S.JAMES PTY. LTD.
Geotechnical Engineers

Location: 601 Point Nepean Rd MCCRAE VIC
Job No. 116269
Borehole: 1
SHEET 2
Date: Nov '14

Soil Type	Description	Depth			
	Continue from Sheet 1				
		10.0			
SAND (GC / SC) 10.0-12.0m / GRANITE (RS)	Granitic Fine to Coarse Grained Grey Brown Mottled Orange With Silt, Some Clay, Coarse Sand Sized Quartz Particles Moist, Dense / Stiff			+	(10.5 m) N = 24 / 16 for 70 mm /- , w = 14.4%
	BOREHOLE TERMINATED	12.5		+	(12.0 m) N = 22 / 8 for 30 mm /- , w = 13.1 %

+ Standard Penetration Test - N blows/150mm. incr.	c Apparent Cohesion	L.L. Liquid Limit	**Figure 6**
I Undisturbed Sample - Diameter Stated	Ø Friction Angle	P.L. Plastic Limit	
s Vane Shear Strength	P Wet Density	P.I. Plasticity Index	
p Pocket Penetrometer Resistance	w Moisture Content	L.S. Linear Shrinkage	

A.S.JAMES PTY. LTD.
Geotechnical Engineers

Location: 601 Point Nepean Rd, MCCRAE VIC
Borehole: 2
SHEET 1
Job No. 116269
Date: Nov '14
Ground Water: NIL

Soil Type	Description	Depth	Tests	Results
	SUB-HORIZONTAL BORE			
FILL (0.0-1.1 m)	Fine to Coarse Grained Grey Brown **Clayey Sand / Sandy Clay** Accessory Materials: Silty, Gravels Trace Brick / Concrete Fragments, Shallow Organics Moist, Medium Dense	0	.	w = 12.0 %
CLAY (CL / SC) (1.1-2.7 m)	Fine to Medium Grained Low to Medium Plasticity Dark Grey / Intermittent Orange Brown Very Sandy With Silt Moist / Very Moist from 2.0 m, Stiff / Medium Dense	1.1	. .	w = 21.2 % w = 18.5 %
CLAY (CL / SC) (2.7-4.5 m)	Fine to Medium Grained Low to Medium Plasticity Grey Brown Very Sandy, Silty, Small Gravels, Grinding @ 3.6 m Moist, Stiff / Medium Dense	2.7	. . .	w = 17.3 % w = 18.2 % w = 17.3 %
SAND (SC / CL) (4.5-7.5 m)	Fine to Medium Grained Orange Brown Very Clayey, Silty, Trace Small Gravels Moist, Medium Dense / Stiff	4.5	. . .	w = 14.8 % w = 13.2 % w = 15.0 %
CLAY (CL / SC) (7.5-11.0 m)	Fine to Coarse Grained Low to Medium Plasticity Grey Brown Very Sandy, Silty, Small Granitic Gravels observed from 8.0 m Moist, Stiff / Medium Dense	7.5	. . .	w = 11.6 % w = 12.2 % w = 12.3 %

Continue to Sheet 2

+ Standard Penetration Test - N blows/150mm. incr.	c Apparent Cohesion	L.L. Liquid Limit	**Figure 7**
I Undisturbed Sample - Diameter Stated	Ø Friction Angle	P.L. Plastic Limit	
s Vane Shear Strength	P Wet Density	P.I. Plasticity Index	
p Pocket Penetrometer Resistance	w Moisture Content	L.S. Linear Shrinkage	

A.S.JAMES PTY. LTD.
Geotechnical Engineers

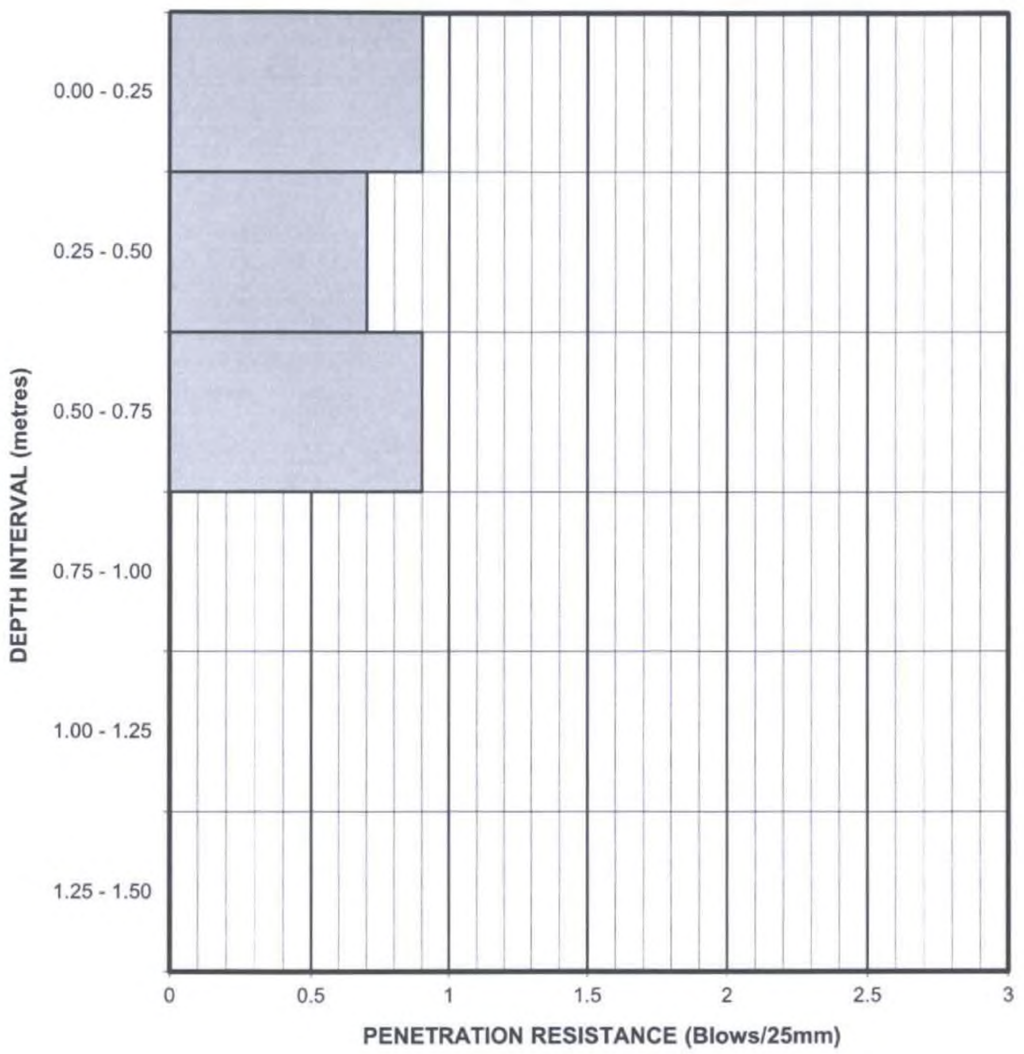
Location: 601 Point Nepean Rd, MCCRAE VIC
Borehole: 2
SHEET 2
Job No. 116269
Date: Nov '14
Ground Water: NIL

