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THE INFLUENCE OF ROCK SLOPE SCALES OF JOINT NETWORK AND SLOPE HEIGHT ON THE STABILITY AND FAILURE MECHANISMS

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ABSTRACT

This paper demonstrates the influence of slope geometry and scale of the joint networks on the stability and failure mechanisms in discontinued rock masses. These analyses are based on the finite element method (FEM) to calculate the factor of safety (FOS) and to simulate the explicit discontinuity deformations for different scales of rock masses. The study was carried out through examples of regular jointed models having different slope heights and irregular jointed dry rock slopes. Joints usually decrease the strength of the slopes and suggestively increase slope stability problems. The result shows that the slope stability decreases with increased block size and shape and that the irregular columnar joints are more unfavorable to the slope stability than the regular joints. Furthermore, with dropping the factor of safety and randomly distributed joints the rock failure mechanism appears to shift from structurally controlled for the 50 m slopes towards a step-like failure path for the 100 m and 200 m slopes.

1. INTRODUCTION

In rock mechanics of both natural and engineering slopes, many studies have provided the influence of discontinues on slope stability (Park et al., 2005). These discontinuities respond significantly to any change in stresses and loadings applied to rock masses (Bandis et al., 1983; Brady & Brown, 2006). This response changes the distribution of stresses to extend further to the next areas of the rock masses and other discrete pre-existing structures (Gao et al., 2019). These residual stresses could allow shear/ tensile through intact rock and shear along discontinuities leading to failures or creating new fractures (Karami & Stead, 2008).

In geotechnical investigations of rock failures, many mechanisms primarily depend on the geometry of geological structure and change in slope scale. It has also been long observed from the slope and other failures that the influence of discontinuities is not the same at different scales of excavations (Hammah et al., 2009). With an increased slope angle or depth of mining, comes an increased risk of slope failure (Sjöberg, 1999). Usually, at smaller slope scales, discontinuities may exert a greater influence on behavior than rock properties (Yanuardian et al., 2020). In small slopes, failure mechanisms, such as planar wedges, which are controlled by joints, are common (Stead & Wolter, 2015). Franz (2009) stated that in large-scale slopes the pre-existing discontinuities and the new fractures created by the

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redistribution of stresses in the slope surface probably congregate to develop a circular failure, combined failure, or unidentified failure mechanisms.

Many failures of rock slopes appear to occur by sliding along major individual discontinuities such as fault planes and bedding planes, or along combinations of these planes (Stacey, 2006). As the scale increases, more complex mechanisms such as step-path failures and rotational shear rock failure, which combine failure along discontinuities with shearing through intact rock bridges, begin to occur (Eberhardt et al., 2004). These complex mechanisms can follow overall curved paths that can be similar to those encountered in soils. Toppling and columnar flexural bending or buckling are other failure mechanisms that can occur with increasing slope scale (Wang et al., 2011).

Rock discontinuities include joints, bedding planes, dykes, fractures, faults, shear zones, cleavages, foliations, stratum contacts, and other geological structures, whose strength is lower than the intact rock. Consequently, these discontinuities increase the permeability of rocks, resulting in even lower strength. Thus, rock discontinuities are typically governing the overall behavior of the rock masses (Zhao, 2004). Sjöberg (1996) indicated that pre-existing fractures in the rock mass and their characteristics were the most important factors contributing to potential slope failures, along with the hydrologic conditions at the mine. A recent study with more details (Xu et al., 2013) emphasized that the failure of rock slope depended on the size of discontinuities, and showed that both tensile and shear damages at the weakest element were the trigger for the failure surface initiation in the rock slope. Chen et al. (2018) stated that collecting and estimating the sizes and other geometrical discontinuity properties in rock slopes for example, the number of discontinuity sets, spacing, roughness, and the frequency at which they occur is the main purpose of studying the rock mass failure probabilities. However, researchers used rock mass characterization to define discrete models, estimate rock mass deformation, and verify slope stability. The rock masses of slope include the discontinuities and intact rock.

This paper inspects the effect of the height of the slope on its stability. In an open-cut mine, the geometry of the slope could be defined by height, width, bench face, interramp, and slope angles. The geological structure in rock masses may vary in different scales, ranging from micro-cracks having a length of a few millimeters to many kilometers, such as the Paroo fault, which strikes in south George Fisher and south-west of Handlebar Hill open cut mines at Mt. Isa (Long, 2010).

