11. Rock slopes in civil and mining engineering

Introduction

Free-standing unsupported rock slopes are important components of open pit mines, large excavations for building foundations, highway cuts in mountainous terrain large and mountain slopes in which landslides can occur. Installation of support or reinforcement in these slopes is uneconomical. Their safety depends upon the inherent strength of the rock masses in which they are excavated or in which they were created by mountain building processes. Hence, the "design" of the slopes is limited to the recognition of weak features, such as through-going joints and faults, the understanding and control of groundwater pressures in the slope and the control of blasting and mechanical excavation methods used in creating the slopes.

Hoek, Read, Karzulovic and Chen, (2000) presented a comprehensive review of the state of the art of the design of these slopes with an emphasis on open pit slopes. Their paper provided much of the discussion in the following text.

In the original version of this chapter, written in approximately 2003, it was stated that there was a lack of current research in rock slope stability and rock mass properties, and that it was not clear where suitably qualified researchers could be found and how such research on open pit stability would be funded. In April 2005 an inaugural meeting was held in Santiago, Chile, to finalize plans for the Large Open Pit Project, an international research and technology transfer project on the stability of slopes in open pit mines. This project was funded by 12 mining companies and managed on behalf of the CSIRO in Australia by Dr John Read. Some of the publications resulting from, and following on, from this project are as follows:

Read, J. and Stacey. P. 2009, Guidelines for Open Pit Slope Design, CSIRO Publishing, Australia and CRC Press/Balkema, The Netherlands, 496 pages.

Beale, G. and Read, J. 2013. Guidelines for Evaluating Water in Pit Slope Stability, CSIRO Publishing, Australia and CRC Press/Balkema, The Netherlands, 600 pages.

Hawley, W. and Cumming, J. 2017, Guidelines for Mine Waste Dump and Stockpile Design, CSIRO Publishing, Australia and CRC Press/Balkema, The Netherlands,

Martin, D, and Stacey, P. 2018. Guidelines for Open Pit Design in Weak Rocks. CSIRO Publishing, Australia and CRC Press/Balkema, The Netherlands, 398 pages.

Sharon, R. and Eberhardt, E, 2020, Guidelines for Slope Performance Monitoring, Taylor and Francis Group, CRC Press. 330 pages.

There are numerous additional publications which can be found in an Internet search for rock slope topics. Hence, it is clear that there is now an abundance of publications which deal with many of the issues related to rock slope design and the reader will find that many of the topics in this Chapter have been covered in great detail in these publications.

The geological model

In an open pit mine a significant effort is devoted to defining a reliable geological model in order to locate and quantify the ore reserves. Consequently, the refinement of these models to include features, such as the distribution of weak features like joints which can cause slope instability, is both logical and feasible. This is not the case for slopes for foundations and highways where the profile of the slopes in the excavation is of fundamental importance and where detailed geological investigations are generally limited to those locations which show signs of potential instability. Similarly, in the case of landslides, detailed investigations are only undertaken when there is a history of slope instability.

Rock slope failures are geological events controlled by natural physical processes. Geological-geotechnical models that can be used to understand and analyse these processes must include structural data as well as information on lithology, mineralization and alteration, weathering, hydrogeology and rock mass characteristics such as joint persistence and the condition of the joints. The most important information is the structural data which should include information on major structures and on structural domains. This information must be sufficiently detailed that meaningful structural trends can be derived and used as input for analytical models.

Routinely, these models are stored and manipulated electronically. The availability of a number of computer geological models is an important aspect of modern rock slope design. Packages such as Minex-Horizon, Vulcan and Minescape provide three-dimensional solid modelling systems which permit the construction and visualization of comprehensive models that can include geological and structural geology information, ore grade distributions, groundwater distributions and a variety of geotechnical details. The construction of these models is itself a useful exercise since it highlights deficiencies in the database and forces the user to consider the inter-relationships between the various types of information displayed in the model.

These modelling systems are already fully operational and are used routinely by most large- scale open pit mining operations. However, they are very seldom used in civil engineering projects which is a serious deficiency that I hope to see remedied in the years to come. Under development are interfaces which will allow a direct transfer from these three-dimensional models to other types of limit equilibrium and numerical models used for slope design.

Geological and geotechnical data collection

The tools and techniques for geological data collection are well developed and have been widely used by the mining and civil engineering industries. The definition of what information to collect and the efficient use of the collected data is another matter. There is a fundamental need to collect good quality geological data in the form of original and site lithological, structural and hydrological information, detailed core logs and photographs that can be interpreted by an engineering geologist at any time. Currently, there are a few standards for data collection. The type and quality of information gathered depends very much upon the personal opinion and preferences of the individuals or organizations carrying out the work. Disadvantages of this approach are that the reasons for collecting the data become unclear and it becomes difficult to maintain continuity when staff or organizational changes occur.

