Expert Opinion Report - Rectification

10-12 View Point Road, McCrae

PSM5226-005R   11 June 2024

PRIVILEGED AND CONFIDENTIAL

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1. Introduction

1. This report provides my opinion on rectification of a landslide (**the Landslide**) at 10-12 View Point Road, McCrae (**the Site**). The Landslide affects the following properties:
   (a) 10-12 View Point Road (referred to herein as property "**P1**").
   (b) 2 Penny Lane (referred to herein as property "**P2**").
   (c) 3/613 Pt Nepean Road (referred to herein as property "**P3**").
2. A plan showing the properties referenced in my report is presented in Inset 1.
3. I have been requested to prepare this report by Ms Tanya Cimino of Harwood Andrews (**HA**), who act for Mornington Peninsula Shire Council (MPSC).
4. My brief and supporting documents were provided by HA on 9 November 2023. Appendix A presents the letters and document index of the brief (**the Brief**).
5. This report has been prepared by Mr Dane Pope, resume attached in Appendix B. I have 17 years of experience in the Civil and Mining industries with the following experience I consider relevant to this project:
   (a) Bogong Village temporary access cut in deeply weathered granite.
   (b) Deviation Road landslide risk assessment of a significant escarpment with an extensive history of landslide events.
   (c) Cliff Road landslide risk assessments, Frankston.
   (d) Great Ocean Road and inland routes slope remediation projects.
6. In preparing this report I have been provided with a copy of the Expert Witness Code of Conduct (refer to the Brief) and the VCAT Practice Note (PNVCAT2). I have read both the Expert Witness Code of Conduct and the VCAT Practice Note and agree to be bound by them. I have made all the enquiries that I believe are desirable and appropriate, and that no matters of significance which I regard as relevant have, to my knowledge, been withheld from the Court/Tribunal.

Inset 1: Plan of properties affected by the Landslide (Aerial Image from Nearmap dated 25 August 2023)

2. Work Undertaken

7. I have undertaken the following work in providing my opinions on this matter:
   (a) I reviewed the Brief.
   (b) I assembled my understanding of facts as they relate to the opinions I provide. I prepared this information with the assistance of the following staff under my direct supervision:
      i. Mr Andrew Wilson (Associate Geotechnical Engineer) who assisted with:
         (A) Completing a Site visit to characterise the Landslide and map local slope exposure.
         (B) Reviewing documents.
         (C) Compilation of facts.
         (D) Undertaking stability modelling of my preferred rectification option.
   (c) I reviewed all work undertaken under my direction, and notwithstanding the assistance provided by my colleague under my instruction, the opinions in this report are my own and ones that I believe to be true and correct.
   (d) I considered the questions I have been asked to address in the Brief in the light of my experience and understanding of engineering principles.
   (e) I prepared this report presenting my opinions.

3. Parties

8. My understanding of the relevant parties is described below.
   (a) MPSC who with regards to the Building Act act as the Municipal Building Surveyor, and with regards to the landslide risk assessments act as the Regulator. The following consultants have provided advice to MPSC:
      i. Stantec Australia (**Stantec**)
   (b) Mr Gerry and Bronwyn Borghesi (**Borghesi**) who are the owners of property P1. The following consultants have provided advice to Borghesi:
      i. CivilTest Pty Ltd (**CivilTest**).
      ii. Rexicon Consulting Engineers (**Rexicon**).
   (c) Paul and Denise Willigenburg (**Willigenburg**) who are the owners of property P3. The following consultants have provided advice to Willigenburg:
      i. A.S. James Pty Ltd (**AS James**).

4. Document Review

4.1 CivilTest Documents

9. I have reviewed a series of CivilTest Pty Ltd (**CivilTest**) documents. Document 2 of the Brief is the CivilTest report 1222044-1 Issue 2 (5 December 2022). The report provides:
   (a) A cross section from a drone and feature survey, Inset 2. I note that:
      i. In my opinion the section cut from the drone survey point has too much width to understand the shape of the slope (i.e. the section includes trees and landscaping).
      ii. In my opinion it is likely that vegetation is obscuring the true ground surface, and therefore the presented shape of the slope does not accurately reflect the actual site conditions.
      iii. There is a significant difference between the feature survey (the blue line of Inset 2) and the drone survey.

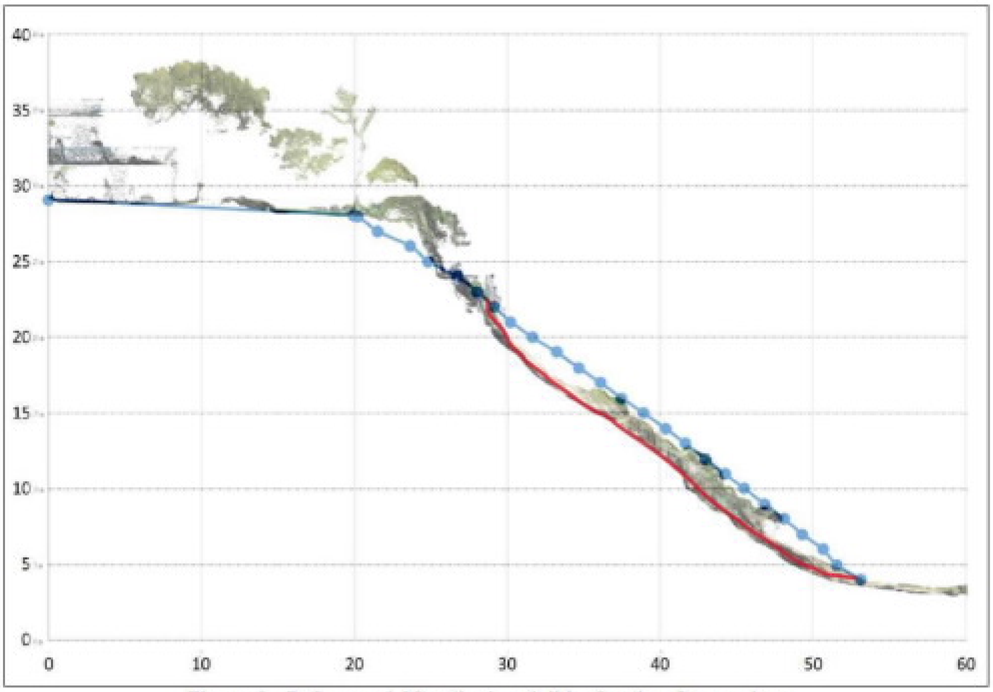


Figure 6 - Before and After the Landslide, Section Comparison

Inset 2: Excerpt from CivilTest report 1222044-1 Issue 2 (5 December 2022) (Document 2, pdf page 4)

10. Documents 9 of the Brief is the CivilTest Land Stability Assessment report 1222044-3 (24 March 2023). The report indicates to me:
    - *(a)* Boreholes drilled at the toe of the slope in Penny Lane encountered landslide debris, Inset 3 (Section 2.1, pdf page 3).
    - *(b)* Boreholes 1 and 2 encountered landslide debris 1.2 m and 0.7 m thick respectively (Appendix C, pdf pages 29 to 30). I have assumed that all fill reported at the toe of the slope is landslide debris.
    - *(c)* Geotechnical laboratory testing completed on borehole 1 (Appendix D, pdf page 33 to 36) indicates to me that all four samples (with depths of 3m, 10m, 15m and 19m) are a Sandy CLAY of low plasticity with between 36 to 48% fines and fine to coarse sand (typically medium grained).
    - *(d)* The boreholes were drilled on 1 March 2023.
    - *(e)* Wet soils were reported in:
        - i. Borehole 1 at 2.6 m below ground level (**bgl**).
        - ii. Borehole 2 at 2.8 m bgl.
        - iii. Borehole 3 between 1.8 m and 5.2 m bgl.
11. I have relied on the accuracy of the borehole log reports and the laboratory testing reports except for the assigned geotechnical units and the laboratory description of the soils. Details of my adopted geotechnical units are included in Section 6.4.

2.1 Soil Profile

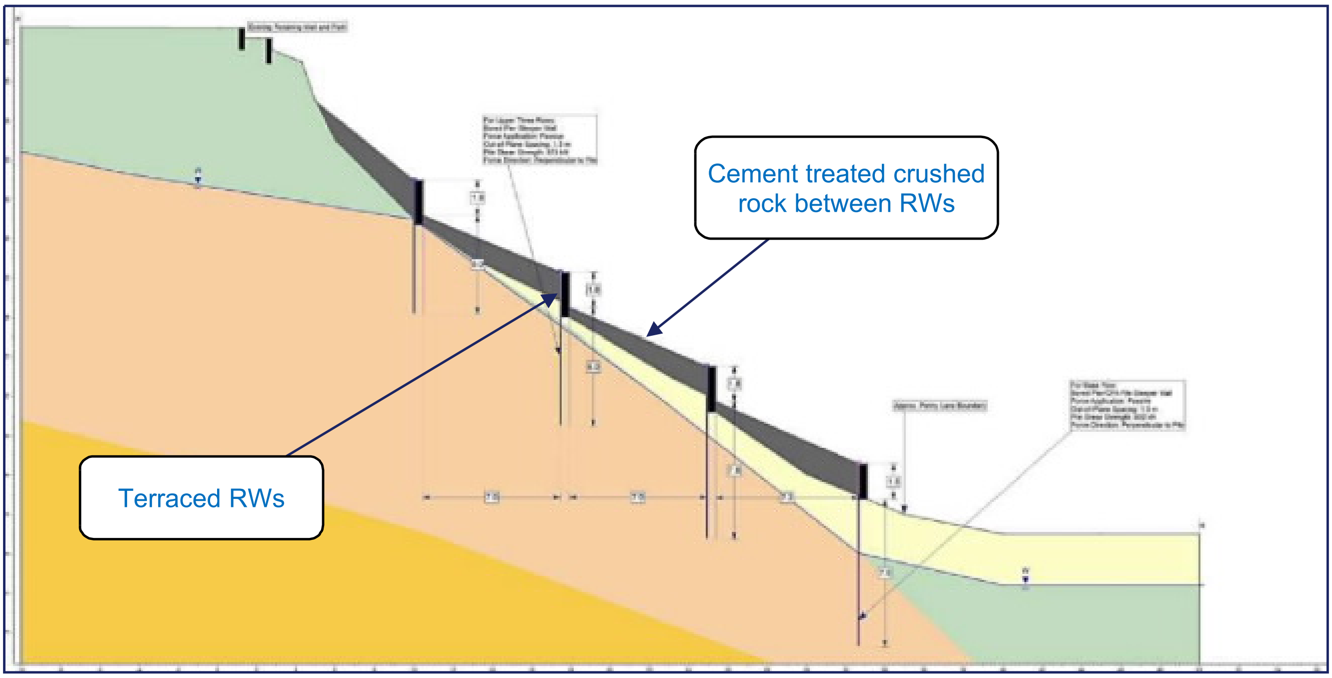
Three boreholes (BH) were drilled by a mechanical auger at the approximate locations shown on the attached plan. The two boreholes drilled at the toe of the slope on Penny Lane revealed that the soil profile consists of residual material from the landslip made up of silty SAND FILL and sandy CLAY FILL, overlying Colluvial material consisting of natural sandy CLAY, SAND and gravelly SAND. This is further underlain by Aeolian silty SAND.

The borehole drilled at the top of the slope revealed that the soil prpfile consists of silty SAND FILL, overlying natural Aeolian SAND followed by sandy CLAY and silty CLAY with sand.

Groundwater was encountered in the boreholes at depths of 2.6 metres in borehole 1 and 2.8 metres in borehole 2.

Inset 3: CivilTest description of debris flow as "fill" (Document 9, pdf page 3)

12. Document 19 of the Brief is the CivilTest report 1222044-3 Issue 5 (2 August 2023). Section 2.1 (pdf page 3) indicates the following geotechnical models were adopted in CivilTest stability modelling:
    (a) A sub-surface model at the crest of the escarpment at the Site comprising:
        i. A thin layer of sand FILL, overlying,
        ii. Aeolian Sands (slightly indurated), overlying
        iii. Residual granite (sandy/silty CLAY).
    (b) A sub-surface model at the toe of escarpment at the Site comprising:
        i. Debris from the landslide (logged by CivilTest as fill), overlying,
        ii. Colluvium (Sandy CLAY, SAND and Gravelly SAND) overlying
        iii. Aeolian Silty SAND.
13. Document 12 of the Brief is the CivilTest report 1222044-3 Issue 4 (6 June 2023).
    (a) Section 6.4 (pdf page 11) describes the proposed *"Slope Stabilisation System"* to remediate the Landslide and includes:
        i. Four rows of retaining walls (**RWs**).
        ii. Cement treated crushed rock (**CTCR**) placed between retaining walls with a maximum 22 degree slope angle.
        iii. "Loose colluvium/landslip material should be removed from the slope".
    (b) Appendix E (pdf page 29) illustrates the proposed remedial works in section view, Inset 4.



Inset 4: CivilTest section with proposed RW remediation (Document 12, pdf page 29). I have added my commentary in blue.

4.2 Stantec Documents

14. Document 4 of the Brief is the Stantec Geotechnical Assessment ((V220600Report01.1, 7/12/2022) referred to herein as the "Stantec GA"). The Stantec GA indicates:
    (a) Two landslide mechanisms were observed:
        i. Translational slide of the upper slope (Section 4.1, pdf page 3) in *"the upper soils overlying the underlying completely weathered granite"*.
        ii. Debris flow of the lower slope (Section 4.2, pdf page 7) initiated:
            (A) "by a significant increase in ground moisture".
            (B) "within the accumulation zone of the upper landslide". I note that the upper landslide refers to the Translational slide mechanism.

(b) The thickness of the Landslide was possibly less than 0.5m (pdf page 4)

(c) Seepage was observed in the head scarp, Inset 3.

15. Figure 4-7 of Document 4 (pdf page 9) includes a photograph of the debris flow zone. I note that there is no scale in the photograph however the photograph indicates to me that the depth/thickness of material evacuated from the debris flow zone is inferred to be on average less than 0.5 m.

> Figure 4-2 shows the main scarp of the landslide where it has undermined the existing stairs. It can be seen that the failure surface of the landslide runs parallel to the ground surface. The weathered granite is exposed in the failure surface. Looking at the side flank it can be seen that the thickness of soil that would have overlain the weathered granite is relatively shallow, possibly less than 0.5m.
>
> Water was observed to be seeping from the head scarp at several locations more than 24 hours after the storm occurred. These seeps appear to be associated with natural springs further up slope.

Inset 5: Excerpt from Section 4.1 of the Stantec GA (pdf page 22 of the Brief)

4.3 Rexicon Design Documents

16. Document 13 of the Brief is Rexicon Structural Drawings for a proposed retaining wall stability treatment (ref. 23031-S00 to 23031-S06, dated 16/06/2023) referred to herein as the Rexicon Drawings. I understand that these drawings are based on the *"Slope Stabilisation System"* proposed by Civil Test (refer paragraph [14]) on behalf of Borghesi.

17. The Rexicon Drawings show:

(a) The landslide area is to be stabilised by the construction of four terraced RWs in P1. The RWs are located between the Penny Lane site boundary and the existing stairs at the head of the landslide (Drawing No. 23031-S04, pdf page 5).

(b) The RWs are to be steel post and reinforced concrete panel soldier pile with the following typical details (Drawing No. 23031-S04 to 23031-S06, pdf page 7):

i. Lengths between 7.5 m and 12.0 m.

ii. A maximum retained height of 1800mm.

iii. 250 UB37.3 Structural steel posts at 1500mm centre to centre spacing

iv. Bored pile footings with 450 to 600 mm diameters and depths of between 6000 mm and 7500 mm

v. CTCR is to be placed between RWs.

vi. A subsurface drainage system comprising "*Polythene* (sic) *membrane*" and *"agi drains"* at the rear face of the retaining wall. The "*agi-drain*" for each wall connects to a longitudinal "*agi-drain*" that discharges into Penny Lane at the toe of the slope.

(c) General notes ((Drawing No. 23031-S01 to 23031-S02, pdf page 2-3) that include:

i. Temporary works to "*retain earth banks, roads, pavement, walls and footings"* is the responsibility of the Contractor (Bulk Excavation Note 4, Drawing No. 23031-S01, pdf page 2)

ii. Reference to the Civil Test Report No. 1222044-3 Issue 3 as the Geotechnical Report and that:

(A) "The Contractor is to allow for the engagement of a geotechnical engineer to verify the found material prior to placement of concrete"

(B) "The contractor is to implement all the recommendations contained in the Geotechnical Report"

18. I note that the Rexicon Drawings do not show elevations or levels of the RWs or how the RWs are sited relative to the landform, i.e. it is not clear to me if they are proposed to be cut into or built above the existing slope.