Soil Type	Description	Depth	Tests	Results
	Continue from Sheet 1			
		10.0		
SAND	Fine to Coarse Grained	11.0	.	w = 11.6 %
(SC /GC)	Low to Medium Plasticity			
11.0-12.0m	Light Brown Grey			
/ GRANITE	Very Clayey, Silty, Gravelly (Granitic)			
(RS)	Moist, Medium Dense / Stiff			
	BOREHOLE TERMINATED	12.0		

+ Standard Penetration Test - N blows/150mm. incr.	c Apparent Cohesion	L.L. Liquid Limit	**Figure 8**
I Undisturbed Sample - Diameter Stated	Ø Friction Angle	P.L. Plastic Limit	
s Vane Shear Strength	P Wet Density	P.I. Plasticity Index	
p Pocket Penetrometer Resistance	w Moisture Content	L.S. Linear Shrinkage	

A.S. JAMES PTY LTD Geotechnical Engineers	JOB: 601 Point Nepean Road MCCRAE, VIC	JOB No 116269
		DATE: Nov 14'

DEPTH BELOW GROUND SURFACE AT THE COMMENCEMENT OF PENETRATION: 0.00

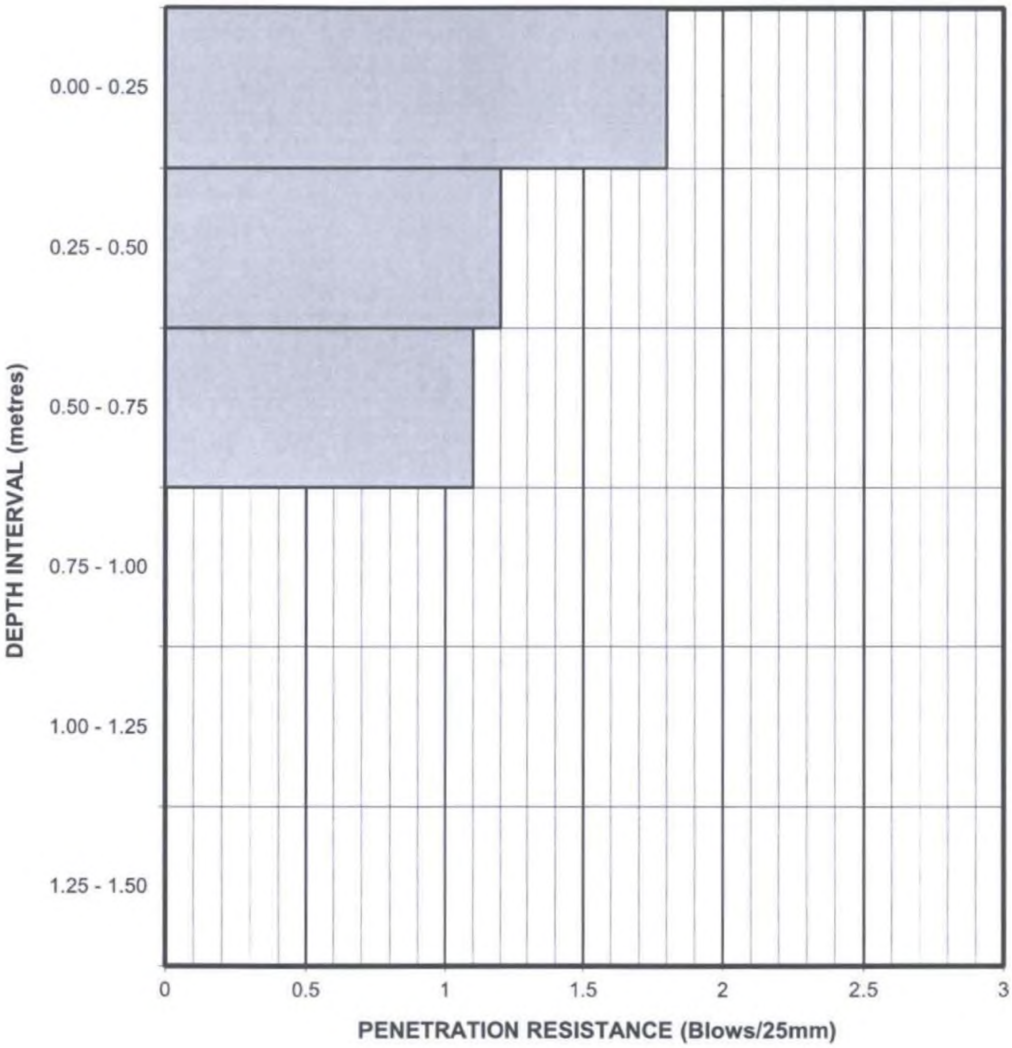


TEST LOCATION: DCP 1 (REFER TO FIGURE 1)

