ABSTRACT
This paper presents a methodology whereby statistically representative fracture patterns can be used for an accurate representation of structural discontinuities in rock. This implies that field data can be used to generate characteristic fracture systems for a rock mass. It is then possible to introduce any range of slope geometry in the generated rock mass. The stability of such excavations can be evaluated using traditional limit equilibrium analysis. The advantage of this approach is that it can consider the influence of both large-scale (major) fractures whose relative location in the rock mass is well defined and minor fractures whose location and orientation is defined by probabilistic algorithms. This is demonstrated in this paper with reference to a rock slope susceptible to wedge-type failures.

KEYWORDS
Fracture systems; Slope stability; Wedge failure

CITATION

This is the author's version of the original manuscript. The final publication is available at Taylor & Francis Link Online via https://doi.org/10.1080/13895260500106589

1 INTRODUCTION

A critical issue in the engineering of slopes in rock is the ability to predict stability with a certain degree of confidence. The reliability of any such predictions is influenced by the quality and quantity of field data as well as the applicability and limitations of the employed design tools.

The interdependence of these factors is difficult to quantify and account in the final design. In traditional deterministic analysis, the inherent uncertainty in defining the geomechanical regime is often reflected in the use of a factor of safety. Hoek (1991) suggests a mean factor of safety of 1.3 for temporary slopes and 1.5 for permanent slopes.

Probabilistic techniques, on the other hand, treat all input parameters affected by uncertainty as random variables. This allows all input properties (slope configuration, material properties, etc.) to be defined by their specific probability density functions. Priest and Brown (1983) demonstrated the potential of this approach by employing Monte Carlo simulations. They further demonstrated the use of probabilistic design criteria to account the consequence of slope failure (table 1). Table 2 illustrates how this information can be used to interpret the stability of a given rock slope, depending on design purpose and whether the defined probabilistic design criteria are met.
In practice, there are several ways of improving confidence in any specific rock slope design. The first deals with improving the data collection process. This is reflected in a series of recent works including the development of a digital face mapping methodology that speeds up data collection and employs image processing algorithms to construct fracture trace maps (Lemy and Hadjigeorgiou 2003). Slob et al. (2002), Kemeny et al. (2003) and Lemy and Hadjigeorgiou (2004) report successful applications of laser scanning to quantify the orientation of fracture surfaces found within 3D point clouds representing a rock exposure. Another interesting development has been the representation of structural field data as fracture systems. An excellent review of fracture systems has been presented by Dershowitz and Einstein (1988). The last 15 years have been witness to many interesting developments in the development of computerized tools for the design of rock slopes. The major challenge is, however, to link these design tools to quality field data. In other words, to provide the necessary input data to take full advantage of the sophisticated analytical models. A successful integration will result in improved design of rock slopes.

This paper presents a methodology whereby statistically representative fracture patterns can be used for an accurate representation of structural discontinuities in rock. This implies that field data can be used to generate characteristic fracture systems for a rock mass. It is then possible to introduce any range of slope geometry in the generated rock mass. The stability of such excavations can be evaluated using traditional limit equilibrium analysis. The advantage of this approach is that it can consider the influence of not only large-scale (major) fractures whose relative location in the
rock mass is well defined but also minor fractures whose location and orientation is defined by probabilistic algorithms. This is considered in this paper with reference to a rock slope susceptible to wedge-type failures.

2 FRACTURE CHARACTERIZATION

The analysis and design of excavations in rock requires characterization of the rock mass. Piteau (1972) argues that ‘…stability of slopes in rock is determined principally by structural discontinuities in the mass and not by the strength of the rock itself’. It follows that a prerequisite for a rational analysis of rock slopes is the reliable application of a series of geological premises:

1. The characteristics of the joint population (generic structural type, orientation and position in space, intensity, joint wall rock hardness and asperities, joint size, infilling and alteration materials between joint planes). As access to all fractures is not possible it is necessary to extract the maximum information. The description of the joint population must be done on a sampling basis, and statistical analysis and judgement must be applied to decide whether the best estimate is made of the whole population.

2. Structural regions can be defined. This implies that the joint population within the region has similar characteristics.

3. A reliable model of the joints in the mass can be constructed. This implies that a statistical sampling of any property will give a representative picture of the entire portion of the rock mass it is meant to depict.