4.4 Chronology

19. Document 22 of the Brief is a chronology of events to date. The Chronology includes some of the MPSC remediation design requirements including the 9 May 2023 email that stated:

(a) *"…Council's position [is] that a 'Factor of Safety (FoS) of at least 1.5' is council's standard practice as the minimum value required to protect against long term instability of batters / slopes, and Council requires that standard to be met by Borghesi"*.

4.5 A.S. James Advice

20. Document 24 of the Brief is AS James Preliminary Comments (ref. Report No. 122573, dated 13 October 2023).

21. The document includes the following AS James advice:
    - *(a)* *"Rectification works proposed for 10-12 View Point Rd … are not necessary to render the residences on Point Nepean Road habitable"*
    - *(b)* *"There are available more cost-effective measures … to reduce the risk to property and life on [the Point Nepean Rd] residences to acceptable levels"*
    - *(c)* *"… whatever is done to rectify the property at 10-12 View Point Rd… will do little to prevent the possibility of a similar slide occurring adjacent to [the existing landslide] that may also potentially affect those properties on Point Nepean Road"*
    - *(d)* A debris flow barrier (also known as a debris fence) installed at the base of the escarpment is an appropriate mitigation to *"… protect Penny Lane and the rear of properties on Point [Nepean] Road"*.
    - *(e)* The debris fence could be designed and supplied by Geobrugg.

4.6 Published Information

4.6.1 Coastal LiDAR

22. I have relied on the accuracy of the Coastal LiDAR elevation data captured by the Department of Sustainability and Environments between April 2007 and October 2008 and published on www.data.vic.gov.au as *VicMap Elevation Coastal 1m DEM and 0.5m Contours.* This data provides a 0.5 m contour in coastal areas.

23. In my experience (refer to Wye River Landslide Assessments in my CV, Appendix B) the Coastal LiDAR is an appropriate survey tool to use for the purposes of preliminary remediation design and landslide assessments.

4.6.2 Rainfall data

24. The Bureau of Meteorology (**BOM**) Rosebud weather station climate data (Station ID: 086213, Climate Data Online - Map search (bom.gov.au), accessed 31 October 2023) indicates to me:
    - *(a)* On 14 November 2022 approximately 80mm of rainfall was recorded and reported to 9am over the preceding 24 hour period.
    - *(b)* The 30-day cumulative rainfall on the 14 November 2022 was 133 mm.
    - *(c)* The 30-day cumulative rainfall on the 1 March 2023, when the CivilTest boreholes were drilled was 47 mm.
    - *(d)* The 30-day cumulative rainfall on 23 October 2023, when the PSM site visit was undertaken was 12.5 mm.
    - *(e)* The dataset commenced in 1927 (albeit is missing significant data) and there are at least 19 events where the 30-day cumulative rainfall has exceeded 150 mm.

5. Site Visit

25. A Site visit was completed by Mr Andrew Wilson on 23 October 2023. Selected photographs are included in Appendix C. The Site Visit was conducted during dry weather and with no rainfall reported by the BOM Rosebud Country Club weather station in the 7 days prior to the Site Visit.

26. The Landslide had the following characteristics:
    - *(a)* It initiated in the upper to middle portion of the slope, with the rear scarp approximately at the base of the Stairs, Photo 1 Appendix C.
    - *(b)* It was inferred to have initiated as a translational slide followed by mobilisation of failed material into a debris flow which was deposited at the toe of the slope, Photo 2 Appendix C.
    - *(c)* The Landslide had three distinct zones being:
        - i. A steep "**Upper Zone**" where the initial translational sliding occurred with approximate dimensions of 8 to 10 m wide x 8 to 10 m long x 0.3 m thick, Photo 3 Appendix C.

ii. A steep "**Middle Zone**" approximately 15 m long by 3 m wide through which the debris flow travelled, with some scour and erosion, Photo 3 Appendix C.

iii. A flatter "**Lower Zone**" of debris runout where the debris flow deposited at the toe of the slope, Photo 3 Appendix C. The approximate dimensions of deposited debris are 8 to 10 m wide, 9 to 10 m long, and 0.2 to 0.7 m thick.

(d) Disturbed ground that had undergone translational sliding but did not mobilise into a debris flow was observed on the right (western) flank of the Landslide, Photo 4 Appendix C.

(e) The total area of instability was not able to be mapped in detail due to poor access and vegetation. There is still uncertainty as to the width of the unstable ground. It is possible that there is additional unstable ground to left (east) of the observed Landslide.

(f) A lack of prominent backscarp, Photo 5 Appendix C, with minor steepening observed in the backscarp area and with a slope angle of approximately 45 degrees.

27. The following soils were observed and logged in the Landslide area.

(a) Residual Granite on the failure surface in the Upper Zone. This material was logged as Sandy CLAY, low to medium plasticity, pale grey brown to mottled orange grey brown, fine to coarse grained granitic sands, dry to moist, very stiff to hard. I note it is possible this material is cemented Surficial Sands, Table 1, as in my opinion this unit is derived from eroded Residual Granite.

(b) Surficial sands were found to cover the escarpment slope. This material was logged as Silty SAND to Sandy SILT, fine to medium grained SAND/low plasticity SILT, brown to pale grey brown, dry, weakly cemented.

(c) Possible older (pre 2022 Landslide) Colluvium was observed in the lower slopes. It was logged as Silty/Clayey SAND, fine to coarse grained granitic sand, brown, trace 10-100 mm granitic gravel/cobbles, dry to moist, loose to medium dense.

(d) Newer Colluvium was observed in the debris flow deposits and logged as Silty SAND, fine to medium grained, pale brown, dry, loose.

28. I note that the Landslide characteristics observed during the Site visit were in general agreement with those described in the Stantec GA (refer Section 4.2).

29. The Site had the following characteristics:

(a) Located on a prominent escarpment. The escarpment is approximately 25 m high, with an overall slope angle of 35°. The escarpment has a concave profile, with slope angles of approximately 30° in the lower slope and 40° in the upper slope. The ground above and below the escarpment has flat to gentle slopes with typical slope angles of 0 to 5°.

(b) No evidence of current or historic large-scale landslide features that affect the full height of the escarpment, e.g., stepped ground, hummocky ground, landslide scarps, etc.

(c) Groundwater was observed to be seeping from the slope to the east of the stairs, Photo 6 Appendix C

(d) A variety of water infrastructure was observed across the site, Photo 7 Appendix C including:

i. Subsurface 'agi-drains'

ii. Water pipes including taps.

(e) A series of paths had been constructed across the slope to provide access from the top of the escarpment to the bottom of the escarpment, Photo 8 & 9 Appendix C. The paths are constructed from varying materials. Other infrastructure associated with the paths include minor RWs, board walks and stairs.

(f) The slope above the Landslide area was consistent with adjacent slopes outside of the Landslide area, with an approximate slope angle of 40°. I note that a combination of minor RWs and vegetation have been constructed/planted in this area.

(g) The condition of RWs across the Site was generally poor, with overturning and bulging RWs observed. A section of RW to the east of the Landslide had significant tilts. This indicates to me possible instability in the ground above the RW and possible structural or geotechnical failure of the RW, Photo 10 Appendix C.

(h) Numerous fallen trees were observed across the escarpment slope, Photo 11 Appendix C. I note that most of the failed trees appear to have failed from causes unrelated to the Landslide, i.e., wind or poor root embedment. I note at least one tree appears to have fallen because of the Landslide.

30. Additional observations were made in the broader Site area to understand larger scale slope processes. These observations include:

(a) Anthony's Nose, approximately 600 m to the northeast of the site, is a headland where the escarpment protrudes into Port Phillip Bay. It is the only coastal exposure of Dromana Granite as such it provides useful insights into ground conditions and slope performance. Key observations include:

i. Natural voids and internal erosion (i.e. piping) is common in upper soil profile, Photo 12 Appendix C.

ii. A sub vertical cliff profile in extremely weathered and highly weathered granite, Photo 13 Appendix C. I note this sub-vertical may be the result of road construction activities in the 1920's and 1930's.

iii. Granite rock is exposed in shore platform below the road.

iv. Steep to sub-vertical upper slopes are inferred to fail by undercutting and erosion of the lower slope leading to toppling style failures, Photo 12 Appendix C.

(b) A new stormwater drainage system has been constructed in View Point Rd. I note that a constant flow of water was observed to be running in this drainage system.

(c) The Site is located on the lower slopes of Arthurs Seat where those slopes meet Port Phillip Bay and have formed an escarpment, Photo 14 Appendix C. In proximity to the Site the general topography of the areas slopes to the north west. There is extensive residential development above the Site on the lower slopes of Arthurs Seat.

6. Geotechnical Model

6.1 Aerial Photography

31. I have considered readily available Nearmap images which indicate:

(a) The translational slide scarp has approximate dimensions of 8 m x 5 m, Appendix D1.

(b) The debris flow had approximately 35 m of runout from the translational slide scarp. The debris flow runout area has approximate dimensions of 10 m x 10 m. Run out extended approximately 5 m into properties P2 and P3, Appendix D1.

32. I have considered readily available historical images which indicate to me:

(a) In 1939, Appendix D2:

i. The escarpment had several exposed slopes with sparce vegetation.

ii. The property at P1 is visible, therefore its age is greater than 84 years old.

iii. There are northwest trending incised gullies along the escarpment.

(b) In 1951, Appendix D3:

i. Residential development including roads has occurred above and below the escarpment.

ii. The escarpment still has exposed slopes with sparce vegetation.

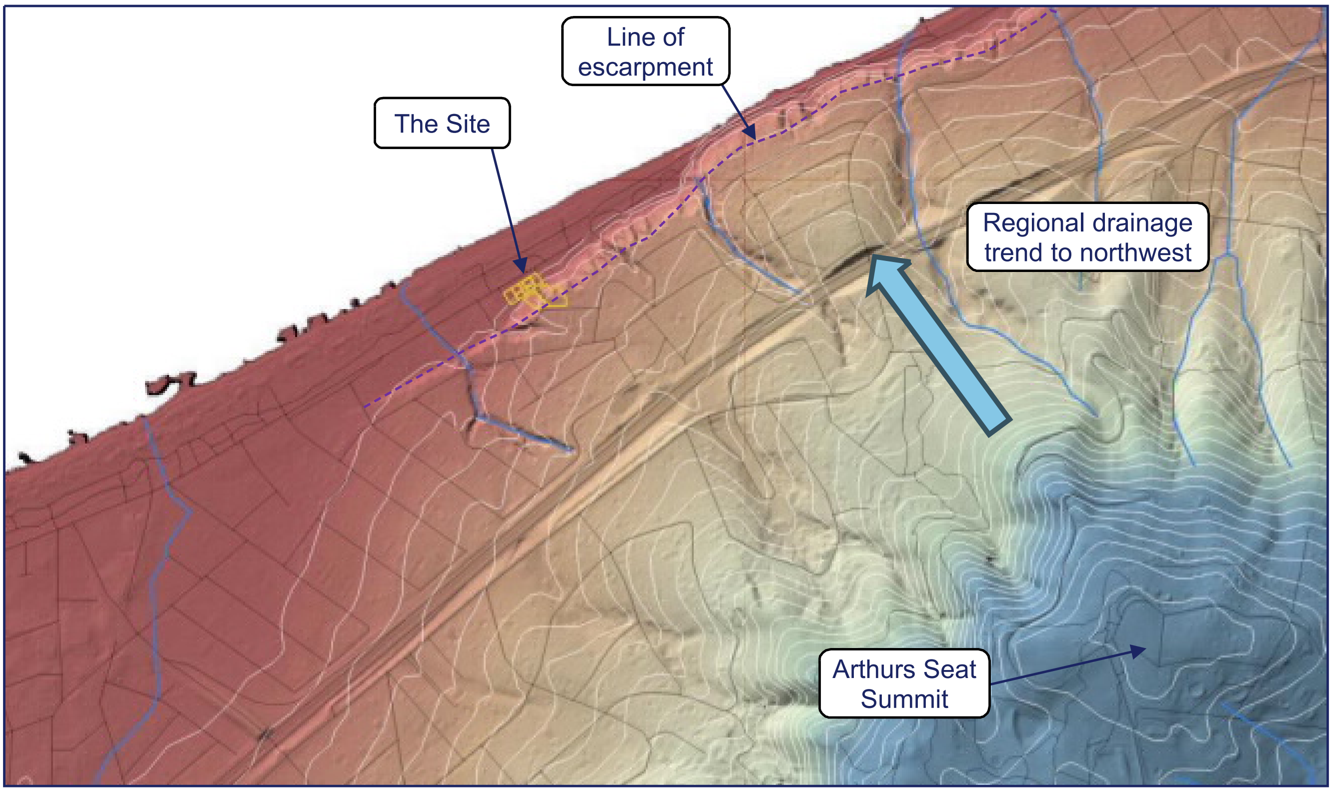
(c) In 1984, Appendix D4

i. There has been substantial residential development in the area.

ii. The exposed escarpment slopes are no longer visible as they are now covered by extensive vegetation.

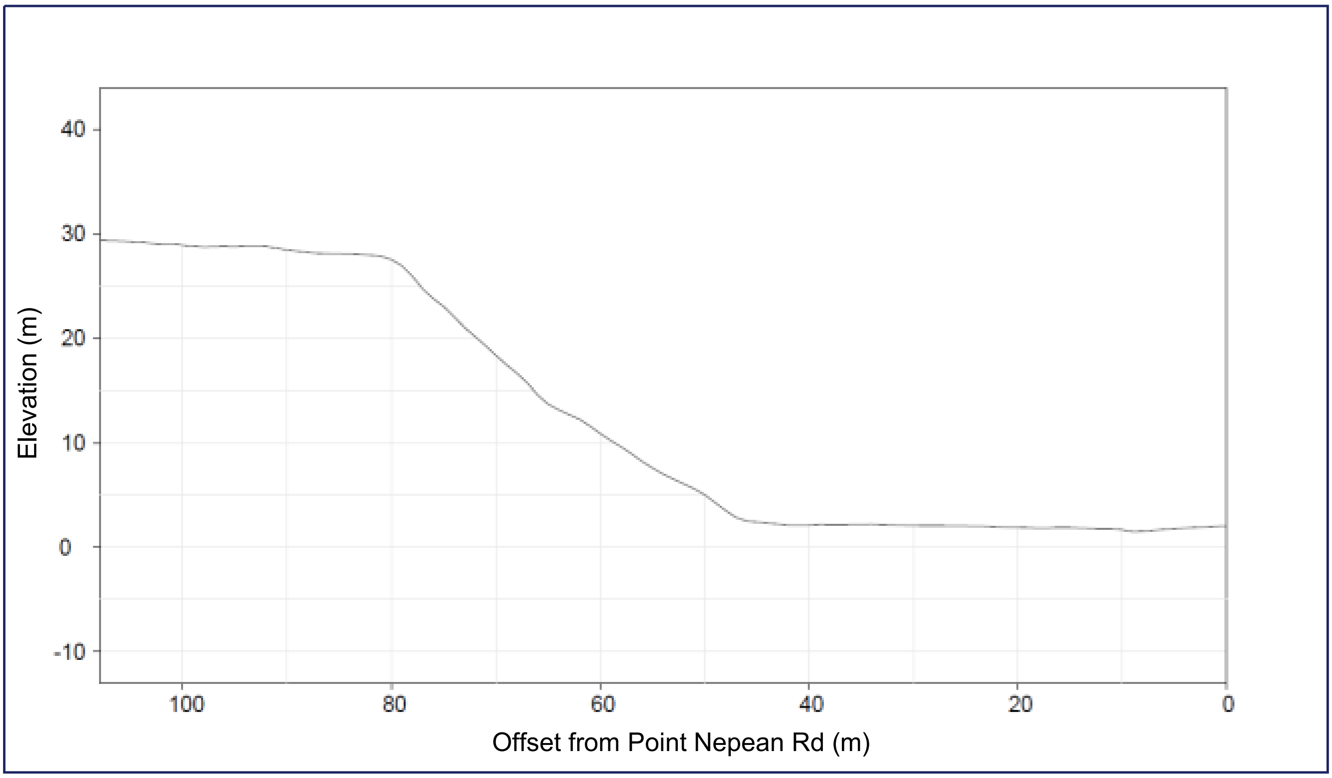
6.2 Topography and drainage

33. The Coastal LiDAR indicates that the Site is located at the lower escarpment of Arthurs Seat, Inset 6. There is approximately 270 of metres of relief measured in a northwest direction from the summit of Arthurs Seat to the escarpment at the Site. I note that several drainage paths strike in a north to northwest direction and that the Mornington Peninsula Freeway provides significant disruption to surface and sub-surface water flows in the region.