Hekmatnejad & Crespin (2022) concluded that the factor of safety in rock slopes is sensitive to the geometrical rock parameters of discontinuities when the number of enclosed joints is more than the discrete elements in the rock mass. In addition, the sensitivity of joint parameters affects the factor of safety and is used to predict the behavior of discontinuities in rock masses (Alonso et al., 1996).

2. THE SLOPE STABILITY ANALYSIS METHOD

The numerical models in this investigation are two-dimensional finite element stress analyses using finite element software Rocscience/ Phase^2. Application of the finite element method (FEM) to problems of blocky rock masses as explicit modeling of the behavior of individual joints can be used for practical engineering in blocky rock masses (Hammah et al., 2009). Geotechnical engineers have used different failure criteria for rock slopes. One of the techniques is the shear strength reduction (SSR) approach. SSR is simple in concept, which systematically reduces the shear strength envelope of material by a factor of safety until deformations are unacceptably large or solutions do not converge (Hammah et al., 2008).

The SSR technique for slope stability analysis can be used to verify the stress reduction factor (SRF). The critical SRF is the one that brings the slope to failure, namely the factor of safety. The shear strength parameters of all slope materials will be reduced until the critical SRF is found. Use at most three levels of headings that correspond to chapters, sections, and subsections.

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2.1. Finite Element Analyses for Models at Different Scales

The effect of slope scale on its stability has been numerically experimented with through five models. The change in a safety factor and the failure mode of rock masses were verified in these samples when the slope dimension increased. The strength parameters required to define the material properties dialog for discontinuities and intact rock are explained in Table 1. The (SRF) was established within a tolerance of 0.001 for an initial estimate of SRF = 1 when setting the shear reduction dialog of the project.

The rock mass is assumed to be homogenous and rock slopes are dry. The five models have different slope heights and the same discrete joint sets networks as shown in Table 1. One joint set dips at an angle of 36° (clockwise from the horizontal axis), while the other dips at 80°. The spacing of each joint set was assumed to follow a normal distribution with a mean of 4 m and a standard deviation of 1.0 m. Both joint sets have infinite lengths that can form discrete elements through the rock block. The slope angle is 76° and the heights are 10 m, 20 m, 50 m, 100 m, and 200 m, respectively, for the five models. The FEM meshes for the five slopes are shown in Figure 1.

The factor of safety was calculated during the numerical simulation and the failure mechanism was analyzed for each of the models. Table 2 shows the factors of safety obtained for each slope with the same infinite two joint sets and rock mass parameters.

2.2. Failure Mechanisms of Slopes with Regular Joints

A 2D finite element software program for dry soil and rock was run to predict the behavior of intact rock and fracture developments in these models. Figures 2a to 2e are the modeling results of FEM-SSR.

The distributions of shear strains obtained from the numerical simulations are shown in Figures 2a through 2e. Contours of shear strains in discontinuous rock mass in Figure 2a indicate the highest shear strain zone above the toe at the intersection of the two joint sets, namely the largest shear deformation in this zone for the 10 m slope. Figure 2b reveals the high shear strain zone at the toe controlled by two parallel 36° joints near the toe and an 80°

joint at the toe. In Figure 2c, for the 50 m slope, the contours indicate large blocks of shearing extending up through the intact rock and controlled by the joints. While in Figures 2d and 2e, there are indications of more shearing through intact rocks and a step-wise shear strain contour at the bottom of the high strain band that is parallel to the 36° joints and the steps are parallel to the 80° joints, suggesting a step failure coupled with intact rock shearing. The band in Figure 2d expands much larger than in Figure 2e.

It was found that the shear strain level formed above the toe is much higher, suggesting higher shear stress levels in those locations. Thus, unstable zones would be progressively initiated there. At lower slope heights, the slopes are stable (Figures 2a and 2b), refer to Table 2.

Table 1: Mechanical property of intact rock and joints used in Phase 2 models.

