TECHNICAL MEMORANDUM

To	Claudio Flores Deputy Municipal Building Surveyor Mornington Peninsula Shire	**From**	Davin Slade Senior Principal Geotechnical Engineer Stantec
Project	10-12 View Point Road, McCrae	**Date**	29 June 2023
Subject	Review of CivilTest Report	**Ref No.**	V220600Report02.1

1 INTRODUCTION

At the request of Claudio Flores of Mornington Peninsula Shire in an email dated 20 June 2023, Stantec has been engaged to review a 'Land Stability Assessment' report prepared by Civiltest with regard to the remediation of a landslide that occurred at 10-12 View Point Road (Ref. 1222044-3 Issue 4 dated 6 June 2023).

Stantec was initially engaged by Derek Rotter of Mornington Peninsula Shire on 15 of November 2022 with regard to a landslide that had recently occurred in the vicinity of Penny Lane, McCrae. The landslide initiated on the property of 10-12 View Point Road, flowed across Penny Lane and into the rear of two properties at 3 Penny Lane and 3/613 Pt Nepean Road. It was requested that an engineer from Stantec attend the site to inspect the landslide and to make recommendations with regard to the suitability of emergency orders that had been issued to residences in the nearby area.

Subsequent to the inspection, a geotechnical assessment report was issued on 7 December 2022. The report identified that the landslide was most likely a complex landslide initially occurring as a translational slide on the upper slopes and subsequently after a period of approximately 12 hours as a debris flow that flowed down and across Penny Lane.

The report concluded that the landslide most likely occurred due to an excessive water build up on the upper slope, whether from natural or human causes.

The report identified that in the current condition of the site while the risk to life for a majority of the houses in the area was 'Tolerable' but that the risk to life for 3 Penny Lane and 3/613 Pt Nepean Road as well as the steeper section of 10-12 View Point Road was 'Not Tolerable' and that significant rehabilitation works were required to make the risk 'Tolerable'.

It is understood that the owner of the 10-12 View Point Road property has subsequently engaged CivilTest to conduct a geotechnical investigation and analysis of the site to enable the design of the rehabilitation works. The report prepared by CivilTest, *Land Stability Assessment at 10-12 View Point Road, Mcrae* dated 6 June 2023, is the subject of this review.

2 REVIEW OF CIVILTEST REPORT

An initial review of the CivilTest report identifies that the report generally follows the expected conventions and that the general solution provided is a potential practical solution. However, there are some issues that need to be highlighted in order to ensure that the solution is appropriately designed and constructed. As such, this review will only discuss the issues and will not discuss in details the parts of the report that are considered appropriate.

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2.1 GROUNDWATER MODELLING

The slope stability model identifies groundwater sitting at the interface between the sands and clays. It is considered that this is incorrect. This is because the landslide that occurred was within the upper sands and occurred due to the saturation of these sands. However, at the same time it is possible that the underlying clays are not saturated, with the water that is present running within the sands due to the lower permeability of the underlying clays.

It would be appropriate for any design solution to model an elevated groundwater table within the sands.

It would also have been appropriate to install a piezometer in the upper borehole to establish the groundwater depth at depth (i.e. not within the upper sands) within the clays, if present.

2.2 SLOPE STABILITY MODEL PARAMETERS

The borehole logs in the report identified that the site is overlain by colluvial sands on the slope and aeolian sands at the toe. There is most likely also residual granitic sands at the crest. These sands are then shown to be underlain by residual granitic sandy clays. The borehole logs did not identify weathered granite rock within their depth. These conditions are fairly consistent with those previous identified for nearby sites in the area.

It is noted that the BH3 generally identified the sands to be SP to SW, meaning they had minimal fines content (<12%), but the single laboratory test conducted on a 'sand' identified the soil to have 36% fines which actually would mean the sample is a clay.

Understanding of the fines content is critical as SP and SW sands as well as SM (silty sands) would typically have little to no drained cohesion while SC (clayey sands) would have a higher drained cohesion. Clays would have a higher drained cohesion than the clayey sands.

The model adopted the following parameters for the sands and clays respectively. The colluvial and aeolian sands are considered together for simplification due to their similar properties.

Soil	Drained Friction Angle (deg)	Drained Cohesion (kPa)
Colluvial/Aeolian SAND	38	5 – 6
Granitic CLAY	27	18

The drained friction angle parameters adopted appear appropriate generally appropriate for the clays. For the sands the drained friction angle is appropriate provided that the sands are medium dense. If the sands are loose then the drained friction angle would be under conservative.

The drained cohesion values for both the sands and clays seem under conservative. For the sands, if they are SP, SW or SM then a drained cohesion in the order of 0-2 kPa would be appropriate but if they are SC then a value of 5kPa is appropriate. For the clays, 18 kPa is quite unconservative and a value of 10 kPa is more consistent with typical values.