Inset 6: Topography and drainage paths of Arthurs Seat

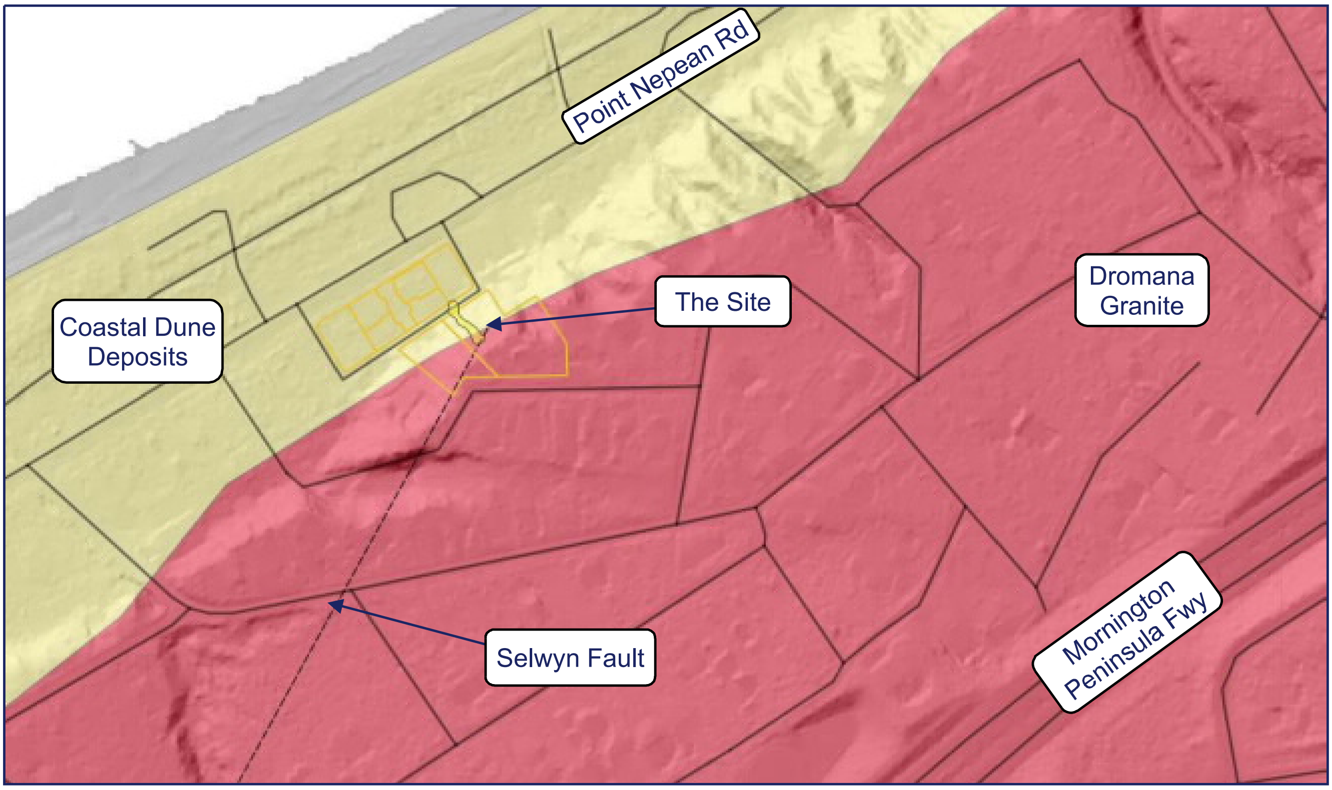
34. In the area of the Landslide the Coastal LiDAR data indicates:
    (a) 25 to 30 metres of relief between the toe of the escarpment and the crest of the escarpment.
    (b) A typical overall slope angle of 30 to 35°.
    (c) A concave slope profile with the upper half of the escarpment being steeper (typically 35 to 40°) than the lower half of the escarpment (typically 25 to 30°).
    (d) Pre-failure slope geometry of the Landslide is shown in Inset 7.
    (e) A lack of large-scale features other than gullies, that may indicate the presence of a large, full height slope failure mechanism.
35. I note that the slope trends in the Coastal LiDAR are consistent with my Site observations, Section 5.



Inset 7: Prefailure slope geometry through the centre of the Landslide from Coastal LiDAR 1 DEM

6.3 Regional geology

36. The Victoria Seamless Geology (Earth Resources publications (efirst.com.au), (2014)) model indicates that the Site is close to the boundary of Quaternary aged coastal dune deposits (with siliceous and calcareous sands) and Devonian aged Dromana granite. The Earth Resources mapping portal (GeoVic Anonymous (gsv.vic.gov.au), accessed 1 November 2023) indicates that the inferred location of the Selwyn Fault traverses the Site, Inset 8.



Inset 8: Earth resources seamless geology map of the area, with Selwyn Fault highlighted.

6.4 Sub-surface conditions

37. The conditions documented by others in the boreholes and slope exposures indicated subsurface conditions generally consistent with those described on the geological map. Table 1 presents my interpretation of the geotechnical units.

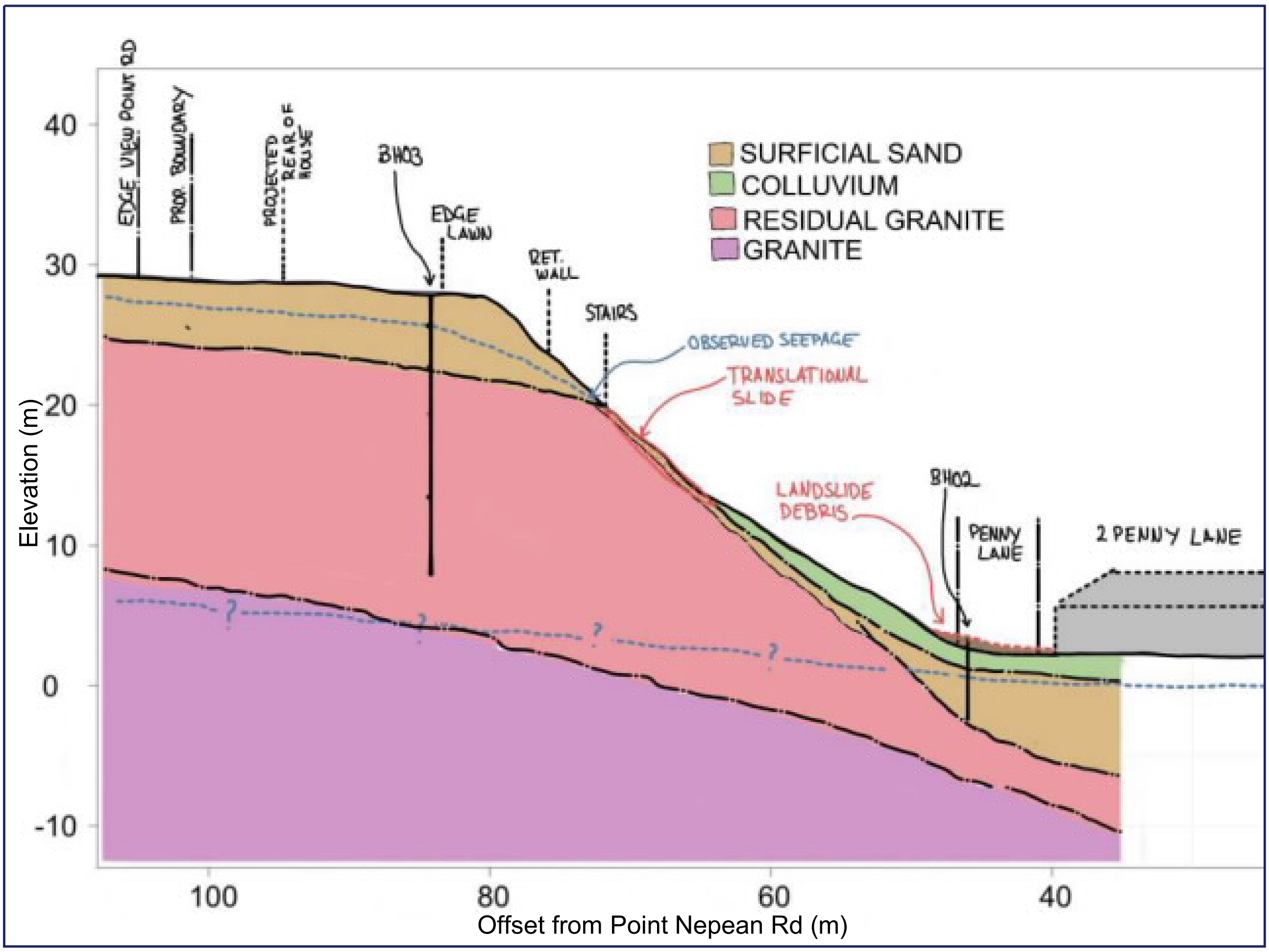
Table 1 – Geotechnical units

Unit	Description
SURFICIAL SAND (1)	SAND and Silty/Gravelly SAND, fine to coarse grained, brown to pale grey brown, moist to wet, inferred medium dense. SPT N value of 10 at 1.5 m bgl. Contact with underlying Residual Granite (3) is difficult to define.
COLLUVIUM (2)	Inferred to be a mixture of Units 1 and 3. Recent Colluvium (the debris flow from 2022 landslide) is Silty SAND/Sandy CLAY. Old Colluvium buried by 2022 landslide is Sandy CLAY and SAND. This unit has no strength testing.
RESIDUAL GRANITE (3)	Sandy to Silty CLAY/Clayey SAND, low plasticity, pale grey brown to mottled orange grey brown, fine to coarse grained, wet at contact with overlying Surficial Sands otherwise moist, typically medium dense to dense/stiff to very stiff. SPT N values vary from 12 to 34 with a mean of 25 from 3 m to 20 m bgl.

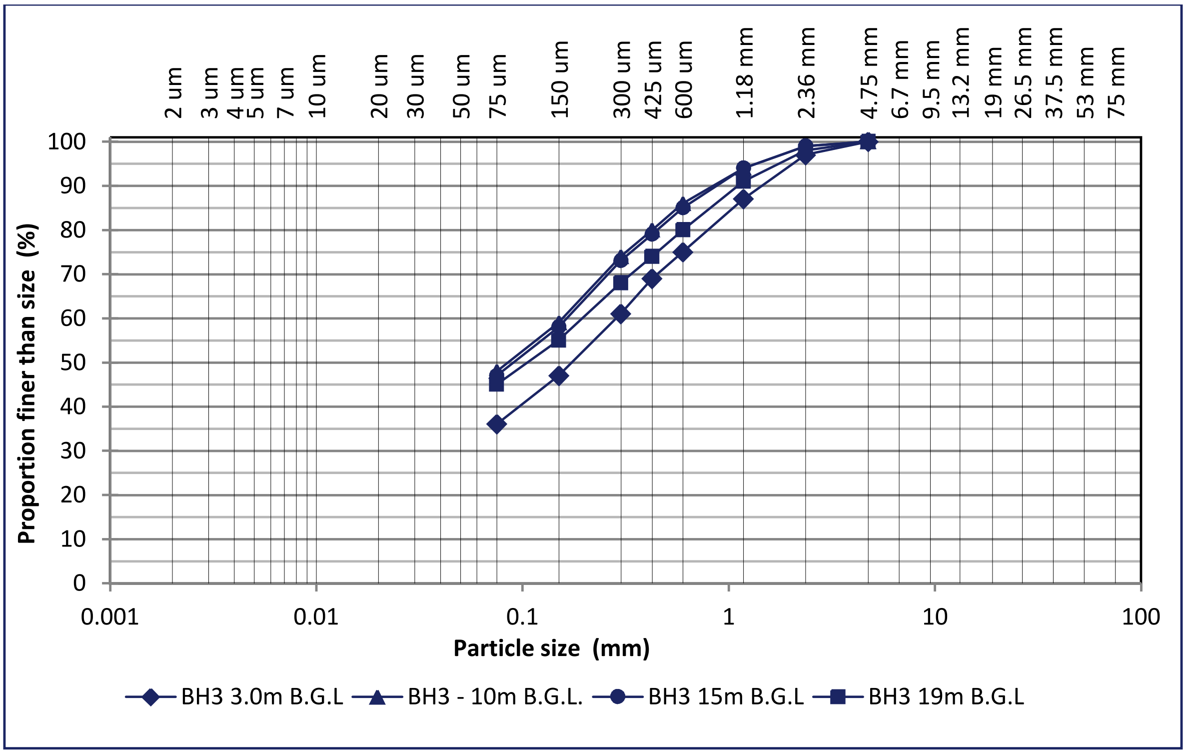
38. My interpretation of the geological conditions is presented in Inset 9. With regards to the geotechnical model, I note the following key observations:

    (a) There is uncertainty regarding the contact between the SURFICIAL SAND and the RESIDUAL GRANITE owing to the likelihood of some of the parent material of the SURFICIAL SAND being derived from erosion of the Dromana Granite. I have assumed that the wet soils are an indicator of the contact between the two geotechnical units.

    (b) The laboratory testing indicates that all samples between 3 m and 19 m bgl have very similar Particle Size Distributions, Inset 10, and Atterberg limits indicate low plasticity CLAY fines.



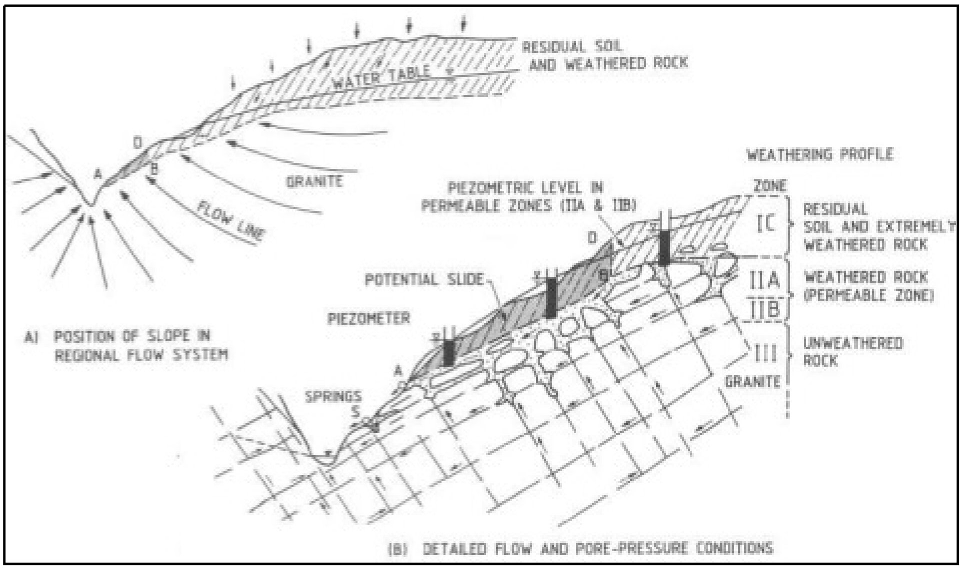
Inset 9: **Cross section of my geotechnical model**



Inset 10: **CivilTest Particle Size Distributions**

6.5 Groundwater

39. I note that no groundwater monitoring has been conducted on the Site.
40. Based on published literature and the observations on the CivilTest borehole logs, in my opinion, a perched water table is likely to exist at the contact of the SURFICIAL SAND and the underlying RESIDUAL GRANITE. This is supported by:
    - *(a)* Wet soils observed in CivilTest borehole 3 (refer to paragraph [10(e)iii]).
    - *(b)* Erosion pipes in the SURFICIAL SAND, (refer to paragraph [30(a)i], which indicate to me a pathway of past and preferential sub-surface water flow.
    - *(c)* Observations in the Stantec report (refer to paragraph [14(c)]).
    - *(d)* Observations of seepage during the Site Visit (refer to paragraph [29(c)])
41. In my opinion the presence of the perched water table will not necessarily be limited to periods of wet weather due to:
    - *(a)* The size of the catchment of Arthurs Seat and slopes and drainage paths that fall towards the northwest, the Site and the escarpment (refer to paragraph [33]).
    - *(b)* Local sources of water common to residential development and subdivisions (garden watering, street catchment run off, leaky pipes (private and public sources).
42. This is supported by:
    - *(a)* The observation by CivilTest of wet soils encountered in borehole 3 between 1.8 m and 5.2 m bgl in March 2023. I note that this was not during or following a period of high rainfall, Section 4.6.2.
    - *(b)* Furthermore, the CivilTest observation is consistent with published groundwater models in weathered granitic profiles, Inset 11.
    - *(c)* Observations of seepage during the Site visit (refer to paragraph [29(c)]).