In more recent years the term fracture has gained acceptance over joint. In this context a fracture is any geological discontinuity in a given rock mass, while a fracture system is a set of individual fractures that may or may not intersect. The properties of a fracture system will dictate the mechanical, structural and hydrological response of a rock mass to external loads. In practice, information on the in situ system is usually collected from boreholes and surface outcrops using linear and area-based sampling methods. Area-based sampling methods, although less popular as they are more labour intensive, provide more representative spatial information. This allows the construction of structural models that form the basis of kinematic stability analysis. Numerical representations of in situ structural conditions, fracture system modelling, provide more representative rock characterization. The developed methodology integrates random fracture system generation and individual rigid block limit equilibrium stability analysis. In particular it investigates the influence of fracture orientation, location and frequency variability on wedge stability at the crest of mining benches, as well as mechanical properties such as cohesion and friction.

3 SINGLE WEDGE STABILITY ANALYSIS

In rock slopes it is often assumed that failure is confined to preexisting fracture planes or combinations of planes. This assumption implies that there are various modes of failure that are kinematically possible, i.e. planar, wedge, toppling, circular and buckling. The resulting failure mechanism is defined by the prevalent structural regime. A qualitative kinematic analysis can identify the potential failure mode and a limit equilibrium analysis can determine the resulting factor of safety for a selected excavation.

The mechanics of a sliding wedge in a rock slope have been defined by Hoek and Bray (1981). A generalized geometry is presented in figure 1, where the failure surfaces are identified as 1 and 2, the upper ground surface by 3, the slope face by 4 and the tension crack (if present) by 5. \( H_1 \) is the slope height referred to in plane 1.

A preliminary analysis is required to establish if the necessary conditions for wedge failure are met. Assuming that inter-fracture cohesion is negligible and following the conventions defined in
figures 1 and 2, slope is susceptible to wedge failure when $\psi_{fi} > \psi_i > \phi$ where $\psi_{fi}$ is the inclination of the slope face, $\psi_i$ is the dip of the line of intersection and $\phi$ is the angle of friction along the failure plane.

Figure 1. Typical wedge geometry: 1, 2, intercepting fractures; 3, upper ground surface; 4, slope face; 5, tension crack. $H_1$ is the slope height and $L$ the distance of the tension crack from the crest (after Hoek and Bray 1981).

Figure 2. Stereographic representation of the necessary kinematic conditions for failure.

If kinematic analysis suggests that the investigated fracture configuration can result in wedge failure, a more detailed analysis (also developed by Hoek and Bray (1981)), is undertaken to determine the factor of safety, $F = \frac{\text{resisting forces}}{\text{driving forces}}$.

A basic assumption is that the wedge behaves like a rigid body either sliding along a plane, or along the intersection of two planes. As the sliding block does not undergo any rotation, it follows that all acting forces pass through the centroid of the wedge. This type of wedge analysis is now incorporated in commercially available software. Furthermore, in the most advanced software packages, such as Swedge by Rocscience (2004), variability in mechanical and orientation properties of fractures can be incorporated in the analysis.
A common limitation of most current engineering analyses of wedge instability is the inability to account for fracture location, frequency, and size distributions that in fact define all possible wedges. Fracture size controls the volume and extent of unstable wedges. If the spatial variability of the structural regime is not considered, i.e. fractures being considered ubiquitous, it is not possible to determine the probability of occurrence of wedge-type instability. In traditional analyses, this implies that all wedges have an equal probability of being present at a particular site. Obviously, this is not necessarily true for most applications.

4 MULTIPLE WEDGE ANALYSIS USING FRACTURE SYSTEMS

The influence of adequately represented fracture systems on the stability of an excavation in a jointed rock mass is explored by means of a worked example.

4.1 System geometrical properties

The first step in a stability analysis is to define the volume of interest. In this example a rock mass of 32 000 m$^3$ is defined: easting, 40 m; northing, 40 m; elevation, 20 m. A structural regime is defined by both deterministically defined major fractures and smaller scale statistically generated fractures. A convenient terminology is to define a structure as large scale if it is of the same magnitude as the region of interest, and minor or small scale if it is considerably smaller than the region of interest. Figure 3 identifies two parallel major features at 45° dip and 45° dip direction in the volume of interest.

Figure 3. Location of major fractures in the region of interest.

Figure 4. Fracture system for the region of interest.
Table 3. Summary of minor fracture properties (mean value and standard deviation).

