



# Slope Stability Analysis of a Surface Portal: A Case Study of a Platinum Mine

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**Abstract.** The risk of portal slope collapses due to the inherent nature of semi to highly weathered soil profiles and discontinuous rock mass conditions near the surface is ever present. An optimum slope design for soil and rock formations is therefore critical to obviate major risks arising from slope collapses. This paper presents the use of a geotechnical model for portal slope stability analysis at a platinum mine. Drill core logging data was analysed using rock mass classification techniques and integrated with laboratory Uniaxial Compressive Strength (UCS) data to classify three geotechnical domains: the soft/transitional zone, moderately weathered zone and competent zone. Generated critical slip surfaces were relatively identical and isolated to the upper bench consisting of relatively weak materials. Limit Equilibrium (LE) techniques were applied for stability analysis using Slide 2D software. The Shear Strength Reduction (SSR) technique based on finite element (FE) method was employed as a verification to enable comparison of the factor of safety (FS) with the Strength Reduction Factor (SRF) values from RS2 software. A sensitivity analysis performed to assess the effects of varying slope angle with a corresponding probability of failure (PF) yielded an optimum slope angle range of 35-40°. The obtained FS and SRF values were above the 1.3 acceptance criterion for slope angles between 35-40°. The findings demonstrated that optimization of soil-rock portal slope geometries can stabilize formations thereby mitigating slope failure and the consequential hazards. Adverse joint orientation and basic friction angle were also proven to have a considerable influence on kinematically-controlled failures based on analysis done using Dips software.

**Keywords:** Slope stability analysis, slope design, geotechnical model, geotechnical domain.

## 1 Introduction

The stability of portal slopes forms an integral part of the mining process. Portal slopes are excavated as the surface entrance providing link to underground workings. Developing a slope portal frequently entails enormous financial outlays and as such, the design process should consider consequences of failure from the onset [Wesseloo & Read, 2009]. Ideally, development should be done in zones where the ground is competent and requires minimal support. However, due to the near-surface, inherent nature of semi to highly weathered soil profiles and discontinuous rock mass conditions, portals often pose ground control problems as they traverse weak geotechnical conditions [Puchner, 2009]. An optimum mining slope design for soil and rock formations is therefore critical to obviate major risks that arise from slope collapse.

## 2 Background

The study area is located on the southern part of the Great Dyke of Zimbabwe. Platinum Group Metals (PGM) mineralization occurs at shallow depths of ~120m. The orebody outcrops on the side of a hill and is variably oxidized up to a vertical depth of 40m. Main access to the orebody requires portal slope development which links to the boxcut and underground mine plan. The designed slope geometry traverses weathered materials, at the slope crest, but lies within moderately weathered gabbro-norites and competent host-rock pyroxenite (at toe). Faults of varying magnitudes truncate the orebody with prominent joint sets forming parallel to fault orientation. Site-specific data was used to construct the models considering geological and structural detail for both slope design and kinematic analysis.

## 3 Review of Concepts

### 3.1 Limit Equilibrium and Finite Element Methods

Slope stability problems are statically indeterminate hence the application of simplifying assumptions being made to arrive at a unique factor of safety [Abdulla et al., 2000]. Slope stability studies typically designate factor of safety (FS) and Probability of Failure (PoF) to define the stability of a slope and for this study, the FS was the primary design acceptance criteria applied.

A number of numerical models, including limit equilibrium (LE) and finite element (FE) methods have been developed for slope stability evaluation with both techniques making use of deterministic approaches for stability analysis of slopes. Despite several limitations, LE methods remain the most commonly used approaches in slope stability due to their simplicity and accuracy. LE techniques applied for stability analysis in this study included the Bishop Simplified method (BS), Janbu Simplified Method (JSM), Spencer Method (SM) and Morgenstern & Price (M-PM). These are typically rigorous methods which satisfy various failure shapes and slip surface conditions. Complete

equilibrium conditions are satisfied under the M-PM and SM. According to Wubalem [2022], the fundamental differences in their approaches are based on: i) statics equations and interslice forces included and satisfied and ii) assumed relationship between inter-slice shear and normal forces.