Inset 11: Possible piezometric conditions in weathered granitic soils (Fell et al, 2004)[1].

43. Groundwater was observed on Penny Lane between 2.4 m to 2.6 m bgl (paragraph [10(e)]). I note that this is consistent with water levels of the adjacent Port Phillip Bay and that these levels are anticipated to fluctuate with tidal levels.

[1] Fell,R. MacGregor, P.Stapledon, D. Bell, G. 2005. Geotechnical Engineering of Dams. CRC Press.

7. Mechanisms of Failure

44. In my opinion the key mechanisms at the Landslide include:
    (a) Translational slide of the Surficial Sand and Colluvium unit. Key controls of translational sliding at the Site include:
        i. Uncontrolled deposition of Colluvium on the slopes either through erosion or landscaping.
        ii. Depth of Surficial Sand.
        iii. Steep slope angle, Inset 9.
        iv. Steep basal contact angle at the interface with underlying Residual unit.
        v. Saturation of Surficial Sands, and possible excess groundwater pressure from upslope seepage (Section 6.5).
        vi. Transient cohesion of the unit, with cohesion possibly being affected by loss of vegetation and loss of suction from high saturation.
    (b) Debris flow. Key controls of debris flow landslides include:
        i. Periods of high antecedent rainfall, i.e., high cumulative 30-day rainfall totals, leading to saturated ground conditions.
        ii. Intense rainfall event(s).
        iii. Concentrated source of water inflow, i.e., groundwater seepage, surface water, damaged drainage or water infrastructure, etc.
        iv. Loose or disturbed ground, i.e., existing colluvium or ground that has been disturbed from other instability mechanisms such as translational sliding.
45. I note that the key mechanisms I have identified are consistent with the Stantec GA (refer to Section 4.2).
46. I have not considered a circular failure of the full height of the escarpment because in my opinion:
    (a) This mechanism is not observed in the escarpment adjacent to the Site and is not feasible.
    The regression and failures observed at Arthur's nose is non-circular. In my opinion progressive erosion of the escarpment is controlled recently by rainfall, sub-surface water flow and stormwater and less recently by ancient higher sea levels. This process results in incised gullies and sub-vertical cliff lines. The cliff line then fails by progressive undercutting of cemented sand and weathered rock blocks in wedge and toppling failures (refer Paragraph [30(a)iv]).
    (b) Sub-vertical slopes are not present at the Site or the Landslide. In my opinion it may take 1000's of years for this type of feature to develop by the erosion processes described in Paragraph [46(a)].
    (c) The colluvium at the Site is formed from ongoing erosion, Figure D2 of Appendix D, and similar failures to that observed at the Site. i.e., relatively thin translational slide and debris flows which create the colluvium fan neither of which are a circular mechanism.
    (d) The steep head scarp depicted in CivilTest drone survey (Inset 2) and models (ref) is an artefact of creating a thick cross section line from the drone model. i.e. the head scarp feature depicted by CivilTest at the crest of the escarpment does not exist on Site (refer to Paragraph [9(a)]).
47. In my opinion a large full height slope mechanism is not active at the Site as this would be controlled by structure in the granite or relic structure in the RESIDUAL. This is supported by the lack of obvious large scale landslide features in the Coastal LiDAR (refer to Paragraph 34(e)).

8. Conceptual Rectification

8.1 Rectification Options

48. I have developed three options that in my opinion may be suitable for rectification of the Site. These options are summarised in Table 2 along with a brief opinion on each option.

Table 2 – My Opinion on remedial works

Option	Type	My Opinion on Option
1	Debris fence, earthworks, drainage, revegetation	• Earthworks to trim slope to remove all loose and disturbed ground in Landslide including flanks. • Installation of debris fence at toe of slope to contain landslides from impacting properties at toe of slope. • Horizontal sub-surface drains installed at interface of Residual Granite and Surficial Sand to control seepage at this interface. Drains to be directed to a piped stormwater drainage system at toe of slope. • Surface drains to control and redirect flows away from susceptible areas. Coir logs placed: – At the rear of the Landslide to capture and deflect water away. – Diagonally across trimmed Landslide to direct water off the slope to reduce erosion and sheet flow. • Modify/upgrade path and stairs to reduce concentration of water flow into the Landslide area. • Revegetation of the site. Revegetation to include a mixture of trees, shrubs, and grasses. Revegetation techniques to ensure the new vegetation establishes a strong and healthy root network across the site. Input from an arborist/horticulturist likely required. • Re-establishment of vegetation may take 2 to 5 years. • Will require the development of an appropriate ongoing maintenance and inspection plan. This plan must be strictly adhered to. Maintenance of vegetation, drainage infrastructure and debris fence will be required. • Landslide risk is likely to be reduced to tolerable levels for properties P2 and P3. Landslide risk is unlikely to be reduced for garden areas of P1. • Design life of 50 years can be achieved for the debris fence. • The long-term effectiveness of this option is uncertain, even where there is strict adherence to a maintenance plan. Periodic inspection required to assess effectiveness of option; further controls may be required. As such, the design life of the overall system is currently uncertain. The effectiveness of the revegetation effort is likely to control the design life.
2	Terraced RWs	• As proposed by Borghesi (refer Section 4.3 of this report) • RWs improves stability of the Site as: o Bored piles at close spacing increases shear resistance in slope. o Allow for reduced slope angle. • Lateral extent may need to be extended which may be difficult to execute and expensive. • The system, if possible to install as intended, is likely to reduce landslide risk to tolerable levels. • Constructability of the system is very difficult, due to steep slopes and limited access. Difficult access increases costs and time of project. • The diameter and depth of the bored piles requires a medium to large size excavator with a pendulum mounted auger. Smaller excavators are unlikely to be appropriate. Long reach excavators may be required. • Construction has potential to create additional hazards due to steep temporary excavations and fill slopes and placement of heavy plant, materials on the slopes. • Placement and compaction of CTCR not required. Single size gravel backfill considered more appropriate.

Option	Type	My Opinion on Option
		• Remove CTCR from design so that vegetation can be established. • Surface water drainage to intercept surface water flow and reduce sheet flow across the RW terraces. • Design life of 50 years
3	Soil nail and netting slope reinforcement system	• Soil nails are installed in the slope by drilling and grouting to improve the stability of the slope and retain the steel netting. Soil nails are typically a 20 to 40 mm bar grouted into an 80 to 150 mm hole. The length of soil nails is typically 3 to 8 metres. • A proprietary steel netting is placed and tensioned across the surface to restrain surficial sliding. • Minimises excavation required as only steep overhangs and landslide debris is required to be trimmed or removed. • Ability to efficiently treat the entire Landslide area including the flanks and rear scarp. The system can be readily extended in area. • Re-vegetation, by direct planting, is possible through the netting and should be limited to grasses and small shrubs to ensure large diameter trunks/branches do not damage the netting. • Soil nails may be installed by rope technicians using handheld drills or a lightweight drill. No large plant is required on the slope. The use of a drill mounted to the mast of a telehandler positioned at the toe of the slope may also be possible allowing increased efficiency and safety of construction. • Option is likely to inhibit access across the slope without construction of additional structures (e.g., boardwalks or stairs) • Water pipes and irrigation should be removed from the slope. • Installation of sub-surface horizontal drains at interface of Residual unit and Surficial Sands to control seepage and reduce development of excess pore pressures in the Surficial Sand unit. Drains to be connected and piped to appropriate outlet at toe of slope. • Design life of 50 years can be achieved through use of stainless steel netting and glass fibre reinforced polymer (GFRP) soil nails. • Design life of 50 years

8.2 Proposed Rectification Solution

49. My recommended rectification solution is based on my opinion of the most cost-effective and practical solution which achieves long-term stability of the slope (as defined by MPSC in Section 4.4)

50. I recommend that Option 3 is adopted. In my opinion Option 3 is preferred over Option 1 & 2 as it:

    (a) Provides a reliable and durable engineering control of landslide risk with low maintenance requirements over the design life.

    (b) Is comparatively cost effective.

    (c) Has low aesthetic impacts where revegetation is undertaken.

    (d) Avoids or minimises the creation of additional hazards and risk during construction.

51. I have not recommended Option 1. In my opinion Option 1:

    (a) Is not appropriate in a residential setting. The effectiveness of Option 1 is uncertain and reliant upon successful revegetation of the slope and active maintenance and inspection. In my opinion Option 1 is only suitable for adoption by an organisation with advanced asset management capabilities.

    (b) Is unlikely to control landslide risk to P1. Further controls to limit access and development of P1 would be required. In my opinion this is likely to be unacceptable to MPSC or Borghesi.

52. I have not recommended Option 2. In my opinion Option 2 is likely to effectively control landslide risk when applied to the full extents of the Landslide. However, I do not recommend it when compared with Option 3 as I consider that Option 2:

    (a) Is comparatively expensive. In my experience (Refer to Great Ocean Road and inland routes landslide remediation in my CV, Appendix B) soil nails and slope netting are a lower cost option compared to RWs for retention of steep slopes and stabilisation of landslides.

(b) May create additional hazards during construction from steep temporary cuts.

(c) The system is not as efficient to extend where the extent of the hazard is greater than mapped (refer to Paragraph [26(e)]).

(d) Requires the development of a detailed construction staging plan including further analysis and design of temporary works.

(e) Is likely to constitute significant betterment of the slope. That is, in my opinion the Borghesi design includes treatment of a circular mechanism which was not observed at the Site and I do not believe is a credible mechanism (Paragraph [46]) as well as the near surface mechanisms of translational slide and debris flow observed at the Site.

8.3 Preliminary Design of Proposed Rectification Solution

53. I have prepared a preliminary design of my proposed rectification solution to support feasibility and costing analyses only. Although the design is preliminary, I do not envisage a significant amount of work is required to finalise the design so that it is suited for construction.

8.3.1 Assumptions

54. I have assumed:

(a) A target Factor of Safety (**FOS**) of 1.5 is required to rectify the Site. I note that MPSC required the Rexicon RW solution to achieve a minimum Factor of Safety (FOS) of 1.5 for static loading and 1.3 for seismic loading. I assume that this target FOS applies to any rectification solution for the Site. I assume that the FOS applies to the slope treatment only for an appropriate slope mechanism as identified in Section 7. In my opinion a full height circular failure is not a valid mechanism (refer to Paragraphs [46] & 47]). In my opinion the solutions must be designed for the active mechanisms at the Site and not for betterment of larger mechanisms which may have very low probability of detachment.

(b) The stability of the Surficial Sand slopes above the existing stairs at the crest of the Landslide is excluded, as this is controlled by the performance of P1 RWs. Where the RWs are showing signs of failure, the owner of P1 should address this separately.

(c) A target design life of 50 years.

(d) The design is undertaken in accordance with the requirements of Australian Standard AS 5100 (2017) – Bridge Design. In my experience (Refer to Great Ocean Road and inland routes landslide remediation in my CV, Appendix B), AS 5100 is a commonly adopted design standard for the design of soil nail systems. I note whilst Australian Standard AS 4678 (2002) – Earth-retaining structures can be adopted for the design of soil nail retaining structures AS 4678 specifically states that it does not apply to structures with a slope less than 70° (Cl. 1.1 of AS 4678 (2002) and to structures founded in unusual ground conditions such as “land slips” (Cl.1.2.1 of AS 4678 (2002). The soil nail and netting system is not a RW as such and not all requirements of AS 5100 (2017) and readily available soil nailing specifications (e.g., VicRoads Standard Specification Section 683) are applicable.

(e) Static and dynamic loading scenarios are to be checked. A seismic earthquake loading coefficient of 0.10 is to be applied to the slope.

55. I have adopted the Coastal LiDAR 0.5 m contour to develop my design slope profile. This is because in my opinion:

(a) The CivilTest drone survey shows an over-steepened back scarp that was not observed during the Site visit.

(b) The feature survey contours show possible inaccuracies in the middle and lower slopes, with an overall convex slope profile that is inconsistent with the concave slope profile observed during the Site visit.

(c) The typical and maximum observed thickness of the Landslide was 0.3 m and 0.5 m respectively. The comparison between the CivilTest drone and feature survey, Inset 2, overestimates the thickness of the Landslide by as much as 1.5 m.

8.3.2 Engineering Parameters

56. Table 3 provides a summary of the engineering parameters I have adopted for the preliminary design. I have only provided parameters for my preferred rectification option.

57. The parameters adopted in my assessment are based on:
    (a) The in-situ and laboratory testing completed by CivilTest (refer to Section 4.1)
    (b) Observation of escarpment slope performance immediately adjacent to the Site (29(b)).
    (c) My experience with soil nail tests in colluvial and residual granitic soils (refer to "Bogong Village" projects in my CV, Appendix B).

Table 3 – Geotechnical Parameters

Unit	Unit Weight	Effective Cohesion	Effective Friction Angle	Ultimate bond stress
	γ' (kN/m^3)	c' (kPa)	ϕ' (°)	σ_{ult} (kPa)
SURFICIAL SANDS	18	0	34	-
COLLUVIUM	18	2	30	-
RESIDUAL	20	20	30	140

Table 4 – Support Parameters

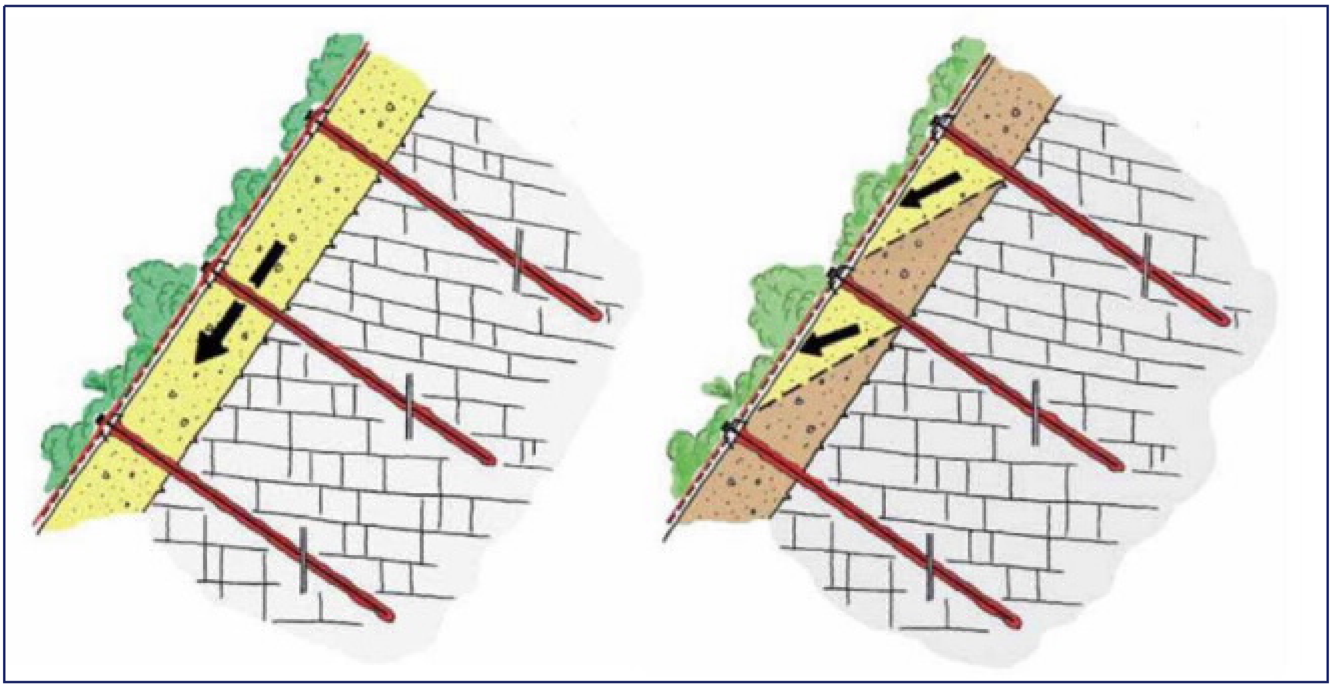
Unit	Out of Plane Spacing	Spacing Along Slope	Tensile Capacity	Plate Capacity	Shear Capacity
	(m)	(m)	(kN)	(kN)	(kN)
Soil Nail	2.5	2.5	140	100	N/A

8.3.3 Ruvolum Analysis

58. I have checked for sizing and spacing of the soil nail and steel netting system using Geobrugg's Ruvolum dimension tool (www.geobrugg.com). Ruvolum assesses superficial slope-parallel instabilities as well as local instability between individual nails, Inset 12. I have adopted a thickness of unstable material of 0.5 m parallel to slope.