<table>
<thead>
<tr>
<th></th>
<th>Dip (deg)</th>
<th>Dip direction (deg)</th>
<th>Area (m²)</th>
<th>Intensity (m²/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Set 1</td>
<td>85 (5)</td>
<td>175 (5)</td>
<td>6</td>
<td>2.0</td>
</tr>
<tr>
<td>Set 2</td>
<td>60 (5)</td>
<td>045 (5)</td>
<td>5</td>
<td>5.0</td>
</tr>
<tr>
<td>Set 3</td>
<td>60 (5)</td>
<td>135 (5)</td>
<td>7</td>
<td>2.2</td>
</tr>
<tr>
<td>Set 4</td>
<td>45 (5)</td>
<td>025 (5)</td>
<td>6</td>
<td>2.0</td>
</tr>
</tbody>
</table>

It is the influence of minor structures, defined by dip, dip direction, fracture area and fracture intensity, that is often overlooked. Dip and dip direction are obtained during scanline or face mapping. Fracture area is computed based on field measurements of fracture trace length as described by Villaescusa (1991). Fracture intensity is fracture area per unit volume and is based on fracture length per unit area. The latter is determined during face mapping, and is converted to fracture intensity by a procedure developed by Dershowitz and Herda (1992). In the worked example, minor structures are defined by four fracture sets (table 3) and the resulting system is shown in figure 4.

A total of 100 fracture systems were generated to cover the range of statistical variations that are likely to be encountered. Each generated system was then used to evaluate the stability of a given slope configuration.

4.2 Fracture mechanical properties and rock density

For ease of presentation a geologically homogeneous rock mass was used with a rock density of 2.7 t/m³. The mechanical properties of a fracture can be estimated through random sampling over any given statistical distribution for every set, or deterministically. In more recent works the case has been made for the use of distributions. In the example, fractures were assigned an angle of friction of 30° and zero cohesion. This was a conscious decision in order to isolate and demonstrate the influence of discontinuity systems on the overall stability of an excavation.

4.3 Slope geometry

Once a realistic representation of the rock mass in the region of interest is established it is possible to investigate the influence of different slope configurations. Figure 5 provides a representation of a particular slope configuration in the region of interest. The analysed slope has a dip/dip direction of 75°/90°; the upper face is at dip/dip direction 00°/000° and a bench height of 10 m.

4.4 Fractures intersecting the crest of the excavation

It is possible to identify all structures that intersect any part of the slope for a series of system simulations. In fact, 100 simulations were undertaken for the same input data, described in figure 4 and table 3. This has allowed a statistical representation of input data. This is illustrated in figure 6 showing the intersection of the fracture system with the crest of the slope for a particular simulation.

A quantitative analysis due to the variability of the geometrical properties of the systems is presented in figure 7. 140 – 210 fractures intersect the 40 m long crest (figure 7(a)). The relative importance of individual fracture sets is shown in figure 7(b), where the number of fractures intersecting the crest is clustered by fracture sets. It is evident that for this rock mass and slope configuration, fracture set 1 is dominant.
Figure 5. Slope configuration in the region of interest.

Figure 6. Minor fractures intercepting the crest of the slope.

Figure 7. (a) Number of fractures intersecting the slope crest. (b) Number of fractures, identified by fracture set, intersecting the slope crest.
4.5 Tetrahedral wedges along the crest of the excavation

A number of minor and major fractures that intersect the crest of the excavation may combine to create tetrahedral wedges. These are potentially instable. In reviewing the 100 simulations it is possible to quantify the frequency of the number of wedges that intersect the crest of the excavation (figure 8) where the number of kinematically feasible wedges varies from 75 to 185.

A source of useful information is the frequency of the different fracture sets, along with the slope geometry, in the formation of wedges (figure 9). It can be seen that while fracture set 1 was the most dominant set in the rock mass along the crest of the excavation (figure 7(b)), fracture sets 3 and 4 are of equal importance in defining wedges in the excavation.

Figure 8. Number of wedges intersecting the crest of the slope.

Figure 9. Fracture set contribution in the wedges defined along the crest of the slope.

Figure 10. Maximum volume wedge distribution for 100 simulations.
4.6 Wedge geometrical properties

For practical purposes two volumetric parameters can be used to quantify the geometrical characteristics of wedges. The first one identifies the biggest wedge formed for every simulation. This is represented in figure 10 for 100 simulations of the fracture system. Another way to address the influence of volume is to trace the total volume of all wedges formed along the crest of the excavation (figure 11).

