Variation of Failure Mechanisms of Slopes in Jointed Rock Masses with Changing Scale

R.E. Hammah and T. Yacoub Rocscience Inc., Toronto, Canada J.H. Curran

Rocscience Inc. & Department of Civil Engineering, University of Toronto, Toronto, Canada

Abstract

This paper demonstrates the ability of Shear Strength Reduction (SSR) analysis based on the Finite Element Method (FEM) to model the scale effects of discontinuity networks on the stability and failure mechanisms of slopes in blocky rock masses. Through three examples, it shows how these mechanisms depend on joint network geometry and change with increasing slope height.

Introduction - Scale Effects and the Stability of Slopes in Rock Masses

It has been long recognized in rock mechanics that discontinuities (geological structures) significantly influence the response of rock masses to loadings and excavation (Goodman et al, 1968, Manfredini et al, 1975, Cundall et al, 1975, Bandis et al, 1983). It has also been long observed from slope and other failures that this influence is not the same at different scales (excavations sizes). Generally, at smaller scales discontinuities exert greater influence on behaviour than do intact rock properties. In small slopes, failure mechanisms such as planar wedges, which are controlled by joints, are common. As the scale increases more complex mechanisms such as step-path failures and rotational shear failure, which combine failure along discontinuities with shearing through intact rock bridges, begin to occur. 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.

At intermediate and large scales, anticipation or prediction of the stability of rock slopes and the manner in which they can fail can be very difficult. This is because at such scales stability is affected by the strength and deformation properties of both intact rock and joints, the geometry and distribution of joints throughout a rock mass, and stress and groundwater conditions.

Of the numerical methods used for the stress analysis today, the family of Discrete Element Methods (DEMs) and Discontinuous Deformation Analysis (DDA) have been considered to be the most wellsuited to the problems of blocky rock masses. Recently however, it has been demonstrated that the Finite Element Method (FEM) with explicit representation of discontinuities with joint elements is a credible alternative (Hammah et al, 2009, 2008 and 2007). This paper examines the ability of the FEM to capture the variation of factor of safety and failure mechanism of blocky rock mass slopes with scale. The paper will show that such modelling can help engineers to understand the behaviour of blocky rock slopes at different scales much better, and to more accurately predict failure mechanisms and factors of safety.

Application of the Finite Element Method (FEM) to Problems of Blocky Rock Masses

Due to the widespread availability of powerful desktop and laptop computers, the FEM with explicit modelling of the behaviour of individual joints can be used for practical engineering in blocky rock masses. This has also been facilitated by the development of techniques for generating networks of discrete fractures, and the development of the Shear Strength Reduction (SSR) method. SSR analysis (Dawson et al, 1999, Griffiths and Lane, 1999, Matsui and San, 1992) allows factors of safety of slopes to be calculated with numerical methods. Studies have confirmed the accuracy of the FEM-SSR technique in general and for the variety of failure mechanisms encountered in rock slope engineering (Dawson et al, 1999, Griffiths and Lane, 1999, Hammah et al, 2005 and 2007).

Although FEM-based SSR analysis is an alternative to conventional limit equilibrium methods in many cases, its ability to readily combine slip along joints with failure through intact material offers several advantages in the modelling of blocky rock mass problems. The method can model the broad range of behaviours of slopes at different scales, from wedge sliding to toppling and rotational failures (Hammah et al, 2007). As well it can easily handle cases in which fractures intersect in a manner such that discrete blocks may not necessarily be formed, i.e. cases in which joints may terminate within intact rock and not only at intersections with other joints (Hammah et al, 2008). Perhaps, the greatest benefit of FEM-based SSR analysis is that it can automatically determine the broad variety of failure mechanisms with no prior assumptions regarding the type, shape or location of these mechanisms. These advantages will be demonstrated on simple slope examples to be described next.