Recent advances in technology have seen finite element (FE) methods gaining increased usage due to the method's accuracy and computational ability in handling more complex problems including stress-strain behaviour of slopes [Matthews et al., 2014]. The FE method proffers the ability to model complex slopes and better visualize insitu soil deformations.

### 3.2 Slope Failure Mechanisms

These can be broadly classified into two; factors causing a decrease in shear strength and factors causing an increase in shear stress [Duncan and Wright, 2005]. Several causes of slope failures may exist simultaneously in most field cases. Although continuum failures exist in soils, weak and heavily fractured rocks, most failures are typically governed by discontinuities such as faults and joints [Duncan and Wright, 2005]. The shear strength of discontinuities mainly governs failure which may be developed as a single discontinuity, two discontinuities or intersecting combination of discontinuities [Duncan, 2014]. In rocks, orientation of discontinuities relative to the face and the number of sets of discontinuities is a key factor in slope stability. Conditions for discontinuity related failures include planar sliding, circular rotational failures, non-circular failures, wedge failures, flexural and direct toppling failures. .

A number of critical considerations govern optimum slope designs, and these include:

1. Virgin stress - a state of equilibrium under virgin stresses exists for typically undisturbed rock or soil masses. As excavations are made into the ground, stress redistributions occur [Indian Bureau of Mines, 2014].
2. Rock or soil mass quality parameters - Hoek and Brown strength criterion has been widely used for fractured rock masses given that most slope stability problems can be conveniently analysed in terms of shear and normal stresses.
3. Kinematic analysis - will assist to forecast structurally controlled, kinematically probable failures within the rockmass.
4. Other critical factors such as slope geometry, groundwater condition, shear strength parameters (angle of internal friction and soil cohesiveness), and soil unit weight are all crucial components of slope stability analysis [Wubalem, 2022].

## 4 Methods and Materials

### 4.1 Methodical Summary

An integrated approach involving onsite geotechnical investigations, laboratory UCS testing of rock samples, numerical modelling and kinematic analysis using Dips software was applied in this study. Vertical and inclined boreholes were drilled around the portal area. The oriented core was analysed, logged and borehole data grouped into geotechnical domains to characterise areas with similar material properties, strength parameters and hydrogeological factors:

- Soft/transitional zone – consisting of overburden loose topsoils and severely weathered gabbro/norite exposures,
- Moderately weathered – grading of transitional material into moderately weathered gabbro and
- Competent zone – competent rock exposures which are moderately jointed.

For limit equilibrium analysis, Slide 2D® software was used to obtain the FS. Shear strength reduction (SSR) technique based on finite-element (FE) method was employed as a verification to enable FS comparison with the obtained Strength Reduction Factor (SRF) values from RS2®. Kinematic analysis using Dips software was performed to evaluate potential for wedge, planar and toppling failure mechanisms based on structural data obtained from re-oriented core.

Laboratory Uniaxial Compressive Strength (UCS) and UCS with Elastic Moduli (UCM) tests allowed for compressive strength, elastic modulus, unit weight ( $\gamma$ ) and Poisson’s Ratio ( $\nu$ ) determination which formed material parameters into the slope design. Empirical estimation of cohesion ( $c$ ) and friction angle ( $\phi$ ) involved use of the Mohr-Coulomb strength model and Barton’s relationship.

The general flow chart towards formulation of the geotechnical model is shown in Fig. 1 and material properties used in the slope designs are presented in Table 1.

