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Selection of appropriate strength envelopes for open pit slope stability analyses in soils and weak rocks

DR Wines *Itasca Australia Pty Ltd, Australia*

W Zhang *Rio Tinto, Australia*

J van Rensburg *Rio Tinto, Australia*

A Lucarelli *Itasca Consulting Group, Inc, USA*

Abstract

Soils and weak rocks are often encountered in the upper benches of open pit mines. The exposure depth of these materials is often insignificant and therefore limited effort may be committed to their characterisation and analysis. For example, these materials may be represented using a simple linear Mohr–Coulomb model with the properties based on limited testing data. However, observations of soil and weak rock behaviour (including laboratory testing) indicate that the strength envelopes are often non-linear. When deeper exposures of these relatively weak materials are encountered, providing a more rigorous numerical representation of their expected behaviour may be important to the safety and economic viability of an operation.

Compared to linear strength envelopes, non-linear strength envelopes can provide an improved understanding of the expected behaviour of soils and weak rocks. In this paper, numerical analyses were used to illustrate the difference in slope stability analysis results obtained using linear and non-linear strength envelopes. The paper also discussed case studies from three different open pit mines where non-linear strength envelopes provided more plausible and realistic representations of observed slope behaviour.

Keywords: *soil, weak rock, non-linear strength, Mohr–Coulomb, confining stress*

1 Introduction

Open pit slope stability is often influenced by soils and weak rocks, which often attracts minimal effort in material characterisation due to their limited thickness of exposure. However, in many cases, multiple benches with an overall depth of greater than 100 m may be exposed in these materials, dominating stability of these slopes as well as the safe and economic success of an operation.

Weaker materials will often be represented using a linear Mohr–Coulomb strength envelope. However, laboratory testing and the literature indicate that soil strength envelopes are often non-linear (Fredlund et al. 1987; Gao et al. 2015; Atkinson 2017; Wu et al. 2019), especially under low confining stresses. Failure criterion for rocks, such as the Hoek–Brown criterion (Hoek et al. 2002) are non-linear. Other techniques have also been proposed specifically to estimate the strength of weak rock masses (e.g. Castro et al. 2013).

Using linear Mohr–Coulomb strength envelopes can be beneficial because it simplifies the analysis process and some slope stability analysis software may be limited to only these simple inputs (i.e. cohesion and friction angle). Traditionally, in order to model materials of non-linear strength behaviour, linear approximation may be conducted for selected ranges of confining stress. The adopted confining stress over which the linear envelopes are fitted to the non-linear curve is an important consideration, because this will influence the applicability of the adopted properties for the slopes (and potential failure mechanisms) being analysed. Renani & Martin (2020) investigated how the results of slope stability analyses may be significantly affected by the range of confining stresses. Wines (2020) also illustrates how analysis results may be influenced by the maximum confining stress used to define a linear strength envelope.

A conceptual example of the potential variability in linear approximations is provided in Figure 1, which shows the following:

- An example non-linear strength envelope. See Figure 1a. A non-linear envelope may be estimated using an empirical technique such as the Hoek–Brown criterion (if applicable for the rock mass being analysed) or using laboratory test results (such as triaxial tests).
- A bilinear strength envelope fitted to the non-linear curve (i.e. two linear segments are fitted to the non-linear curve, and each segment can be defined by a cohesion (c) and friction angle (ϕ)). See Figure 1b.
- A linear Mohr–Coulomb strength envelope fitted to the non-linear curve (based on a maximum confining stress of 0.5 MPa). See Figure 1c. This is achieved by fitting an average linear relationship to the non-linear curve for the relevant range of minor principal stress values, as described by Hoek et al. (2002).
- A linear Mohr–Coulomb strength envelope fitted to the non-linear curve (based on a maximum confining stress of 2.0 MPa). See Figure 1d.

The difference between the strength envelopes is particularly noticeable at low confinement. It is noted that slope failures are often relatively shallow and accurate modelling of material properties at low confinement can be critical for slope stability assessments. Case studies from three different open pit mines are provided below to illustrate the importance of the adopted strength envelopes. In all cases, the mechanisms being analysed are assumed to involve failure through the soil or rock mass only with no structural component (i.e. structures such as fissures, joints and faults are not explicitly included in the analyses).

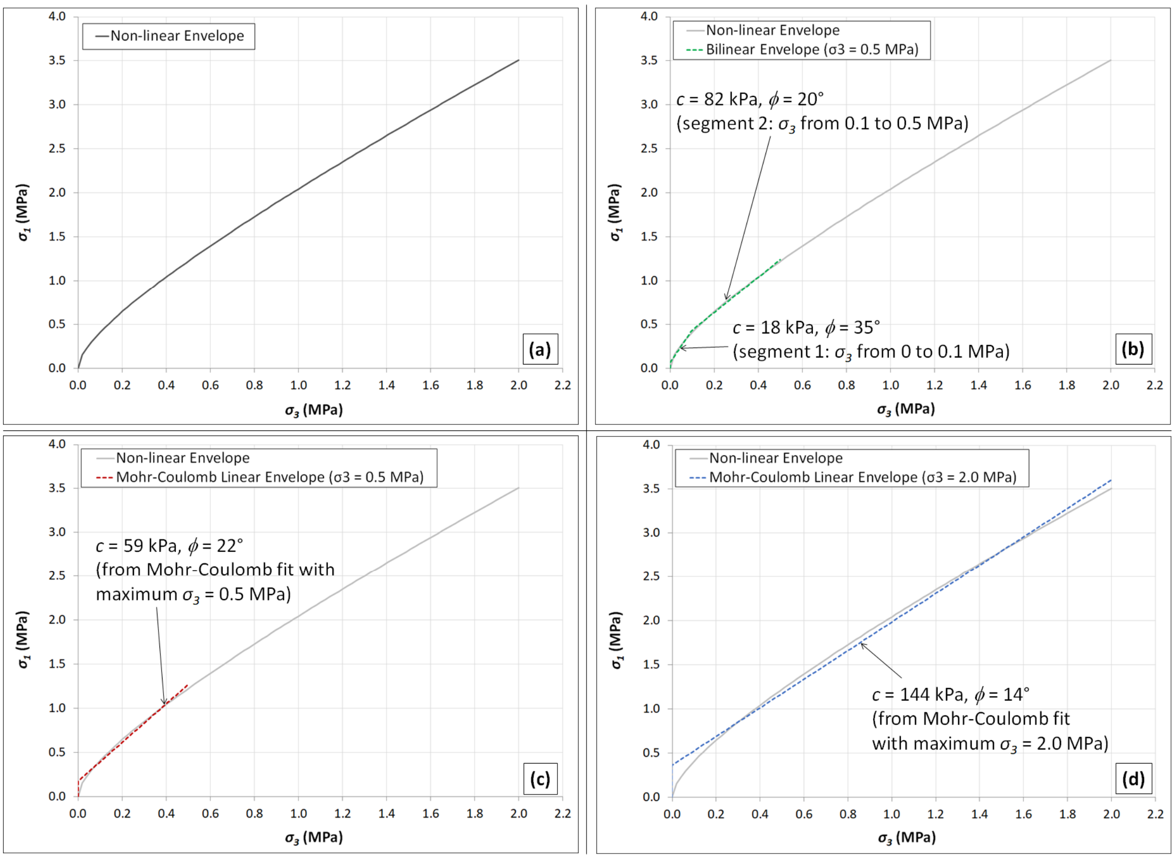

Figure 1 Principal stress charts showing: (a) Example non-linear strength envelope; (b) Bilinear envelope fitted to the non-linear envelope based on a maximum confining stress of 0.5 MPa; (c) Linear Mohr–Coulomb envelope fitted to the non-linear envelope based on a maximum confining stress of 0.5 MPa; (d) Linear Mohr–Coulomb envelope fitted to the non-linear envelope based on a maximum confining stress of 2.0 MPa