Three Test Examples

The ability of FEM-SSR to capture the effects of scale on stability (factor of safety) and mode of failure of slopes in blocky rock masses was tested on three simple examples. The strength and deformation properties of intact rock and joints are provided in Table 1. *Phase*², a two-dimensional finite element program for modelling geotechnical excavations, was the numerical analysis tool used. Factors of safety were determined within a tolerance of 0.05. This degree of accuracy facilitated a good balance between reasonably fast computation and accurate factor of safety values.

Material	Properties				
Intact Rock	Unit weight = 0.027 MN/m3				
	Young's Modulus = 20000 MPa				
	Poisson's ratio $= 0.3$				
	Tensile strength = 0 MPa				
	Cohesion = 1 MPa				
	Friction angle $= 30$ degrees				
	Dilation angle $= 0$ degrees				
Joints	Dip (of Joint Set 1) = 0 degrees				
	Dip (of Joint Set 2) = 45 degrees				
	Spacing $= 3m$				
	Normal stiffness = 100000 MPa/m				
	Shear stiffness = 10000 MPa/m				
	Tensile strength $= 0$ MPa				
	Cohesion = 0.5 MPa				
	Friction angle $= 20$ degrees				

Table 1: Strength and deformation properties of intact rock and joints.

In all three examples, a slope with a face angle of 1:3 (ratio of one horizontal to three vertical) was assumed. Each example had a different discrete fracture (joint) networks. Example I involved a rock mass with two sets of infinite, parallel joints. One joint set dipped at an angle of 36 degrees (measured clockwise from the horizontal axis), while the other dipped at -45 degrees, i.e. at 45 degrees anticlockwise from the horizontal. The spacing of each joint set was assumed to follow a normal distribution with a mean of 2.5m and standard deviation of 0.5m. Lastly, the Example I joints were assumed infinitely long, leading to the formation of discrete blocks throughout the rock mass. The resulting joint network is shown on Figure 1a.

Example II involved a similar joint network except that this time the joints were assigned finite lengths that were normally distributed with a mean length of 10m and standard deviation of 1m. Each of the two joint set was assumed to have a constant length persistence of 0.8. The Example II joint network pattern is shown on Figure 1b.

The third example modelled a rock mass comprising irregularly-shaped polygonal blocks, which can be represented by a Voronoi tessellation. It was assumed in this example that the Voronoi polygons had a density (number of polygons per unit area) of $0.2 / m^2$. The joint network pattern for this example is provided on Figure 1c.

For each of the three examples two conditions were examined for joint ends exposed at the surface (slope face, and upper and lower slope surfaces). In the first scenario, joints ends were assumed 'open' at the surface, meaning that the two opposing faces of a joint were free to slip relative to each other at exposed faces. Under the second scenario, joint end conditions were assumed 'closed' - this opposing joint faces at exposed faces from slipping relative to each other.

Six different slope heights - 10m, 20m, 30m, 60m, 90m and 120m - were considered in each joint network example. A factor of safety was calculated for every slope and the failure mechanism noted. Table 2 lists all the calculated factors of safety. Due constraints on paper length, images of the failure mechanisms are provided in only a few instances.





Figure 1a: Joint network pattern comprising two sets of infinitely long joints used in Example I



Figure 1c: Joint network pattern comprising Voronoi polygons used in Example III

Figure 1b: Joint network pattern comprising two sets of finite joints used in Example II

Figure 1: Geometry of joint networks used in the three examples in the paper.

Slope Height	Example I		Example II		Example III	
(m)	'Open' at Slope Face	'Closed' at Slope Face	'Open' at Slope Face	'Closed' at Slope Face	'Open' at Slope Face	'Closed' at Slope Face
10	2.8	4.4	5.6	9.35	6.85	15.55
20	1.3	1.75	2.4	3.1	4.6	5.45
30	1.05	1.3	1.35	1.85	2.4	2.9
60	0.7	0.75	0.9	1.05	1.5	1.6
90	0.6	0.6	0.8	0.85	1.15	1.2
120	0.55	0.55	0.7	0.65	1.15	1.2

Table 2: Factors of Safety

Discussion

From the factor of safety values given in Table 2, it can be seen that FEM-SSR analysis captures scale effects induced by the presence of fractures in rock masses. For each slope condition, factor of safety reduced with increasing slope height, and failure mechanism varied with slope height.