The current state of practice includes a variety of rock mass classification schemes, some of which have been specifically adapted for rock slope engineering (Haines and Terbrugge, 1991, Romana, 1995, Chen, 1995). Unfortunately, the use of these classification systems in rock slope studies is questionable since most of them were developed for the confined conditions that apply in the rock masses surrounding underground excavations. These classifications have been found to produce unreliable results in the low confining stress conditions in slopes. The use of classification systems, in which the properties of the rock mass are characterized by a single number, also tends to produce a false sense of security. At best, it may be possible to define ranges of values of these rock mass quality indices. These ranges may be used to delineate zones that can be used to complement all the other geological, hydrogeological and geotechnical information. Where classification systems are used, the need to understand their limiting assumptions, and calibrate them to the regional and local site conditions, is emphasized.

The input of the results of the geological data collection process into a geological model should preferably be carried out by an engineering geologist. Deficiencies and anomalies in the data become obvious during this model construction process which provide useful guidance to the development of future site investigation programmes.

The role of major structures

No one questions the role of major geological discontinuities such as lithological boundaries, bedding planes, or faults in controlling the stability of large slopes. Clearly, it is essential that the data gathering process embraces the development of regional and local geological models, which include these major structures, and that on-going mapping of these structures as they are exposed in the slopes, be used as a means of continually updating the geological model.

From a practical point of view, the role of second order structures, such as joints, is more of a problem. Their importance in controlling the behaviour of the rock mass is clearly recognized but, because of the large number of such features, a question that arises is - how much data should be collected and how should it be used in the design of rock slopes?

The current state of practice tends to separate slope designs into two distinct categories. The first of these categories is for those designs that can be dealt with in terms of kinematically possible structurally controlled failures. For example, failures that involve wedges, sliding along the line of intersection of two intersecting faults can be analyzed using limits equilibrium models. This type of failure is commonly seen in slopes of up to 20 or 30 m high in hard, jointed rock masses. The design of such slopes can sometimes be based upon analysis of simple wedge failures.

The second category is that which includes non-structurally controlled failures in which some or all the failure surfaces pass through the rock mass which has been weakened by the presence of joints or other second order structural features. An assumption commonly made is that the second order structural features are randomly or chaotically distributed and the rock mass strength can be defined by a simple failure criterion which is 'smeared' or 'average' non-directional strength properties that are assigned to the rock mass. This approach is frequently used for the analysis of the overall stability of large slopes where no obvious failure mode presents itself.

Figure 1: Slope failure in which some structural control by faults is evident at the top of the failure, but where the mechanisms involved in the low part of the failure are unclear.

Figure 2 is an approximately 350 m high mine slope failure. The rock mass within the failed material is altered and closely jointed. There is evidence of faults at the back of a failure, but what triggered the failure? Was it the faults at the back of the failure or was it a non-structurally control failure where most of the failure surface passes through an altered rock mass which has been weakened further by the presence of joints or other second order structural features? Can the failure be analyzed and the slope redesigned?

Figure 3 illustrates a joint map of the foundation for the Three Gorges Dam, published by Wang, Chen and Jia, (1998) and Chen (1999). This map was generated by Monte Carlo analysis using probability density functions for the joint dip, strike, spacing and trace length (Priest and Samaniego, 1983). The map was then used in stability analysis of this 60 m high slope by searching for joint and rock bridge combinations that result in minimum shear strength along potential failure surfaces. A similar process was used to investigate the seepage and drainage processes in the rock mass by considering the conductivity of the joint fabric.

Figure 2: A large scale slope failure in an open pit mine.

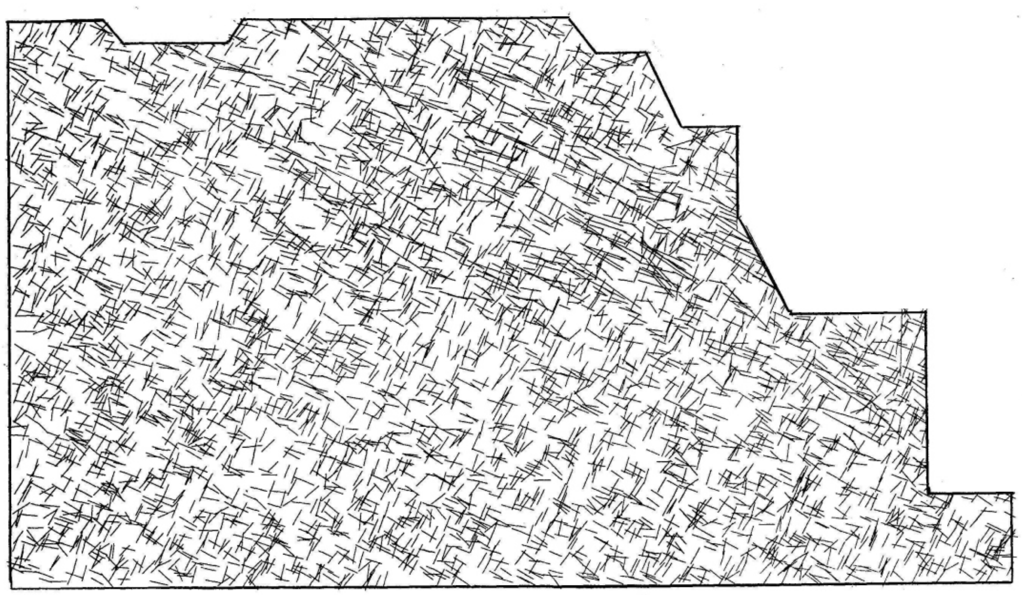

Figure 3: Joint map for the rock mass forming the foundation of the Three Gorges Dam in China.