DYNAMIC CONE PENETROMETER TEST (AS1289, 6.3.2, 1997)	Drawn / Tested: J. Holt	Figure
325 sq.mm Cone - 9 kg Weight Falling 510 mm	Checked: T.Holt	9

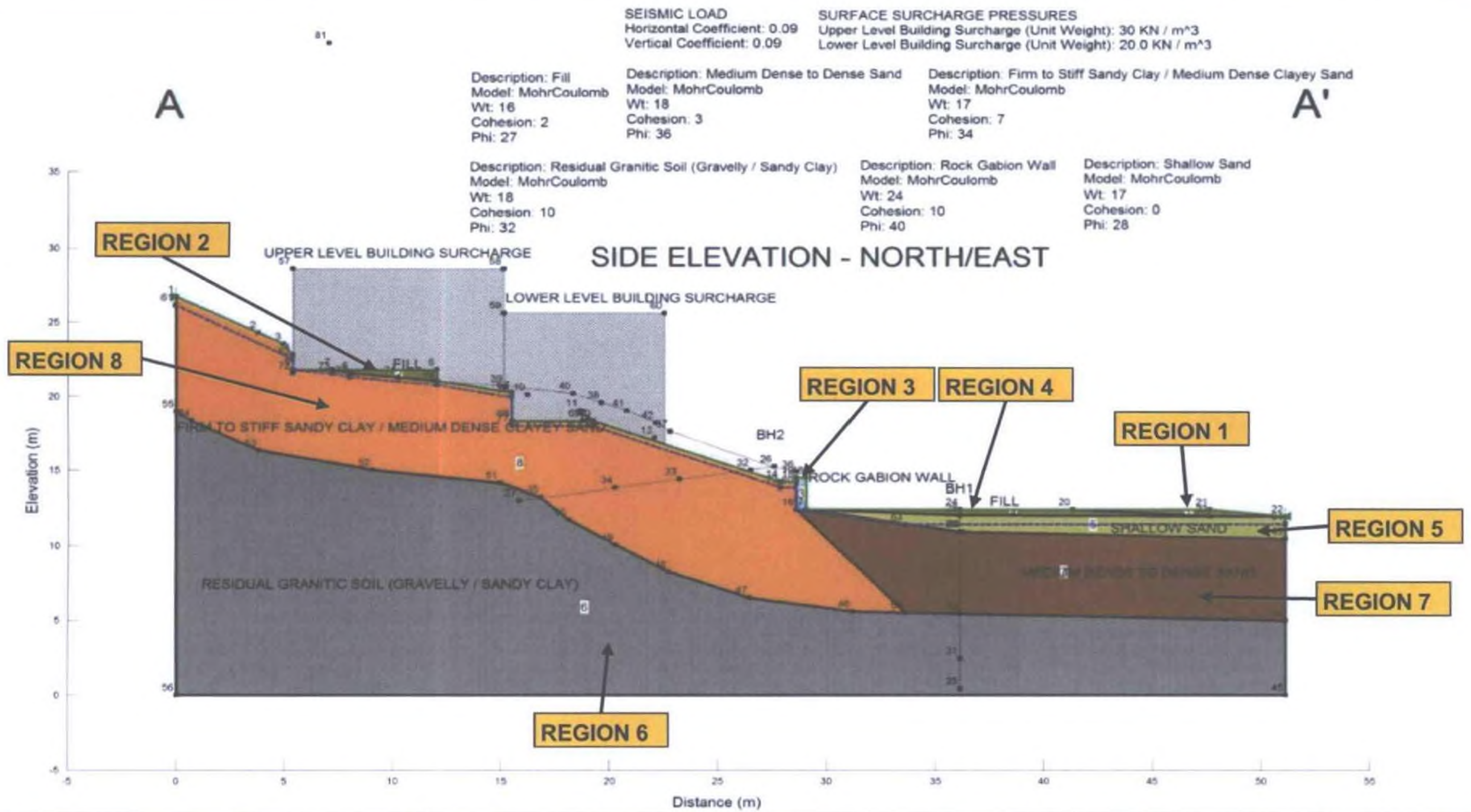
A.S. JAMES PTY LTD Geotechnical Engineers	JOB: 601 Point Nepean Road MCCRAE, VIC	JOB No 116269 DATE: Nov 14'

DEPTH BELOW GROUND SURFACE AT THE COMMENCEMENT OF PENETRATION: 0.00



TEST LOCATION: DCP 2 (REFER TO FIGURE 1)

DYNAMIC CONE PENETROMETER TEST (AS1289, 6.3.2, 1997) 325 sq.mm Cone - 9 kg Weight Falling 510 mm	Drawn / Tested: J. Holt Checked: T.Holt	Figure 10