4.7 Wedge stability analysis

Once all possible wedges have been identified, it is possible to determine the factor of safety for every wedge. A traditional limit equilibrium analysis is applied as described by Hoek and Bray (1981). The resulting factor of safety distribution, for 100 simulations, is presented in figure 12(a) where 35% of all generated wedges have a factor of safety lower than one. The proportion of unstable wedges may be used as an indicator of global slope stability for this excavation. For design purposes these data are best reviewed with reference to the geometrical properties of the unstable wedges.

What constitutes an acceptable factor of safety has been an active topic of discussion for many years. Traditional approaches result in a factor of safety distribution for a single wedge. This type of analysis may account for variations in fracture orientation and/or mechanical properties. The approach presented in this paper and illustrated in figure 12(a) is larger in scope in that it produces a factor of safety analysis for every individual wedge in the rock slope. The resulting factor of safety distribution not only accounts for variations in fracture orientation and/or mechanical properties but also takes into consideration the spatial probability of occurrence and wedge dimensions.

The presented analysis in figure 12(a) can be used to select a suitable factor of safety depending on the application. In this example, for a factor of safety less than unity, the number of unstable wedges varies between 23 and 68 for 100 simulations for the same slope configuration (figure 13). This factor of safety analysis by itself is not indicative of the magnitude of the stability problem for this worked example. Applying the same volumetric analysis as in figures 10 and 11 provides a much better understanding of the actual situation.

Two volume distributions were then evaluated for the unstable wedges. Figure 14 presents the maximal volume distribution of wedges along the crest for 100 simulations. This distribution has wedge volume between 0.5 and 15.5 m³. The total unstable wedge volume distribution varies from 1 to 30 m³ over the 40 m long crest for 100 simulations (figure 15).

The influence of the different fracture sets in the creation of unstable wedges is shown in figure 16. The relative importance of sets 2 and 3 in unstable wedge creation can be seen clearly. Fracture set 2, while of minimal importance in defining fracture intensity and kinematically feasible wedges,
is critical for stability purposes. Figure 16(b) shows that almost 80% of the fractures from set 2 that intercept the crest of the slope result in the creation of wedges.

Figure 12. (a) Factor of safety cumulative distribution for wedges along the crest of the slope. (b) Factor of safety distribution.

Figure 13. Number of unstable wedges along the crest of the slope (for a factor of safety less than one).

Figure 14. Maximal unstable wedge volume distribution for 100 simulations.
5 CONCLUSIONS

Referring to the guidelines reported by Priest and Brown (1981), for a category 1 slope (individual benches, small temporary slopes not adjacent to haulage roads) and if the consequence of failure was not serious, the mean factor of safety was 1.71 with a \( P(F < 1.0) = 0.37 \) and \( P(F < 1.5) = 0.67 \). In this case the mean factor of safety exceeds the minimum mean value but violates both probabilistic criteria. This would suggest that this is a marginal slope requiring modifications.

This recommendation can, however, be reviewed in light of the information available for this example on the volume of the unstable wedges. This additional information would probably be used in re-interpreting the consequence of failure, given the relatively small volumes involved.

If one employs the recommendations by Hoek (1991) where a mean factor of safety of 1.3 is acceptable for temporary slopes and 1.5 for permanent slopes, in the presented case study it is possible to identify that 58% of all formed wedges have a factor of safety less than 1.3 and 67% fall under the 1.5 factor of safety threshold. Again, this information will be reviewed in the future to better account for the volume of potentially unstable wedges and consequences of failure.

This paper has presented a methodology with which to better assess the stability of rock slope in jointed rock. The basis of this approach is the use of discontinuity systems that provide a better spatial and geometrical representation of the in situ structural system. This developed methodology has been programmed to facilitate a series of simulations to accommodate the statistical variability of fractures mechanical, geometrical and spatial variability. The results can be interpreted with the aid of a series of acceptability criteria for site-specific rock slope conditions.
ACKNOWLEDGEMENTS

We acknowledge financial support from the Natural Sciences and Engineering Research Council of Canada (NSERC) and the Fonds Québécois De La Recherche Sur La Nature Et Les Technologies (FQRNT).

REFERENCES


