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# Fundamentals of slope design

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## ABSTRACT

The management of the slopes in an open pit mine depends, amongst other things, upon the assessment of the stability of these slopes. The recognition and analysis of the different modes of failure are the main issues that will be addressed in this paper on the fundamentals of slope design. A critical component of any stability analysis is estimation of the strength of the rock mass and the discontinuities which control sliding and this issue will be discussed in some detail. Other topics, such as the creation of reliable geological and groundwater models and the assessment of risk and economic factors, will be dealt with by other speakers at this Symposium and in the “Guidelines for Open Pit Slope Design”.

## INTRODUCTION

The overall management of the slopes created during the development of an open pit mine requires an ongoing assessment of the stability of these slopes. This assessment depends on good geological, geotechnical and groundwater models as well as an understanding of the risks and economic consequences of slope instability. A good open slope design is one that integrates all of these factors to produce a balanced compromise between safety, on the one hand, and operational and economic efficiency on the other.

The quality of the geological and groundwater models and of the geotechnical information available cannot be over-emphasised. Without reliable background information a slope stability analysis becomes a meaningless exercise. The methods for collecting this information and setting up the appropriate models are discussed in detail by other speakers at this Symposium and also in the "Guidelines for Open Pit Slope Design" (1) which will be published at this Symposium. These topics will not be dealt with in this paper which will concentrate on the recognition and analysis of the different modes of slope failure and on the estimation of the rock mass and discontinuity properties required for this analysis.

Figure 1 presents simplified illustrations of the four basic failure modes that have been observed in open pit slopes. These failure processes may occur on either a bench scale or as a failure of the overall pit slope and it is not unusual to encounter several of these failure types or combinations of failure types in an open pit slope. All of these failure modes, with the exception of toppling failure, involve simple gravitationally driven sliding along planes or zones that are significantly weaker than the remaining rock mass. In the case of toppling failure, in-dipping discontinuities create release surfaces that allow columns of rock to topple away from the slope face.

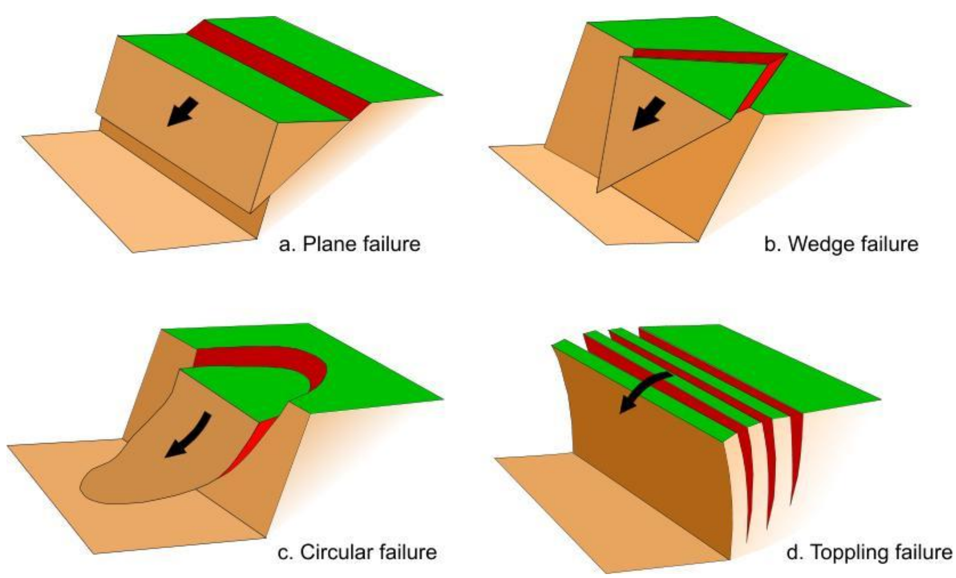



Figure 1- Simplified illustrations of most common slope failure modes.

## METHODS OF SLOPE ANALYSIS

Limit equilibrium methods for the analysis of slope stability have been available for decades and have now been developed to very effective design tools that permit incorporation of the most complex geological and groundwater conditions. Automatic searches for the most critical failure surface are included in many of these models and probabilistic analyses are also available in some cases. A well designed limit equilibrium program is probably the best tool for “what if” type analyses at a conceptual slope design stage or to investigate failures and possible remedial actions. These models run very efficiently on most personal computers and permit the designer rapidly to explore a wide range of options.

The results produced by these models are only as good as the input data. A very good geological model is essential and realistic estimates of rock mass and discontinuity strengths are required. The advantages and limitations of limit equilibrium analyses are best illustrated by means of a practical example.

### Structurally controlled failure

Consider the very simple case of a wedge failure in a 15 m high open pit mine bench. As illustrated in Figure 2, the wedge slides along the line of intersection of two planes and the back of the wedge is defined by a steeply dipping tension crack. It is assumed that water pressures act on the sliding surfaces and in the tension crack and that the wedge is subjected to a horizontal acceleration, due to an earthquake or production blasting, of 0.08 g acting outwards in the direction of the line of intersection.

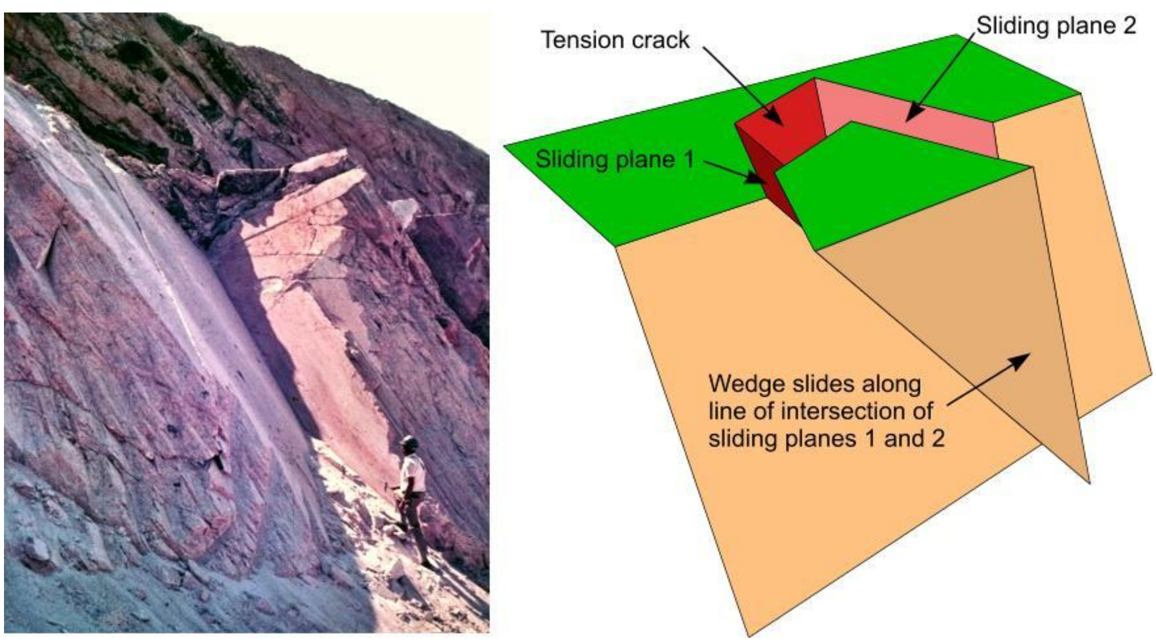



Figure 2 - Example of wedge failure in a bench, analysed using the program Swedge (2).

In order to demonstrate the principles of probabilistic analysis, the variables defining the slope and wedge geometry, the shear strength of the sliding surfaces, the groundwater pressures and the horizontal acceleration have all been defined in terms of truncated normal distributions. A Monte Carlo analysis has been run to produce a distribution of factors of safety and a probability of failure as shown in Figure 3.