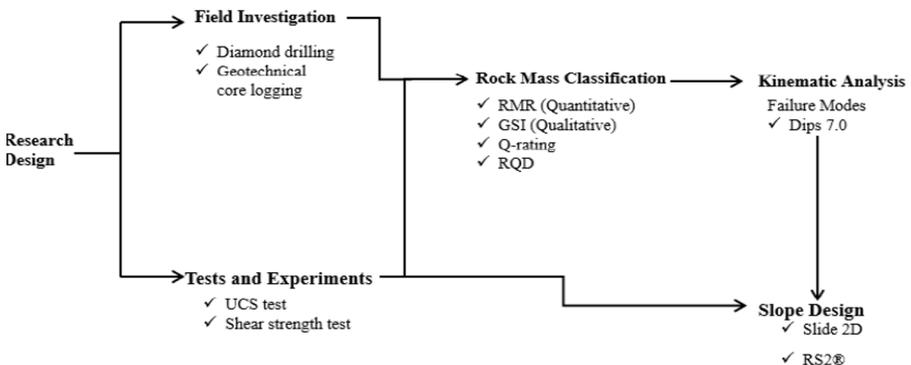


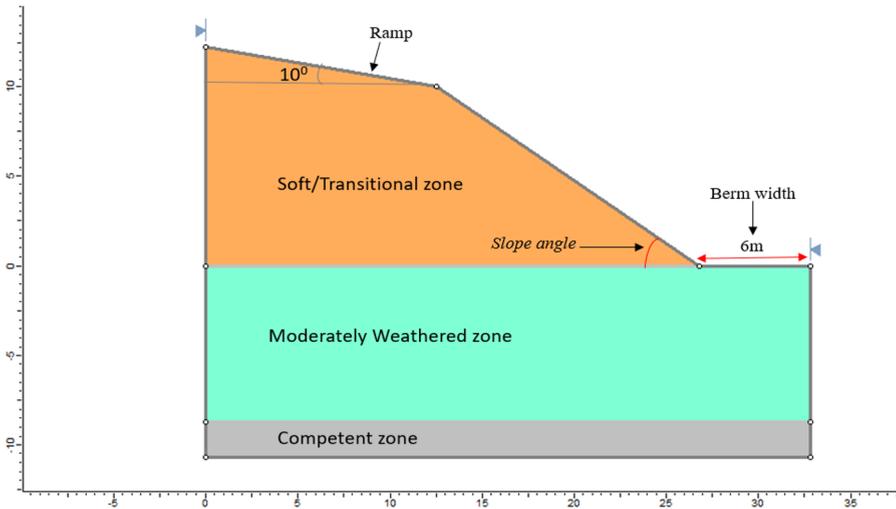
Fig. 1. Process flow towards geotechnical model and slope design

**Table 1.** Slope input parameters

Geotechnical Zone	Colour Code	Unit Weight $\lambda$ (kN/m <sup>3</sup> )	Saturated Unit Weight $\lambda$ (kN/m <sup>3</sup> )	Strength Type	Cohesion (Mpa)	$\Phi$ (deg)
Soft/Transitional		17.2	18.5	Mohr-Coulomb	0.01	29.6
Mod. Weathered		23	23.5	Mohr-Coulomb	18	26
Competent Zone		25		Mohr-Coulomb	25	31.5

## 4.2 Slope Geometry and Input Data

The idealized simulated slope geometry included a 12.2m slope height which incorporated a 10° ramp at the top bench (see Fig. 2). All three geotechnical domains were incorporated with the upper bench constituting mainly soft/transitional materials. The soils were modelled utilizing the Mohr-Coulomb (M-C) yield criterion under both dry and wet conditions. Given that the portal slope provides main access to an underground orebody, failure consequences would be high. A minimum FS of 1.3 or PoF 5% was therefore selected as acceptance criteria based on Wesseloo & Read's [2009] recommendations.

**Fig. 2.** Portal slope geometry

## 5 Results and Analysis

### 5.1 Rock Mass Quality Results

A statistical analysis of the Q-index, Rock Mass Rating (RMR), Geological Strength Index (GSI) and Rock Quality Designation (RQD) was conducted for the portal boreholes. The statistical summary included the arithmetic mean, minimum, maximum,

percentiles (25<sup>th</sup>, 50<sup>th</sup>, and 75<sup>th</sup>) and standard deviation for each quality parameter. The mean distribution of geotechnical parameters was used to indicate rock quality within each geotechnical domain.