Material	Properties	Comments
Spears Siltstone	Unit weight = 0.0272 MN/m^3 Young's Modulus = 22 MPa Poisson's ratio = 0.2 Tensile strength residual = 0.1 MPa Dilation angle = 2^o Internal friction angle = 33^o Cohesive force = 0.1 MPa	Based on the Mohr-Coulomb criterion.
Joint set 1	Dip angle = 36^o Spacing = 4m (mean) Normal stiffness = 100 MPa/m Shear stiffness = 10 MPa/m Tensile strength = 0 MPa Cohesive force = 0.1 MPa Internal friction angle = 18^o Joint end condition = All closed	Parallel statistical joint model.
Joint set 2	Dip angle = 80^o Spacing = 4m (mean) Normal stiffness = 100 MPa/m Shear stiffness = 10 MPa/m Tensile strength = 0 MPa Cohesive force = 0.1 MPa Internal friction angle = 18^o Joint end condition = All closed	Parallel deterministic joint model.

Table 2: Factor of safety of the homogeneous cross-jointed slopes.

Slope height (m)	Factor of safety
10	2.46
20	1.34
50	0.81
100	0.54
200	0.32

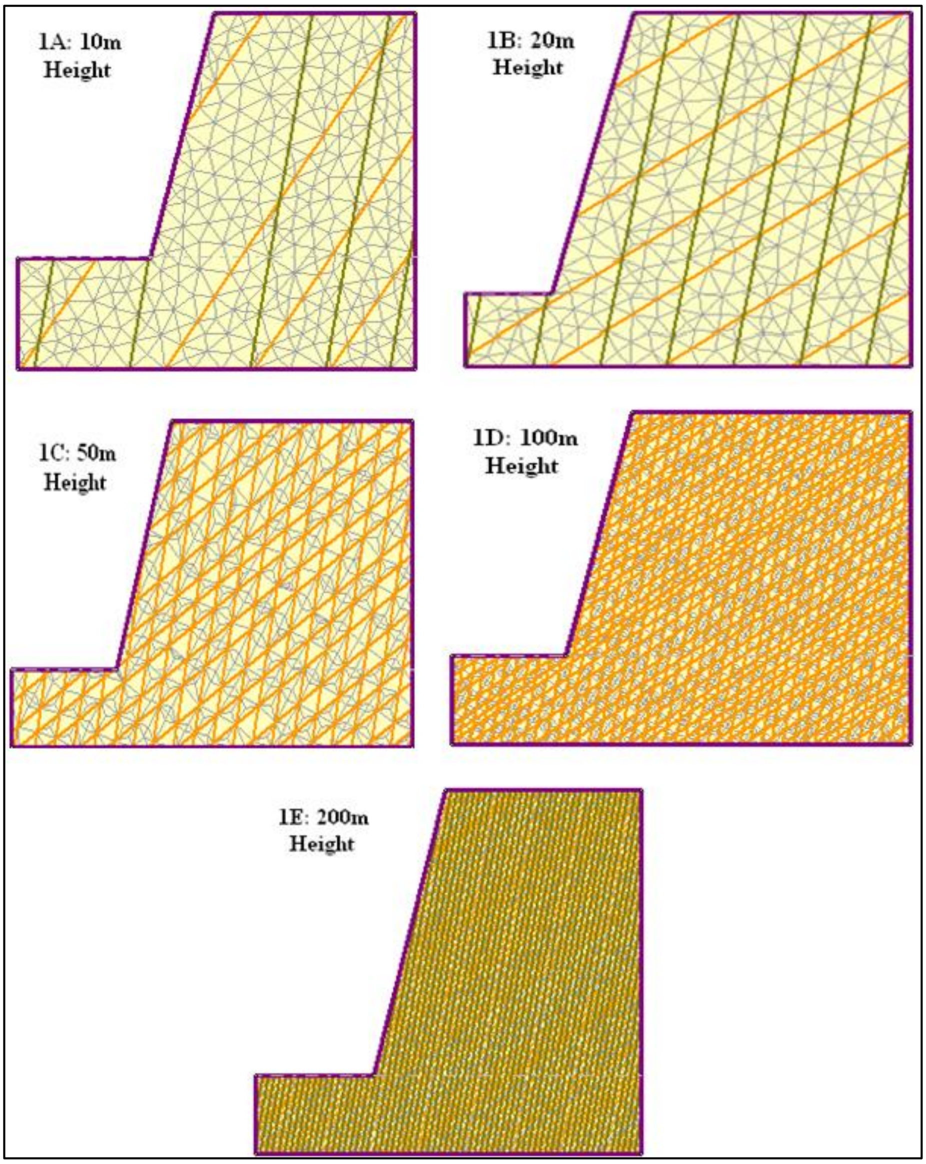

Figure 1: Models of different scales used in this study.