Sainsbury and Sainsbury (2013) reported on three-dimensional slope stability models, developed in Australia, for the analysis of complex failures in jointed rock masses. Their papers are highly recommended reading for anyone seeking a deeper understanding of these analyses.

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Determination of rock mass properties

The determination of rock mass strength is a significant challenge in current rock slope design practice. Even in terms of the state-or-the-art, there are many unanswered questions and many opinions on how this task should be performed in the low stress environment that is characteristic of rock slopes, particularly where weak/altered rocks are present.

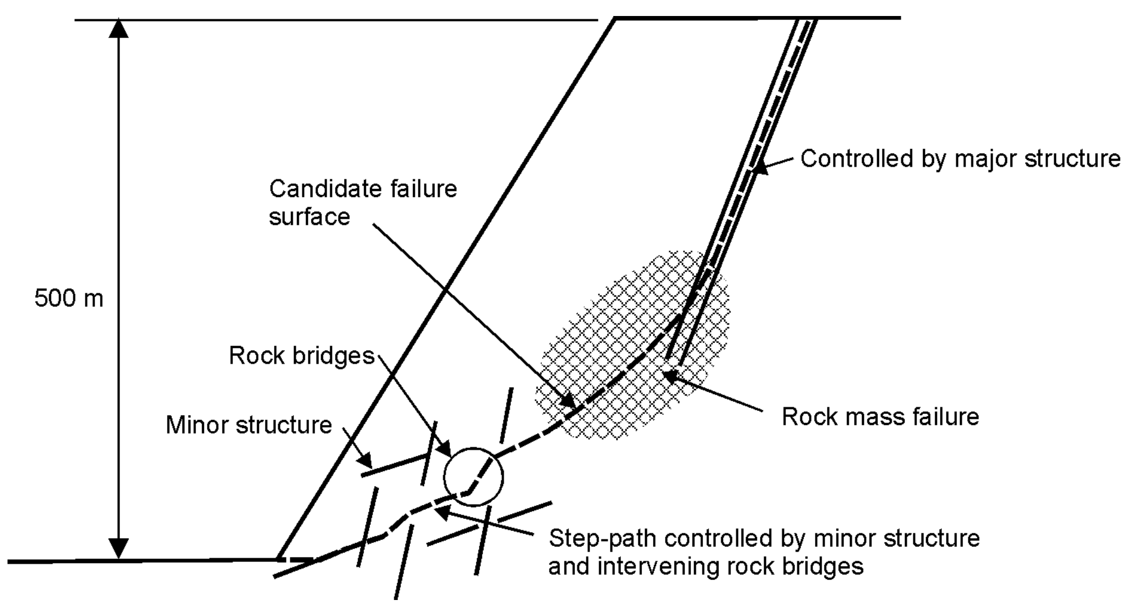

Figure 4: Candidate failure surface involving a number of different shear failure mechanisms.

In terms of the state of practice, most slope designers use some form of 'smeared' failure criterion to estimate the shear strength properties of the rock in the blocks or domains defined by major structural features such as faults. The Hoek-Brown failure criterion is commonly used for estimating the properties of these 'homogeneous and isotropic' rock masses. An alternative is to estimate mass shear strength value from the component parts along a given candidate failure surface, shown in Figure 4. For the failure surface illustrated, the component shear strengths could be obtained from (i), direct shear tests of samples taken from the fault that defines the upper part of the surface (ii), the Hoek-Brown failure criterion, for the central part of the failure and for the rock bridges in the step-path surface at the toe of the slope and (iii), either direct shear tests of samples taken from the joints or by applying a method such as the Barton-Bandis shear failure criterion to the joints forming the step-path surface at the toe of the slope.

Impact of alteration

In open pit mining, copper porphyry deposits are associated with a zone of altered rock as a result of the ore emplacement process. In some areas this alteration is relatively mild and does not have a major impact on rock mass strength. On the other hand, in most of the open pit mines associated with the South American Andes, the orebodies are surrounded by a halo of strongly altered rock. The impact on rock mass strength and slope stability is significant

Results of tests at the Chuquicamata Mine in Chile suggest that potassic alteration has the least influence on rock strength, chloritic alteration has a significant impact and quartz-sericitic and argillic alteration have a major impact. These strength reductions are illustrated in Figure 5.