2 Case studies for mined slopes in weak rocks and soils

Three different case studies involving weak rock and soil slope stability are presented below, including:

- Case Study 1: an open pit mine where a 40 m high failure occurred in laterite materials.
- Case Study 2: an open pit mine where significant cracking developed behind the crest of an 85 m high weathered rock slope.
- Case Study 3: an open pit mine involving a relatively deep (up to approximately 100 m) sequence of detrital materials, including clay.

It should be noted that the authors and their current affiliations may not be directly associated to all case studies described in this paper.

The numerical analyses have been performed using *FLAC3D* (Itasca Consulting Group 2023) for each case study. For simplicity, only two-dimensional (2D) modelling results were discussed in this paper. Note that, in many cases, three-dimensional (3D) analyses are required to provide a rigorous assessment of slope stability, as illustrated by Wines (2016).

2.1 Case Study 1 (laterite slope)

Case Study 1 involves an open pit mine where slope failure occurred in the laterite materials as shown in Figure 2a. Failure occurred through the rock mass. The failure was relatively shallow and has an overall vertical failure height of approximately 40 m, affecting four 10 m high benches. Part of the *FLAC3D* model used to back-analyse the failure is provided in Figure 2b, which shows the pre-failure slope geometry and the approximate failure extents.

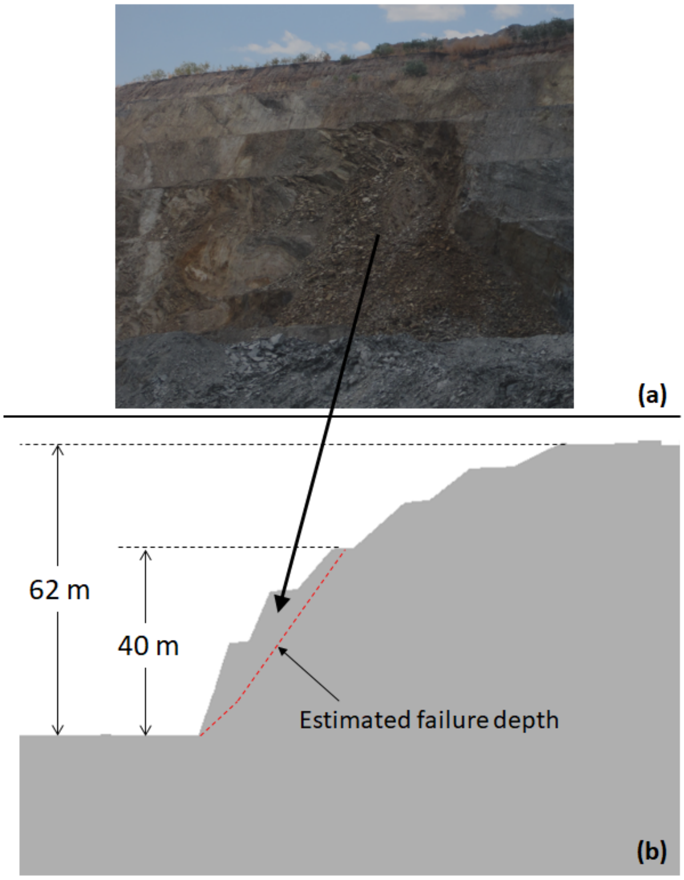

Figure 2 (a) Photograph of Case Study 1 laterite failure; (b) Pre-failure slope geometry and approximate failure extents

A non-linear strength envelope was developed based on all available information and back-analyses to represent the laterite material, together with three other strength envelopes derived using linear approximations. As described below, four separate *FLAC3D* models have been run to investigate the influence of the strength envelopes on the quality of the back-analysis:

- Model CS1_NL: a non-linear strength envelope.
- Model CS1_BL: a bilinear strength envelope (fitted to the non-linear envelope with a maximum confining stress of 0.5 MPa).
- Model CS1_L05: a linear Mohr–Coulomb strength envelope (fitted to the non-linear envelope with a maximum confining stress of 0.5 MPa).
- Model CS1_L10: a linear Mohr–Coulomb strength envelope (fitted to the non-linear envelope with a maximum confining stress of 1.0 MPa).

The principal stresses are monitored in the models during pit excavation at two locations, as shown in Figure 3 (labelled 'Stress Path CS1a' and 'Stress Path CS1b'). Strength reduction analyses have been run to estimate the Factor of Safety (FoS) for each model and the resulting FoS contours are presented in Figure 4. The resulting stress paths for each model are shown in relation to the adopted strength envelopes in Figure 5. Note that the stress paths monitor the stresses during pit excavation, but not during the strength reduction analyses. The following observations were made based on the analysis results:

- The most favourable recreation of the failure (see red contours, FoS < 1.0) is provided by Model CS1_NL (non-linear strength envelope) and Model CS1_BL (bilinear strength envelope). These models provide the most accurate representation of the strength at low confinement. Note that the failure was relatively shallow (less than around 12 m deep) and the confining stress in the failure zone after pit excavation was less than 0.2 MPa.
- The failure is not recreated by Model CS1_L10 (linear envelope with a maximum confining stress of 1.0 MPa), which produces an FoS of at least 1.2 in the failure zone. This is because the adopted strength is too high at low confinement, which is a result of the linear envelope being fitted to the non-linear curve based on an unsuitable maximum confinement that is too high (σ_3 = 1.0) for the expected failure mechanism. Consequently, the stress paths at the two monitoring points do not intersect the linear strength envelope in this model (see Figure 5).
- Failure occurs in Model CS1_L05 (linear envelope with a maximum confining stress of 0.5 MPa) however the failure extents (including depth) are significantly greater than the observed failure. This is because the linear envelope passes below the non-linear curve around the location of the deeper monitoring point ('CS1b'), potentially underestimating material strength at this depth (Figure 5).
- Note that slope failure is not predicted by any of the models until the final 10 m bench is excavated. The slope design steepens in the lower benches, with a 20 m high double-bench being included at the base of the slope. This indicates that the relatively weak laterite extended deeper than expected at the failure location and the pit design was based on the assumption that stronger materials would be exposed in the lower benches.