It is noted that the report makes no mention of the methods used to determine these parameters besides Civiltest's previous experience. Furthermore, no in-situ testing was conducted within BH01 and BH02 to establish the density of the soils. Methods such as Stark and Choi (ASCE May 2005) or Appendix D of AS 4678 could be used to better define these parameters. For example Table D4 of AS4678 identifies sandy clays to typically have a drained angle of friction in the range of 26 to 32 degrees and a drained cohesion in the range of 0 to 10 degrees and sands to typically have a drained angle of friction of 32 to 37 degrees and a drained cohesion of 0 to 5 degrees. Should higher values than the typical values be adopted it would be appropriate to justify the values with laboratory testing such as a Consolidated Undrained triaxial test.

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It is also noted that the report itself is inconsistent in providing parameters. While the above parameters were adopted in the slope stability model, the following parameters were provided for design of the retaining walls. These values are closer to the typical values for these types of soils.

Soil	Drained Friction Angle (deg)	Drained Cohesion (kPa)
Silty SAND	35	0
Silty/Sandy CLAY	25	10

It would normally be appropriate to adopt the same parameters for the retaining wall design as for the slope stability analysis, especially as the slope stability model has been used to justify the size of the retaining walls and the loads applied to the walls.

In addition, use of the slope stability model to determine design strengths for the anchors is not considered best practice. The slope stability analysis software should first, as has been done, determine the required shear strength capacity of the piles to achieve the appropriate Factor of Safety. It is then appropriate to use a Finite Element Analysis package to determine the actual actions on the piles and retaining walls which are used by the structural engineer to assess the capacity of the structural elements.

2.3 SLOPE STABILITY ANALYSIS

The report has provided three different analyses as follows:

- Post landslide before rehabilitation (i.e. existing conditions)
- Remediation with seismic loading
- Remediation without seismic loading

All three of these analyses are considered appropriate as part of the design of the rehabilitation solution. However, differences in the soil parameters and groundwater regime have the potential to alter the solution. For example, including the elevated groundwater may identify that a drainage solution could be used to improve the stability of the site. Such a model may even identify that less substantial retaining walls are required.

In addition, it would be appropriate to model the pre landslide condition to confirm that the model identifies that a failure of the upper sand profile occurs with an elevated groundwater condition (i.e. Factor of Safety of less than 1 in the upper sands) but that the deeper slope remained relatively stable (i.e. worst case Factor of Safety of greater than 1.2 – 1.3 but unlikely to be greater than 1.5). This is because there is no significant evidence that ground movement is occurring at depth.

Using this approach would confirm that the adopted parameters match with the failure conditions and the current conditions but also allow the rehabilitation design to consider the conditions that led to the failure in the first place.

2.4 STRUCTURAL DRAWING REVIEW

Considering further updates to the model are required, the structural drawings have not been reviewed in great detail. However, a brief review has also been conducted. It is noted that the structural drawings do not mention the geotechnical conditions that are required for the founding of the piles and when the piles should be terminated. As a minimum the drawings should indicate the soil type, soil strength parameters and minimum penetration into the foundation medium.

In addition, the drawings also indicate that the retaining walls are only required where the limits of the debris flow occurred. There is no indication of any stabilizing works on the eastern side of where the landslide occur were significant tension cracks are present. As indicated in the previous reports by Stantec any design solution should address the issues in those areas as well.

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3 CONCLUSIONS & RECOMMENDATIONS

While the report has included a significant amount of appropriate analysis and the proposed design solution may be appropriate, it is considered that there are some issues that need to be resolved in order for the design solution to be approved.

It is recommended that the following be undertaken:

1. Preferably a piezometer should be installed in the vicinity of BH3. The piezometer should be screened at depth within the granitic clays and sealed off from the upper sands. This will allow it to be determined where the local groundwater table lies.
2. During installation of the piezometer additional undisturbed samples should be taken. Laboratory consolidated undrained triaxial tests (with pore water measurements) should then be conducted on these samples to confirm the drained shear strength parameters of both the sands and clays.
3. The model should then be updated based on the findings of the additional investigation and lab analysis. This should include modelling of the pre-landslide condition to better calibrate the model against the observed conditions.
4. A finite element model should then be developed to assess the actions being applied to the piles ad retaining walls.
5. The structural drawings should then be reviewed based on the updated analysis. This should include ensuring that the retaining wall systems cover the area to the east of the existing landslide.

It is possible that this additional work will identify that the same solution is still appropriate for the site. However, the solution would then be appropriately justified. It is also possible that an alternative solution may be apparent, such as improved drainage works, constructing a buttress at the toe or a combination including retaining walls. Considering that a Factor of Safety of at least 1.5 has been defined by the Shire some form of stabilization is likely to be required as the natural slope would most likely have a Factor of Safety of less than 1.5.

We trust this meets with your requirements.

Yours sincerely,

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