In the soft/transitional domain, RMR was 37% and classified as poor rock while obtained RQD value of 50.4% marginally classified in the region between poor to fair rock . Computed Q-index value of 1.21 was categorised as poor rock mass. In contrast, the competent zone core yielded Q-22.85, RMR-68% and RQD -87.6% which all indicated relatively competent rockmass. The mean rock mass quality results are presented in Fig. 3.

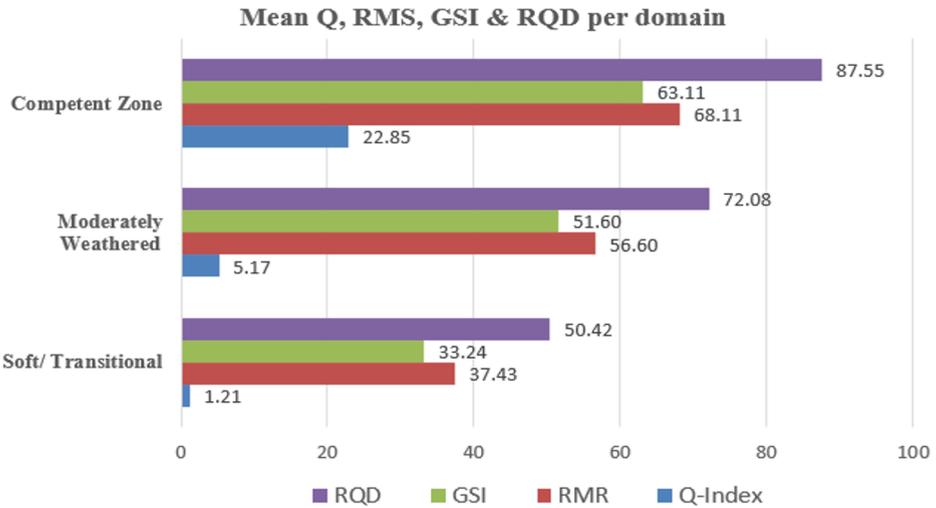


Fig. 3. Mean Q, RMR, GSI and RQD per geotechnical domain

Rock mass quality results depicted an increase in rockmass strength with increasing depth. Potential instability problems are anticipated in the upper areas characterized by weak slope-forming material.

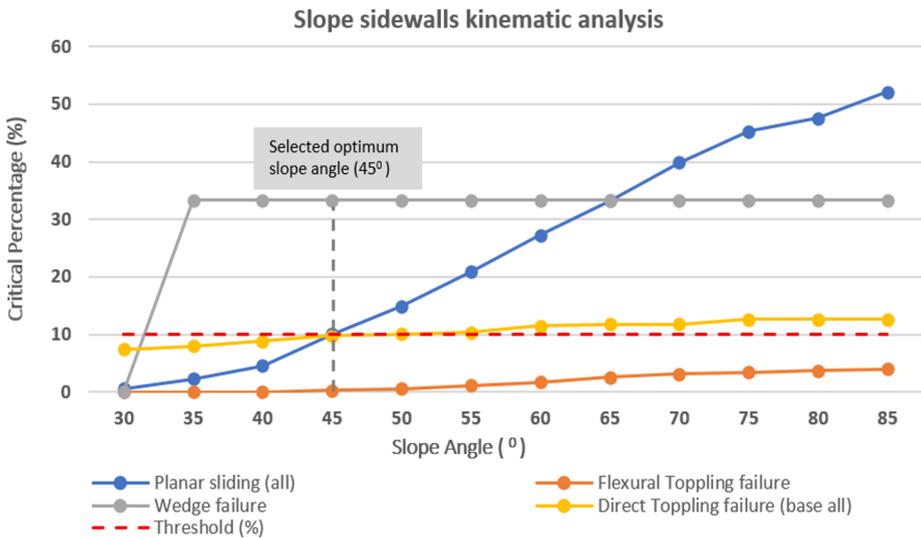
### 5.2 Kinematic Analysis

The stability of slopes comprising moderately weathered material is influenced by the major discontinuity orientations and condition in relation to the pit wall geometry. Dips software was used to produce stereographic projections which allowed for the analysis of potential failure modes that occur due to unfavourable wall geometry and discontinuity orientation; namely planar failure, wedge failure and toppling failure (direct and flexural). Drilling dataset was used for analysis and average spacing of the discontinuity sets ranged between 0.35 - 0.69 m. The friction angle ( $\phi$ ) for joint surfaces was estimated using the Barton [2002] relationship:

$$\varphi = \tan^{-1} \frac{J_r}{J_a} \tag{1}$$

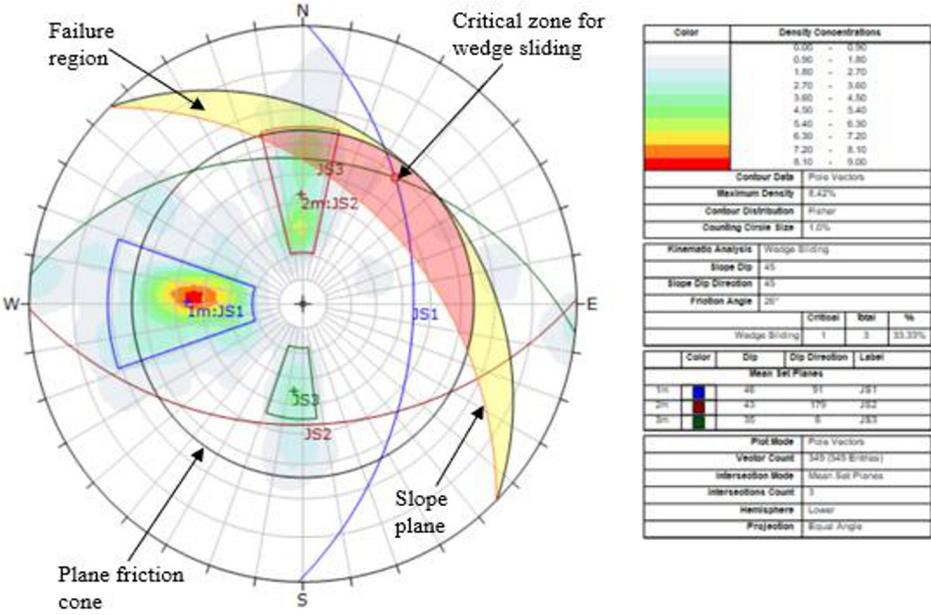
where  $J_r$  and  $J_a$  represent joint roughness and joint alteration numbers respectively. Friction angles were estimated per domain and a conservative lower mean friction angle of  $26^\circ$  was selected from transitional zone domain.

**The sensitivity analysis of slope angle on kinematic admissibility of failure.** A sensitivity analysis was conducted to quantify the likelihood of joints intersecting with increased slope angles. The analysis involved plotting critical percentage against varied slope angle to establish optimum slope angle as illustrated in Fig. 4. The frequency of data points which plot in the critical zone of the analyses was expressed as a percentage of the total number of data points analysed. A conservative probability cut-off of 10% was adopted. The optimum slope angle range yielding tolerable risk levels for all failure mechanisms except wedge failure mode was between  $35\text{--}40^\circ$ .