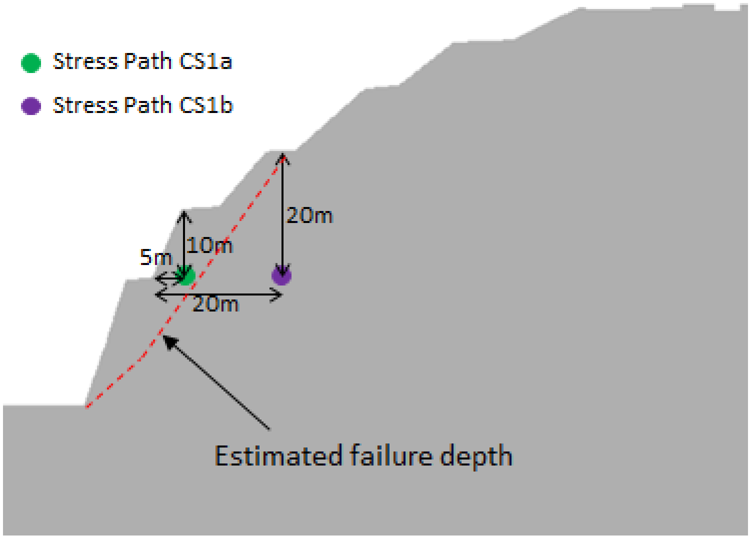

Figure 3 ***FLAC3D* stress path monitoring points for Case Study 1**

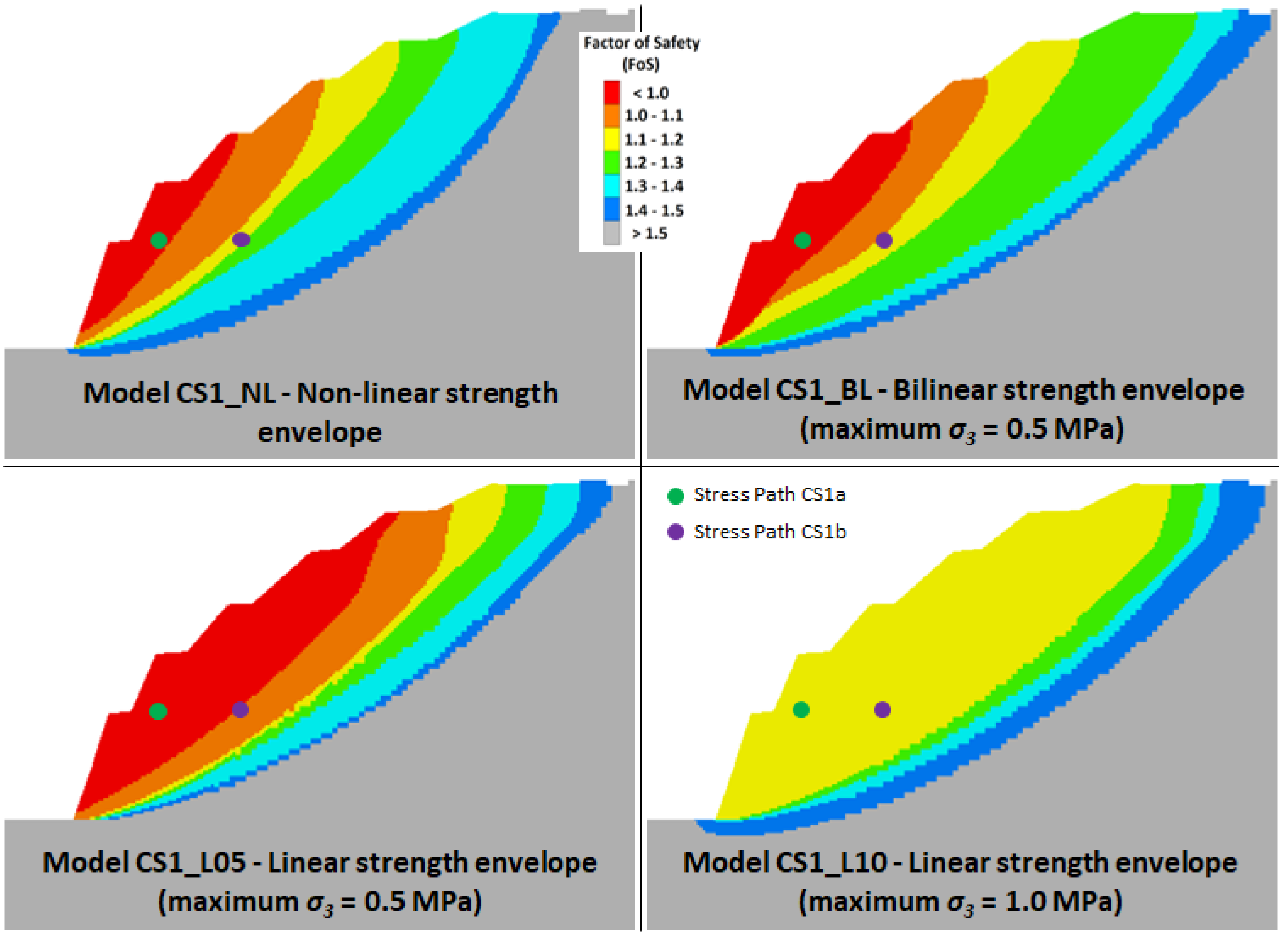

Figure 4 **Factor of Safety contours produced by each of the four *FLAC3D* models for Case Study 1**