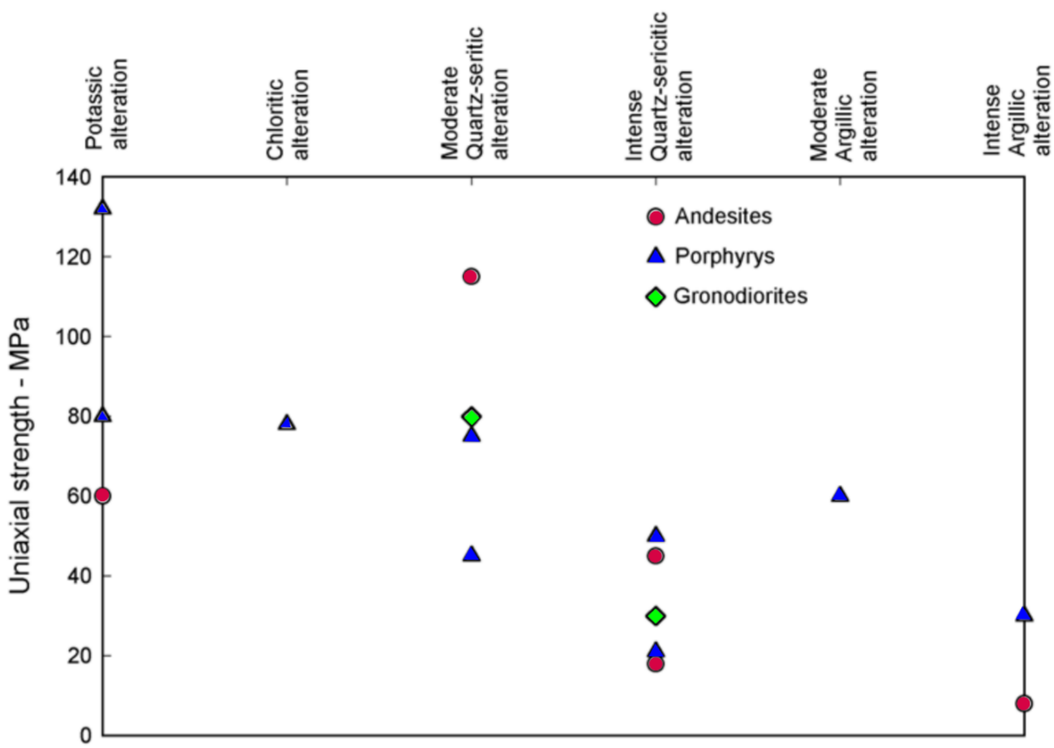

Figure 5: Relationship between intact rock strength and degree of alteration.

Role of groundwater

The presence of groundwater in a rock slope is a critical factor in any assessment of the stability of that slope. Water pressure, acting within discontinuities in the rock mass, reduces effective stresses with a consequent reduction of shear strength. Depressurisation using horizontal or vertical wells or drainage galleries is a powerful tool in controlling slope behaviour.

The technology and tools for groundwater pressure, flow evaluation and control are well developed. It is considered that no further research into this area is required. While it is difficult to maintain piezometers and drains during the excavation of a slope, this is often used as an excuse for not maintaining adequate control or monitoring of groundwater conditions. It is also important that water from horizontal drains should be collected and piped or pumped to a disposal area away from active slope stability problem areas.

As for the geological and geotechnical models discussed earlier, the development of a good groundwater model is an important component in the rational design of large slopes. It is important that resources be provided to ensure that sufficient information is collected to permit the construction of such a model.

On the question of drainage versus depressurisation, it is water pressure that creates slope stability problems and, provided that these water pressures are reduced, it is not necessary for a 'drainage' hole or well to produce large water flows. This is a common misconception which leads operators to abandon 'drains' that do not appear to be working because they do not produce much water. The judgement should be based upon the response of piezometers, which reflect water pressure change, rather than on volume flow. Sub-horizontal drainage holes can be very effective in hard rock slopes provided that they are long enough to depressurise the rock mass in the vicinity of potential failure surfaces. In large rock slopes, holes of 200 to 300 m in length may be required to achieve this goal.

Drainage galleries can have an important function, not only because of the depressurisation that can result from their construction, but also because of the valuable geological information that can be collected from locations that are not normally accessible. Typical drainage gallery construction costs are in the range of US$ 1500 per meter which can sometimes give an overall depressurisation scheme that is comparable in cost to one based upon horizontal holes and/or vertical pumped wells. More serious consideration has to be given to galleries for slope depressurisation than is currently the case in practice.

An example of the use of depressurisation to control slope instability is illustrated in Figure 6. This shows a section through a potential landslide in a hillside known as Dutchman's Ridge, immediately upstream of the Mica Dam on the Columbia River in British Columbia, Canada. This 700 m high 155-million-ton potential slide was recognised during site investigation work for the Mica Dam, constructed in the 1960s. However, it was decided that no remedial action would be taken at that time but that the slope would be monitored by means of electro-optical distance measurements from stable observation posts across the valley. Movements averaging approximately 1 cm per year were measured over a 20-year period and, during a re-evaluation of dam safety in the early 1980s, it was decided that some form of stabilisation was required in order to reduce the downward movement of the slope. A detailed geological and geotechnical investigation established that the slide mass was moving on a basal fault surface, dipping parallel to the slope and it was concluded that depressurisation was the only feasible stabilisation option. The aim of the depressurisation programme was to reduce the water levels in the slope to approximately the equivalent of the levels that existed before the slope toe was submerged by impoundment of the reservoir. It was argued that the original slope had been stable for approximately 10,000 years since the last ice age and that it had withstood several large earthquakes known to have occurred in this area of British Columbia. Hence, it was felt that restoring the groundwater conditions to the pre-reservoir conditions would also restore the stability of slope.