59. Ruvolum is used to assess working soil nail loads, mesh load and to perform checks against component capacities. Ruvolum also provides a first pass of soil nail spacing. I note that this software package is used iteratively with SLIDE2, Section 8.3.4.

60. The analysis indicates for the spacings in Table 4, unfactored soil nail loads of 38 kN.

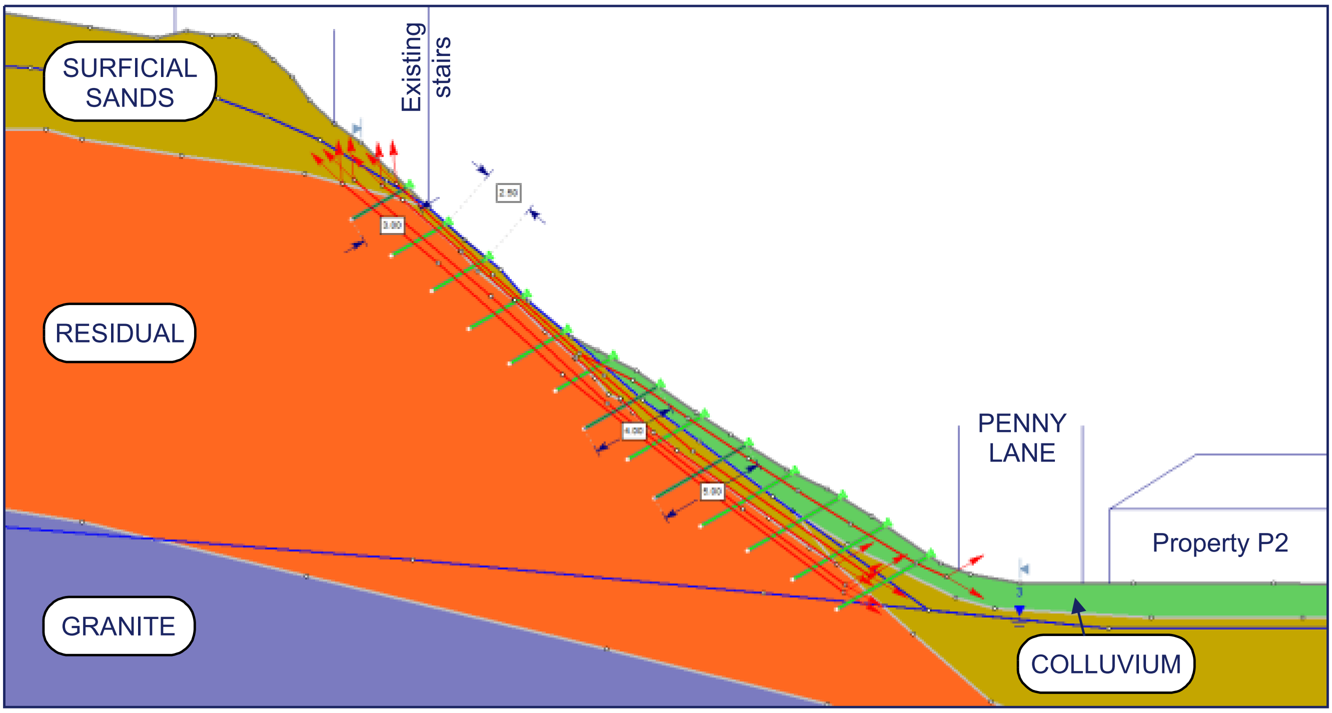


Inset 12: Diagram of mechanisms assessed by Ruvolum (Geobrugg, 2019)[2]

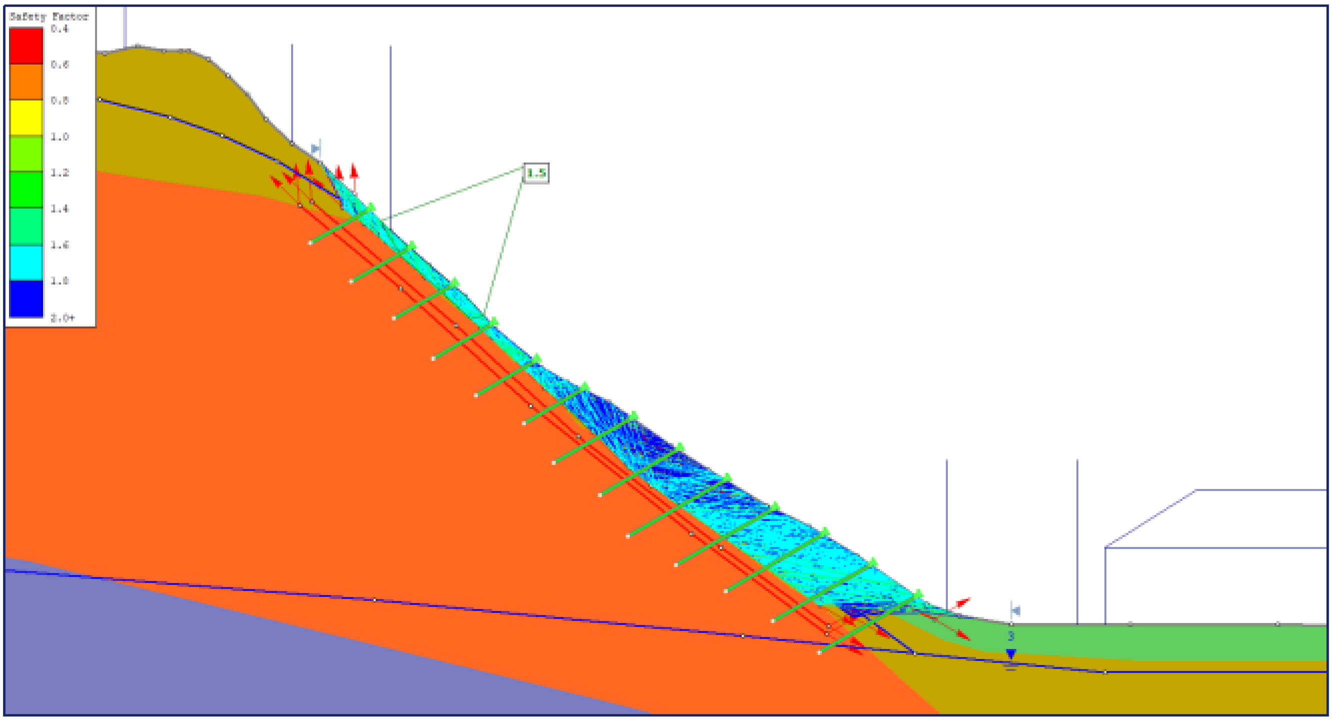
8.3.4 Slide Analysis

61. I have checked the overall stability of the system using Rocscience limit equilibrium software SLIDE2 (version 9.020). The analysis has checked for stability of the soil nail and netting system for the identified mechanisms It is not reflective of the FOS of the overall slope (refer to Section 7)..
62. The design cross section used in the analysis is based on the interpreted geotechnical model (refer to Inset 9) and is presented in Inset 13.
63. The analysis included the following:
    - *(a)* A non-linear polyline search method to model the translational sliding mechanism of failure. The method consisted of a series of polylines roughly parallel to the slope within the Landslide area. The polylines were approximately 0.3 m, 0.6 m, 1.1 m and 1.5m below surface level.
    - *(b)* Exclusion of shallow surficial failures between soil nails as these failures are contained by the slope netting which is accounted for in the Ruvolum Design (refer Section 8.3.3)
    - *(c)* A perched water table within the Surficial Sand unit to represent the observed seepage in this unit. Conservatively, in the area of the Landside below the existing stairs the Surficial Sand unit was assumed fully saturated with the piezometric line applied at the ground surface.
    - *(d)* No surface surcharge loads.
    - *(e)* A geotechnical strength reduction factor of 0.55 applied to soil nail bond strengths. An importance category reduction factor was not applied to soil bond strengths as this is accounted for in the target FOS.
    - *(f)* Shear capacity of the soil nail was conservatively not included in the analysis.
64. The output for static and seismic loading conditions is presented in Inset 14 and Inset 15.

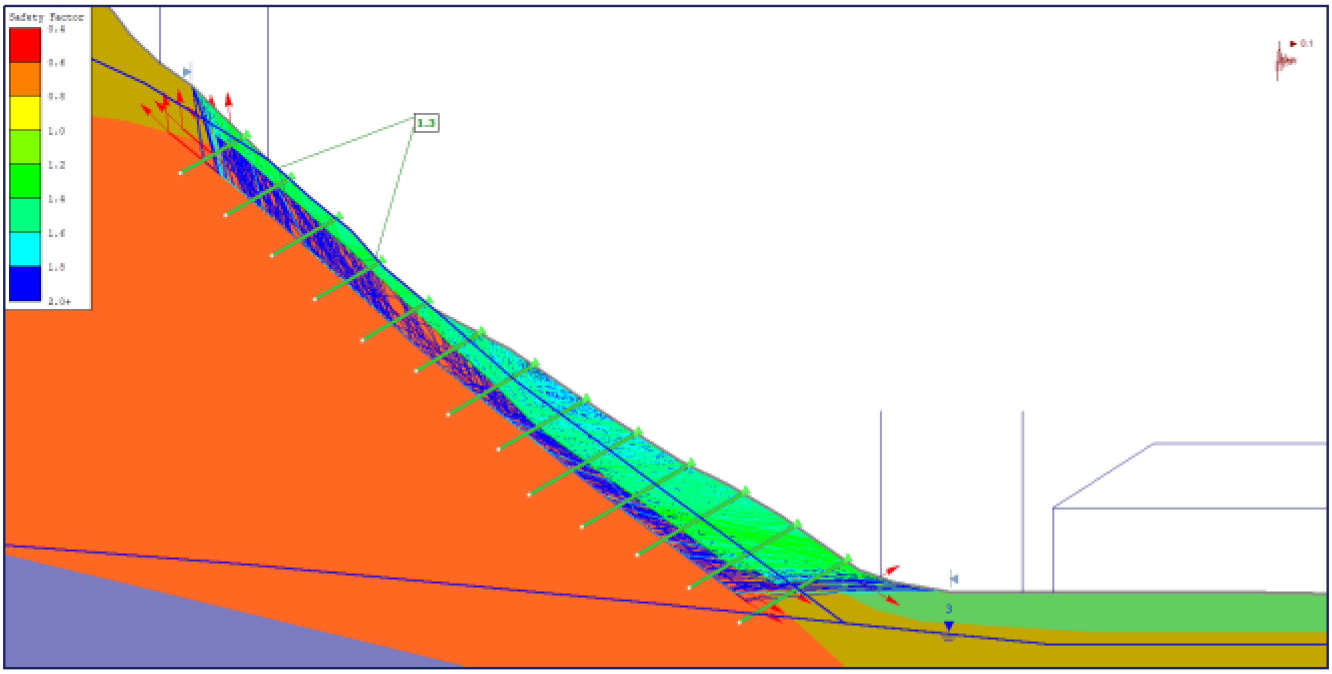
[2] Geobrugg (2019). Tecco System Publications 1998-2019



Inset 13: Slide 2 design cross section



Inset 14: Slide 2 analysis output for static loading conditions



Inset 15: Slide 2 analysis output for seismic loading conditions

8.3.5 Soil Nail Design

65. I have designed the soil nails for pullout from slope netting loads in accordance with AS 5100.3 (2017). I have adopted the design procedures in the standard used for anchorages.

66. The following reductions factors were adopted in the soil nail design:
    (a) Importance category $\emptyset_n$ = 0.70
    (b) Geotechnical Strength $\emptyset_g$ = 0.55 assuming site-specific pull-out tests will be undertaken during detailed design
    (c) Structural strength $\emptyset_n$ = 0.9.

67. The design soil nail load of 38 kN was adopted based on the critical load case from the Ruvolum analysis.

68. The soil nail design is summarised in Table 5.

Table 5 – Adopted soil nail design

Parameter	Value
Nail Length (m)	Minimum 2.5 m embedment into RESIDUAL
Bar Diameter (mm)	25
Min Bar Tensile Strength (kN)	65
Hole Diameter (mm)	115

8.3.6 Netting Design

69. Geobrugg Tecco G45/2 is to be used for the netting along with P33 spike plates that are used to connect the netting to the soil nails. Alternative suitable netting products are available in the market and could be considered during detailed design.

8.3.7 Durability

70. I have used the Australian Standard AS 4312 (2019) Atmospheric corrosivity zones in Australia for corrosion loss rates. The Site is classified as Category C3 (medium) in accordance with AS 4312, as the Site is between 50 m to 1 km from the shoreline of an inland sheltered bay.

71. I have assumed soils are non-aggressive. This should be confirmed during detailed design.

72. In my experience (refer Refer to Great Ocean Road and inland routes landslide remediation in my CV, Appendix B), it may not be possible to achieve a 50 year design life for a galvanised coated soil nail and slope netting system is installed in close proximity to the coastline. As such for the purposes of preliminary design I have adopted:
    (a) Soil nails are to be Glass Fibre Reinforced Polymer (GFRP) bars.
    (b) Slope netting is to be stainless steel.
    (c) Spike plates, and all fixings are to be stainless steel.
73. During detailed design it may be possible to develop a galvanised system that would achieve a 50-year design life. In my opinion careful consideration of the environmental conditions as well as collaboration with a manufacturer would be required to assess the design life of a galvanised coasting system.

8.3.8 Extent of Rectification

74. The approximate extent of rectification required is shown in Inset 16. The approximate extents in plan are 15 m wide by 25 m long with 370 m^2 of total area (in plan) to be rectified. I note that the angle of the slope will mean that the actual ground surface area to be rectified is approximately 480 m^2.
75. The indicated area of rectification extends beyond the observed boundaries of the Landslide to provide stability to the flanks, and to allow for possible uncertainty in the extent of unstable ground (refer to paragraph [26(e)]).

Inset 16: Indicative extent of rectification shown in pink (Aerial image from Nearmap)

9. Opinion

76. I have assumed that any rectification works are designed and constructed in a manner that reduces the risk to life and property to a tolerable level in accordance with AGS (2007c). The extents of treatment are restricted to areas where the assessed risk level has changed as a result of the Landslide. The extent of treatment is as highlighted in pink in Inset 16. I have not included a residual risk assessment.
In my experience (refer to Wye River Landslide Assessments in my CV, Appendix B) an engineer designed treatment, for the appropriate mechanism of failure, constructed in accordance with the design drawings typically reduces the probability of detachment to "unlikely" to AGS (2007c). When considering spatial and temporal probabilities as well as vulnerabilities with these systems in place, the risk to life and property is typically at least tolerable to AGS (2007c). In my experience in remediation of landslides this is typically acceptable to Regulators (council or road authorities).

9.1 Instruction 3.3.1

"The most cost-effective and practical solution which achieves long-term stability of the slope".

77. In my opinion, a soil nail and slope netting slope stability treatment is the most cost effective and practical solution for rectification of this slope. My reasoning is set out in Table 2 and Section 8.2.
78. In forming this opinion, I have assumed the following:
    (a) Long-term stability requires:
        i. A design life of at least 50 years as indicated by AGS (2007c) and the National Construction Code.
        ii. A factor of Safety of greater than 1.5 for the appropriate mechanism of failure, I have excluded full height failures of the escarpment.
        iii. A reliable reduction of risk to life and property to at least a tolerable level, with minimal ongoing inspection and maintenance requirements.
    (b) Most cost effective and practical solution requires:
        i. The lowest cost option that reliably achieves the required outcomes.
        ii. The solution can be safely constructed with readily available materials and contractors.

9.2 Instruction 3.3.2

"Where the proposed solution differs from the solution propose by Borghesi (being the retaining wall solution) why your solution is preferrable."

79. Refer to my opinion in Section 8.2. In my opinion that the solution proposed by Borghesi (refer to Section 4.3) may not achieve a satisfactory level of risk reduction as it may not extend across the full extents of the Landslide, Inset 16. However, with minor amendments, most notably an increase to the lateral extents of the RWs, in my opinion a satisfactory RW design could be achieved.
80. However, in my opinion the soil nail and slope netting slope stability system is preferrable to the RW as outlined in paragraph [52] and Table 2.