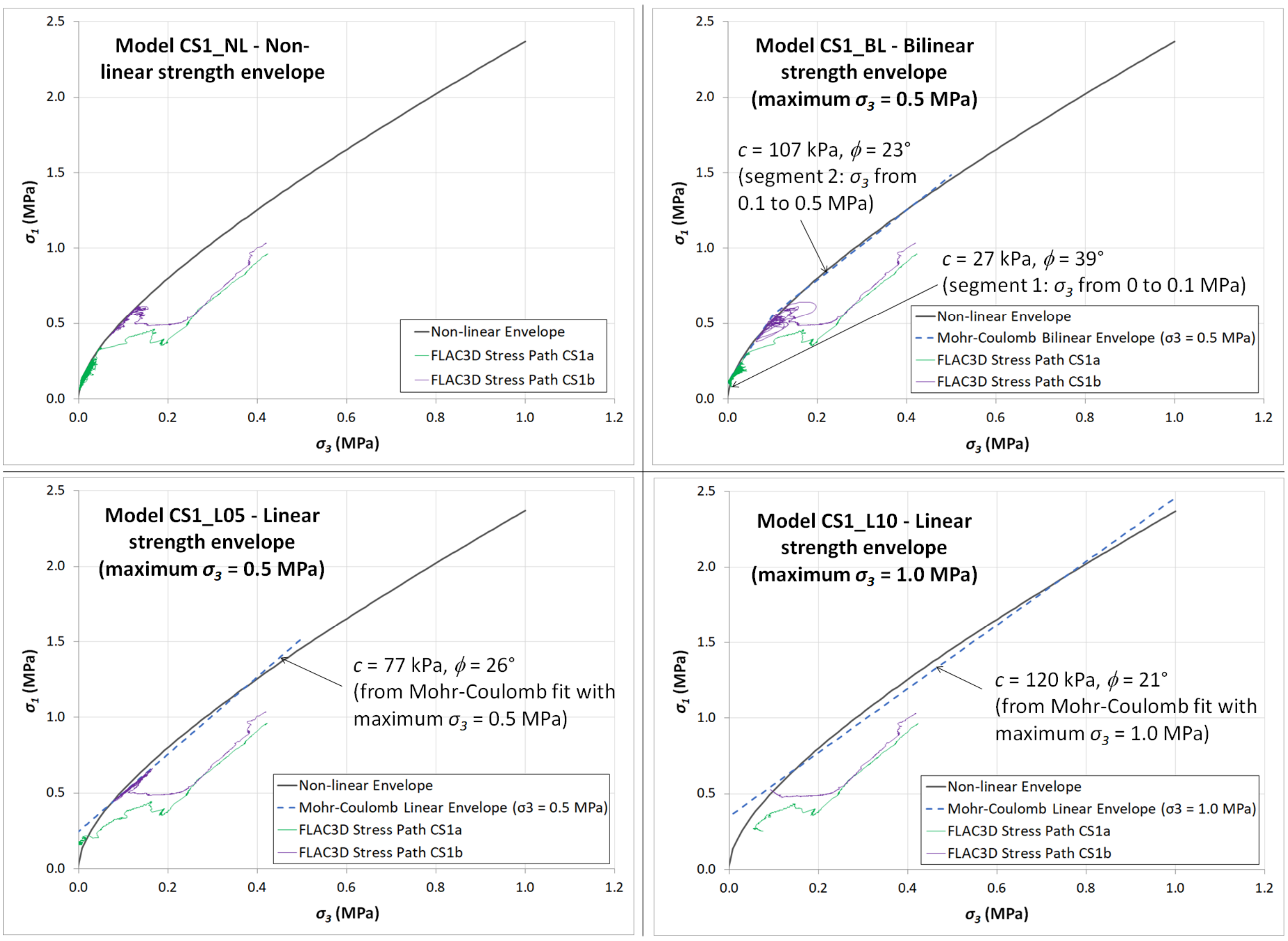

Figure 5 Strength envelopes and *FLAC3D* stress paths for each of the four Case Study 1 *FLAC3D* models. The stress paths monitor the stresses during pit excavation, but not during the strength reduction analyses

2.2 Case Study 2 (weathered rock slope)

Case Study 2 involves an open pit mine where cracking occurred behind an 85 m high weathered rock slope, as shown in Figure 6. The assessed failure mechanism is rock mass failure through the weathered rock. A non-linear strength envelope was developed based on all available information and back-analyses to represent the weathered rock material. As described below, four separate *FLAC3D* models have been run to investigate the influence of strength envelopes on the quality of the back-analysis:

- Model CS2_NL: a non-linear strength envelope.
- Model CS2_BL: a bilinear strength envelope (fitted to the non-linear envelope with a maximum confining stress of 0.5 MPa).
- Model CS2_L20: a linear Mohr–Coulomb strength envelope (fitted to the non-linear envelope with a maximum confining stress of 2.0 MPa).
- Model CS2_L50: a linear Mohr–Coulomb strength envelope (fitted to the non-linear envelope with a maximum confining stress of 5.0 MPa).

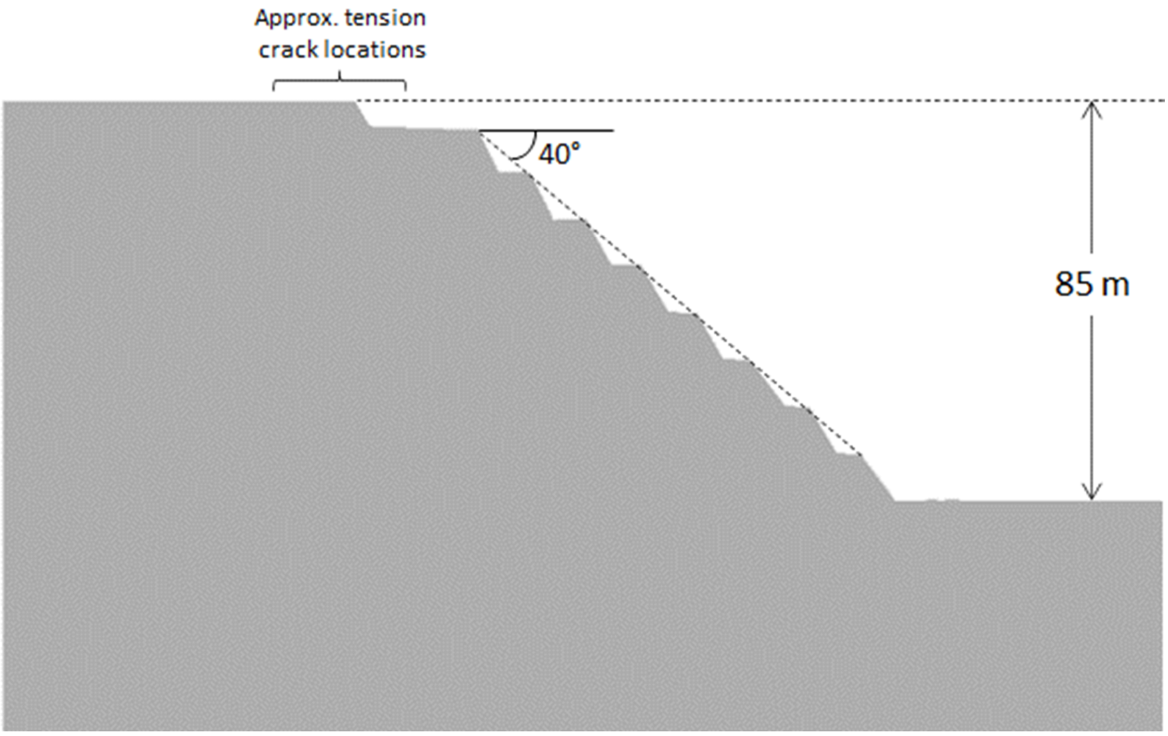

Figure 6 Case Study 2 slope geometry

The principal stresses are monitored in the models during pit excavation at two locations, as shown in Figure 7 (labelled 'Stress Path CS2a' and 'Stress Path CS2b'). Strength reduction analyses have been run to estimate the FoS for each model and the resulting FoS contours are presented in Figure 8. The resulting stress paths for each model are shown in relation to the adopted strength envelopes in Figure 9. Note that the stress paths monitor the stresses during pit excavation, but not during the strength reduction analyses. The following observations were made based on the analysis results:

- Given the extensive cracking behind the pit crest, it is likely that the FoS for the slope is only marginally above unity. Based on this assumption, the most plausible recreation of the slope behaviour is likely provided by Model CS2_NL (non-linear strength envelope) and Model CS2_BL (bilinear strength envelope), both of which produced a FoS of between 1.0 and 1.1. Horizontal displacement and strain contours from Model CS2_NL are presented in Figures 10a and 10b, respectively. Movement into the excavation was predicted and elevated strains were predicted at the location of the observed tension cracks.
- The FoS produced by Model CS2_L20 (linear envelope with a maximum confining stress of 2.0 MPa) is between 1.1 and 1.2. In this case, the adopted strength envelope is above the non-linear envelope at both stress path monitoring points (Figure 9).
- The FoS produced by Model CS2_L50 (linear envelope with a maximum confining stress of 5.0 MPa) is above 1.4. The adopted strength envelope is significantly above the non-linear envelope at the confinement levels that are relevant for this slope (see Figure 9). This is a result of inappropriate linear approximation of the non-linear material strength behaviour using a maximum confinement that is too high (σ_3 = 5.0) for the potential failure(s) being analysed. Note that historic triaxial testing for this project were undertaken using relatively high confining pressures (i.e. minimum 5 MPa).

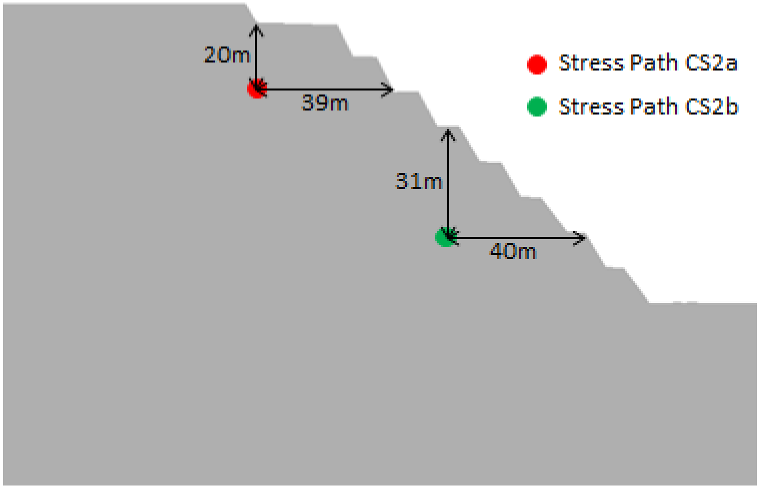

Figure 7 *FLAC3D* **stress path monitoring points for Case Study 2**

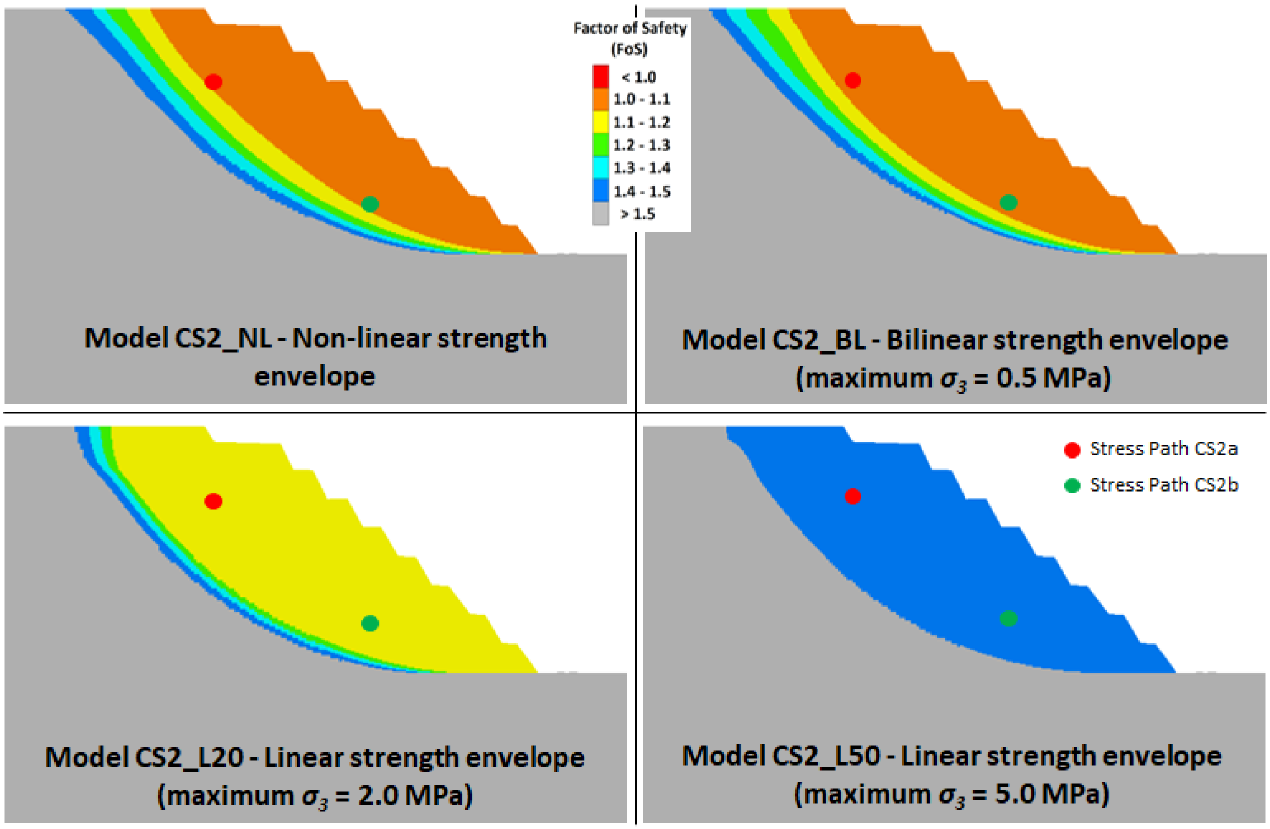

Figure 8 **Factor of Safety contours produced by each of the four** ***FLAC3D*** **models for Case Study 2**