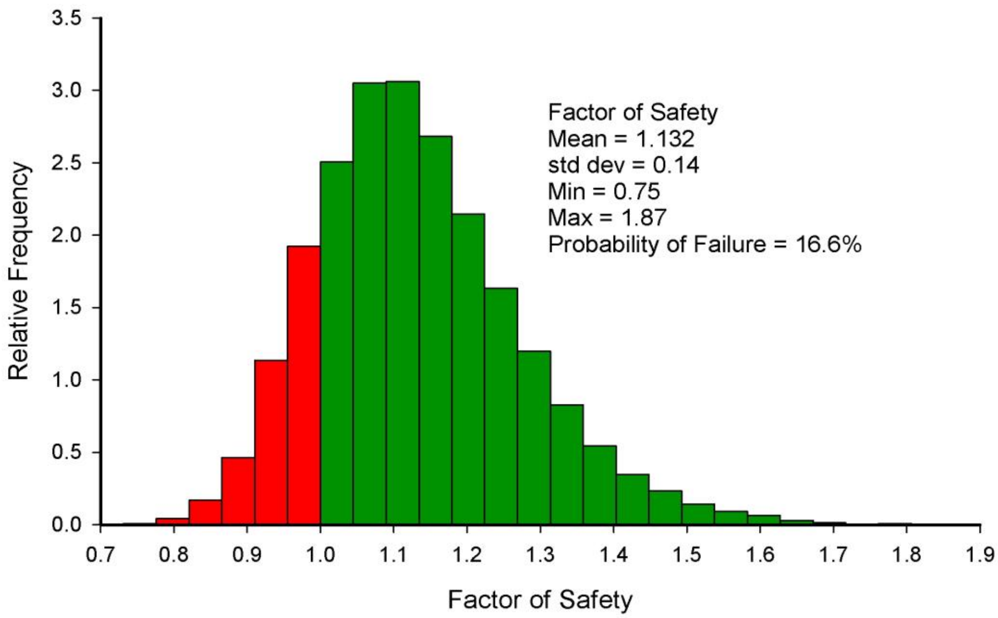



Figure 3 - Histogram of factors of safety and probability of failure for wedge.

The mean factor of safety of 1.13 and the corresponding probability of failure of 16.6% given by this analysis are used to assess the impact of potential bench scale failures on the overall slope design.

The factor of safety and the probability of failure of a slope are not, in themselves, an adequate basis for judging the acceptability of a slope design. This decision should include a consideration of the costs of cleanup and limitation of access versus the cost of remedial action such as changing the slope geometry to reduce the probability of failure. In some cases a relatively high probability of failure may be acceptable while in others, for example benches adjacent to main haul roads, even small scale failures may result in costly production delays. A comprehensive discussion on acceptance criteria for slopes is presented in Chapter 9 of the "Guidelines for Open Pit Design" (1).

The advantage of a simple limit equilibrium such as that discussed above is the ease and speed with which the user can investigate the sensitivity of the slope to changes in slope geometry, shear strength parameters, groundwater conditions and dynamic loading. There is no direct relationship between the factor of safety and probability of failure of a slope. A plot such as that reproduced in Figure 3 will give the user an understanding of the relative importance of different input parameters when using such a tool for sensitivity studies. It is highly recommended that anyone embarking on slope stability studies for the first time should carry out a range of such sensitivity studies in order to gain an appreciation of how the software operates and how small changes in the input parameters can result in dramatic changes in the calculated results.

One major disadvantage of limit equilibrium analyses in open pit mining is the fact that the method does not include any analysis or prediction of displacements. Since the monitoring of slope displacements is the most reliable tool for the prediction of developing instability and for checking the effectiveness of remedial measures, this limitation of the program restricts its uses to relatively simple tasks described above.

### Shear Strength Reduction analysis

The shear strength reduction method was first used for slope stability analysis in 1975 by Zienkiewicz et al (3) and is discussed in detail in Chapter 10 of the "Guidelines for Open Pit Design" (1). The method involves the construction of a numerical model of the slope using programs such as FLAC (4), Phase2 (5), UDEC (6) and other continuum or discontinuum codes. Once the model has been constructed the shear strengths of all the component materials are iteratively increased or decreased by a Strength Reduction Factor (SRF) until the slope fails. This is judged on the basis of monitoring a target point on the slope and where a sudden change in displacement coincides with the onset of instability. Dawson et al (7) show that the shear-strength reduction factors of safety are generally within a few percent of limit analysis solutions when an associated flow rule, in which the friction angle and dilation angle are equal, is used.

The shear strength reduction method is now widely used in open pit slope stability studies because it includes all the benefits of limit equilibrium analyses and it allows the user to study slope displacements that are critical in the evaluation of open pit stability. An example involving the use of this method will be discussed later.

## ESTIMATION OF ROCK MASS PROPERTIES

All early forms of limit equilibrium analyses, which had their origins in soil mechanics, assume that the component materials in a slope are homogeneous and isotropic. Even if the program allows the incorporation of specific structural features such as faults, the other materials in the slope are generally assumed to be homogenous. Recent developments in modelling discontinuous rock masses will be discussed later in this paper.

In the case of rocks, most of the methods for estimating the shear strength of component materials for incorporation into continuum models are based on some form of rock mass classification. Classifications by Bieniawski (8), Barton (9), Laubscher (10) and Hoek and Brown (11) are the best known in the mining industry and are the most commonly used for rock mass property estimates.

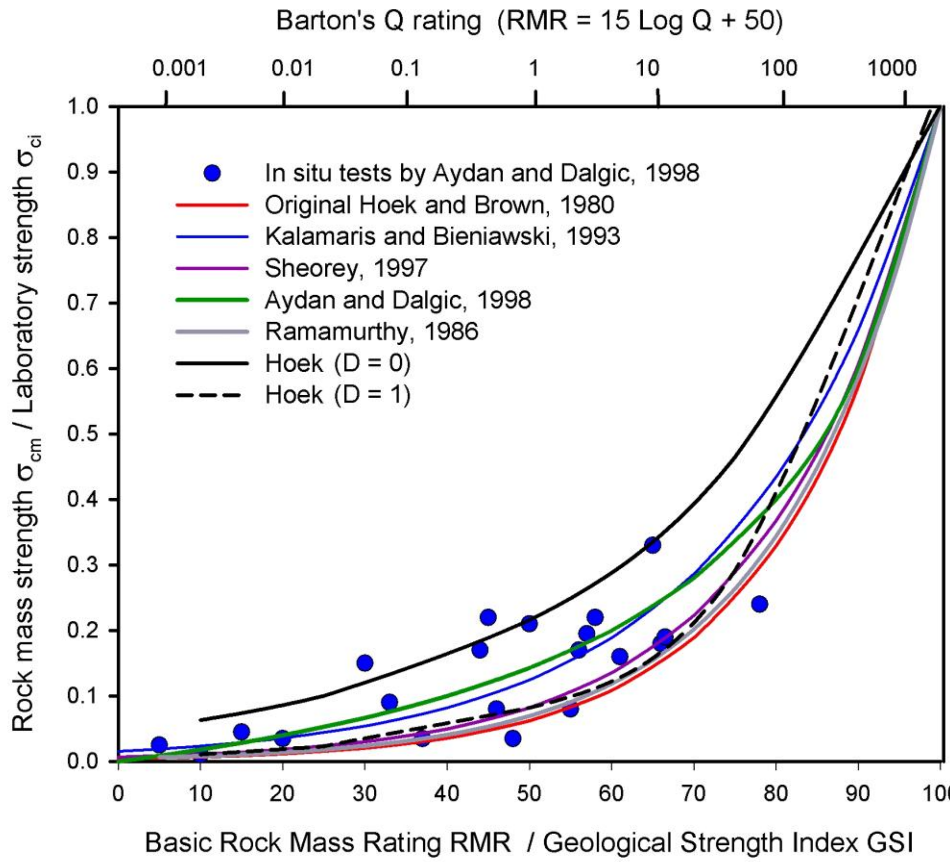



Figure 4 - Estimates of rock mass strength.

Figure 4 gives a selection of the empirical curves relating rock mass strength/intact rock strength for some of these classifications. The two curves marked Hoek (D = 0) and Hoek (D = 1) define the approximate range of typical rock mass strengths $\sigma_{cm}$ determined according to the procedure described by Hoek et al in 2002 (12), implemented in the program RocLab (www.rocscience.com). The original Hoek-Brown criterion, published in 1980, was found to give unrealistically low strength values when applied to confined conditions such as those surrounding a tunnel. This resulted in a gradual refinement of the criterion with the 2002 version giving higher strengths for undisturbed confined rock masses (D = 0) and significantly lower strength values for blast damaged rock masses such as would occur in a typical open pit mine slope (D = 1). Note that the blast damage only extends a limited distance into the slope. This may range from one or two metres for small civil engineering blasts to many tens of metres for open pit mine production blasts. Downgrading the properties of the entire rock mass by the factor D will lead to unrealistically large slope failure predictions.