Figure 7 shows the layout of the drainage gallery and the boreholes drilled from underground to target zones of high groundwater pressures. These zones had been determined from an array of 256 piezometer measuring points in the rock mass. The reduction in groundwater levels is also illustrated by means of the contours included in Figure 7.

Although analysis of the slope, using two- and three-dimensional limit equilibrium methods, indicated that the factor of safety was only increased approximately 6% by the depressurisation programme, measurement of displacements indicated that these had been reduced to negligible levels and the slope stabilisation program was judged to have been successful.

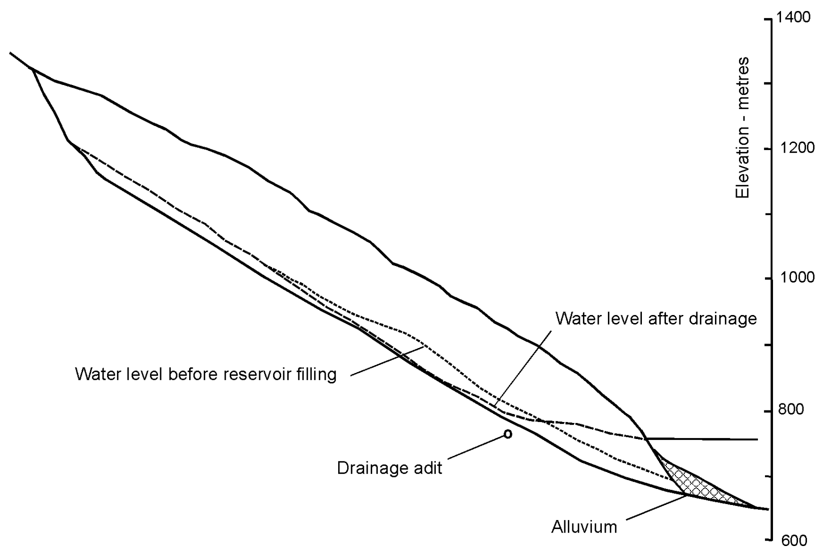

Figure 6: Section through Dutchman's Ridge slide showing the location of the drainage gallery and the phreatic surfaces before impoundment of the reservoir and after impoundment and drainage.

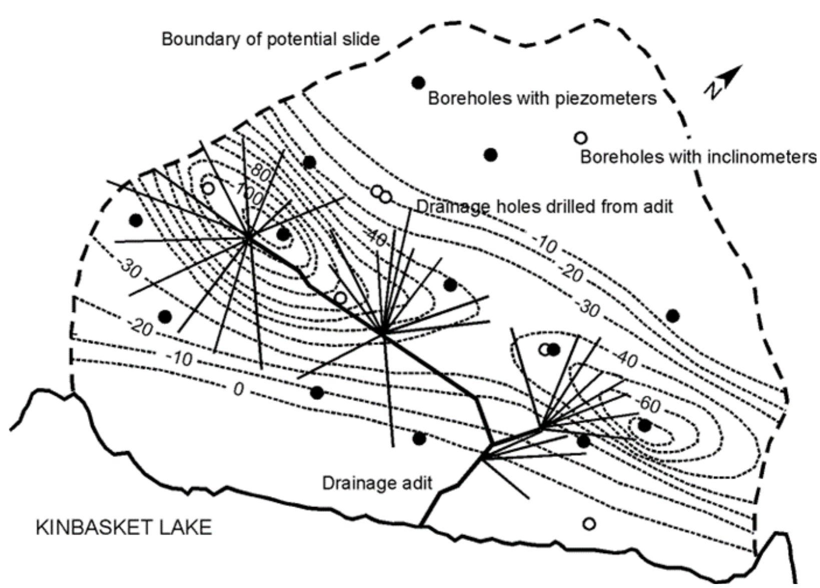

Figure 7: Extent of potential slide with layout of drainage gallery and drain holes drilled from underground. Contours indicate the drawdown of the groundwater levels achieved by the drainage.

Figure 8: Collection of water from the boreholes drilled from the Downie Slide drainage gallery.

A similar depressurisation programme was used to stabilise the Downie Slide on the Columbia River, between the Mica and the Revelstoke Dams. The effectiveness of the underground drainage holes drilled from the Downie drainage gallery is illustrated in Figure 8. In both the Dutchman's Ridge and Downie cases, the drainage galleries have required routine maintenance but the drain holes have continued flowing and have kept the groundwater levels under control for the past 30 or more years.

In situ rock stress

It is well known that rock noses or slopes that are convex in plan are less stable than concave slopes. This is generally because of the lack of confinement in convex slopes and the beneficial effects of confinement in concave slopes. These observations provide practical evidence that lateral stresses, in the rock in which slopes are excavated, can have an important influence on slope stability.