A.S. JAMES PTY LTD
Geotechnical Engineers

JOB: 601 POINT NEPEAN ROAD
MC CRAE, VIC

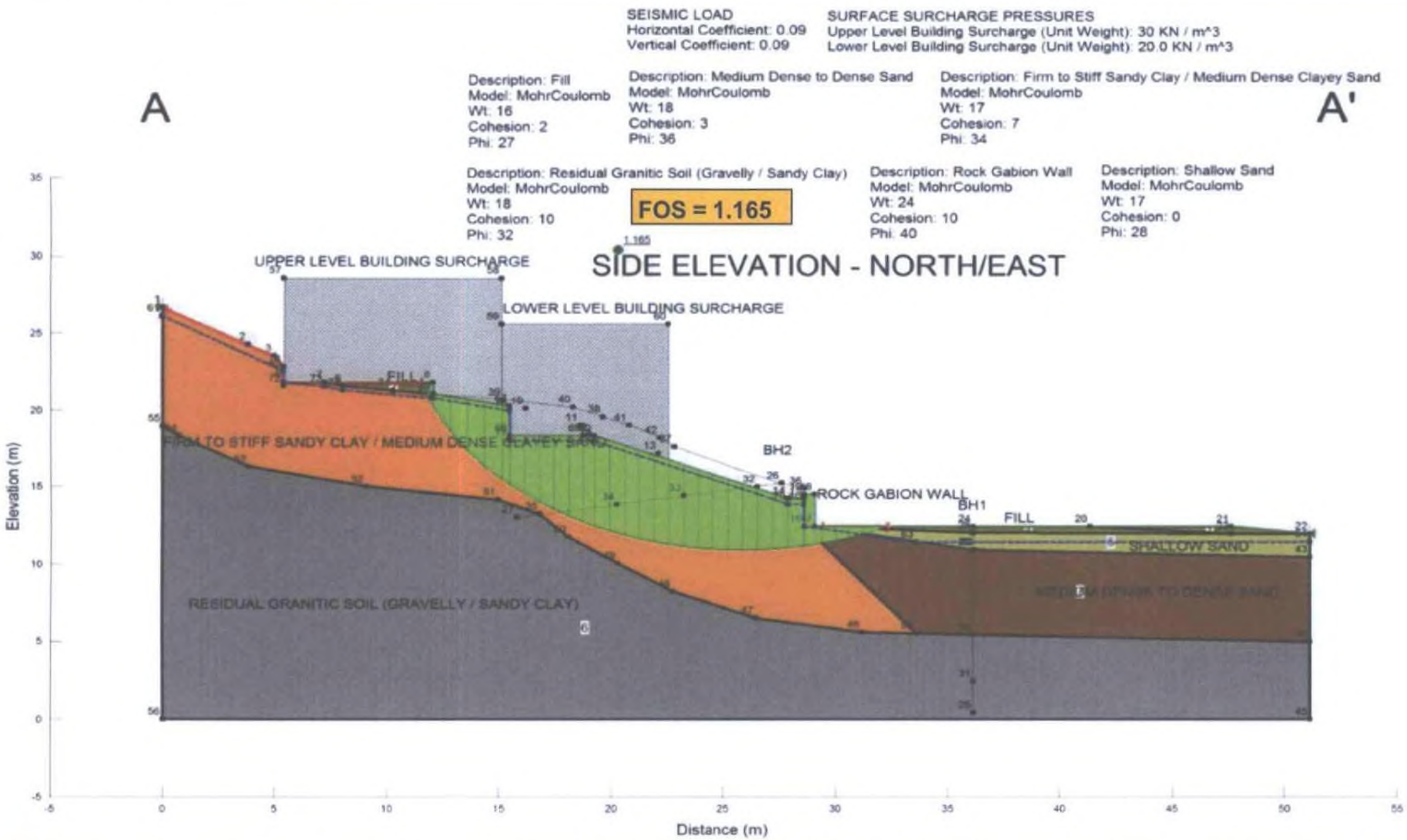
JOB No: 116268 **DATE: Nov '14**

Refer to Figure 2 for section Location.