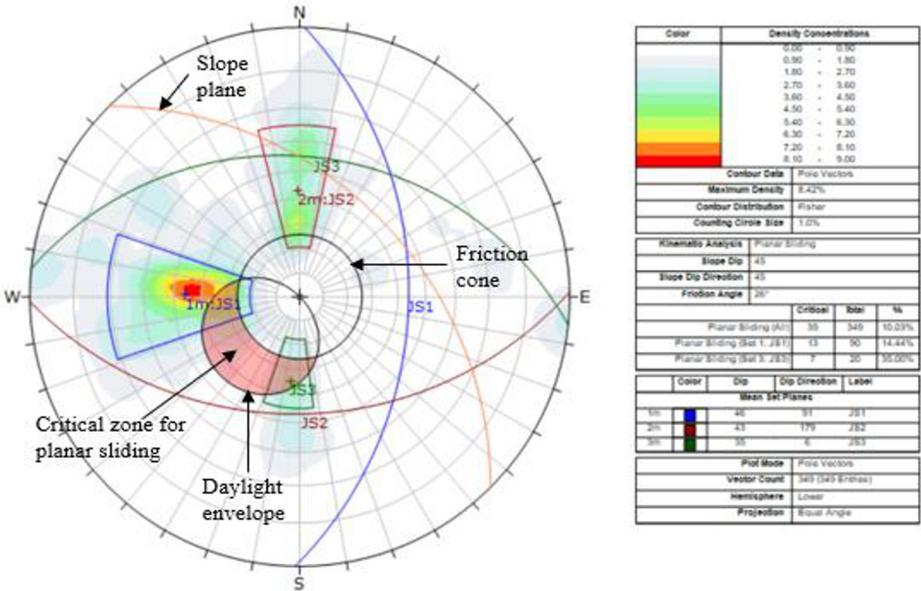


**Fig. 4.** Sensitivity analysis showing effects of slope angle on probability of failure

**Stereoplots.** Conditions for wedge formations were satisfied due to intersection of two planes (JS1-blue & JS2 - red) striking obliquely across the slope face (Fig. 5). The line of intersection was greater than the internal friction angle and daylighted into the slope face. On planar failure pole vector mode, the red shaded zone defines the critical zone for planar sliding where it falls outside the friction cone but inside the daylight envelope.



(a)



(b)

Fig. 5. Stereonets of (a) wedge sliding and (b) planar sliding kinematic analysis

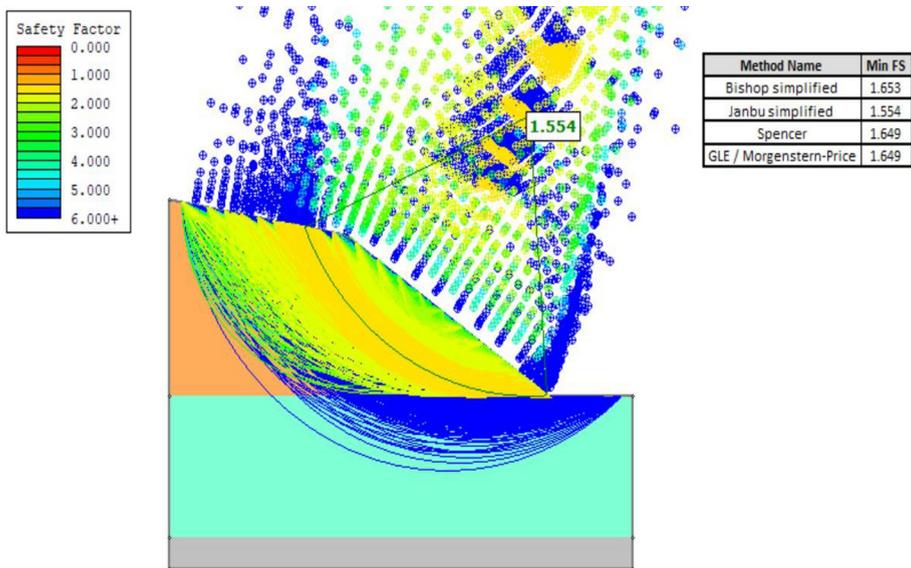
Poles which plot within the red shaded area represent critical zones for wedge and planar sliding risk. Any poles falling within this envelope are kinematically free to slide if

frictionally unstable. For JS3 (circled in green), there is 35% likelihood of planar sliding with sliding risk being significantly lower along JS1 (-14%) where there are fewer critical joint planes.

Flexural and direct toppling failures modes were also analysed in this study. Toppling failure mechanism involves column or block rotation about a fixed base and conditions are satisfied when joint sets dip in the opposite direction to the slope, usually by angles  $>75^\circ$ . Flexural toppling failure risk observed was low at  $<1\%$ . However, the analysis showed direct toppling risk at 4.4% and 25% occurring along JS1 and JS3 planes respectively.