Having estimated the rock mass strength $\sigma_{cm}$ as described above, the corresponding Mohr Coulomb cohesive strength $c$ and friction angle $\phi$ can be determined from the program RocLab or, in a simplified form, from:

$$\sigma_{cm} = \frac{2c\cos\phi}{1-\sin\phi} \tag{1}$$

There are two unknowns in equation (1) and Figure 5 gives a plot of cohesive strength *c* versus friction angle $\phi$ for a rock mass with intact rock strength $\sigma_{ci}$ = 70 MPa, a Geological Strength Index GSI = 45 and a blast damage factor D = 0.3 (from RocLab). It is usual to assume the angle of friction $\phi$, which is the easier of the two to estimate realistically, in order to calculate the cohesive strength *c*.

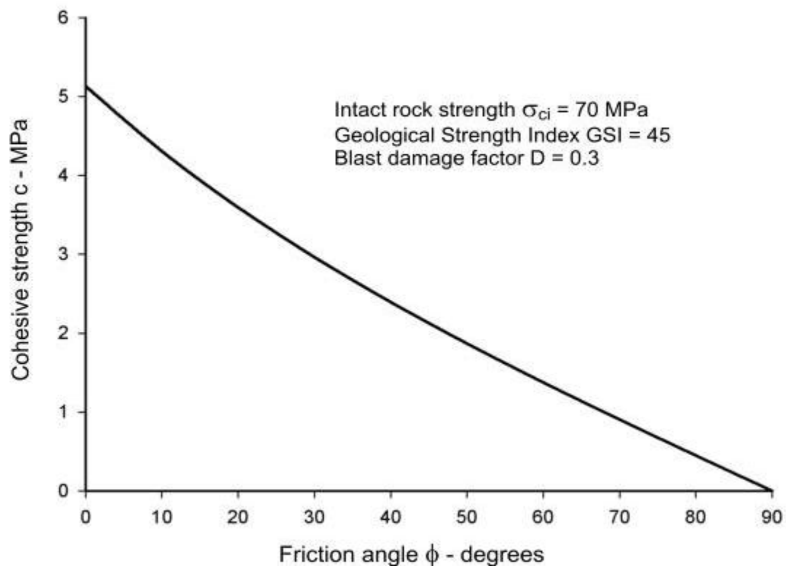



Figure 5 - Relationship between cohesion and friction for assumed rock conditions.

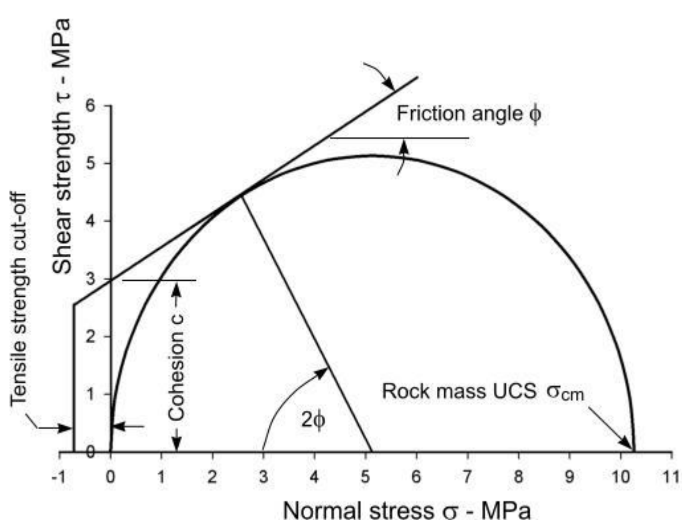



Figure 6 - Definition of cohesion for a chosen friction angle.

Figure 6 illustrates the relationship between the friction angle $\phi$, the cohesive strength $c$ and the uniaxial compressive strength $\sigma_{cm}$ of the rock mass. Also included in this plot is the tensile strength cut-off which is required for some slope stability calculations. The rock mass tensile strength can be estimated as about 8% of the uniaxial compressive strength $\sigma_{cm}$.

Figure 7 gives a number of empirical curves relating rock mass deformation modulus E to rock mass classifications. Hoek and Diederichs (13) proposed that the relationship between the rock mass deformation modulus $E$ and the Geological Strength Index ($GSI$) for different degrees of disturbance ($D$) is best represented by a family of sigmoid curves defined by the equation:

$$E(GPa) = 100\left(\frac{1-0.5D}{1+\exp\left((75+25D-GSI)/11\right)}\right) \tag{2}$$

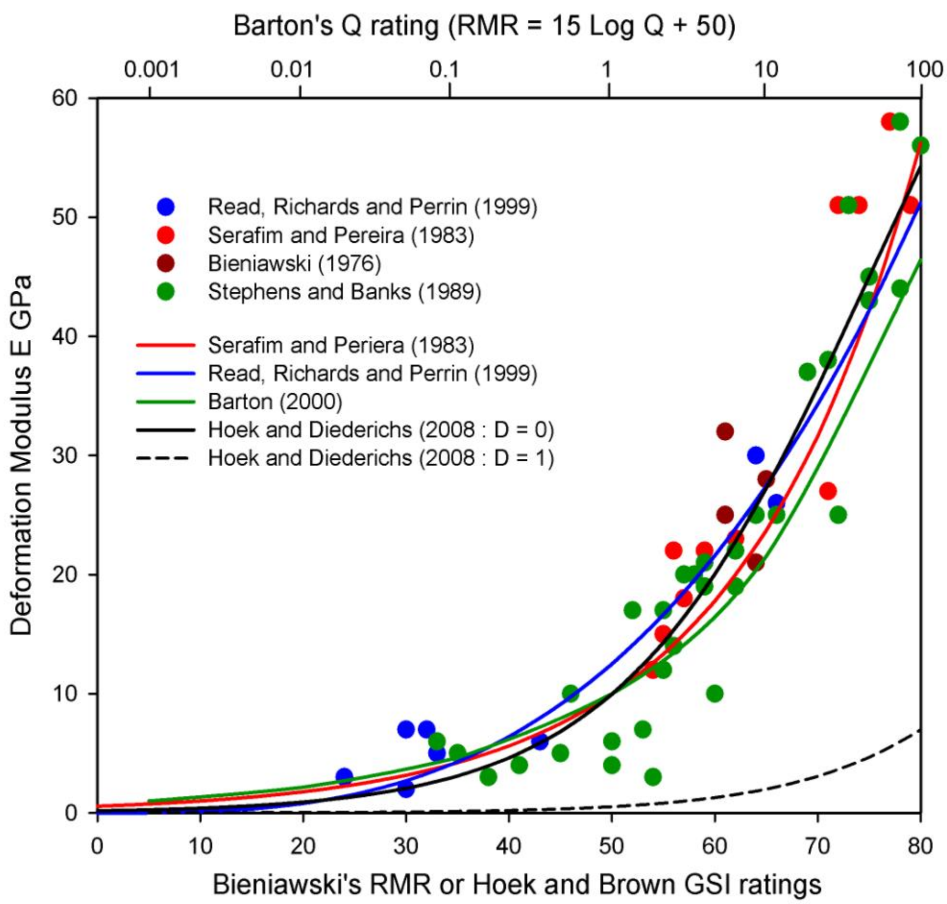



Figure 7 - Estimates of rock mass deformation modulus.

Equation (1) gives a first very crude estimate of the cohesion and friction angle required as input for a limit equilibrium analysis of a slope consisting of homogeneous and isotropic rock or soil masses. If displacements are considered, as in the case of a numerical analysis, then the deformation modulus of each rock mass component can be estimated from equation (2). This approach and refinements of it lie at the heart of all classification based methods for estimating rock mass properties.

Hoek and Marinos (14) have described the efforts that have been made, over a period of about 30 years, to refine the Hoek-Brown criterion and the GSI classification system to cover a wide range of rock masses and to improve its accuracy. Other authors in this field have made similar efforts and the geotechnical literature abounds with papers and discussions on this topic. The question is whether all of these efforts have resulted in significant improvements on the reliability of the estimates of rock mass properties. In retrospect, and bearing in mind that the rock masses considered are limited by the assumptions of homogeneity and isotropy, it is apparent that these efforts have reached the point of vanishing returns. This is not a critical comment since there were very few options available when these methods were developed. However, it is now time to see whether developments in computer technology and numerical methods can help us calibrate or move on from the empirical methods, such as the Hoek-Brown criterion, that have been used for so long.