CHECKED: T. HOLT
DRAWN: J. HOLT

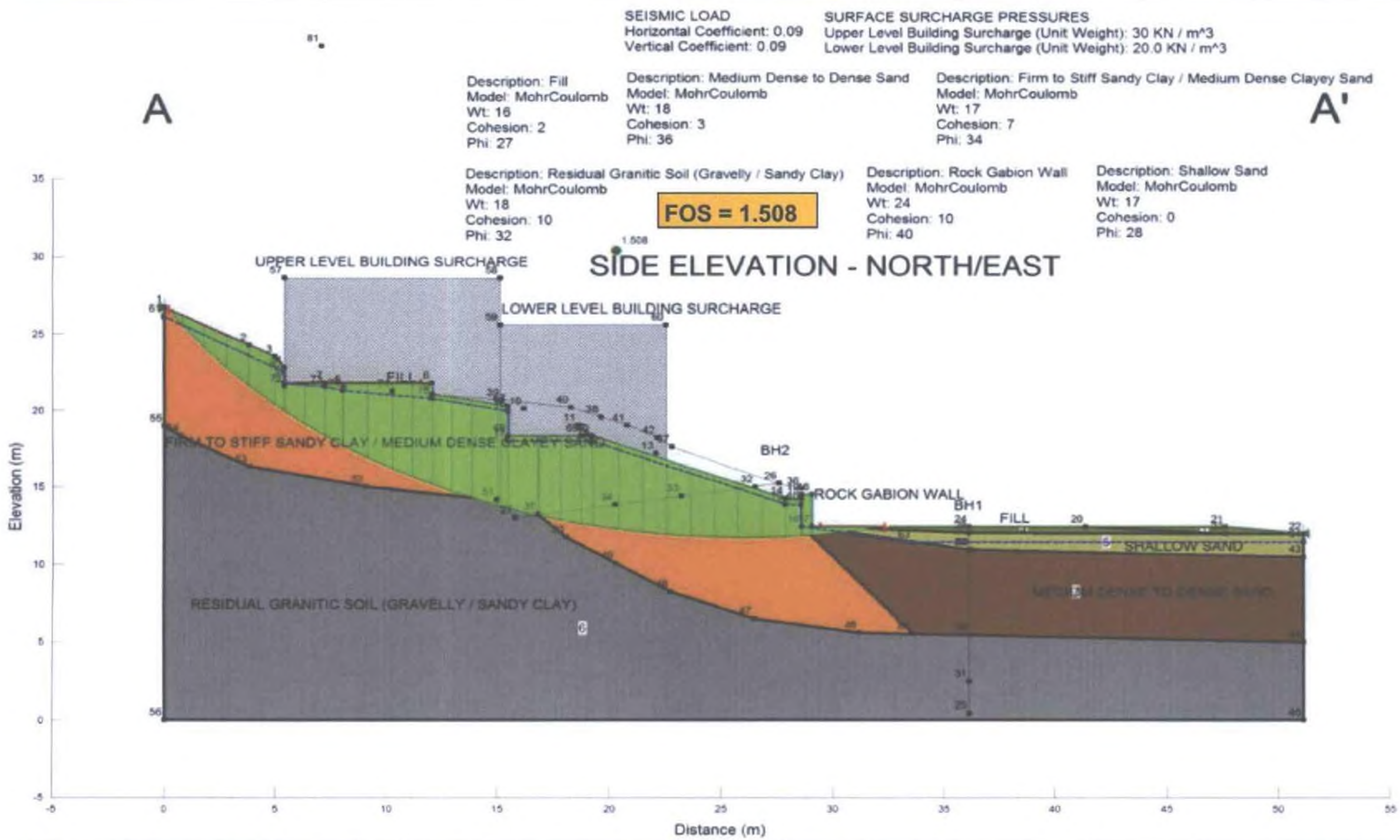
STABILITY MODEL SECTION

FIGURE 11

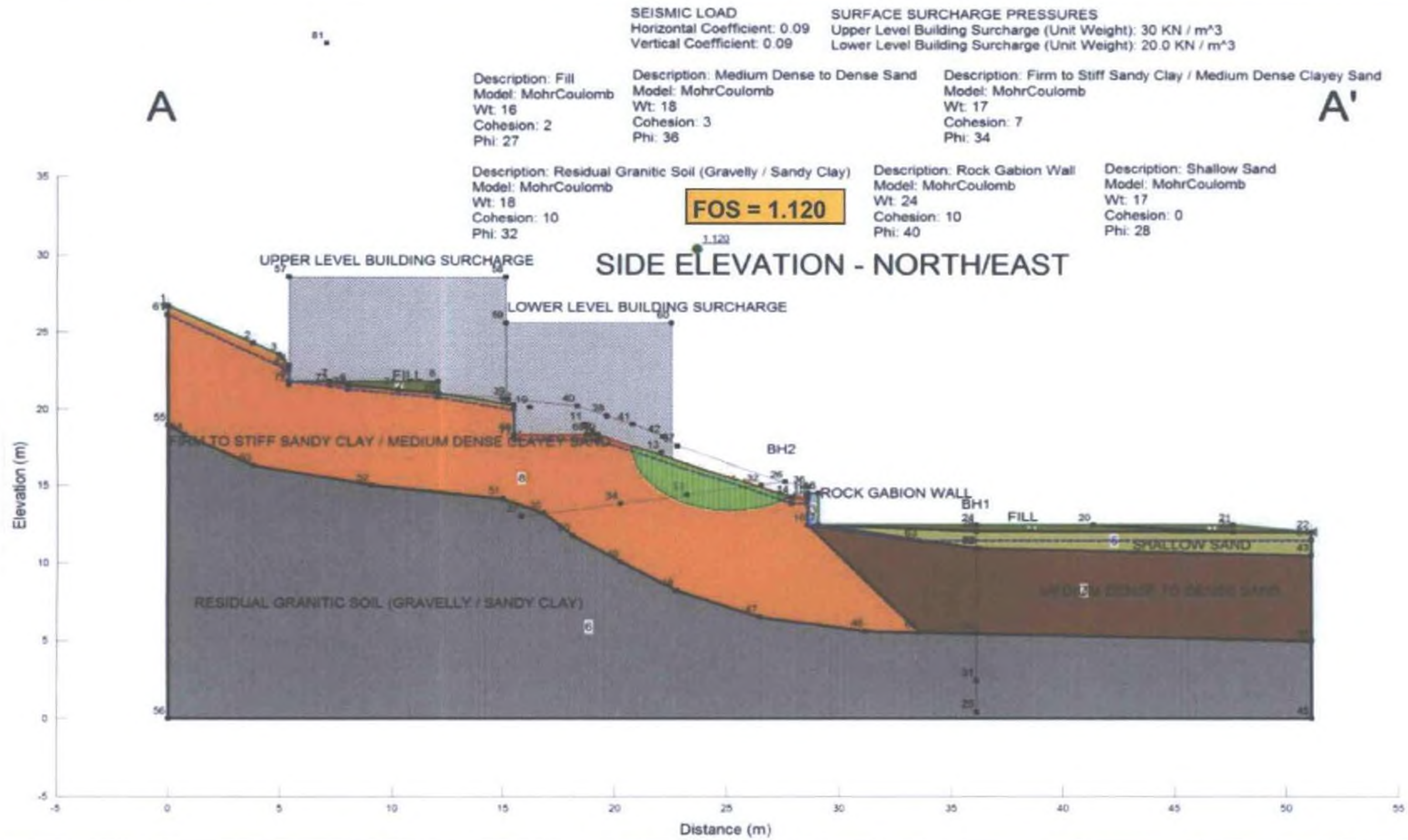


A.S. JAMES PTY LTD Geotechnical Engineers	**Refer to Figure 2 for section Location.**	**CHECKED:** T. HOLT **DRAWN:** J. HOLT
JOB: 601 POINT NEPEAN ROAD MC CRAE, VIC		**STABILITY MODEL SECTION CRITICAL OVERALL**
JOB No: 116268 **DATE: Nov '14**		**FIGURE 12**

MSC.5067.0001.0177



A.S. JAMES PTY LTD Geotechnical Engineers	Refer to Figure 2 for section Location.	CHECKED: T. HOLT DRAWN: J. HOLT
JOB: 601 POINT NEPEAN ROAD MC CRAE, VIC		STABILITY MODEL SECTION OVERALL FOS = 1.5 WITH SEISMIC LOAD
JOB No: 116268 DATE: Nov '14		FIGURE 13



A.S. JAMES PTY LTD
Geotechnical Engineers

JOB: 601 POINT NEPEAN ROAD
MC CRAE, VIC

JOB No: 116268 **DATE:** Nov '14

Refer to Figure 2 for section Location.