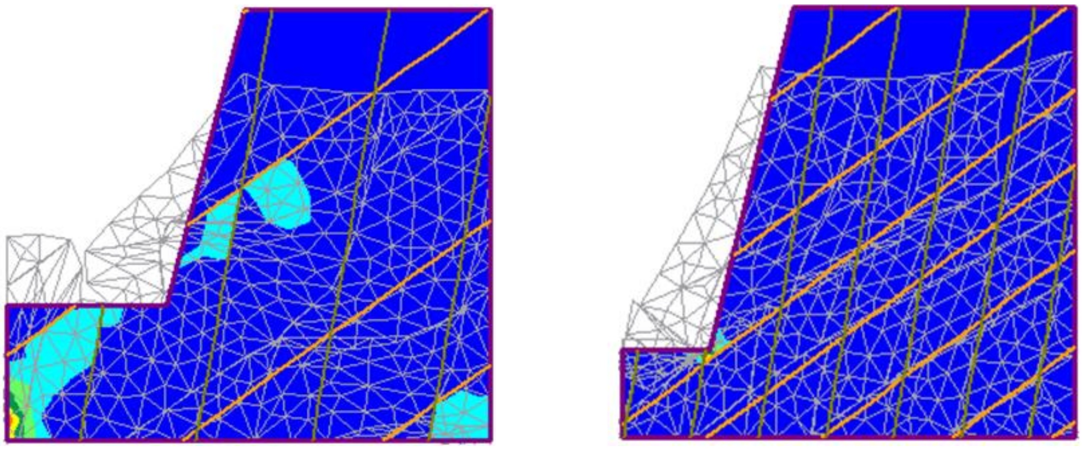

Figure 2a: Shear strain development of 10 m height slope.

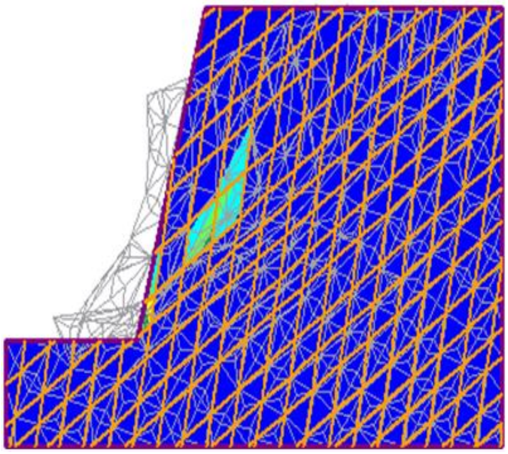

Figure 2b: Shear strain developments of 20 m height slope.

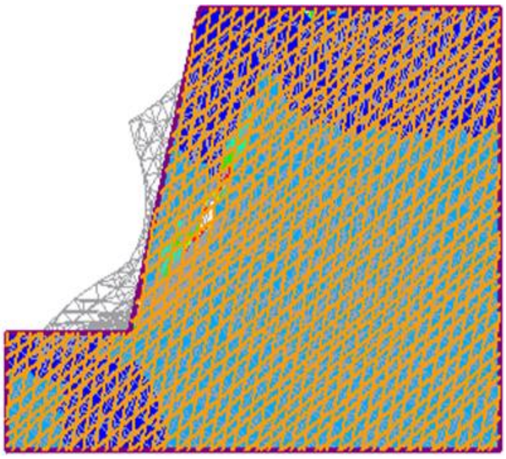

Figure 2c: Shear strain development of 50 m height slope.

Figure 2d: Shear strain development of 100 m height slope.

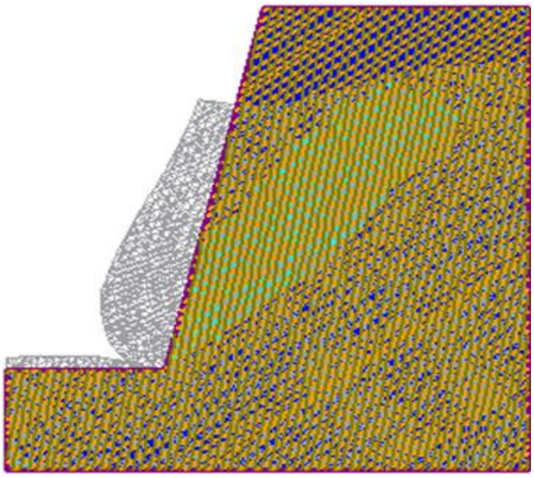

Figure 2e: Shear strain development of 200 m height slope.