## MODELLING THE BEHAVIOUR OF DISCONTINUOUS ROCK MASSES

In 1971 Cundall (15) published one of the earliest papers on the modelling of discontinuous rock masses and this led to the development of the Itasca programs UDEC (6), 3DEC (16) and several other programs for modelling jointed rock masses and granular materials. The most recent development in this group of programs is the Synthetic Rock Mass, SRM (17) which holds out the promise of being able to incorporate all significant discontinuity and intact rock properties into a discontinuous rock mass model without having to resort to classification based estimates. Other discontinuous models such as the combined finite-discrete element models described by Munjiza (18), although not yet as well developed for rock mechanics applications as the SRM, show a great deal of promise. The combined finite-discrete element program ELFEN (19) was developed for the dynamic modelling of impact loading on brittle materials such as ceramics, but has been increasingly used in rock mechanics.

It is clearly beyond the scope of this paper to discuss the relative merits of these numerical models, their advantages and limitations and the developments that are still required before they can be considered routine design tools. Participants in this Symposium will find a discussion on these methods in Chapter 10 of the Guidelines for Open Pit Design (1) and in some of the other presentations. The discussion that follows is limited to the use of discontinuous models to estimate rock mass properties and to analyse complex structurally controlled slopes.

**Analysis of rock mass behaviour**

A rock mass comprises blocks of intact rock separated by discontinuities such as faults, shear zones, bedding planes and joints. The failure of such a rock mass involves sliding on multiple discontinuity sets as well as tensile and shear failure of rock bridges and intact rock blocks. How do we develop a realistic representation of the behaviour of such a rock mass?

Potyondy and Cundall (20), in discussing this challenge, point out that systems composed of many simple objects commonly exhibit behaviour that is much more complicated than that of the constituents. They list the following characteristics that need to be considered in developing a rock mass model:

- Continuously non-linear stress–strain response, with ultimate yield, followed by softening or hardening.
- Behaviour that changes in character, according to stress state; for example, crack patterns quite different in tensile, unconfined- and confined-compressive regimes.
- Memory of previous stress or strain excursions, in both magnitude and direction.
- Dilatancy that depends on history, mean stress and initial state.
- Hysteresis at all levels of cyclic loading/unloading.
- Transition from brittle to ductile shear response as the mean stress is increased.
- Dependence of incremental stiffness on mean stress and history.
- Induced anisotropy of stiffness and strength with stress and strain path.
- Non-linear envelope of strength.
- Spontaneous appearance of microcracks and localized macrofractures.
- Spontaneous emission of acoustic energy.

One the tools that has been applied to the problem of analysing the behaviour of rock masses is the Synthetic Rock Mass (SRM) approach developed by Itasca. Cundall (21) explains that the SRM represents a jointed rock mass by an assembly of joint set elements embedded in a matrix that allows new fractures to initiate and grow dynamically according to the imposed stress and strain level. This approach uses the Bonded Particle Model, shown in Figure 8 and described by Potyondy and Cundall (20), for the rock matrix and the Smooth Joint Model, described by Mas Ivars et al (22), for pre-existing fractures.

The Bonded Particle Model is based on the Itasca discrete-element code PFC3D (23) and it represents the rock as an assembly of spherical particles bonded together at their contacts. These bonds can break, depending on the stress level, representing fractures that originate and grow naturally within the assembly. The Smooth Joint Model, which allows slip and opening of planar surfaces independently of the local particle geometry, is used to represent pre-existing joints and fractures. Note that the particles "pass through" each other in order to respect the given sliding direction. These joints and fractures are defined by a Discrete Fracture Network (24) which is based on joint spacing, trace-length and orientation derived from on site drilling and mapping.

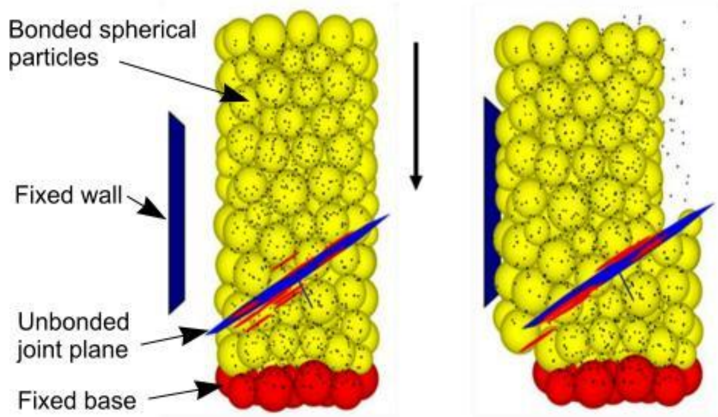



Figure 8 - Illustration of the Bonded Particle Model and the Smooth Joint Model. Modified after Cundall et al (26).

It has been found that most of the characteristics described by Potyondy and Cundall (20), listed above, can be reproduced in a SRM model. These characteristics will vary, depending upon the properties and constitutive relationships assumed for the rock and the discontinuities. Considerable work remains to be done to refine these relationships and to understand all of the complex interactions that occur during the progressive failure of rock masses. However, as illustrated in the paper by Lorig (25), the SRM approach has already been demonstrated to overcome many of the limitations inherent in empirical estimates of rock mass properties.

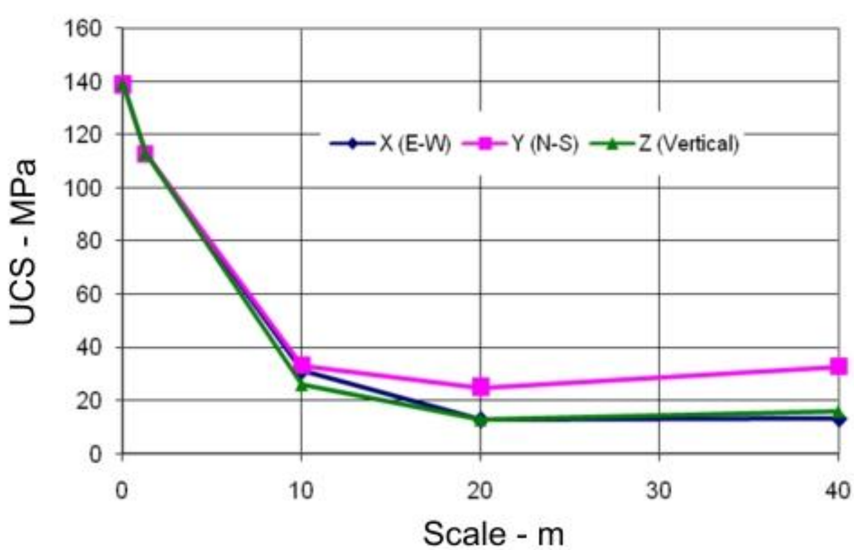



Figure 9 - Numerically obtained values UCS versus sample size for three orientations of the applied axial stress for Carbonatite from the Palabora Mine in South Africa. After Cundall et al (26).

Cundall et al (26) have described the use of the SRM approach to investigate the influence of sample scale on the strength of jointed rock masses. The results of one of these studies, for a Carbonatite rock mass from the Palabora Mine in South Africa, are plotted in Figure 9. This is a long standing problem in rock engineering since it is impossible to conduct physical tests on samples of comparable size to the rock mass into which the slopes of an open pit mine are excavated. While the concept of size effect has been understood for a very long time it is only with the advent of tools such as the SRM that it has been possible to quantify this effect with any degree of certainty.

Cundall (27) suggests that, while the PCF3D approach described above shows great potential, there is still a role for continuum codes which give more reasonable computation times. He discusses the use of one such continuum code called the Ubiquitous Joint Rock Mass model, developed by Sainsbury et al (28), for the study of progressive deterioration of matrix (intact) cohesion and ubiquitous joint failure. The results for one such study are reproduced in Figure 10.