CHECKED: T. HOLT
DRAWN: J. HOLT

STABILITY MODEL SECTION CRITICAL SHALLOW FAILURE WITH SEISMIC LOADING

FIGURE 14

A.S. JAMES PTY LTD
Geotechnical Engineers

JOB: 601 POINT NEPEAN ROAD
MC CRAE, VIC

JOB No: 116268 DATE: Nov '14

CHECKED: T. HOLT
DRAWN: J. HOLT

589 Point Nepean Road
McCrae
Site Photograph

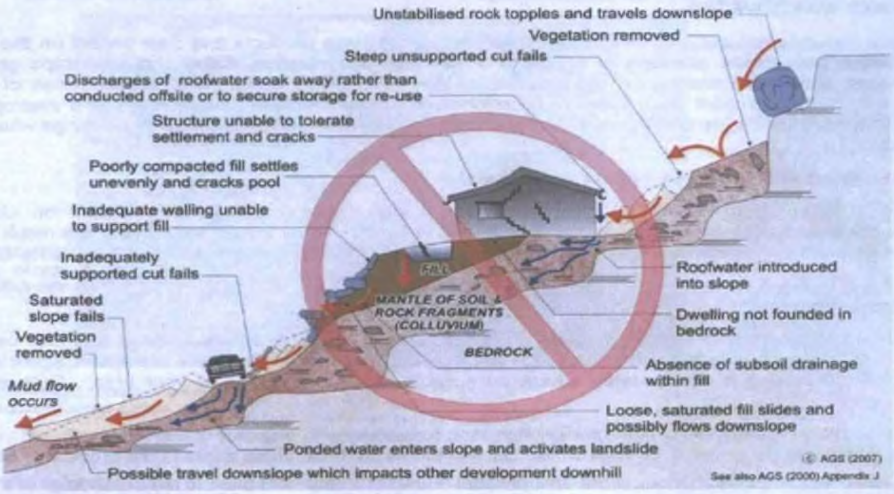
SOURCE: VAST ARCHITECTS

FIGURE 15

A.S. JAMES PTY Geotechnical Engineers	JOB: 601 Point Nepean Rd MCCRAE, VIC	JOB No. 116269
		DATE: NOV '14

AUSTRALIAN GEOGUIDE LR8 (CONSTRUCTION PRACTICE)

EXAMPLES OF POOR HILLSIDE CONSTRUCTION PRACTICE



WHY ARE THESE PRACTICES POOR?

Roadways and parking areas - are unsurfaced and lack proper table drains (gutters) causing surface water to pond and soak into the ground.

Cut and fill - has been used to balance earthworks quantities and level the site leaving unstable cut faces and added large surface loads to the ground. Failure to compact the fill properly has led to settlement, which will probably continue for several years after completion. The house and pool have been built on the fill and have settled with it and cracked. Leakage from the cracked pool and the applied surface loads from the fill have combined to cause landslides.

Retaining walls - have been avoided, to minimise cost, and hand placed rock walls used instead. Without applying engineering design principles, the walls have failed to provide the required support to the ground and have failed, creating a very dangerous situation.

A heavy, rigid, house - has been built on shallow, conventional, footings. Not only has the brickwork cracked because of the resulting ground movements, but it has also become involved in a man-made landslide.

Soak-away drainage - has been used for sewage and surface water run-off from roofs and pavements. This water soaks into the ground and raises the water table (GeoGuide LR5). Subsoil drains that run along the contours should be avoided for the same reason. If felt necessary, subsoil drains should run steeply downhill in a chevron, or herring bone, pattern. This may conflict with the requirements for effluent and surface water disposal (GeoGuide LR9) and if so, you will need to seek professional advice.

Rock debris - from landslides higher up on the slope seems likely to pass through the site. Such locations are often referred to by geotechnical practitioners as "debris flow paths". Rock is normally even denser than ordinary fill, so even quite modest boulders are likely to weigh many tonnes and do a lot of damage once they start to roll. Boulders have been known to travel hundreds of metres downhill leaving behind a trail of destruction.

Vegetation - has been completely cleared, leading to a possible rise in the water table and increased landslide risk (GeoGuide LR5).

DON'T CUT CORNERS ON HILLSIDE SITES - OBTAIN ADVICE FROM A GEOTECHNICAL PRACTITIONER

More information relevant to your particular situation may be found in other Australian GeoGuides:

- GeoGuide LR1 - Introduction
- GeoGuide LR2 - Landslides
- GeoGuide LR3 - Landslides in Soil
- GeoGuide LR4 - Landslides in Rock
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
- GeoGuide LR7 - Landslide Risk
- GeoGuide LR9 - Effluent & Surface Water Disposal
- GeoGuide LR10 - Coastal Landslides
- GeoGuide LR11 - Record Keeping

The Australian GeoGuides (LR series) are a set of publications intended for property owners; local councils; planning authorities; developers; insurers; lawyers and, in fact, anyone who lives with, or has an interest in, a natural or engineered slope, a cutting, or an excavation. They are intended to help you understand why slopes and retaining structures can be a hazard and what can be done with appropriate professional advice and local council approval (if required) to remove, reduce, or minimise the risk they represent. The GeoGuides have been prepared by the Australian Geomechanics Society, a specialist technical society within Engineers Australia, the national peak body for all engineering disciplines in Australia, whose members are professional geotechnical engineers and engineering geologists with a particular interest in ground engineering. The GeoGuides have been funded under the Australian governments' National Disaster Mitigation Program.