9.3 Instruction 3.3.3.1

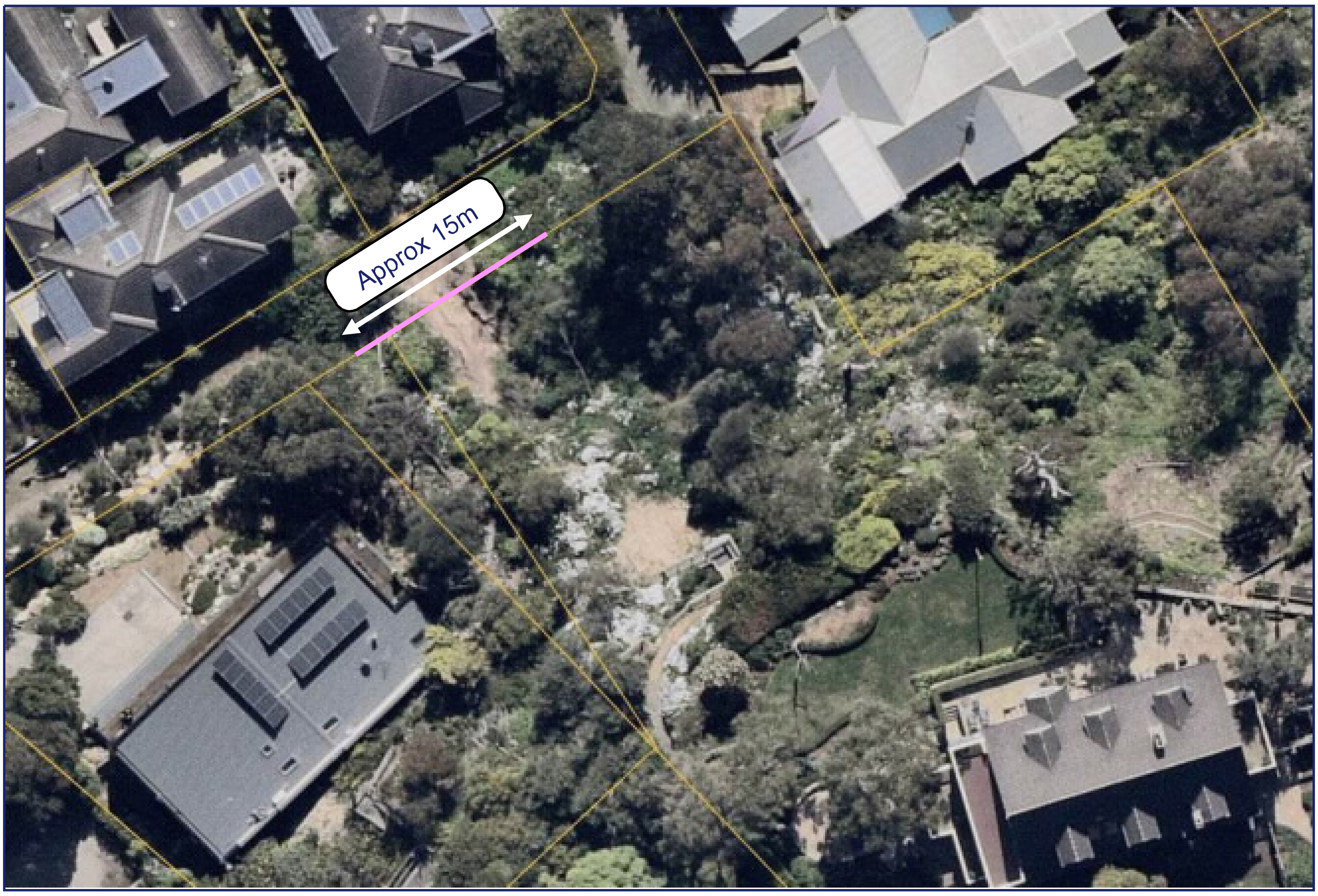
"Will the short-term solution alleviate the risk to life and property in relation to the affected properties."

81. I have assumed the following:
    (a) The short-term solution (**STS**) as per my Brief is to consist of a debris fence installed at the toe of the slope below Property P1 in Penny Lane.
    (b) The debris fence is to provide approximately 15 m in length of protection below Property P1, Inset 17.
    (c) The debris fence is to be an appropriately designed and constructed shallow landslide barrier manufactured by Geobrugg.
    (d) Landslide debris is cleared from the slope prior to the installation of the debris fence.
82. I note that a fundamental principle of the debris fence is to protect people and property below the landslide rather than to prevent a landslide from occurring. In my opinion the STS can be designed to reduce risk to properties P2 and P3 and risk to life of occupants of P2 and P3 to a tolerable level.

83. In my opinion the STS does not reduce risk to life and property for Property P1. As per paragraph [82], a debris fence does not prevent a landslide from occurring, therefore in my opinion landslides may still occur on Property P1. In my opinion a residual risk assessment post clean-up of Landslide and for:

(a) A pedestrian on the slopes of P1 is likely to indicate an unacceptable risk to life for future landslide events as no controls have been introduced to control repeat landslides.

(b) The dwelling at P1 is likely to indicate a tolerable risk to the dwelling and occupants within the dwelling with risks unchanged owing to the offset to the Landslide.



Inset 17: Assumed extent of debris fence shown in pink (Aerial image from Nearmap)

9.4 Instruction 3.3.3.2

"What effect, if any, will the short-term solution proposed by A.S. James have on the final solution you propose."

84. I assume that the debris fence will be constructed on the boundary of Penny Lane and Property P1. My assumed position of the debris fence is shown in Inset 17.

85. In my opinion the STS does not affect my recommended remedial works, except for the following minor construction issues:

(a) The debris fence would reduce construction efficiency as the fence is likely to get in the way of personnel and materials that access the slope from Penny Lane.

(b) The debris fence will have anchor ropes installed upslope from the line of the fence. These anchor ropes may conflict with the installation of the soil nail and slope netting system. The conflict is likely to occur during the placement of the slope netting. For a short period (less than 1 day) the STS would need to be temporarily deactivated whilst the slope netting is installed on the slope.

86. In my opinion these issues could potentially, subject to design requirements, be managed by moving the debris fence to the boundary of Penny Lane and properties P2 and P3.

87. In my opinion the STS would:

(a) Become redundant following completion of both Option 2 and Option 3, Table 2.

(b) Form a critical component of Option 1, Table 2.

9.5 Instruction 3.3.3.3

"Can the short-term solution be undertaken as 'stage 1' of the longer-term works, and if so, what extra expenses will that add to the overall project, above and beyond the solution you propose."

88. In my opinion, the answer is yes. This is supported by there being no major conflicts between the STS and my recommended remedial works. Refer to my response to Instruction 3.3.3.2 in Section 9.4.
89. In the event both the STS and Option 3 are to be installed, assuming similar lead times on ordering of materials, in my opinion the installation of a debris fence would likely delay the installation of Option 3 as it unlikely that both options could be constructed concurrently.
90. I am not a quantity surveyor. However, there are no cost efficiencies in the two designs. i.e. the extra expenses will be the total budget of the debris fence.

Yours Sincerely

Irrelevant and Sensitive

DANE POPE
PRINCIPAL

Appendix A
Brief

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Principal:	Benjamin Broadhead

3 November 2023

Mr D Pope
Pells Sullivan Meynink
Email: Dane.Pope@psm.com.au

Subject to legal professional privilege

Dear Dane,

Mornington Peninsula Shire Council (Council)
Advice regarding landslips at 10-12 View Point Road, McCrae (the property)

As you are aware, we act for the Council in relation to the landslips that occurred at the property on 15 November 2022.

By letter dated 23 October 2023, we provided you with a bundle of documents in relation to the landslips. We now attach a formal brief of documents, which contains the documents previously provided to you, as well as several additional documents.

1. We are instructed to engage you to prepare an opinion in relation to the most appropriate and achievable short and long-term solutions to alleviate the ongoing risk to life and property as a result of the landslips and to remediate the stability of the slope.

Background

2. By way of background:

   2.1. The landslips occurred at the property on 14 and 15 November 2022 after significant rainfall.

   2.2. While emergency orders were issued to 8 affected properties, only two remain in force, affecting properties at 2 Penny Lane, McCrae and 3/613 Point Nepean Road, McCrae (the **affected properties**) as it is considered the risk to life and property with respect to the affected properties is unacceptable.

   2.3. Council has, since shortly after the landslips occurred, been liaising with Mr Borghesi, the owner of the property, to arrive at a solution which addressed the longer-term stability of the slope.

      2.3.1. Borghesi has engaged Civil Test to assist with that process. Council has engaged Stantec (Cardno) to peer review the work undertaken by Civil Test. The various revisions of reports prepared by both are in your brief and have been previously provided to you. A full timeline setting out the various iterations of the reports, with references to the brief, is attached.

   2.4. On 28 March 2023, Building Order MW-127/23 (**BOMW**) was issued directing Borghesi to do particular works to the property. The BOMW supported a ‘retaining wall’ solution. In accordance with the BOMW, Borghesi then obtained structural computations and a design for the retaining all solution, which was peer reviewed by both Civil Test and Stantec.

2.5. In August 2023, Council confirmed Borghesi should proceed with the retaining wall solution.

2.6. In September 2023, Borghesi advised he would not be proceeding as his insurer would not accept his claim on the basis that it considered Council was liable for the works.

2.7. On 13 October 2023, the solicitor for one of the owners of the affected properties obtained preliminary comments from Tim Holt at A.S. James, regarding a solution that alleviates the unacceptable risk to life and property (the **short-term solution**) so that the affected property owners can return to their homes safely, setting to one side the longer term stability of the slope. We understand that Geobrugge has estimated the short-term solution will cost I&S for materials and labour. The owners of the affected property have asked that Council pay for the short-term solution.

Instructions

3. We are instructed to request that you prepare a fee proposal to:

3.1. Consider this letter and its attachments.

3.2. Undertake any site inspection or testing required as necessary to form your expert opinion.

3.3. Prepare a report, in expert witness report format, in relation to:

3.3.1. The most cost-effective and practical solution which achieves long-term stability of the slope.

3.3.2. Where the proposed solution differs from the solution proposed by Borghesi (being the retaining wall solution) why your solution is preferable.

3.3.3. Separately:

3.3.3.1. Will the short-term solution alleviate the risk to life and property in relation to the affected properties?

3.3.3.2. What effect, if any, will the short-term solution proposed by A.S. James have on the final solution you propose?

3.3.3.3. Can the short-term solution be undertaken as 'stage 1' of the longer-term works, and if so, what extra expenses will that add to the overall project, above and beyond the solution you propose.

4. Your report must comply with VCAT Practice Note PNVCAT2. Please have particular regard to the duty of an expert witness to the Tribunal at paragraphs 8-10 of that practice note, the mandatory inclusions at paragraph 11 and the report you should also:

4.1. identify any assumptions made;

4.2. confine your opinions to matters which are within your professional expertise;

4.3. when expressing an opinion, clearly set out the reasons and basis for that opinion, showing that the opinion is one which has been reached by you bringing your expertise to bear;

4.4. consider whether there are any limitations in your opinion and describe those together with the potential impact those limitations have on your opinion.

5. It may be that you require the assistance of others in forming your opinions. If so, please identify those persons and clearly explain their role in your report.

Fees

6. Before you commence substantive work on preparing this opinion, please provide us with an estimate of your fees in this matter as well as an estimate of the time it will take you to prepare the report.

Please contact Tanya Cimino on 5225 5232 or Ben Broadhead on 5226 8549 with any query.

Yours faithfully,

Harwood Andrews
HARWOOD ANDREWS

Encl.

Appendix B
Resume

Curriculum Vitae

Dane Pope
Principal Geotechnical Engineer

Dane Pope is a Principal Geotechnical Engineer at Pells Sullivan Meynink. He graduated from Griffith University, Gold Coast in 2006 with Bachelor of Engineering in Civil Engineering (Honours 1) and was awarded the University Medal. Dane joined PSM in November 2011, during which time he completed his master's degree in geotechnical engineering at UNSW in 2015.

Dane moved to Victoria in early 2016 and has actively been involved in civil infrastructure and property development projects throughout Victoria. Dane re-joined PSM in late 2019 to help to establish PSM's Victorian office.

Educational Qualifications:

- BE Hons Bachelor of Engineering (Civil), Griffith University, Gold Coast, 2006
- MEngSc. in Geotechnical Engineering, University of New South Wales, 2015

Professional Associations:

- Chartered Professional Engineer (CPEng)
- Registered Professional Engineer Queensland (RPEQ)
- Engineers Australia

Experience:

- 2020 – Current: Principal Geotechnical Engineer, Pells Sullivan Meynink
- 2019 – 2020: Associate Geotechnical Engineer, Pells Sullivan Meynink
- 2015 – 2019: Senior to Associate Geotechnical Engineer, P.J. Yttrup & Associates
- 2011 – 2015: Senior Geotechnical Engineer, Pells Sullivan Meynink
- Mar 2011 – Oct 2011: Geotechnical Engineer, MEC Mining
- 2006 – 2011: Geotechnical Engineer, Golder Associates
- 2005 – 2006: Undergraduate Engineer, Macdonald Sheet Piling

Field of Competence:

- Landslide Risk Assessment for Local Government and Road Authorities
- Unsaturated Soil Mechanics
- Industrial and residential subdivisional geotechnics including pavement design
- Surface Coal Mining and Quarry Operations and slope design
- Detailed instrumentation planning, installation and analysis
- Deep basement excavations

CIVIL PROJECTS

Bogong Village, Temporary Access Track, VIC

Geotechnical assessment and cut slope design for a temporary access track in deeply weathered granite and migmatite.

Great Ocean Road and inland routes, Landslide Remediation, VIC

Ongoing landslide remediation for over 20 sites from mid-2020 onwards. Sites include sideling fill batters, cut slopes and embankments in steep to very steep terrain. Remediation included rock bolt/anchor systems, rock fall netting, catch bunds, light weight fills, bored pile walls with capping beams and reconstruction of fill batters. All projects included the provision of IFC drawings and Construction Supervision Services.

Strzelecki Ranges flood recovery, Landslide Remediation, VIC

Detailed design of landslide remediation for a flood recovery site in the Strzelecki Ranges. Provision of IFC drawings.

Otway Ranges 2016 flood recovery, Landslide Remediation, VIC

Detailed design of landslide remediation for three flood recovery sites in the Otway Ranges in 2016. Designs included post and panel retaining walls, gabion walls and reconstruction of fill embankments. Provision of IFC drawings.

Cliff Road, Frankston, VIC

Landslide Risk Assessments for complex soil profile in existing landslide domain. Detailed field reconnaissance of the area. Managing complexities relating to the application of the Erosion Management Overlay (EMO) to existing properties which predate the recent application of the EMO.

Peer review, Mornington Peninsula, VIC

Peer review of Landslide Risk Assessment for development application in calcareous dune deposits.

Deviation Road, Fyansford, VIC

Landslide Risk Assessment for complex profile of Newer Volcanic Basalt overlying Gellibrand Marl. Groundwater monitoring to identify multiple aquifers.

McCurdy Road, Fyansford, VIC

Regression analysis of escarpment to inform permanent development offsets.

Wye River, Landslide Assessments, VIC

Landslide risk assessments for properties affected by the recent bushfires. Established structural domains of township to aid in better understanding mode of failure across the town. Assessment for proposed new stormwater network.

Cumberland River, Rockfall Assessment, VIC

Rock fall assessment for VicRoads included mapping by hand and photogrammetry methods. Detailed assessment of the structural controls of a 90 m high slope.

Sunshine North, Quarry infill sub-division, VIC

Rock Face Assessment of abandoned Basalt quarry for potential sub-division. Key inputs into landslide risk assessment.

Western Sydney Airport, Pavement Tender

Part of the successful bid team for the Pavement Tender. Worked with the Pavement Designers to assess risk of collapse settlement of engineered fill and differential settlement at cut/fill interfaces.

Geelong & Melbourne, Site Classification, VIC

Managing geotechnical investigations, analysis and reporting for residential developments in highly to extremely reactive soils with a focus on residual Basalt and Limestone profiles. Coordinating activities for a small team of engineers and a technician. Establishing and managing borehole reporting standards. Specialise in measuring total suction profiles to provide ground movement estimates for sites with abnormal moisture conditions.

Geelong Subdivisions, VIC

Geotechnical support from site investigation, pavement design and construction supervision for numerous greenfield sub-divisions in the Geelong region including Manzene Village, Lara West, Armstrong creek, Charlemont Rise, Leopold and Point Lonsdale Golf Course.

Bulk Earthworks Supervision, City of Greater Geelong, Colac Otway Shire, VIC

Provision of Level 1 certification of bulk earthworks for residential and commercial projects. Assessment and re-classification of lots to AS2870-2011.

Wintringham Social Housing, Travancore VIC

Geotechnical investigation and temporary works for basement excavation in Old Volcanics.

Barwon Water Easement Investigations, City of Greater Geelong, Colac Otway Shire, VIC

Forensic investigations into collapse settlement in stormwater and sewer easements at three sites. Development of backfill specification to reduce risk of collapse settlement.