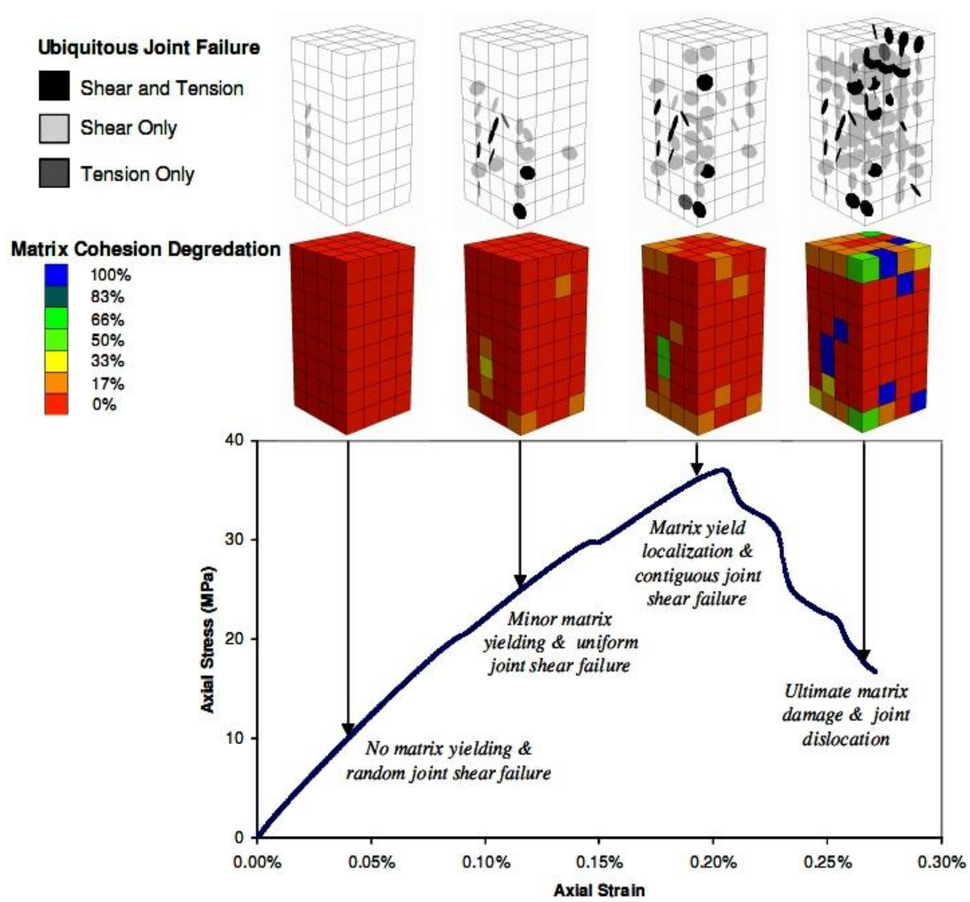



Figure 10 - Stages of damage within a strain-softening ubiquitous joint model including matrix cohesion degradation and ubiquitous joint failure. After Cundall (27).

One of the other advanced numerical codes that has been applied to rock engineering analysis is ELFEN, developed by RockField Software (19). This is a hybrid 2D/3D model that incorporates finite element and discrete element analysis. Failure bands can develop within or between single elements and, when the load carrying capacity across such failure bands decreases to zero, a fracture propagates within the continuum finite element mesh. At this point the mesh is updated and this results in the formation of a discrete element rock fragment. Crook et al (29) showed that, by using a combination of the standard Mohr-Coulomb yield function with a tension cut-off, they could model brittle, tensile axial-splitting fractures and more ductile shear features in ELFEN. This allows the modelling of a continuum and a discontinuum and the transformation of the rock mass from a continuum to a discontinuum. Pre-existing faults and joints can also be inserted into these models.

ELFEN and similar combined finite-discrete element models (18) have not been applied to practical open pit stability problems to anything like the same extent as the Itasca Synthetic Rock Mass models. Hence, the behaviour of rock mass has not been optimised to the same extent. While these models appear to hold out a great deal of promise for applications such as slope stability analyses, it remains to be seen whether future developments can meet the challenges that still have to be overcome.

**Advanced slope stability analysis**

Cundall (27) has described the application of the SRM model to an analysis of the West Wall of the Chuquicamata Open Pit Mine, as defined in Figure 11. This analysis is discussed in greater detail in Chapter 10 of the "Guidelines for Open Pit Design" (1).

The 2 dimensional SRM model is illustrated in Figure 12 and the enlarged inset shows some details of the sliding joints and bonded particle models representing intact rock pieces. The joints were based on data from pit mapping while the intact rock strengths were determined from laboratory testing. The complete model, before excavation, measured 1 km x 500 m and contained 2,890 faults and 37,335 joints. A total of about 330,000 particles made up 38,656 clusters representing blocks of intact rock. The slope was excavated in stages, bench by bench.

The model, shown in Figures 13 and 14, displayed slope behaviour mechanisms consistent with those observed in the actual open pit mine. Figure 13 shows the maximum displacements in the model. The depth of movement, shown in red, is approximately 130 m. A detail of one of the benches, presented in Figure 14, shows the horizontal displacements in the rock mass, the opening up of tension cracks and the toppling in the upper pit walls along a base failure plane. The slope had not collapsed at the end of the model run but it showed slow continued creep which is also consistent with the actual slope behaviour. Figure 15 shows the tension crack formation and a discontinuity surface, in the foreground of the photograph, forming the rear surface of a toppling column. The bench surface at the top of this column, originally horizontal, is now dipping into the pit as a consequence of the toppling process.

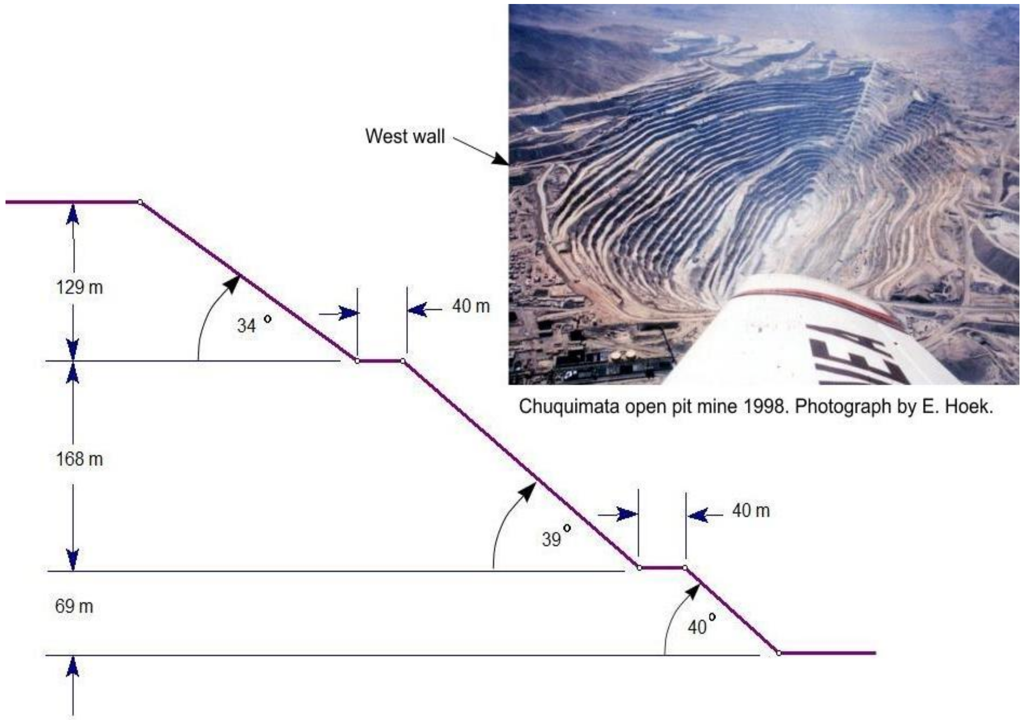



Figure 11 - Simplified cross section of Chuquicamata mine west wall.