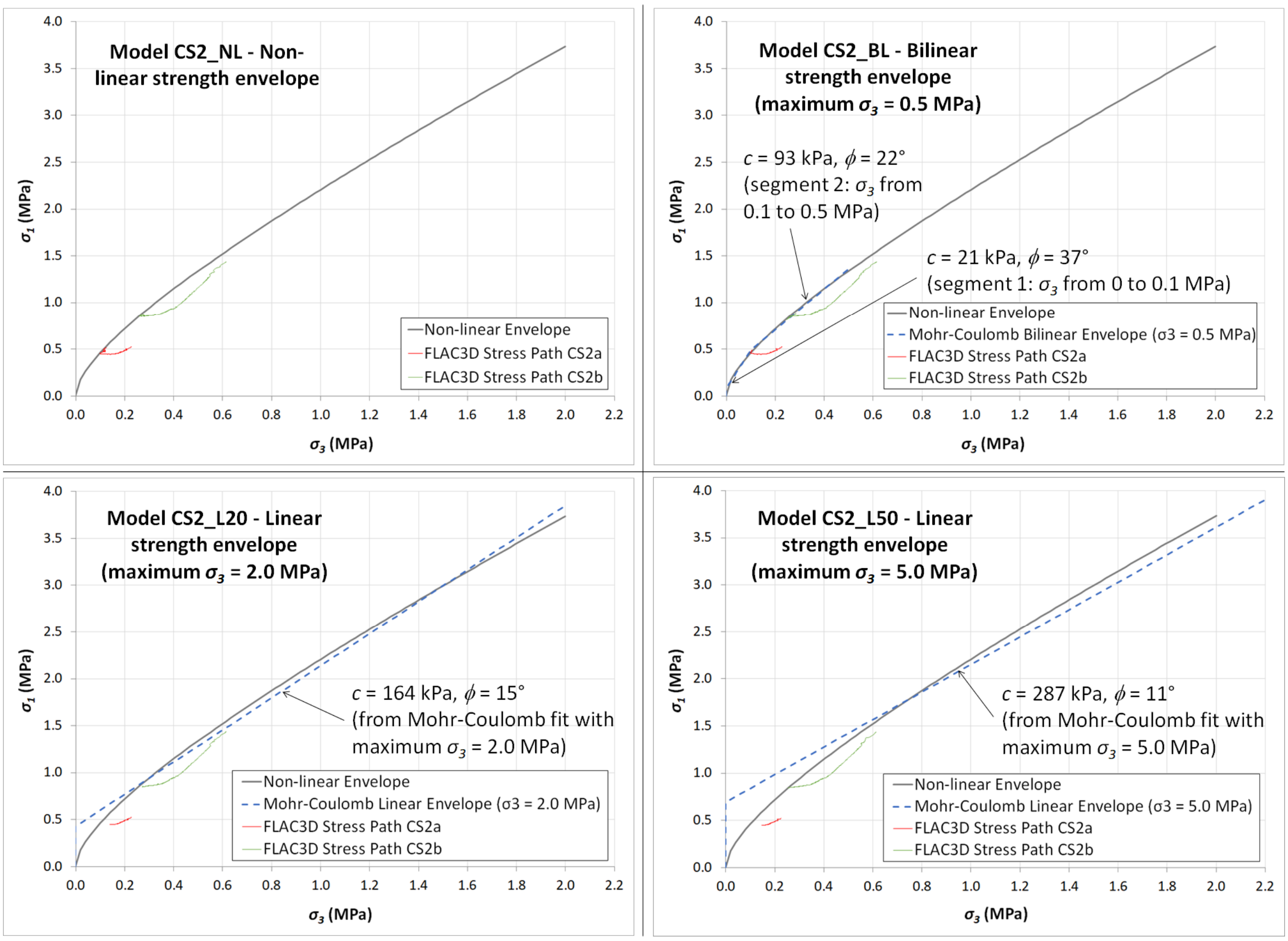

Figure 9 **Strength envelopes and *FLAC3D* stress paths for each of the four Case Study 2 *FLAC3D* models. The stress paths monitor the stresses during pit excavation, but not during the strength reduction analyses**

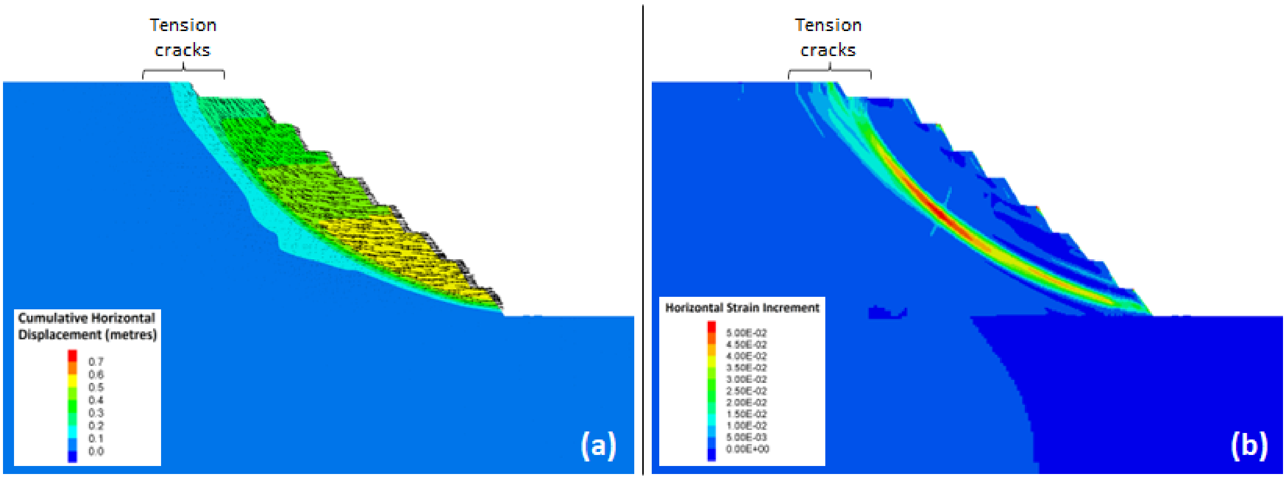

Figure 10 **Results from Case Study 2 *FLAC3D* Model CS2_NL (non-linear strength envelope). (a) Horizontal displacement contours and displacement vectors; (b) Horizontal strain increment**

2.3 Case Study 3 (detrital slope)

The Case Study 3 open pit mine involves a relatively deep (up to approximately 100 m) sequence of detrital materials, including a thick clay unit, underlain by bedded rock units. A photograph of a relevant part of the existing slopes is provided in Figure 11, where no sign of large-scale instability has been observed.

Triaxial testing indicates that the clay strength behaviour is highly non-linear, especially under low confining stresses (as shown in Figure 12). Several phases of slope stability analyses have been performed for the slope to provide a historic match of current operational observations, including the following:

- Model CS3_L: with the clays represented using a linear Mohr–Coulomb strength envelope fitted to the triaxial data.
- Model CS3_NL: with the clays represented using non-linear strength envelopes fitted to the triaxial data (including both strength and modulus softening, with the strength able to reduce from upper to central and then to lower non-linear envelopes based on strain development, shown in Figure 13).