In the current state of practice in rock slope design, these lateral stresses are usually ignored or are dealt with in a very simplistic manner. In fact, all limit equilibrium models are based upon gravity loading only and lateral stresses are excluded from any slope stability analysis that uses these models. Numerical models can incorporate lateral stresses, but most analyses, using these models, are based upon a very simple approximation in which the horizontal stress applied to the model is some proportion of the vertical stress.

Measurement of the in-situ stress field in the vicinity of large slopes is seldom carried out, in spite of the fact that such measurements are entirely feasible. This is because the in-situ stress field is generally considered to be of minor significance. This assumption may be adequate for small slopes, but it needs to be questioned for the design of very large slopes. Lorig (1999), based on the results of numerical analysis, suggests that in situ stresses have no significant effect on the safety factor. However, they do have an influence on deformations, and if the slope is composed of materials that weaken as a result of deformation, then the in-situ stress can have a very important effect in reducing strength and thereby affecting slope stability.

Before embarking on a programme of in situ stress measurement for any specific site, it is probably worth carrying out a parametric study using a three-dimensional model, such as FLAC3D, to determine whether variations in horizontal in situ stresses have a significant impact upon the stresses induced in the near-surface rock in which slope failures could occur. In cases in which lateral stresses in potential failure zones show a large variation, serious consideration would have to be given to a field programme to measure the in-situ stress field.

Blast damage

In the case of large production blasts in open pit mines, blast damage can extend many tens of meters into the rock mass behind the slope face. This blast damage is due to rock fracture and joint opening as a result of the dynamic stresses induced by the blast. In addition, penetration of gas pressure from the blast can open existing discontinuities for considerable distances from the face. This damage causes loosening of the rock mass with a consequent reduction in strength.

Control of the amount of explosive detonated per delay and pre-splitting, smooth-blasting or buffer blasting are methods commonly used to minimise blast damage and, where done correctly on a routine basis, certainly are effective. However, a certain amount of blast damage is inevitable. It is usually necessary to work towards a compromise between effective fragmentation of the rock mass, for easy digging, and leaving the remaining rock face as undamaged as possible. Figure 9 shows the damage to open pit mine benches as a result of uncontrolled large tonnage production blasting. In contrast, the lack of damage to the final walls of the same mine as a result of pre-split blasting and control of the adjacent production blasts is illustrated in Figure 10.

Figure 9: Bench faces created by normal production blasting.

Figure 10: Benches created by pre-split blasting.

One of the consequences of blast damage is that the appearance of the rock mass exposed in the bench faces is not representative of the undisturbed rock mass through which a potential surface may develop. Since most geotechnical mapping is carried out on these bench faces, the shear strength of joints and the overall strength of the rock mass, estimated on the basis of this mapping, may be unrealistically low. Therefore, it is important that observations on diamond drill core and exposures in underground excavations should be incorporated in the evaluation of rock mass strength. This is a largely qualitative process since none of the methods currently used to estimate joint shear strength or rock mass strength incorporate realistic corrections for blast damage.

Slope management

Slope failures, such as that of Mount Toc in the reservoir impounded by the Vajont Dam in Italy, have resulted in major loss of life and destruction of the project. As illustrated in Figure 2, slope failures in the open pit mines, where access is generally restricted, may not result in loss of life, but they certainly have the potential to disrupt the mining operation.

Many ‘slope failures’ are more subtle than the simple cases just described. For example, the gradual deformation of a slope, even when the movement is of the order of 4 m per year, as is the case on the West Wall of the Chuquicamata open pit copper mine in Chile (Figure 11), are not considered as ‘failures’, but rather are regarded as ‘problems’ that must be managed. This is in contrast to civil engineering practice where slope deformations, such as those shown in Figure 11, would certainly be considered as ‘failures.’

One of the threats in open pit mining and civil engineering is the potential for the gradual deformation of a large slope to develop into a fast-moving catastrophic slide, as was the case in the Vajont failure. This is a very poorly understood process. There are few reliable documented case histories, but the process must be considered as a potential threat where large deforming slopes occur. Numerical modelling of these slopes for possible combinations of structural and rock mass failure and comparison of the results of these models with observations and measurements of actual slope behaviour is probably the best hope that we have of understanding this problem.

In any open pit mine, a variety of slope problems may be present at various locations in the mine at any time. The successful management of these failures is the art of good open pit mining. The absence of any failures is a sign of over-conservative slope design. It is, therefore, an absolute requirement that engineering geologists, geotechnical engineers and mine planners work together all the time to ensure that (i), the appropriate data is collected (ii), the appropriate analyses are carried out (iii), the slope designs are clearly conveyed to and understood by the mine planners and operators, and (iv), well-conceived slope monitoring programmes are established to monitor the service performance of the slopes throughout the life of the mine. Contingency plans must also be drawn up to deal with the inevitable surprises that will occur from time to time.

Figure 11: Large rock mass deformations in the West wall of the Chuquicamata Open Pit Mine in Chile.