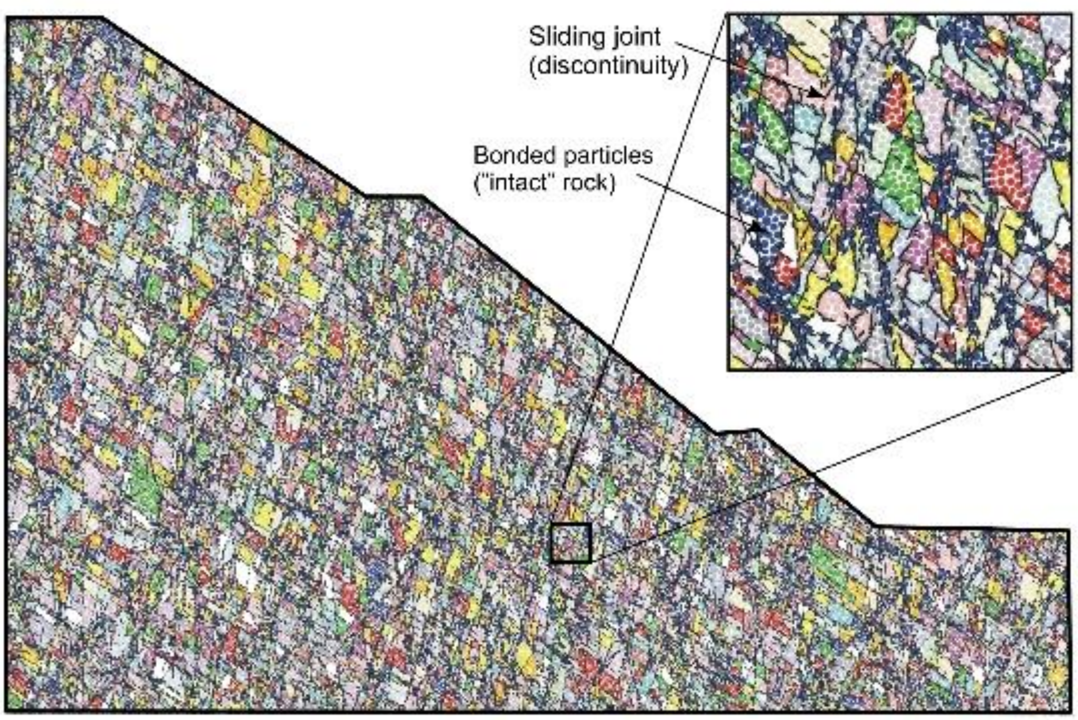



Figure 12 - Two 2 dimensional SRM slope model of Chuquicamata west wall. Modified after Cundall (23).

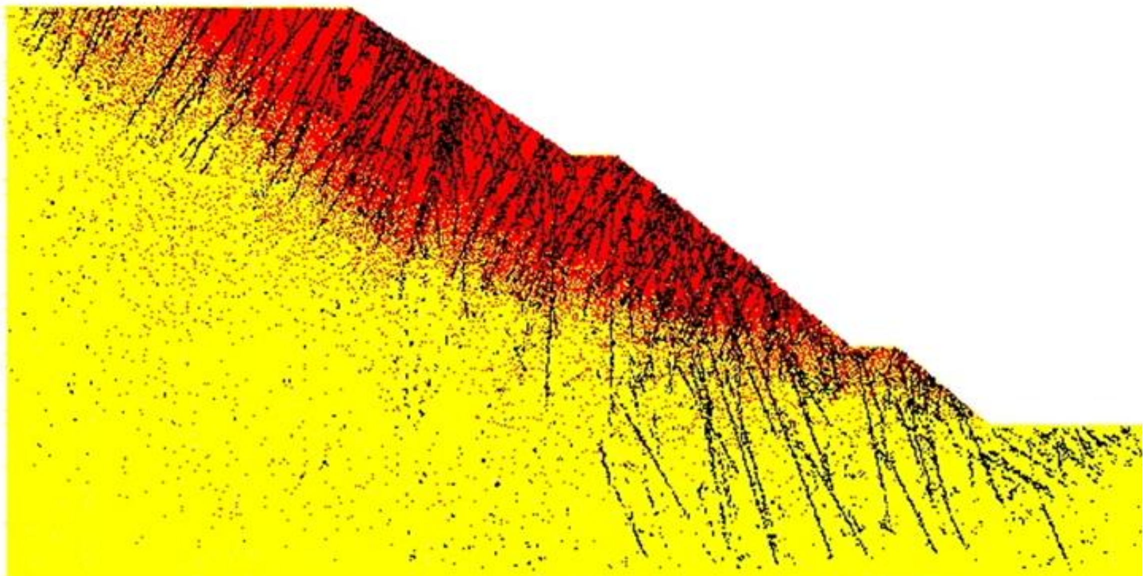


Figure 13 - Movements on the west wall calculated by means of the PFC2D model. After Cundall (27).

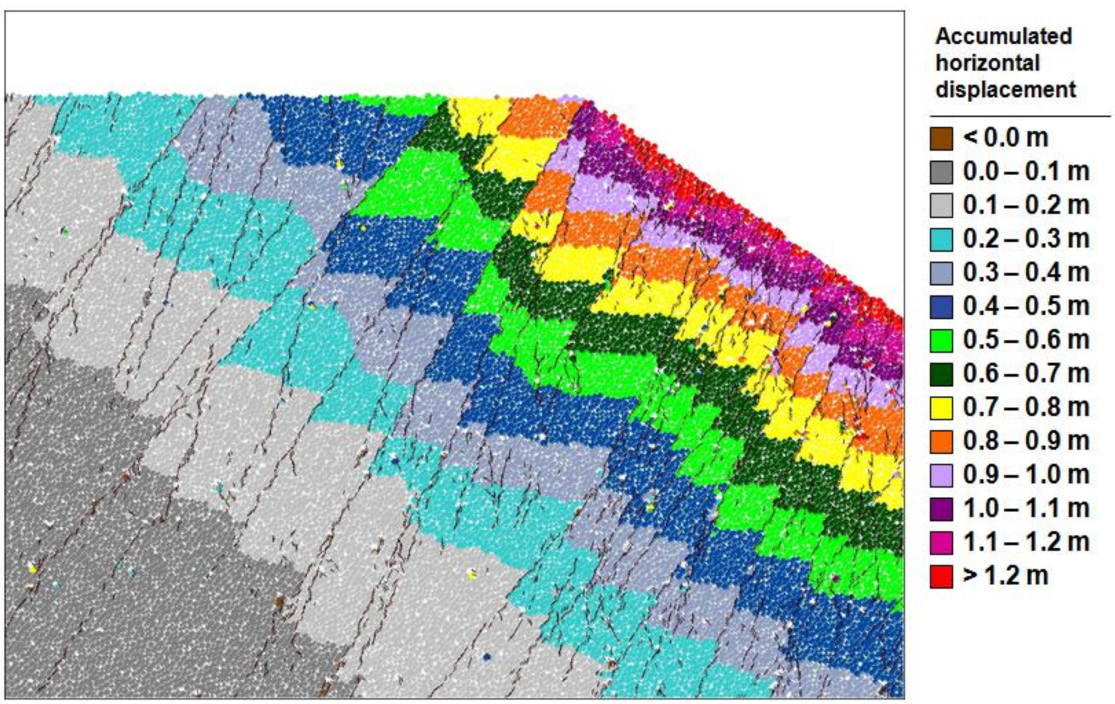
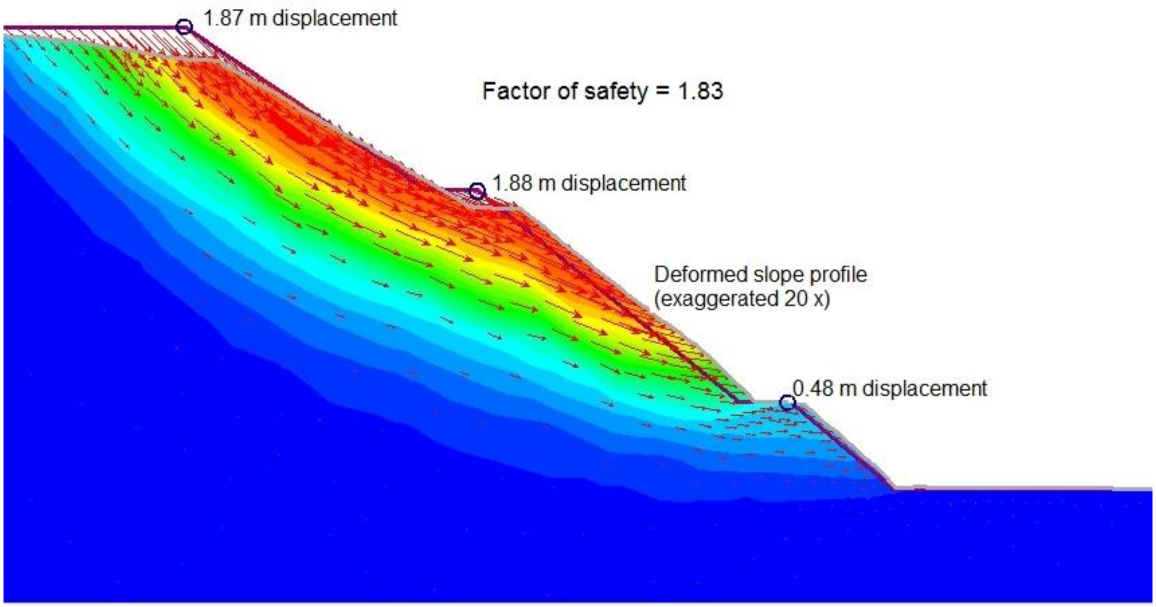



Figure 14 - Detail of horizontal displacements in the open pit wall. After Cundall (27).