POOR HILL SIDE PRACTICE	Drawn: G. Luther	FIGURE 16

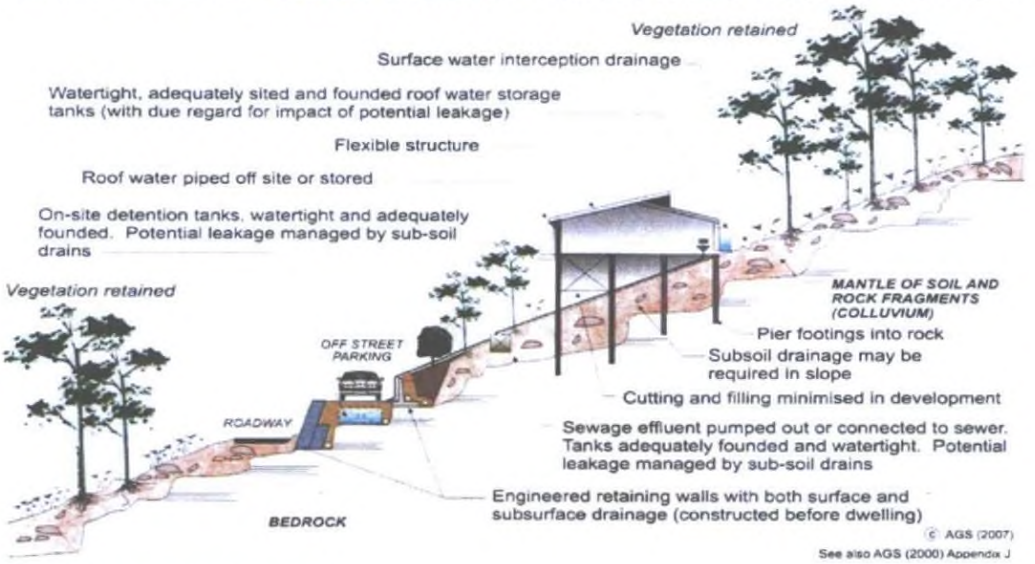
A.S. JAMES PTY Geotechnical Engineers	JOB: 601 Point Nepean Rd MCCRAE, VIC	JOB No. 116269
		DATE: NOV '14

AUSTRALIAN GEOGUIDE LR8 (CONSTRUCTION PRACTICE)

HILLSIDE CONSTRUCTION PRACTICE

Sensible development practices are required when building on hillsides, particularly if the hillside has more than a low risk of instability (GeoGuide LR7). Only building techniques intended to maintain, or reduce, the overall level of landslide risk should be considered. Examples of good hillside construction practice are illustrated below.

EXAMPLES OF **GOOD** HILLSIDE CONSTRUCTION PRACTICE



WHY ARE THESE PRACTICES GOOD?

Roadways and parking areas - are paved and incorporate kerbs which prevent water discharging straight into the hillside (GeoGuide LR5).

Cuttings - are supported by retaining walls (GeoGuide LR6).

Retaining walls - are engineer designed to withstand the lateral earth pressures and surcharges expected, and include drains to prevent water pressures developing in the backfill. Where the ground slopes steeply down towards the high side of a retaining wall, the disturbing force (see GeoGuide LR6) can be two or more times that in level ground. Retaining walls must be designed taking these forces into account.

Sewage - whether treated or not is either taken away in pipes or contained in properly founded tanks so it cannot soak into the ground.

Surface water - from roofs and other hard surfaces is piped away to a suitable discharge point rather than being allowed to infiltrate into the ground. Preferably, the discharge point will be in a natural creek where ground water exits, rather than enters, the ground. Shallow, lined, drains on the surface can fulfil the same purpose (GeoGuide LR5).

Surface loads - are minimised. No fill embankments have been built. The house is a lightweight structure. Foundation loads have been taken down below the level at which a landslide is likely to occur and, preferably, to rock. This sort of construction is probably not applicable to soil slopes (GeoGuide LR3). If you are uncertain whether your site has rock near the surface, or is essentially a soil slope, you should engage a geotechnical practitioner to find out.

Flexible structures - have been used because they can tolerate a certain amount of movement with minimal signs of distress and maintain their functionality.

Vegetation clearance - on soil slopes has been kept to a reasonable minimum. Trees, and to a lesser extent smaller vegetation, take large quantities of water out of the ground every day. This lowers the ground water table, which in turn helps to maintain the stability of the slope. Large scale clearing can result in a rise in water table with a consequent increase in the likelihood of a landslide (GeoGuide LR5). An exception may have to be made to this rule on steep rock slopes where trees have little effect on the water table, but their roots pose a landslide hazard by dislodging boulders.

Possible effects of ignoring good construction practices are illustrated on page 2. Unfortunately, these poor construction practices are not as unusual as you might think and are often chosen because, on the face of it, they will save the developer, or owner, money. You should not lose sight of the fact that the cost and anguish associated with any one of the disasters illustrated, is likely to more than wipe out any apparent savings at the outset.

ADOPT GOOD PRACTICE ON HILLSIDE SITES

GOOD HILL SIDE PRACTICE	Tested:	FIGURE 17
	Drawn: G. Luther	

MSC.5067.0001.0182

A.S. JAMES PTY. LTD. Geotechnical Engineers Clayton Laboratory 16 Libbett Av Clayton South	JOB: 601 Point Nepean Road Mccrae	JOB No. 116269 REPORT No. L001 DATE. 27-Nov-14

FOR Vast Architects
Level 1, 180 Upper Heidelberg Rd
IVANHOE, VIC 3079

Lab Sample No.	Soil Description	Sample Location	Depth of Sample (m)	Insitu Moisture Content (%)
50543	SAND, Clayey - Grey Brown	Bore Hole 2	1.00	12.0
50544	SAND, Clayey - Dark Grey	Bore Hole 2	2.00	21.2
50545	SAND, Clayey - Dark Grey	Bore Hole 2	2.50	18.5
50546	SAND, Clayey - Grey	Bore Hole 2	3.00	17.3
50547	SAND, Clayey - Grey	Bore Hole 2	3.50	18.2
50548	SAND, Clayey - Grey	Bore Hole 2	4.00	17.3
50549	SAND, Clayey - Pale Orange Brown	Bore Hole 2	5.00	14.8
50550	SAND, Clayey - Orange Brown	Bore Hole 2	6.00	13.2

NOTES: Testing Carried Out On Samples As Supplied

NATA Accredited for compliance with ISO/IEC 17025
Accreditation No. 934

Irrelevant / Sensitive

APPROVED SIGNATORY D.Vu 27-Nov-14

Soil moisture content tests - Determination of the moisture content of a soil - AS PER AS1289 .1.1,2.1.1, A.S.JAMES FORM No: LR014 (Fig 1) / REV 9 / 12/12/11	TESTED : D.Vuarchoz REPORTED : D.Vuarchoz	FIGURE 18 1 of 1

A.S. JAMES PTY.LTD	JOB: 601 Point Nepean Road Mccrae	
Geotechnical Engineers Clayton Laboratory 16 Libbett Av Clayton South		JOB No. 116269 REPORT No: L001-1 DATE: 27-Nov-14

FOR Vast Architects
Level 1, 180 Upper Heidelberg Rd
IVANHOE, VIC 3079

Test	Lab Sample No.	Location	Depth (m)	Sample Description	Condition	Preparation
1	50550	Bore Hole 2	6.0	SAND Clayey - Orange Brown	Oven Dried < 50 Deg	Dry Sieved
2	50551	Bore Hole 2	4.5	SAND Clayey - Grey Brown	Oven Dried < 50 Deg	Dry Sieved
3						
4						

Grading of Samples

Aperture Size mm	75.0	53.0	37.5	26.5	19.0	13.2	9.5	6.7	4.75	2.36	1.18	0.600	0.425	0.300	0.150	0.075
% passing Sample No.																
% passing Sample No.																
% passing Sample No.																
% passing Sample No.																