When the slope is higher, the deformation expands till exceeding the limit, or slope failure as in the case of Figure 2c. It may be concluded that the slope scale and joint sets control the development of an unstable rock zone. While the high shear strain contours in Figures 2d and 2e are in more structural rock masses (joint sets) and the dislocations have acquired approximate step path of failure.

The mechanical properties of the rock mass are influenced by the presence of joint sets and, more joint sets provide more possibilities of potential slide planes for rock blocks to slide and fall (Zhao, 2004). Initiations and progression of shear strain along joints were followed by block movements and the involvement of further development of more shearing through intact rock.

The patterns of the deformation vectors for rock masses in terms of the total displacements and joint slip directions are demonstrated in Figures 3a to 3e. Figures 3a to 3e

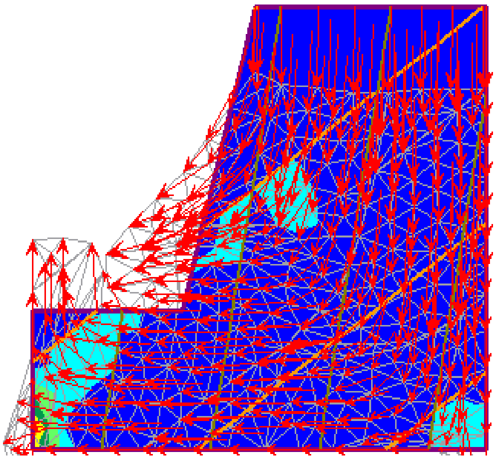

Figure 3a: Displacement vectors and slip directions of joints of 10 m slope.

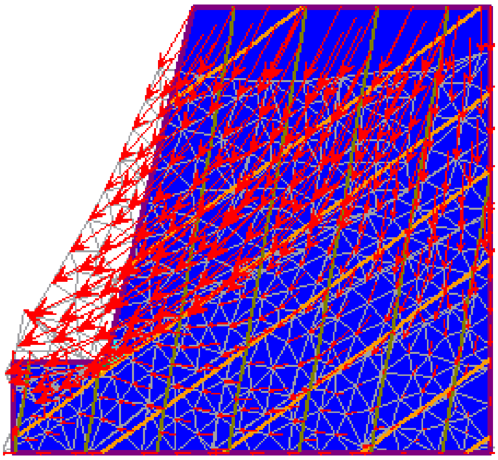

Figure 3b: Displacement vectors and slip directions of joints of 20 m slope.

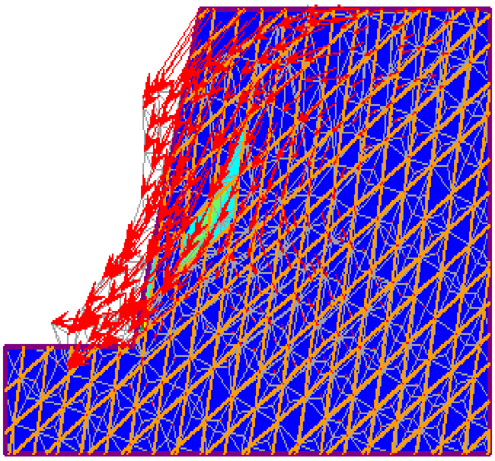

Figure 3c: Displacement vectors directions joints of 50 m slope.

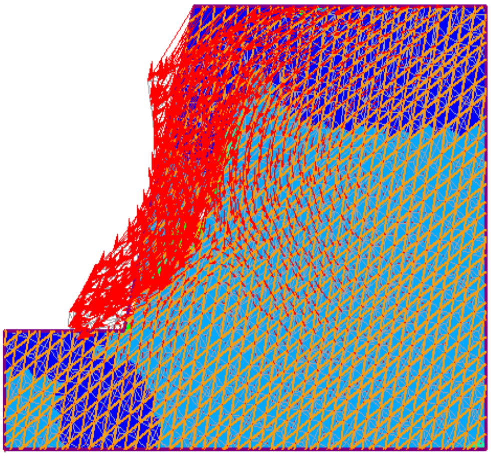

Figure 3d: Displacement vectors directions and slip of and slip of joints of 100 m slope.