### 5.3 Numerical evaluation of portal slope stability

**Limit Equilibrium Analysis.** LE analysis was performed using Slide 2D software. Models for slope angles at  $35^\circ$  and  $45^\circ$  showing generated circular failure surfaces and corresponding safety factors (FS) are shown in Fig. 6. The obtained FS for  $35^\circ$  slope angle simulation was 1.55, indicating a relatively stable slope design. Analysis at  $45^\circ$  slope angle yielded FS of 1.28, which fell marginally below the acceptable 1.3 FS recommended for cases where slope failure consequences are high.



(a)

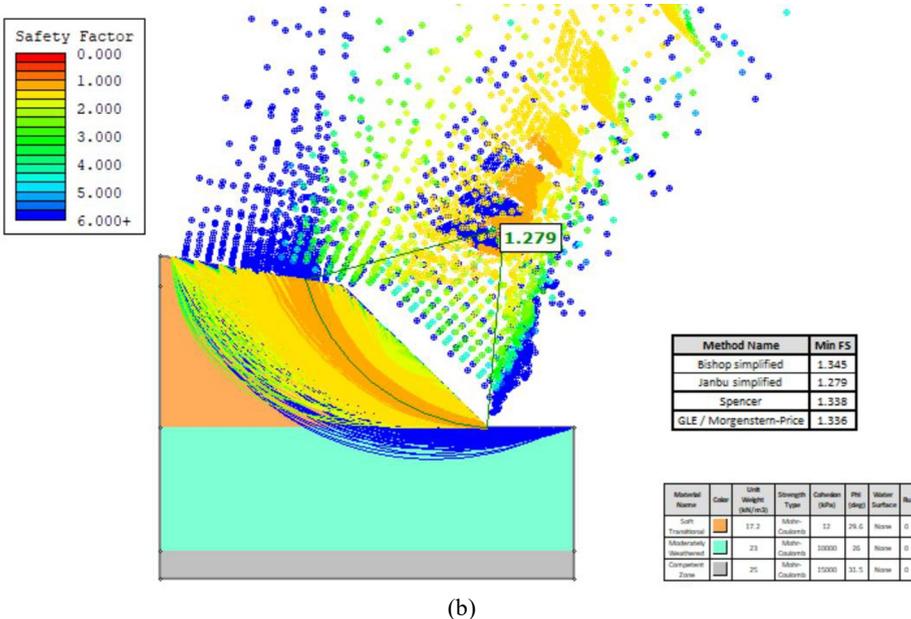
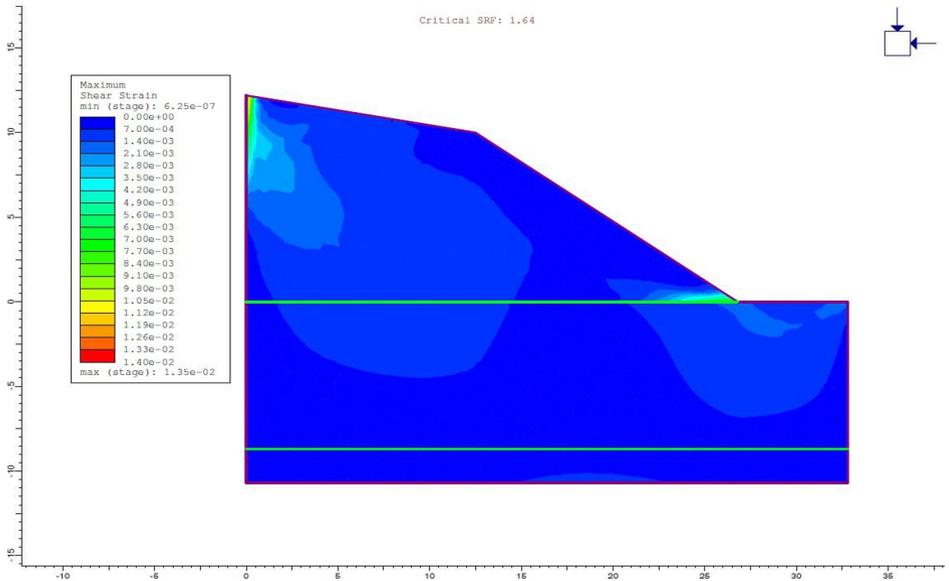
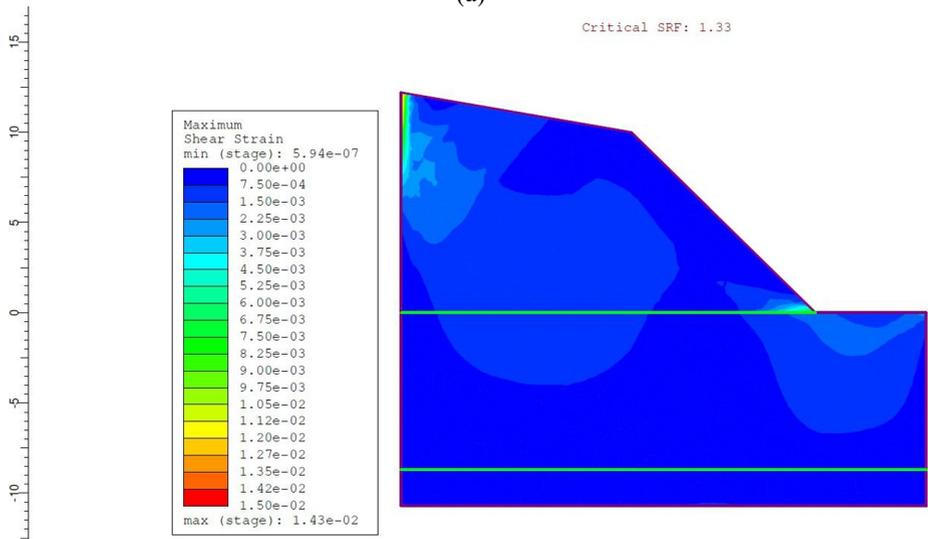