Plastic Index of Samples

Test	Lab Sample No	Liquid Limit	Plastic Limit	Linear Shrinkage	Shrinkage Behaviour	Plasticity Index
1	50550	29 %	13 %	7 %	Cracking	16 %
2	50551	32 %	14 %	7.5 %	Slight Curling	18 %
3						
4						

NATA
Accredited for compliance with ISO/IEC 17025
Accreditation No. 934

Irrelevant / Sensitive

Approved Signatory
D.Vuarchoz (Dip Lab Tech

Notes: Testing Carried Out On Samples As Supplied

REPORT OF TEST RESULTS ON SOILS - SIEVE ANALYSIS & PLASTIC LIMITS AS PER AS1289 .1.1, 2.1.1,3.1.2,3.2.1,3.3.1,3.4.1. A.S.JAMES FORM No: LR007A (Fig 1) / REV 11 / 12/12/11	Tested By : D.Vuarchoz Reported By : D.Vuarchoz	Figure 19 1 of 1

A.S. JAMES PTY. LTD.
Geotechnical Engineers
Clayton Laboratory
16 Libbett Av Clayton South

JOB:
601 Point Nepean Rd
Mccrae

JOB No. 116269
REPORT No. L002
DATE. 27-Nov-14

FOR Vast Architects
Level 1, 180 Upper Heidelberg Road,
IVANHOE, VIC. 3079

Lab Sample No.	Soil Description	Sample Location	Depth of Sample (m)	Insitu Moisture Content (%)
50559	SAND, Clayey - Grey Orange Brown	Bore Hole 1	9.00	17.4
50560	SAND, Clayey - Grey	Bore Hole 1	10.50	14.4
50561	SAND, Clayey - Dark Grey	Bore Hole 1	12.00	13.1
50562	SAND, Clayey - Orange Brown	Bore Hole 2	7.00	15.0
50563	SAND, Clayey - Grey Orange Brown	Bore Hole 2	8.00	11.6
50564	SAND, Clayey - Grey Orange Brown	Bore Hole 2	9.00	12.2
50565	SAND, Clayey - Grey Orange Brown	Bore Hole 2	10.00	12.3
50566	SAND, Clayey - Grey Orange Brown	Bore Hole 2	11.00	11.6

NOTES: Testing Carried Out On Samples As Supplied

NATA
Accredited for compliance with ISO/IEC 17025
Accreditation No. 934

Irrelevant / Sensitive

APPROVED SIGNATORY D.Vu 27-Nov-14

Soil moisture content tests - Determination of the moisture content of a soil -
AS PER AS1289 .1.1,2.1.1,
A.S.JAMES FORM No: LR014 (Fig 1) / REV 9 / 12/12/11

TESTED : B.Wynne
REPORTED : D.Vuarchoz

FIGURE 20
1 of 1

MSC.5067.0001.0184

A.S. JAMES PTY.LTD Geotechnical Engineers Clayton Laboratory 16 Libbett Av Clayton South	JOB: 601 Point Nepean Rd Mccrae	JOB No. 116269 REPORT No: L002-1 DATE: 4-Dec-14

FOR Vast Architects
Level 1, 180 Upper Heidelberg Road,
IVANHOE, VIC. 3079

Test	Lab Sample No.	Location	Depth (m)	Sample Description	Condition	Preparation
1	50567	Bore Hole 1	7.5	SAND, Silty, Clayey - GREY, Orange Brown	Oven Dried < 50 Deg	Dry Sieved
2	50568	Bore Hole 2	8.5	SAND, Silty, Clayey - GREY, Orange Brown	Oven Dried < 50 Deg	Dry Sieved
3	50569	Bore Hole 2	11.5 & 12.0	SAND, Silty, Clayey - GREY, Orange Brown	Oven Dried < 50 Deg	Dry Sieved
4						

Grading of Samples

Aperture Size mm	75.0	53.0	37.5	26.5	19.0	13.2	9.5	6.7	4.75	2.36	1.18	0.600	0.425	0.300	0.150	0.075
% passing Sample No.																
% passing Sample No.																
% passing Sample No.																
% passing Sample No.																

Plastic Index of Samples

Test	Lab Sample No	Liquid Limit	Plastic Limit	Linear Shrinkage	Shrinkage Behaviour	Plasticity Index
1	50567	35 %	15 %	9 %	Cracking	20 %
2	50568	28 %	12 %	7.5 %	Cracking	16 %
3	50569	24 %	14 %	5 %	Cracking	10 %
4						

NATA
Accredited for compliance with ISO/IEC 17025
Accreditation No. 934

Irrelevant / Sensitive

Approved Signatory
B.Wynne (Dip Lab Tech)

Notes: Testing Carried Out On Samples As Supplied

REPORT OF TEST RESULTS ON SOILS - SIEVE ANALYSIS & PLASTIC LIMITS AS PER AS1289 .1.1, 2.1.1,3.1.2,3.2.1,3.3.1,3.4.1, A.S.JAMES FORM No: LR007A (Fig 1) / REV 11 / 12/12/11	Tested By : D.Vuarchoz Reported By : B.Wynne	Figure 21 1 of 1