Brownfield Basalt quarry redevelopment, Tottenham VIC

Geotechnical investigation and design advice for industrial development on complex landfill site. Ground improvement strategies including rigid inclusions.

Armstrong Creek Town Centre, Investigation & Pavement Design, VIC

Geotechnical investigation for $20M town centre including earthworks specification, detailed ground movement assessment in extremely reactive ground and pavement design.

Due Diligence - Dandenong South, VIC

Due diligence assessments for property developers across several large industrial sites throughout Dandenong South. Constraints typically including buildings approaching the end of their design life, poor quality subgrades and one backfilled sand quarry with inferred collapse settlement issues.

Deer Park, Boral, VIC

Ongoing auditing of bulk earthworks for backfill of existing Basalt quarry. Bulk earthworks design and specification for industrial development.

Campbellfield Industrial Development, Campbellfield, VIC

Investigation, settlement analysis and bulk earthworks design and supervision for proposed automated glass manufacturing facility with a high-performance building specification in a Basalt profile.

High Bay Developments and Expansion, Truganina, VIC

Investigation, design advice and specification for three different high bay shed sites in a Basalt profile. Including validation of total suction profile four years after construction of the initial pavement slabs.

High Bay Development, Moorebank, NSW

Investigation, design advice and specification for proposed high bay sheds.

Greystanes Industrial Development, NSW

Investigation, design advice and specification for proposed industrial subdivision.

ACFS Logistics Terminal, Port of Brisbane, QLD

Subgrade remediation in poor soils. Footing and subgrade inspections including plate load testing.

Soleil Tower, Ten Story Basement Excavation, Brisbane, QLD

Monitored excavation activities for a 10 storey basement car park excavation. Completed anchor inspections and review, 'hit and miss' sequencing, detailed instrumentation planning, implementation and reporting.

Vision Apartments, Seven Story Basement Excavation, Brisbane QLD

Geotechnical investigation. Diaphragm wall design using PLAXIS and MSHEET. Supervision of diaphragm wall and secant pile wall construction. Rock bolt design, mapping, anchor supervision and review, 'hit and miss' excavation sequencing on all shoring wal ls.

Infinity Tower, Twelve Storey Basement Design, Brisbane QLD

Geotechnical investigation including pressuremeter testing. Design of shoring walls using PLAXIS.

Springfield to Darra Rail, Pile Design, Brisbane QLD

Successful tender pile design for 6 bridges varying in size from single span to ten span viaducts.

MINING PROJECTS

Lysterfield Quarry, Boral, VIC

Development of photogrammetry model. Geotechnical review of quarry slopes and providing slope stability advice. Review and update of structural model.

Montrose Quarry, Boral, VIC

Geotechnical review of quarry slopes and providing slope stability advice including rock fall mitigation and pit re-design to manage rock fall risk.

Wollert Quarry, Boral, VIC

Geotechnical review of quarry slopes and providing slope stability advice. Biannual inspection.

Clermont Coal Mine, QLD

Western wall review including three dimensional domains using ATV, field mapping and Vulcan. Site visit to calibrate structural model. Stability analysis of structurally complex pit slopes.

Burton Coal Mine, QLD

Maximised coal recovery from large slope failures without incident. Site based geotechnical support for two open cut terrace mines. Maintenance of highwall and lowwall hazard management systems (radar and survey) and monitoring of slope failures. Civil projects included; anchor pull-out tests, wet weather road construction, crane pad selection, plate load testing.

Baralaba Central and North Operations, QLD

Design reviews of pit slopes. Site inspections to provide operational advice for unstable slopes and their interaction with large dams.

Baralaba Expansion, Geotechnical Investigations – Feasibility, QLD

Geotechnical investigation and design of the proposed 200 m deep terrace mining operations. Training of site based rig geologists.

Norwich Park (BMA), Geotechnical Management System, QLD

Seconded to BMA's Norwich Park open cut coal mine. Pit inspections, mapping, radar monitoring and implementation of a revised TARP.

Tutupan Coal Mine, Pressuremeter Testing, South Kalamantan Indonesia

Trained a Jakarta based geotechnical engineer in the use of the pressuremeter at the South Kalimantan Coal Mine.

QC LNG and Pipelines, Pressuremeter Testing and Fieldwork, Gladstone, QLD

Large pressuremeter testing program in various materials from residual clays to high strength rock. Mobilisation of drilling rigs in difficult access conditions for the narrows pipeline project including use of a hover -barge.

TUNNEL PROJECTS

Clem 7 Tunnel, Investigation & Monitoring, Brisbane QLD

Coordinated drilling activities over the tunnel alignment, including permitting, service clearances, supervision and reporting. Installed and monitored settlement monitoring equipment including magnetic and rod extensometers, vibrating wire piezometers, profile gauges and inclinometers.

Burnley Tunnel, VIC

Site based tunnel crack mapping of the tanked section of the tunnel.

Melbourn Metro Tunnel, VIC

Annual inspections and reporting on behalf of the insurer.

EXPERT OPINION/ADVICE

Cut slope instability, Geelong VIC

Geotechnical investigation into wedge failure of cut slope adjacent to a commercial development. Provision of conceptual remediation advice.

Retaining wall settlement, Victoria

Expert Opinion regarding settlement of gravity retaining wall including collapse settlement.

Residential subdivision, Western Sydney NSW

Forensic investigation into collapse settlement including review of property damage and site classification for 100's of dwellings.

Industrial subdivision, Melbourne

Forensic investigation into collapse settlement including review of property damage and remediation.

Preloading soft soils, Pinkenba QLD

Review of settlement controls and effectiveness of preloading activities for deep compressible sediments.

Damaged building assessments, Victoria

Numerous geotechnical investigations to support expert opinion reports for damaged homes on reactive ground. These typically including testing shrink swell, total suction and providing ground movement estimates for seasonal movement and movements due to th e growth or removal of trees and removal of old timber floor dwellings prior to construction.

Publications, Articles and Patents

1. Developments in Engineering Geology the Geological Society (2016). Published Paper: Geological structural controls on stability of footwall slopes, an example from the Bowen Basin.
2. Field Measurements in Geomechanics (FMGM) Sydney, (Sept. 2015). Published Paper: Real -time monitoring of cut slopes and the importance of identifying the mode of failure.

Appendix C
Selected Site Inspection Photographs

Photo 1 - Upper translational sliding area

Photo 2 - Debris flow runout area, Property P2 visible in background

Hardwood Andrews

Expert Witness - Rectification

10-12 View Point Rd, McCrae

Selected Site Photographs (1 of 8)

PSM5226-005R	**Appendix C**

Photo 3 - Landslide Overview, with zones indicated

Hardwood Andrews

Expert Witness - Rectification

10-12 View Point Rd, McCrae

Selected Site Photographs (2 of 8)

PSM5226-005R	**Appendix C**

Photo 4 - Failed material still on slope

Photo 5 - Rear scarp of landslide, note lack of oversteepened backscarp

Hardwood Andrews

Expert Witness - Rectification

10-12 View Point Rd, McCrae

Selected Site Photographs (3 of 8)

PSM5226-005R	**Appendix C**

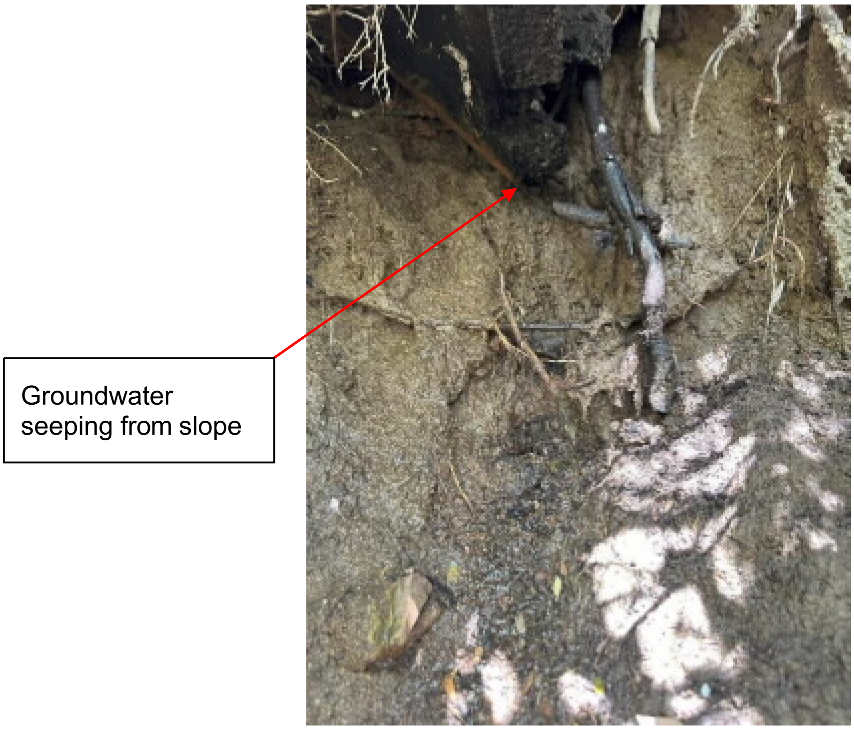

Photo 6 - Groundwater seepage

Photo 7 - Water infrastructure including water pipes (on left), and subsurface 'agi drains' (on right)

Hardwood Andrews

Expert Witness - Rectification

10-12 View Point Rd, McCrae

Selected Site Photographs (4 of 8)

PSM5226-005R	**Appendix C**

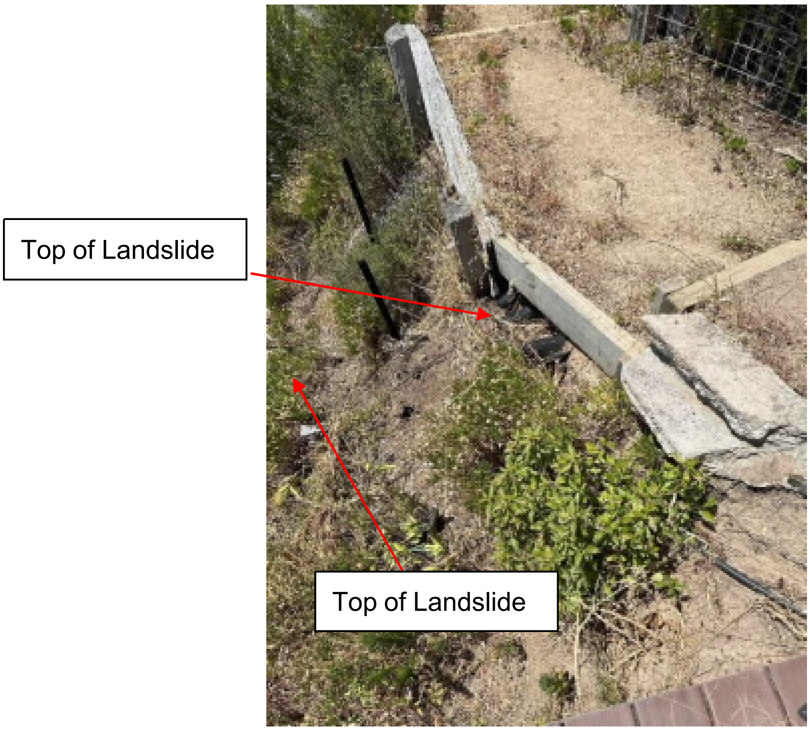

Photo 8 - Path above Landslide, note minor retaining walls and 'agi drains' from Photo 7

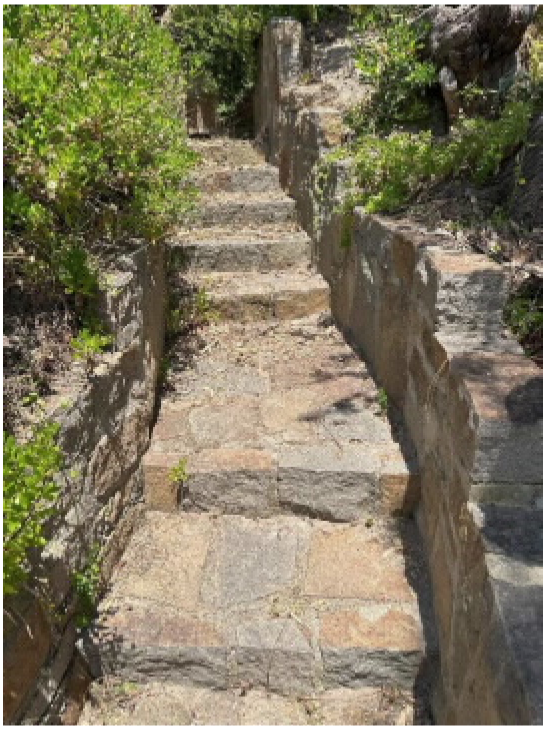

Photo 9 - Granite stairs leading from garden area to path at top of landslide

Hardwood Andrews Expert Witness - Rectification 10-12 View Point Rd, McCrae Selected Site Photographs (5 of 8)	
PSM5226-005R	Appendix C

Photo 10 - Tilitng retaining walls on left side of Landslide

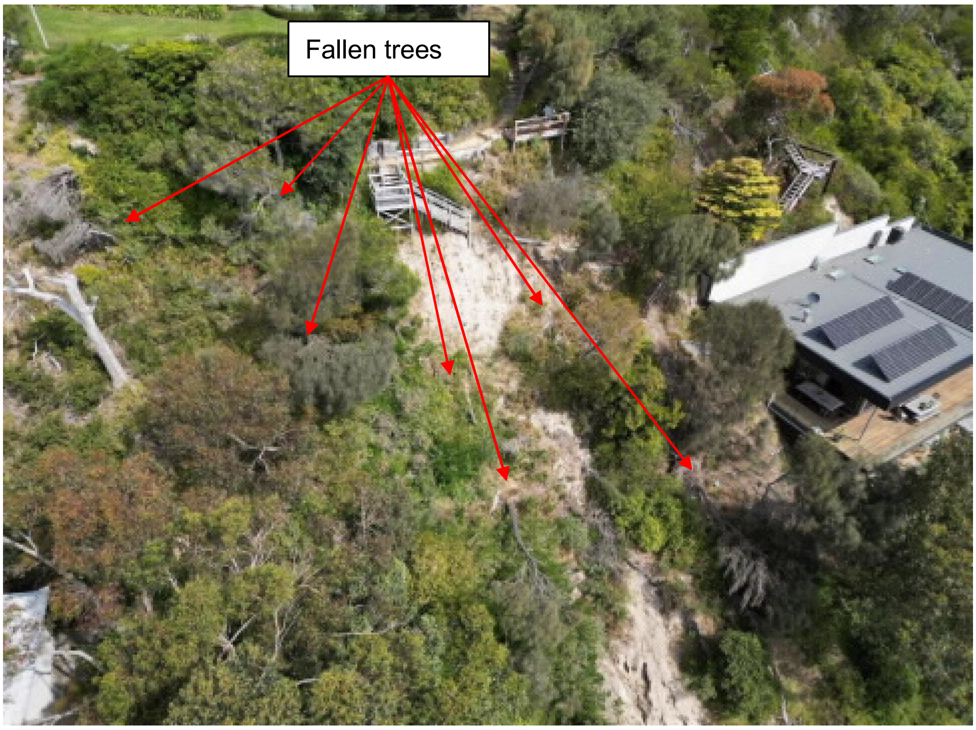

Photo 11 - Fallen trees

Hardwood Andrews **Expert Witness - Rectification** **10-12 View Point Rd, McCrae** **Selected Site Photographs (6 of 8)**	
PSM5226-005R	**Appendix C**