Fig. 6. Factor of Safety for Slide 2D models at slope angles of (a) 35° and (b) 45°

**Finite Element Analysis.** FE analysis was performed to determine the critical SRF and maximum shear strain. Mohr Coulomb failure criteria was adopted for checking stability under plain strain conditions. The critical SRF obtained for the slope angle of 35° was 1.64 with the maximum shear strain at 5.60mm. Simulations done at 45° slope angle yielded SRF of 1.33 which marginally met the acceptance criteria, and maximum shear strain of 5.25mm. The results were comparable to FS obtained from all LE except for JSM which generally yields lower FS values. The highest shear strain occurred at the toe of the slope under both modelled scenarios. This zone denotes boundary between the transitional and moderately weathered zones. Contours of maximum shear strain and corresponding SRF values from RS2 simulations are shown in Fig. 7.



(a)



(b)

**Fig. 7.** Critical Strength Reduction factor for RS2 at slope angles (a)  $35^\circ$  and (b)  $45^\circ$  showing Maximum Shear Strain

## 6 Conclusion

The following conclusions were developed based on results presented in this project:

1. Comparing and evaluating obtained results between LE and FE shows a very close correlation between the two techniques. The factor of safety results were indistinguishable with a variance of  $\pm 1\%$ .
2. However, generated critical slip surfaces showed slight variations. This has been attributable to LE methods predefining arbitrary selection of the search area and shape of potential failure surfaces prior to analysis. This can result in oversight on critical slope failure surfaces. FE analysis computes FS for each element along the critical shear surface which makes the technique more accurate.
3. The JSM yielded comparatively lower FS values (7.25% variance) compared to the FE and other LE approaches due to the method's sensitivity to interslice shear forces.
4. For slope angles between  $35\text{--}40^\circ$ , all FS and SRF values were above the minimum 1.3 threshold. The slope angle of  $45^\circ$  yielding FS of 1.279 is not recommended for design.

The results show that LE produce acceptable results in conventional slope stability analysis. Simulating more cases of complex slopes under various loading conditions can assist in establishing an empirical relationship between FE and LE methods. Overall, although this work only considered Mohr-Coulomb failure criterion for the slope model material, further studies should include other applicable material models incorporated in RS2 such as Generalized Hoek-Brown.

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