Large rock slopes in civil engineering projects can also suffer ongoing displacements, as illustrated in the example of the Dutchman's Ridge and the Downie Slide discussed earlier. These movements are generally much smaller than those observed in open pit mines, but the consequences of slope failure are such that the same requirements for data collection, analysis, interpretation and slope monitoring exist.

A slope design is based upon the best possible evaluations of the rock types and characteristics, the structural geology and the groundwater conditions in the slope. Even the best slope designs require some averaging all of information and local variations in geology or groundwater conditions will not always be incorporated into the design. These local variations can have a significant impact on slope stability and, depending upon the location of this instability, these may have important consequences for the performance of the slope. For example, local failure of benches adjacent to a haul road in an open pit mine can have a major impact on the performance of the mine even if the overall slopes are stable.

Advance warning of these slope instability problems is very important and monitoring of slope movement has proved to be the most reliable method for the detection of slope instability. The more accurate this measurement, the earlier the developing problem can be detected.

Tools for slope displacement monitoring are well developed and are used routinely on most large open pit mines and in many civil engineering projects in which large slopes exist or are being excavated. These are generally based upon observations on numerous targets placed at carefully selected locations on the benches of the mine. Electro-optical distance measuring (EDM) equipment and, more recently, Global Positioning by Satellite (GPS) systems are used to monitor the relative positions of these targets on a daily basis. High quality EDM or GPS systems can give an accuracy of less than one centimetre over measuring distances of a kilometre or more. This order of measurement accuracy is generally sufficient to give advance warning of most slope stability problems.

The use of down-hole inclinometers and extensometers tends to be restricted to local slope instability problems. Maintaining this equipment for any length of time in a deforming operating slope is very difficult, and hence, this type of equipment tends not be used for large-scale slope deformation or failure studies.

Simple visual observation by geologists and geotechnical engineers is a tool that is frequently ignored. The development of tension cracks, the appearance of bench faces and the presence of rocks that have fallen from steep faces are all important signs of slope behaviour. If these are observed routinely and recorded systematically, a 'feel' for the behaviour of the slopes can be gradually developed. This is important information that can be taken into account when signs of significant slope instability appear and when discussions on remedial measures and contingency planning take place.

One of the tools that has been tried unsuccessfully on a number of mines is microseismic monitoring. Background noise from truck and shovel operations, blasting and regional seismic activity tends to mask any measurements of the microseismic noise generated by moving slopes.

Limit equilibrium and numerical modelling of slopes

The design of any slope must involve some form of analysis in which the disturbing forces, due to gravity and water pressure, are compared to the available strength of the rock mass. Traditionally these analyses have been carried out by means of limit equilibrium models but, more recently, numerical models have been used for this purpose.

Limit equilibrium models fall into two main categories: models that deal with structurally controlled planar or wedge slides and models that deal with circular or near circular failure surfaces in 'homogeneous' materials. Many of these models have been available for more than 40 years and can be considered reliable slope design tools.

As illustrated in Figure 4, failure of large rock slopes may involve the combination of several different failure mechanisms. This type is failure cannot be modelled by means of the simple ‘homogeneous’ material models described above and it is necessary to utilise non-circular failure models of the type originally proposed by Sarma (1979).

Significant improvements to Sarma’s original analysis were made by Donald and Giam (1989) and later by Chen (1995), Donald and Chen (1997) and Chen (1999). Many of these advances have been incorporated into commercially available slope stability software and an example is given in Figure 12. This example involves a complex failure surface in an open pit mine slope in which several fault zones and rock types occur. Based upon the orientation of structural features, directional strength properties have been assigned to each of the rock types and the most critical failure surface is generated from an automatic search process.