The results showed that at small slope heights factors of safety differed significantly between the two joint end condition scenarios. Consistently, 'closed' joint end conditions yielded factors of safety considerably higher than those calculated for 'open' conditions. However with increasing slope height these differences tailed off and just about disappeared at slope heights of (around) 90m and beyond for the three examples in this paper. At slope heights of about 90m and higher, the factors of safety for the two joints end conditions became near identical.

Although this influence of joint end conditions is not unanticipated, the fact that it is well modelled by FEM-SSR indicates the usefulness of the technique. The technique correctly captures the increase in stability when exposed joint ends are pinned and therefore force the mobilization of more blocks in order for failure to occur. The nature of this effect, captured by FEM-SSR, becomes more evident when the failure mechanisms of the slopes are examined.

From FEM analysis, slope failure mechanisms can be deduced from plots of contours of total displacement and (exaggerated) outlines of joint deformations. As a result these plots are shown on the remaining figures in the paper for indicating the manner in which analyzed slopes are predicted to fail.

Figures 2 to 7 show the failure mechanisms for Example I, the case of infinite joints. For both the 'open' and 'closed' joint end conditions, the 10m slope failed as a result of sliding of blocks. However, in the case of 'pinned' end conditions more blocks are involved in the sliding.

At a height of 20m the slope fails (Figures 4 and 5) predominantly through sliding along the joint day lighting closest to the slope toe, but also involves more movement through intact rock in the upper parts of the sliding mass. Such movement becomes possible through failure of intact material. Figures 6 and 7 indicate that at heights of 60m and beyond the failure surface acquires rotational-type (curved) features, but retains the tendency to slip along pre-existing joints near the slope toe.

For Example II, because the joint pattern is such that discrete blocks are not readily formed, the slope mechanisms are more complex than in Example I, even for small slope heights as evident on Figures 8 to 13. The failure mechanism for the Example II 60m slope involves a series of local step-path failures that overall have a slightly curved shape. For the 120m slope with open joints (Figure 13) the failure mechanism was step-path at a local scale, but quite linear overall. The 'closed' joint counterpart (not shown) had a more curved overall shape.

In Example III all the failure mechanisms had curved overall shapes. This suggests that when there are no dominant or preferred directions of weakness planes, rock slope failure mechanisms become quite similar to those encountered in soils.



Figure 2: Failure mechanism for 10m slope - case of 'open' joint ends



Figure 3: Failure mechanism for 10m slope - case of 'closed' joint ends



Figure 4: Failure mechanism for 20m slope - case of 'open' joint ends



Figure 5: Failure mechanism for 20m slope - case of 'closed' joint ends



Figure 6: Failure mechanism for 60m slope - case of 'open' joint ends



Figure 7: Failure mechanism for 90m slope - case of 'open' joint ends



Figure 8: Failure mechanism for 10m Example II slope - case of 'open' joint ends



Figure 10: Failure mechanism for 20m Example II slope - case of 'open' joint ends



Figure 9: Failure mechanism for 10m Example II slope - case of 'closed' joint ends



Figure 11: Failure mechanism for 20m Example II slope - case of 'closed' joint ends



Figure 12: Failure mechanism for 60m Example II slope - case of 'open' joint ends



Figure 13: Failure mechanism for 120m Example II slope - case of 'open' joint ends



Figure 14: Failure mechanism for 10m Example III slope - case of 'open' joint ends



Figure 15: Failure mechanism for 10m Example III slope - case of 'closed' joint ends



Figure 16: Failure mechanism for 20m Example III slope - case of 'open' joint ends



Figure 17: Failure mechanism for 20m Example III slope - case of 'closed' joint ends



Figure 18: Failure mechanism for 60m Example III slope - case of 'open' joint ends



Figure 19: Failure mechanism for 120m Example III slope - case of 'open' joint ends

Conclusion

Previous research had shown that numerical methods based on conventional continuum, elasto-plastic assumptions can model the effects of varying scale on the stability of slopes (Hammah et al, 2008). However, these methods still predict only simple, rotational-type failures. The examples provided in this paper demonstrate that the FEM with explicit representation of joints, on the other hand, captures the complicated variation of slope failure mechanisms with changing scale. They show that, combined with SSR, this type of FEM analysis has great potential to help rock slope engineers better understand and predict the stability of slopes in blocky rock.