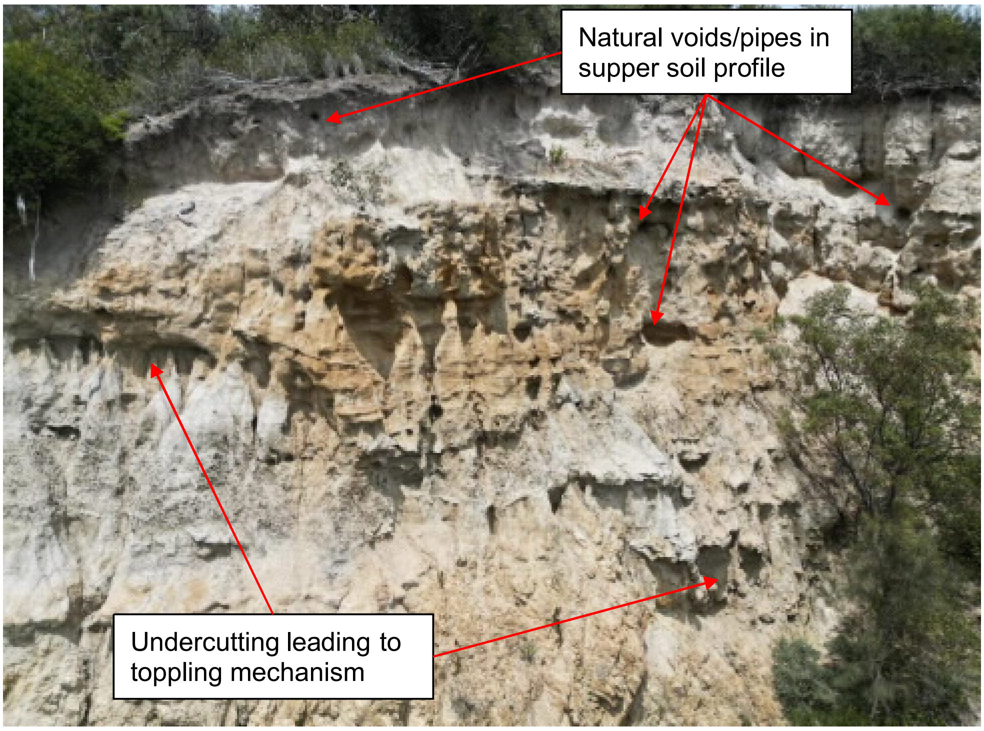

Photo 12 - Natural cliff profile near Anthony's Nose

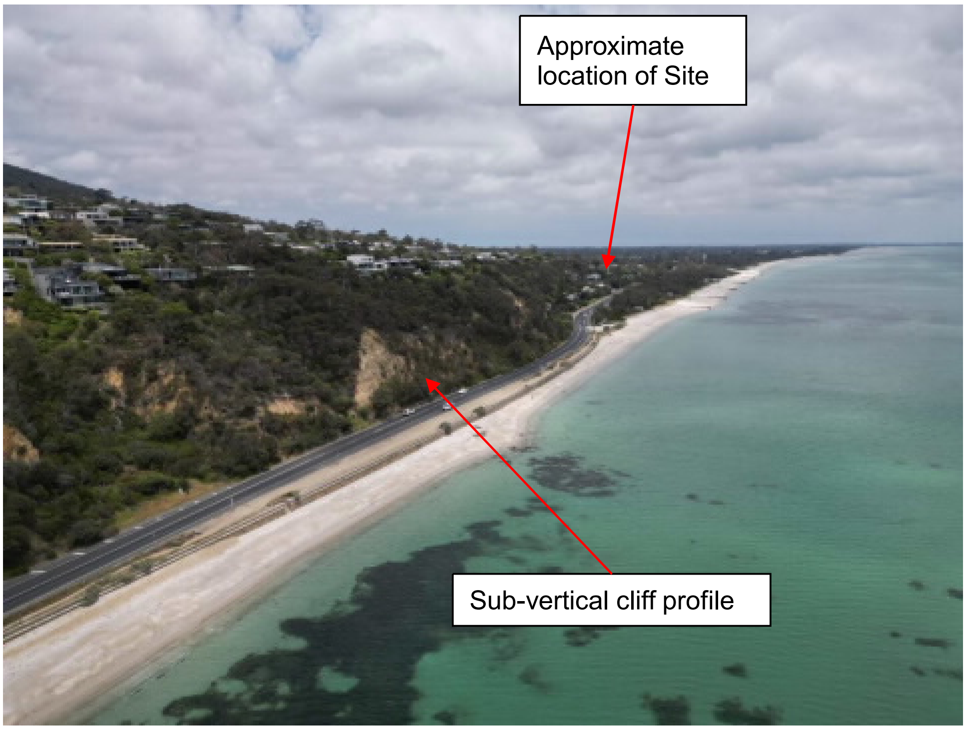

Photo 13 - Anthony's Nose

Hardwood Andrews **Expert Witness - Rectification** **10-12 View Point Rd, McCrae** **Selected Site Photographs (7 of 8)**	
PSM5226-005R	**Appendix C**

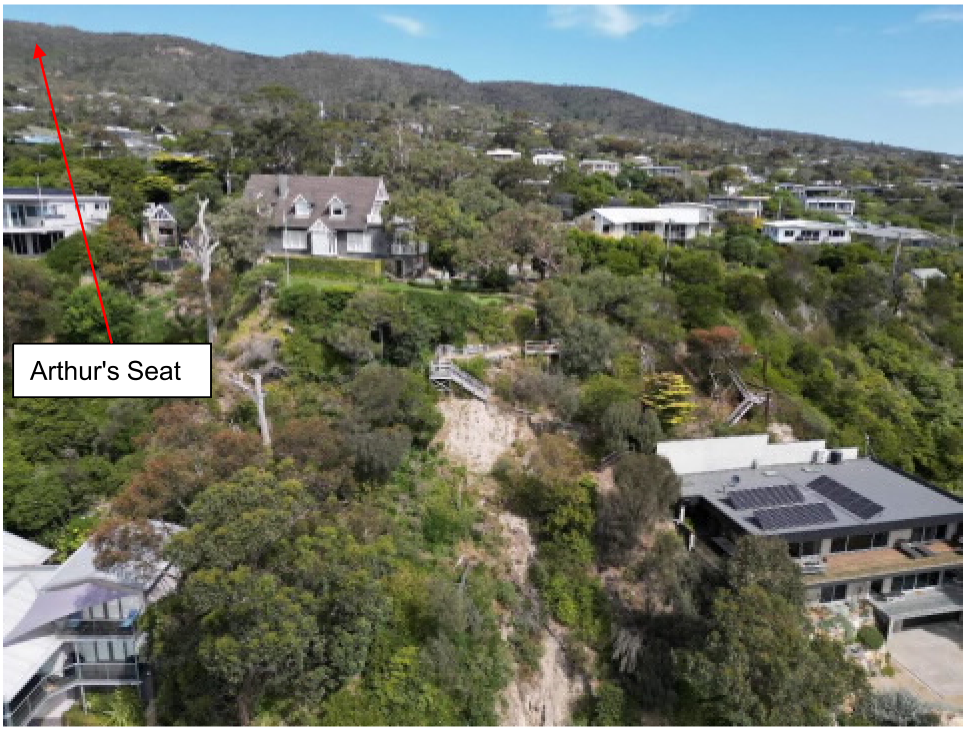

Photo 14 - Arthur's Seat in background

Hardwood Andrews

Expert Witness - Rectification

10-12 View Point Rd, McCrae

Selected Site Photographs (8 of 8)

PSM5226-005R | **Appendix C**

Appendix D
Historical Aerial Imagery

Legend

Site Boundaries

Property

Landslide

NOTES:
1. AERIAL IMAGERY FROM NEARMAP DATED 25 AUGUST 2023

N

Scale 1:500

0 10 20 m

Map Projection:
Horizontal Datum:
Grid: EPSG:7855

PSM

Created By: AW

Revision: A

Date: 01 Dec 2023

Paper Size: A3

HARDWOOD ANDREWS
EXPERT OPINION - RECTIFICATION
10-12 VIEW POINT RD, MCCRAE

HISTORICAL AERIAL IMAGERY
25 AUGUST 2023

PSM5226-005R

FIGURE D1

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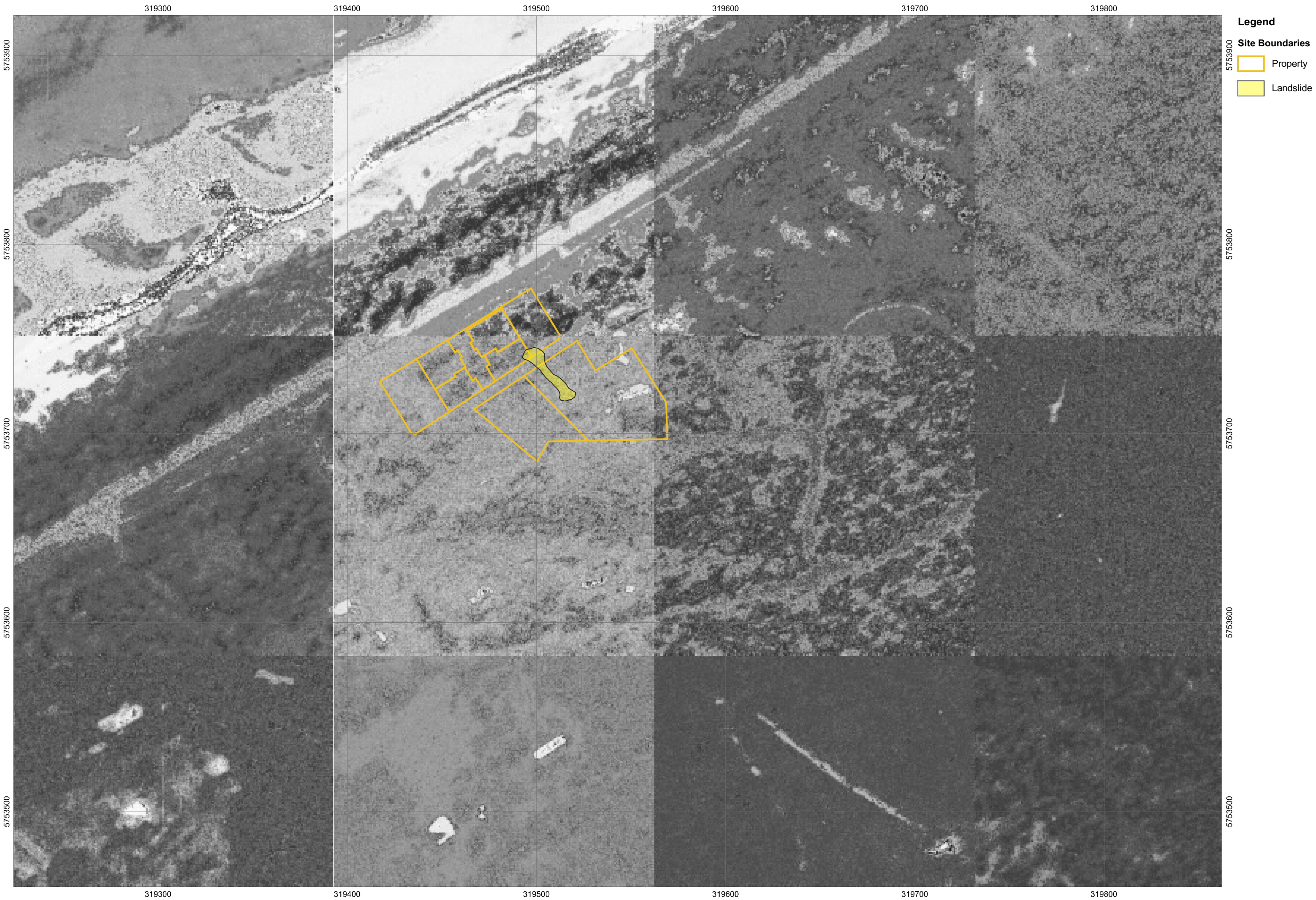

NOTES:
1. Photo from Geoscience Australia (https://aerialphotography-geoscience-au.hub.arcgis.com/)

Scale 1:2,000

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Map Projection:
Horizontal Datum:
Grid: EPSG:7855

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Created By: AW
Revision: A
Date: 01 Dec 2023
Paper Size: A3

HARDWOOD ANDREWS
EXPERT OPINION - RECTIFICATION
10-12 VIEW POINT RD, MCCRAE

HISTORICAL AERIAL IMAGERY
1939

PSM5226-005R

FIGURE D2

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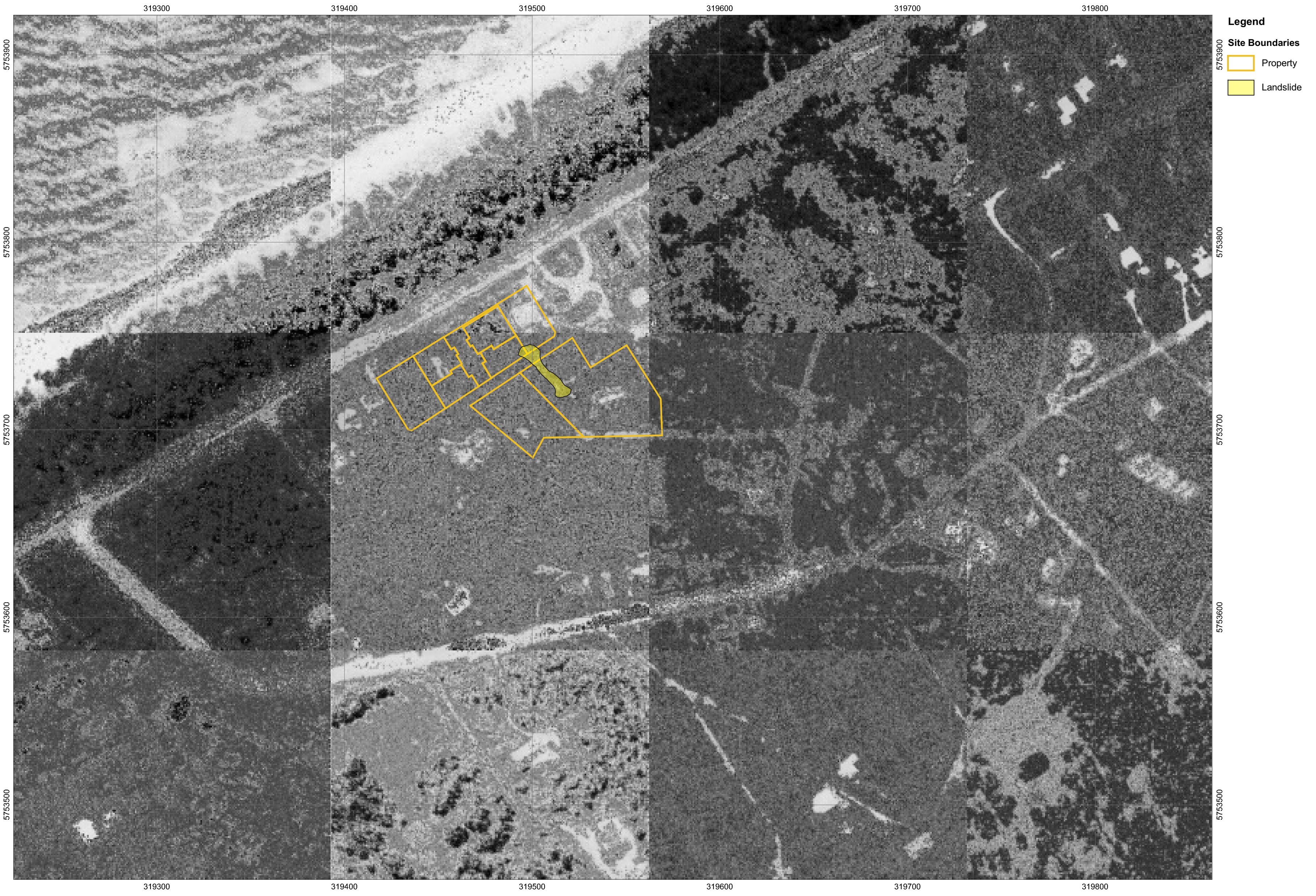

NOTES:
1. Photo from Geoscience Australia (https://aerialphotography-geoscience-au.hub.arcgis.com/)

Scale 1:2,000

N 0 50 100 m

Map Projection:
Horizontal Datum:
Grid: EPSG:7855

PSM	Created By: AW	Revision: A
	Date: 01 Dec 2023	Paper Size: A3

HARDWOOD ANDREWS
EXPERT OPINION - RECTIFICATION
10-12 VIEW POINT RD, MCCRAE

HISTORICAL AERIAL IMAGERY
1951

PSM5226-005R

FIGURE D3

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MSC.5000.0001.2031

Legend

Site Boundaries

- Property
- Landslide

NOTES:
1. Photo from Geoscience Australia (https://aerialphotography-geoscience-au.hub.arcgis.com/)

Scale 1:2,000

N 0 50 100 m

Map Projection:
Horizontal Datum:
Grid: EPSG:7855

PSM	Created By: AW	Revision: A
	Date: 01 Dec 2023	Paper Size: A3

HARDWOOD ANDREWS
EXPERT OPINION - RECTIFICATION
10-12 VIEW POINT RD, MCCRAE

HISTORICAL AERIAL IMAGERY
1984

PSM5226-005R | FIGURE D4

C:\Users\andrew.wilson\Pells Sullivan Meynink\GIS - PSM5226\02_WorkSpace\01_Projects\PSM5226.qgz Layout: Figure D4