Figure 15 - Tension crack formation and flexural toppling in the upper west wall benches of the Chuquicamata mine. Photograph by E. Hoek, June 1999.

It is important to note that the SRM model did not involve the use of any of the rock mass classification schemes, discussed earlier, for the estimation of rock mass properties. No failure mechanisms or failure surfaces were assumed. The entire failure process was generated by the model itself, based on the displacements and failure of individual rock blocks and discontinuities in response to the changing stresses induced by mining.

Before leaving this example it is worth stepping back to consider the results obtained from a more conventional homogeneous continuum model. Figure 16 illustrates the total displacement contours obtained from a Shear Strength Reduction analysis, using the program Phase2 (5). These contours and the displacement vectors suggest a typical circular failure process with maximum displacements of about 1.9 m. A factor of safety of 1.83 indicates that the slope is stable. In fact, when these displacements and the factor of safety are compared with the conditions observed in the field, the results appear to be reasonable. However, as discussed above, the actual slope behaviour suggests that most of the deformation is due to flexural toppling rather than to circular sliding. This toppling process, which is controlled by discontinuities within the rock mass, cannot be reproduced in a homogeneous continuum model.




Equivalent continuum rock mass properties:

| | | | |
|---|---|---|---|
| Unit weight $\gamma$ | 0.026 MN/m$^3$ | Cohesion c | 0.22 MPa |
| Deformation modulus E | 4,600 MPa | Friction angle $\phi$ | 38 degrees |
| Poisson's ratio $\mu$ | 0.25 | Tensile strength $\sigma_t$ | -0.09 MPa |

Figure 16 - Total displacement contours for homogeneous model given by the Shear Strength Reduction method, using the program Phase2 version 7 (5).

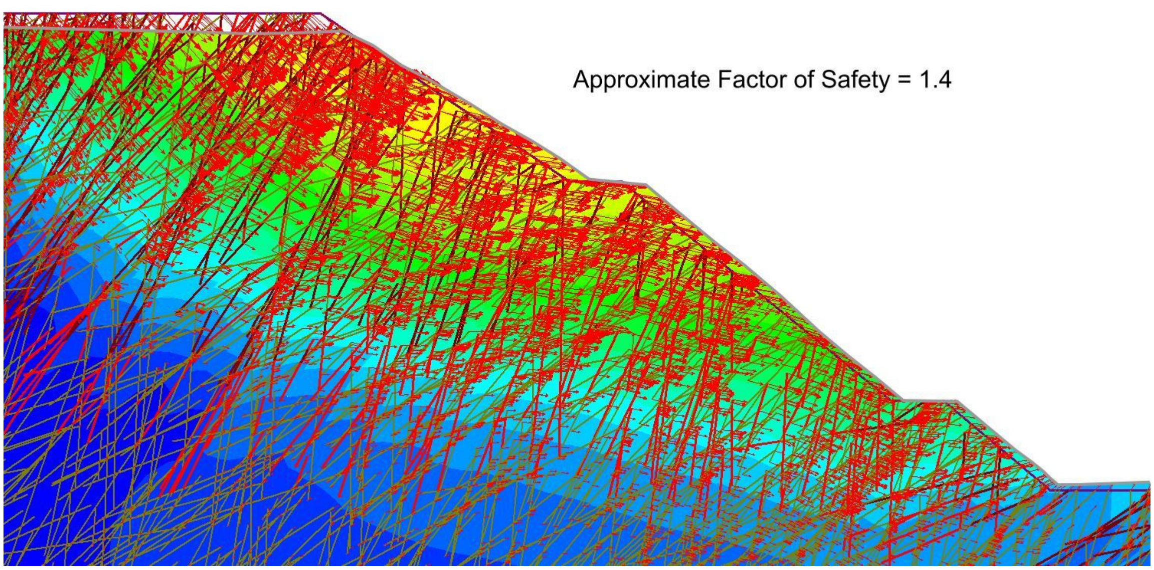



Figure 17 – Total displacement contours for faulted model given by the Shear Strength Reduction method, using the program Phase2 version 7 (5).

The superposition of a pattern of faults in the homogeneous continuum model, used to generate Figure 16, results in a significant change in the rock mass behaviour. As illustrated in Figure 17, the displacement vectors indicate a predominantly toppling failure process which is much more realistic than that for a homogeneous model. However, this process is only qualitative since the detailed mechanical behaviour of both the intact rock pieces and the discontinuities have not been included in the model as for the Synthetic Rock Mass model illustrated in Figure 12

A comprehensive analysis of the failure in a jointed continuum model requires that tensile and shear failure of the "intact" rock pieces have to be considered in addition to the shear failure of discontinuities. It is only relatively recently that jointed continuum models with this capability have been developed (28), as shown in Figure 10. Such models are attractive because of the lower demands on computer capacity than for equivalent SRM models and there appear to be significant advantages in further developing these models.

It can be argued that most slope designers are interested in designing stable slopes and that it is not necessary to model the large displacements and detatchment of rock blocks as can be done in models such as the Synthetic Rock Mass. There is considerable merit in this argument and it is important that development of jointed continuum models should be continued in order that they can be used in cases that do not demand the capabilities of the SRM models.

There are also cases, such as that discussed below, where it is necessary to study the break-up of rock masses and the movement of blocks down the slope. While the SRM model can be used for such analyses there are also simpler alternatives that can be considered.

**Rockfall analysis**

Watson et al (30) and Lorig et al (31) have published the details of the analysis of a potential slide of the Checkerboard Creek slope located 1.5 km upstream of the Revelstoke Dam on the Columbia River in British Columbia, Canada. Measured displacements and visible slope deformation features on the ground surface indicate that the rock mass is moving at a rate of up to 13 mm/year downslope towards the Revelstoke Reservoir. Between 1984 and 2005, British Columbia Hydro carried out an investigation of the risks associated with a sudden rockfall depositing a significant volume of rock into the reservior and creating waves with the potential for overtopping part of the dam.

The photograph reproduced in Figure 18, taken on one of the upper benches during site investigations, shows strongly developed steeply dipping joint sets in the massive to weakly foliated granodiorite. Kinematic analysis suggests that the slope is stable but there was concern that failure of some of the large blocks, possibly during an earthquake, could generate slope failure.

Figure 18 - Checkerboard Creek slope formed by a steep roadcut. The photograph, showing the sub vertical joints, was taken on one of the upper benches during site investigations. Photograph by E. Hoek, 2004.

A cross-section through the slide is shown in Figure 19 which also includes the UDEC model used in this analysis. The rock blocks were represented by rigid polygons with shapes defined by the real joint structure (Figure 18). Artificial (Voronoi) joints were included within the rigid polygons to represent internal flaws within the massive rock blocks. The sizes of the Voronoi blocks were chosen to be representative of the rock blocks that were observed during construction of the rock slope. All of the rock blocks were assumed to be rigid. The shears and conjugate joints were assigned a friction angle of 25° with zero cohesion. The simulated collapse was controlled by downgrading the tensile strength of the Voronoi joints from 10 MPa to 4 MPa to represent the weathering process of intact rock blocks.