The increased factor of safety for 'closed' joint ends cases over their 'open' counterparts for small scale slopes confirms the effectiveness of support measures such as bolting and wire meshes that essentially force exposed joint ends to move together. The modelling in the paper also confirms the loss in efficiency of such support with increasing slope height, and suggests that other stabilization methods be considered for large slopes.

Although we are confident that the predictions of the method would correspond well to real-world behaviour, it would be of great benefit if future research could critically examine and verify the performance of FEM with SSR on real examples and case studies. With increased confidence in the approach, engineers could use its tools to better design blocky rock slopes and regulate their behaviour.

References

- BANDIS, S.C., LUMSDEN, A.C. and BARTON, N.R., 1983. Fundamentals of rock joint deformation. International Journal of Rock Mechanics, Mining Sciences & Geomechanics Abstracts, vol. 20, no. 6, 249-268.
- CUNDALL, P., VOEGELE, M. and FAIRHURST. C., 1975. Computerized design of rock slopes using interactive graphics for the input and output of geometrical data. In Design Methods in Rock Mechanics, Proceedings of the 16th Symposium on Rock Mechanics, University of Minnesota, Minneapolis, American Society of Civil Engineers.
- 3. DAWSON, E.M., ROTH, W.H. and DRESCHER, A., 1999. Slope stability analysis by strength reduction. Geotechnique, vol. 49, no. 6, 835-840.
- 4. GOODMAN, R.E., TAYLOR, R.L. and BREKKE, T.L., 1968. A model for the mechanics of jointed rock. Journal of the Soil Mechanics and Foundations Division, ASCE, 637-659.
- 5. GRIFFITHS, D.V. and LANE, P.A., 1999. Slope stability analysis by finite elements. Geotechnique, vol. 49, no. 3, 387-403.
- HAMMAH, R.E., YACOUB, T.E., CORKUM, B. and CURRAN, J.H., 2008. The Practical Modelling of Discontinuous Rock Masses with Finite Element Analysis. In Proceedings of the 42nd U.S. Rock Mechanics Symposium – 2nd U.S.-Canada Rock Mechanics Symposium, San Francisco, US, 2008.
- HAMMAH, R.E., YACOUB, T.E., CORKUM, B., WIBOWO, F. and CURRAN, J.H., 2007. Analysis of blocky rock slopes with finite element shear strength reduction analysis. In Proceedings of the 1st Canada-U.S. Rock Mechanics Symposium, Vancouver, Canada, 329-334.
- HAMMAH, R.E., YACOUB, T.E., CORKUM, B. and CURRAN, J.H., 2005. A comparison of finite element slope stability analysis with conventional limit-equilibrium investigation. In Proceedings of the 58th Canadian Geotechnical and 6th Joint IAH-CNC and CGS Groundwater Specialty Conferences – GeoSask 2005. Saskatoon, Canada.
- 9. MANFREDINI, G., MARTINETTI, S. and RIBACCHI, R., 1975. Inadequacy of limiting equilibrium methods for rock slopes design. In Design Methods in Rock Mechanics, Proceedings of the 16th Symposium on Rock Mechanics, University of Minnesota, Minneapolis, American Society of Civil Engineers, 35-43.
- 10. MATSUI, T. and SAN, K.C, 1992. Finite element slope stability analysis by shear strength reduction technique, Soils and Foundations, 32(1): 59-70.
- 11. ROCSCIENCE INC., 2008. Phase2 v7.0 Two-dimensional finite element slope stability analysis.