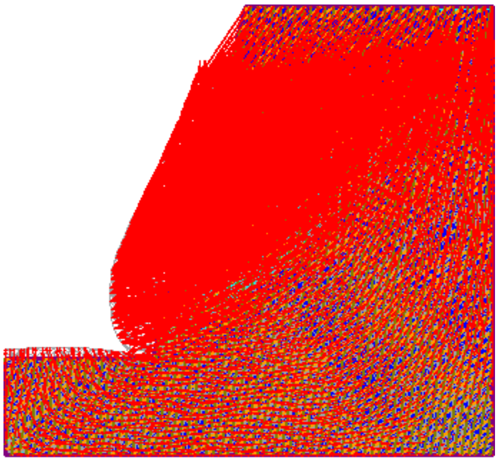

Figure 3e: Displacement and joints slip direction of 200 m slope.

2.3. Mechanisms of Slopes with Irregular Joints

In this section, an irregular columnar joints Voronoi network is introduced to obtain the influence of irregular joint patterns on slope stability. The same material properties composed of the slope of 200 m height and 76° slope angle (Case E in Figure 1) are adapted to analyze the behavior of irregular discontinuity. The numerical analysis result shows a FOS of 0.36 for the irregularly jointed slope. Figure 4a is the shear strain where the green legend represents the largest shear strain and (b) is the total displacements.

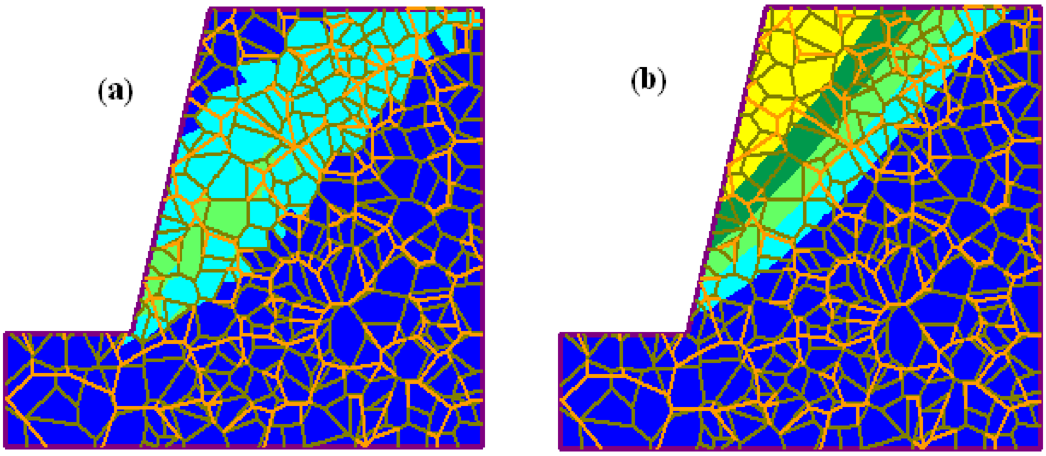

Figure 4: Visualised slope with irregular Voronoi- joints, slope angle 76°, slope height 200 m and FOS is 0.36 **(a)** maximum shear strain, **(b)** total displacement.

3. DISCUSSION AND CONCLUSION

The factor of safety values in Table 2 clarifies that when slope height increases, a factor of safety reduces, and the mechanisms of failures (Figures 2c and d; and Figures 3c and d) have been varied with the slope scales when the slope and the dip angles are the same. The finite element analysis using the SSR method has demonstrated the influence of slope scales and joint patterns on slope stability.

For the examples used in this study, the slope of 10 m height, the simulated results indicate that the area above the toe has the highest shear strain along a joint. Consequently, this area of the 10 m slope is most critical. For the 20 m height slope in Figure 2b, the simulation reveals a high shear strain zone controlled by joints near the toe. More recently, Xu et al. (2013), however, suggested that shear stress level and normal stress level might have a significant effect on the long-term behavior of rock mass with discontinuities of low strength.