Figure 11 Photograph showing existing slopes in Case Study 3 open pit

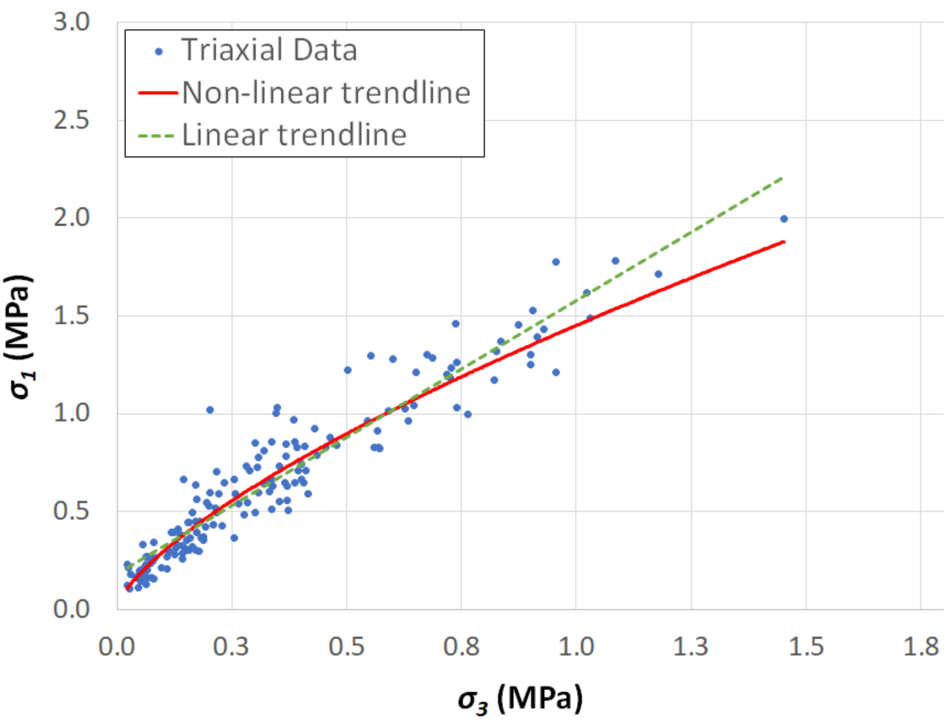

Figure 12 Triaxial test data for detrital clay in Case Study 3 slopes, including linear and non-linear trendlines

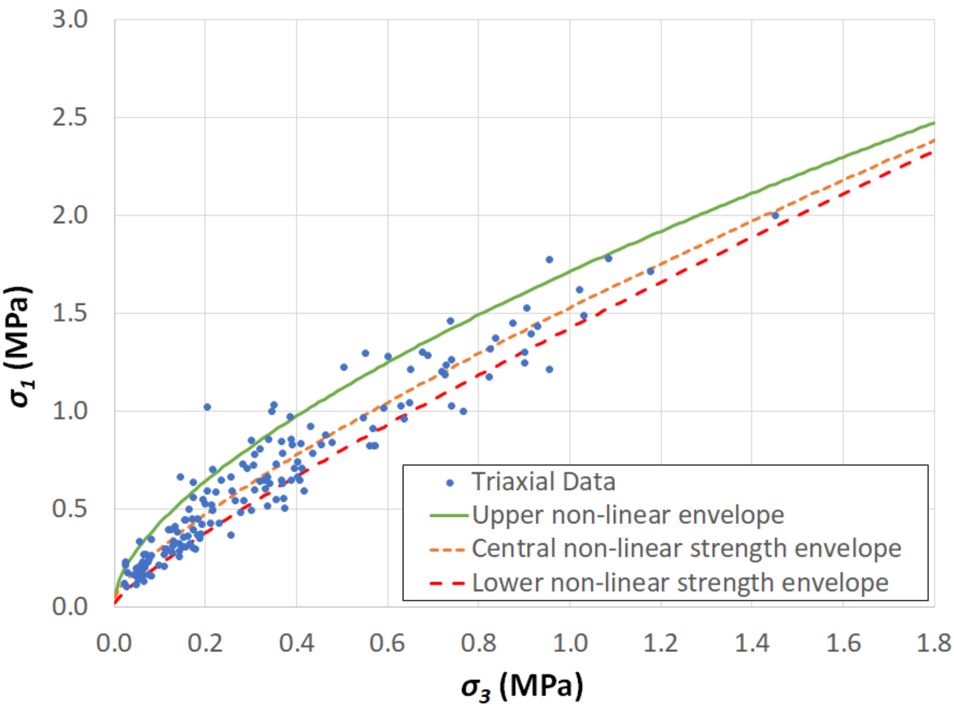

Figure 13 Non-linear strength envelopes adopted for *FLAC3D* model CS3_NL

Results from the two different *FLAC3D* models are presented in Figure 14. If a linear Mohr–Coulomb strength envelope is fitted through the triaxial data for the clay (Model CS3_L), a multi-bench failure is predicted in this material, which was not observed in the pit. Model CS4_NL provides an improved representation of the slope behaviour. Shallow and localised bench-scale instabilities were predicted in the clays, which is consistent with pit observations. A favourable correlation to the historic monitoring data (i.e. prisms and inclinometers) is also achieved by this model, as shown in Figure 15. Note that each instrument was monitored for different excavation stages, and results from only one selected time step is shown against the corresponding inclinometer data, for the purpose of demonstration.

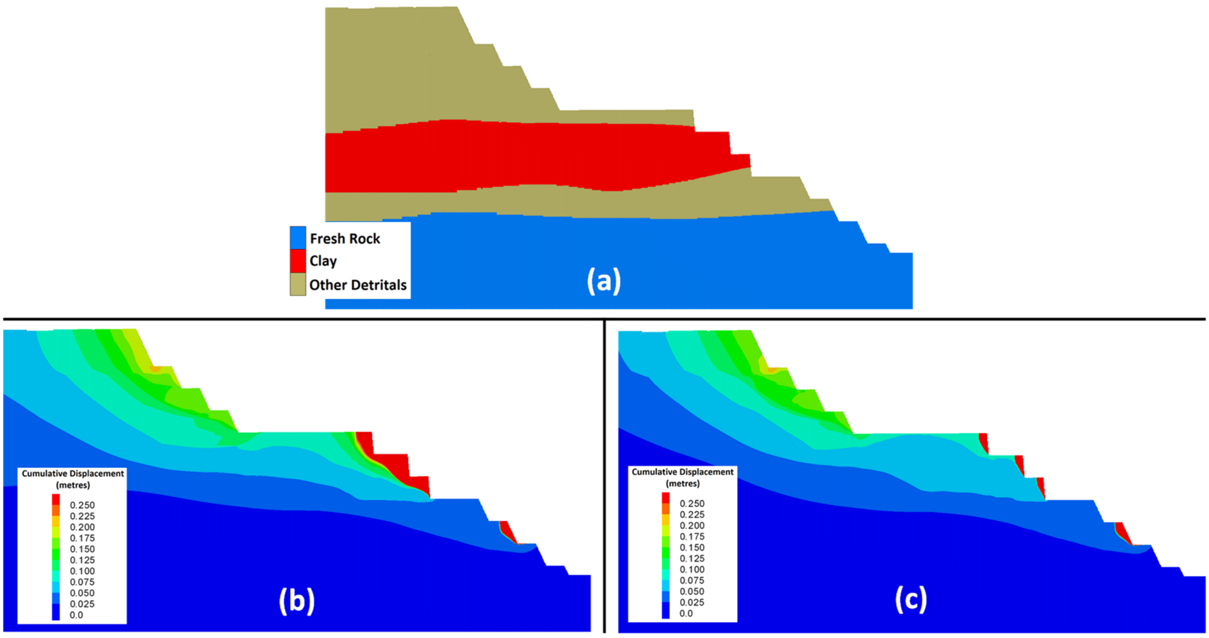

Figure 14 *FLAC3D* model plots for Case Study 3 slopes. (a) Domains; (b) Cumulative horizontal displacement contours from Model CS3_L (linear Mohr–Coulomb strength envelope); (c) Cumulative horizontal displacement contours from Model CS3_NL (non-linear strength envelope)