The failure process in the model involved toppling caused by downward sliding of blocks along shears and conjugate joints leading to bulging of the slope near mid height and tensile failure of joints within the rock blocks. The process of failure of the UDEC model is illustrated in Figure 20.

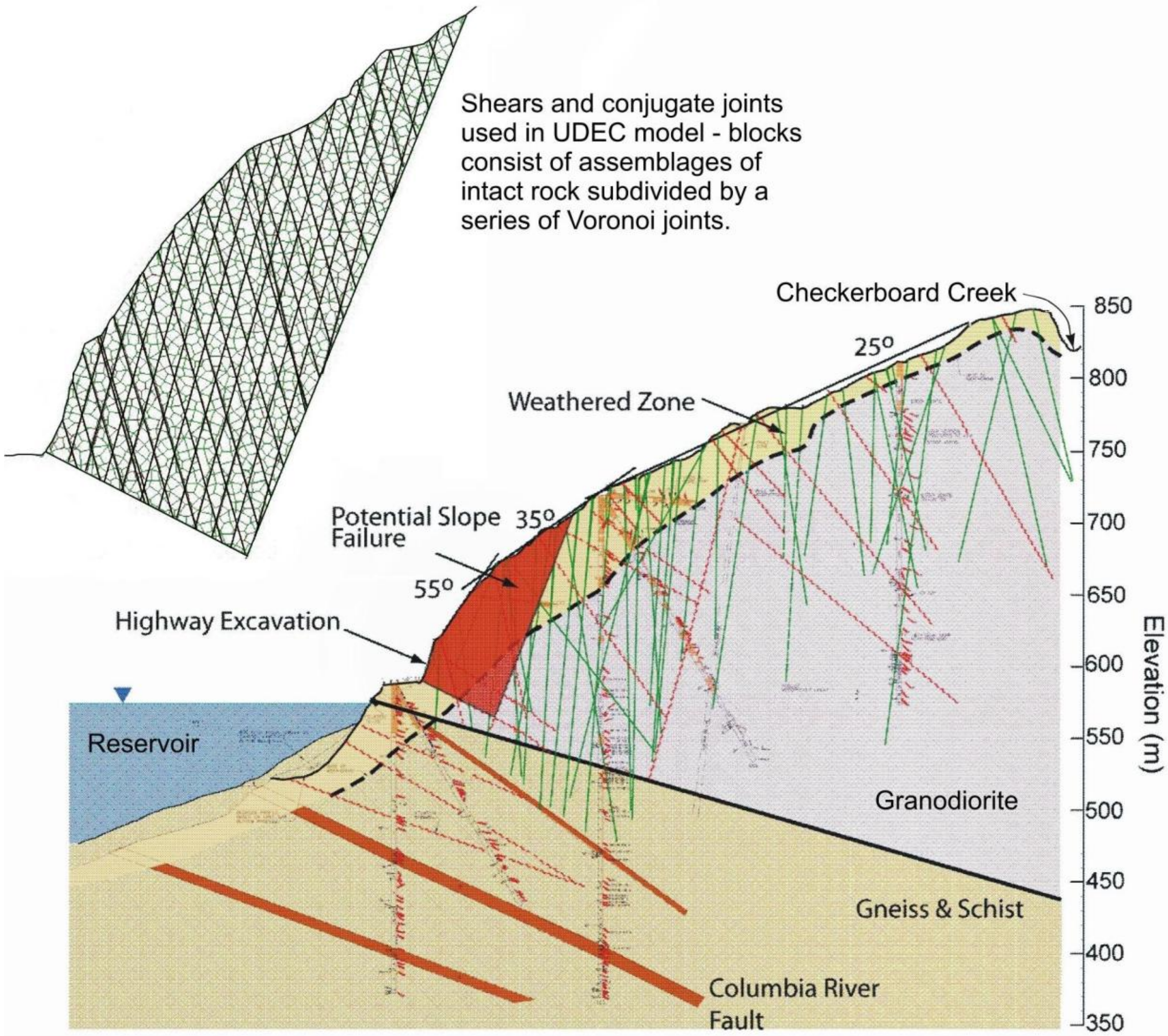



Figure 19 - Checkerboard Creek cross-section and details of UDEC model. Modified after Watson et al (20)


Figure 20 - UDEC model used to assess potential slide dimensions and velocities. Modified after Watson et al (20)

In the UDEC model, used to simulate the failure of the potentially unstable part of the Checkerboard Creek slope, no effort was made to model the precise failure mechanism of the intact rock blocks defined by the intersecting shears and conjugate joints. Failure of the Voronoi joints, when the tensile strength was exceeded, achieved the goal of breaking-up the rock mass into blocks of representative size and freeing these blocks to move down the slope. Einstein is credited with having said "Keep it simple; as simple as possible, but no simpler" and this model is an excellent example of using the capabilities of the model required to simulate the process being studied without over-complicating the model.

The range of slide behaviour was investigated by varying the following five factors: strength of joints and shears, type and amount of damping used to simulate the coefficient of restitution and thus energy loss during runout, friction angle of' contacts during movement, reservoir effects and rock fragment sizes. The UDEC model provided a wide range of results and these were used as input for a scaled physical model used to generate waves in the reservoir. The integration of site investigation, monitoring, numerical modelling and the construction of the physical hydraulic model resulted in a defensible approach to risk assessment at the Checkerboard Creek rock slope. Each stage was essential in determining that there is a negligible risk of a single event rockslide larger than 0.5 million $m^3$ and that the wave from such a slide would not overtop the Revelstoke Dam (30).

## CONCLUSIONS

The economy of scale has made large scale open pit mines or underground large caving operations attractive for the recovery of low grade mineral deposits. These massive operations have placed new demands on rock engineers to design slopes and underground mines on a scale not attempted before. In recognition of deficiencies in available design methods the mining industry has funded a series of large research projects including the International Caving Study (ICS), started in 1997, and its successor the Mass Mining Technolgy (MMT) project and the parallel Large Open Pit (LOP) project. The main outcome of the LOP project has been the "Guidelines for Open Pit Design" which will be published at this symposium.

The development of numerical models to study the failure and deformation of large rock masses has been an integral component of all three programs mentioned above. Itasca has been one of the principal contractors in these programs and the development of the Synthetic Rock Mass and its components has been one of the most significant developments.

One of the major features of the SRM system is the fact that rock mass models can be built up from basic properties of intact rock and of rock discontinuities without having to rely on estimates based on rock mass classification systems such as GSI. The joint spacing, trace lengths and orientations are derived from drilling and mapping on site and are incorporated into the SRM model by means of a statistically based Discrete Fracture Network (24). The properties of the intact rock and of the discontinuities are determined from laboratory tests and are used directly in the models.

At the very least these types of models provide a means of calibrating empirical rock mass classification schemes. As described by Cundall (21), a testing environment has been developed for SRM samples for a standard suite of direct tension and uniaxial and triaxial compression tests. These tests allow the systematic characterisation of the mechanical behaviour of jointed rock masses. The author is hopeful that, in time, a sufficient number of numerical "tests" will have been conducted that the classification schemes can be reorganised and calibrated to provide much more rational and reliable classifications than those which exist today.

At the other end of the spectrum these systems can be used to construct two and three dimensional models of complete rock masses, such as that illustrated in Figure 12, into which slopes can be excavated sequentially to simulate the mining process. Failure processes in such rock masses are generated from the changing induced stresses as the open pit is mined and, provided that the properties of all of the components have been correctly represented, the overall model rock mass behaviour will replicate that of its in situ counterpart.

The Synthetic Rock Mass is currently the most advanced system of its kind for modelling rock mass behaviour. Other models such as the combined finite-discrete element method (18, 19) and the ubiquitous joint rock mass model (28) are alternatives that appear to have great potential and that may offer significant advantages in certain applications. It will probably require many years of development before robust and reliable models emerge as leaders from the large number of competing codes currently under development.

Numerical models, in themselves, do not provide a complete solution to open pit slope design. As demonstrated by the large number of topics covered in the " Guidelines for Open Pit Design", a complete slope design requires consideration of a large number of geological, geotechnical, groundwater and operational factors. The results of a slope stability analysis, by whatever means this is carried out, should always be considered in association with these other factors and should be checked wherever possible by back analysis of monitored displacements and slope failures. A slope stability calculation should never be treated as an end in itself but rather as a contribution to the overall design process.

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