As the height increases in Figure 2c, further blocks are involved in the sliding and the critical failure surface developed from the initial local shear failure along the joint and near the toe. The contours of maximum shear strain indicate a rock failure mechanism involving structural movements and failure through intact rock. With further increasing the slope height, composite failure surfaces become visible through the toe as illustrated in Figure 2d. Figure 2e shows maximum strain contours for the slope of 200 m height, suggesting a step failure coupled with intact rock shearing.

In large-scale slopes, failure may involve two or more interacted failure modes, such as step-path failure coupled with the intact rock shearing found in this model study. In many slope stability cases, the step-path failure dominantly controlled the large-scale rock slopes that coalesce with fractures through intact rock bridges (Franz, 2009). Another approach by Zhang et al. (2006) discussed the instability that occurred with increasing the slope height appears that the entire slope might go through a combination of two failure mechanisms. Sjöberg (1996) stated that rotational shear failure in large-scale slopes, would probably primarily involve failure along pre-existing discontinuities with perhaps some portion of the failure surface going through intact rock. Recently, Ma et al. (2013) simulated numerical models for the progressive failure of rock slopes involving a complex non-linear rock path failure from initiation, through propagation to a major crack. Fractures in regular and irregular joints normally exist in rock masses, and the failure mechanisms are mainly initiated by the crack coalescence pattern and the density of pre-existing flaws combined in the coalescence (Wong et al., 2001). Commonly, the crack coalescence pattern of irregular jointed rock masses has more pre-existing flaws implicated in the coalescence than the regular joint with the same strength parameters (Chen et al., 2013).

This study indicates that the scale of the slope remarkably influences the factor of safety. Stacey et al. (2003) showed a typical relation between slope angle and slope height in an open pit. The higher the slope is, the less stable it becomes as shown in Figure 5. Moreover, the existence of regular joint sets in the rock mass can significantly affect the mechanical behavior of the rock slope and degrade the factor of safety (FOS). The irregular joints further reduce the factor of safety (FOS) and consequently decrease the slope stability as shown in Figure 6.

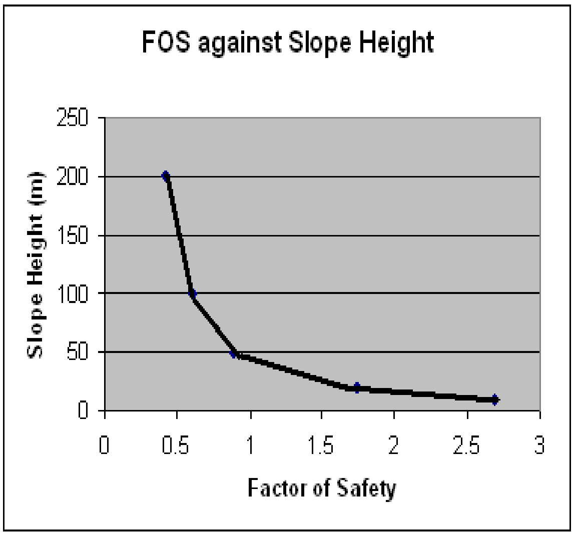

Figure 5: Shows the factor of safety against slope Heights (after Stacey et al., 2003).

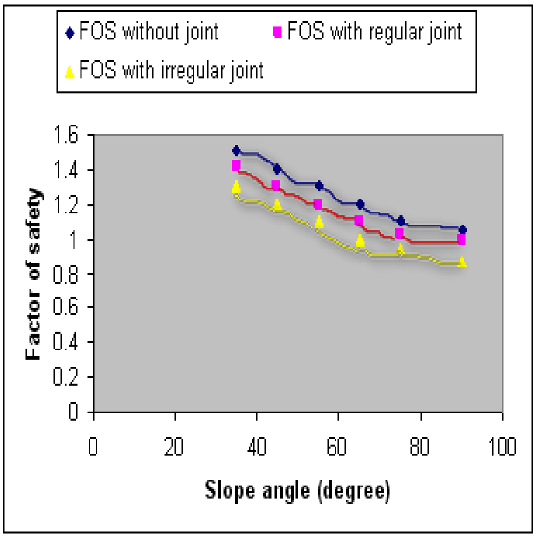

Figure 6: Shows the factor of safety of intact, regular and irregular jointed.

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