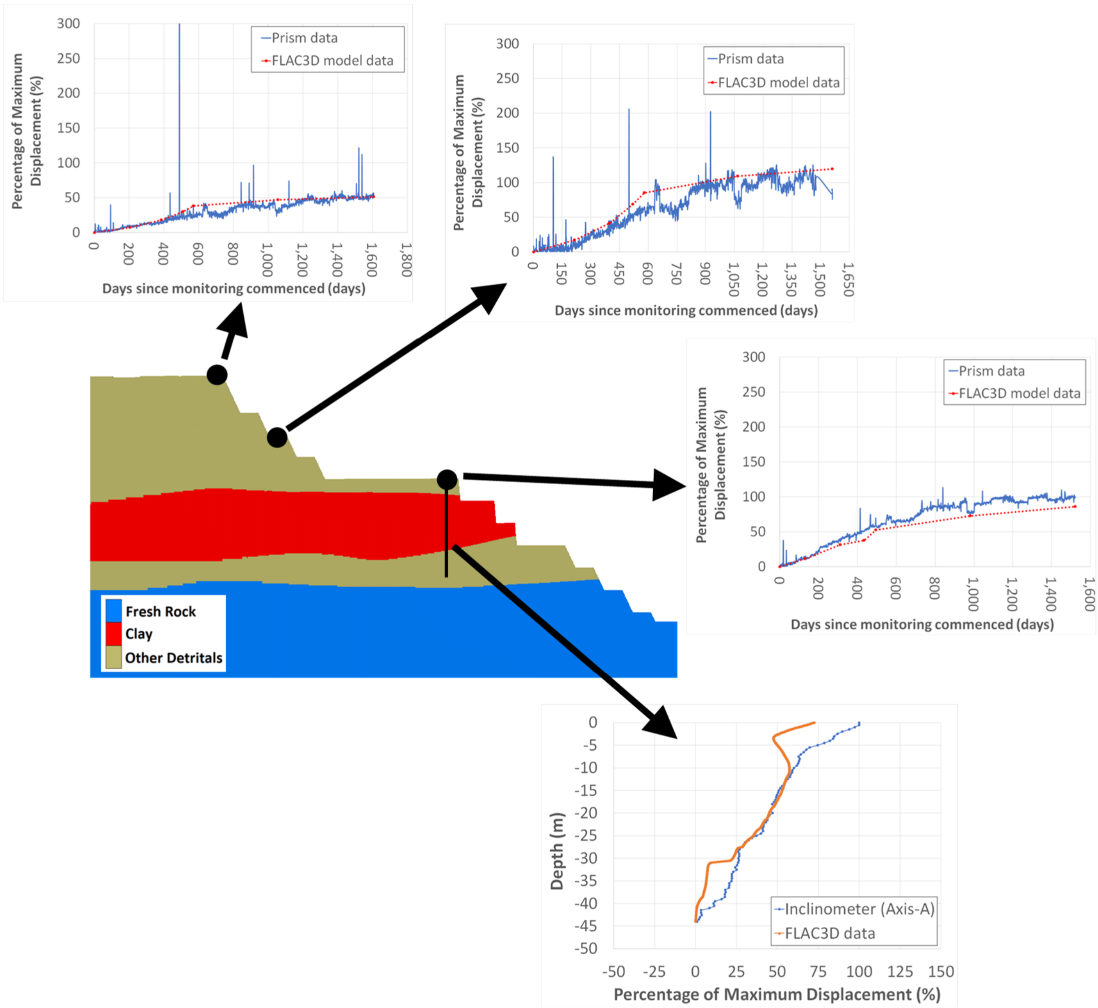

Figure 15 Comparison between monitored displacements and *FLAC3D* model displacements for three prisms and one inclinometer, Model CS3_NL (non-linear strength envelopes)

3 Conclusion

Selection of appropriate strength envelopes for slope stability analyses involving soils and weak rocks can have a significant influence on the accuracy of the analysis results. Whilst non-linear strength envelopes have been commonly adopted for rocks (such as the Hoek–Brown criterion), laboratory testing have indicated that soil strength envelopes are often also non-linear. Where Mohr–Coulomb envelopes are fitted to a non-linear curve, careful consideration should be given to the confining stresses over which this linear approximation is undertaken.

Numerical modelling results from three case studies are presented in this paper. The mechanisms being analysed are assumed to involve failure through the soil or rock mass only with no structural component (i.e. structures such as fissures, joints and faults are not explicitly included in the analyses). The results indicate that the non-linearity of the adopted strength envelope and the confining stresses used to estimate Mohr–Coulomb envelopes for linear approximation can significantly influence the results (including the FoS), and in some cases, have direct impact on operational safety. This is consistent with the work of others (e.g. Renani & Martin 2020).

The potential depth of failures should be considered when selecting appropriate confinement levels and thought should be given to using different confinements when estimating the properties for different geotechnical domains. For example, the potential failure depths (and therefore confinement) for a 15 m deep soil layer encountered at the pit crest will be different to potential failure depths and confinement for a 100 m deep weathered rock unit. Latapie & Lochaden (2016) discussed different options for selecting the maximum confining pressure when fitting Mohr–Coulomb envelopes to Hoek–Brown curves and state that it is preferable to consider the ranges of confining pressures specific to each rock layer being analysed. Renani & Martin (2020) indicate that the appropriate range of confinement is primarily controlled by the slope geometry.

For a deep exposure of a particular domain, both shallow and deep-seated failure modes may need to be assessed. If a linear fit is adopted over a large range of confining stresses, the adopted strength can be too low at some confinement levels and too high at other confinement levels, making it impossible to accurately represent material strength behaviour at all failure scales. Where possible, it is recommended that non-linear strength envelopes be adopted to provide a more accurate (and realistic) representation of the strength at different confining stresses. If this is not possible due, for example, to the analysis method being used, adopting a bilinear (instead of linear) envelope can provide an improved fit. It is noted that the depth of potential failures (and therefore the relevant confinement levels) will often be relatively shallow. For the analyses presented in this paper, the relevant confining stress is generally less than 0.5 MPa.

Consideration should also be given to other important aspects such as the confinement levels prescribed for laboratory testing. For example, triaxial tests should be undertaken at confining stresses that are relevant to potential failures in the material being tested. Lupo et al. (2015) recommend that yield stresses for the material being analysed should be considered when defining test parameters (such as confining pressures) prior to testing. Renani & Martin (2020) indicate that the confining stresses that are appropriate for the slope stability analyses should also be used in bounding the range of relevant stresses for laboratory testing.

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