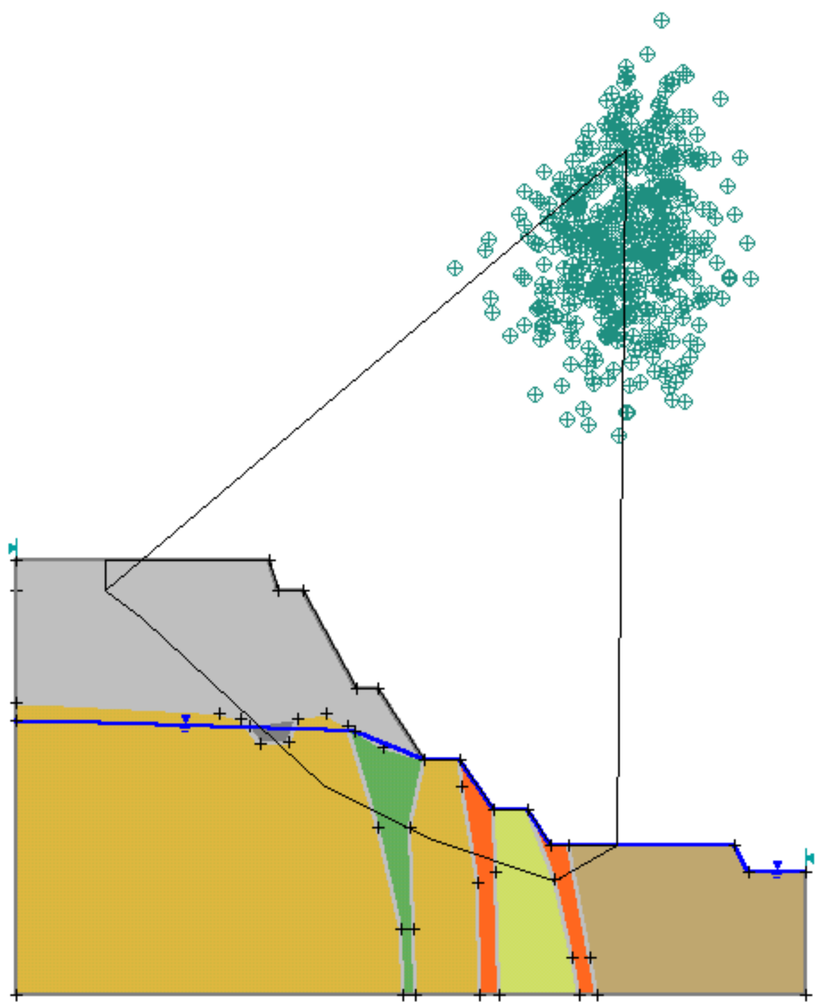

Figure 12: Non-circular failure surface in a rock slope composed of several different rock types separated by fault zones. The factor of safety for the critical failure surface indicated is 2.42. This analysis was performed by means of the program Slide2 [1].

[1] Details available from www.rocscience.com

Numerical modelling of slope deformation behaviour is now a routine activity on many large open pit mines. Programs such as FLAC and UDEC[2] are generally used for such modelling and these codes do not require any further development to meet the needs of slope modelling. Using these codes correctly is not a trivial process and mines embarking on a numerical modelling programme should anticipate a learning process of at least a year, even with expert help from consultants. Obtaining realistic input information for these models and interpreting the results produced are the most difficult aspect of numerical modelling in the context of large-scale slopes.

In practice, both limit equilibrium and numerical modelling tools are used together to generate a range of possible solutions for the range of input parameters that exist for a particular site. While this may be frustrating for mine management and mine planners in that the geotechnical department does not appear to be capable of producing a single definitive design, it is far more realistic to look at the results of a parametric study than to rely on a single analysis.

The advantage of these numerical models over the limit equilibrium models described earlier is that they can be used to model progressive failure and displacement as opposed to a simple factor of safety. This makes them much more useful in managing ongoing slope displacements such as the case illustrated in Figure 11. These numerical models can also be used to determine the factor of safety of a slope in which a number of failure mechanisms can exist simultaneously or where the mechanism of failure may change as progressive failure occurs. Figures 13 illustrates this procedure which has been described by Dawson, Roth and Drescher (1999).

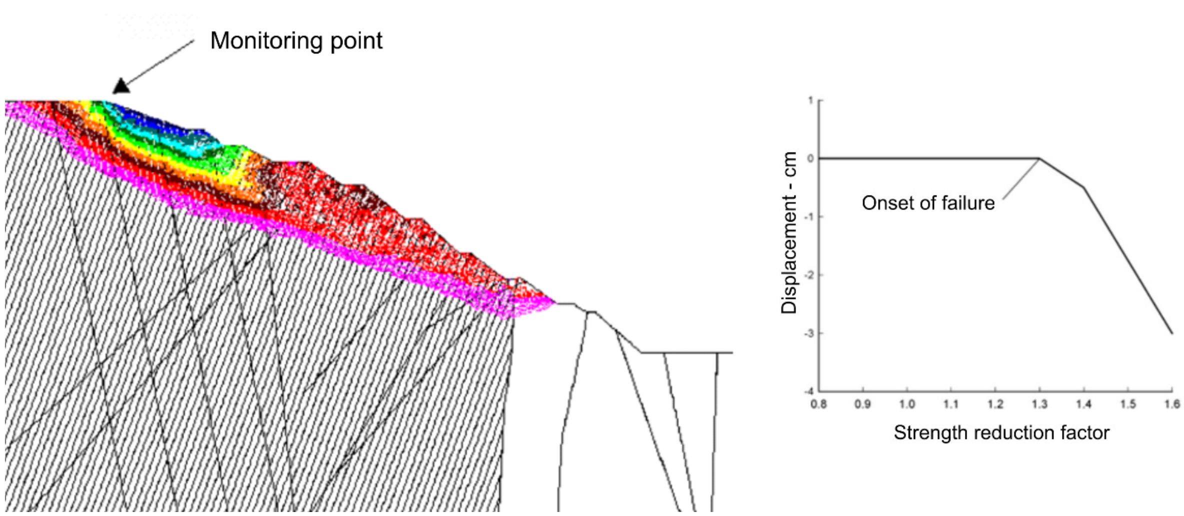

Figure 13: UDEC model of Chuquicamata West Wall showing displacement velocity contours and strength reduction factor.

The Itasca UDEC model is set up to incorporate all the rock types, faults and joint systems as well as groundwater conditions within the slope. The strength of all the rock units, faults and joint systems is then decreased progressively by dividing them by a ‘strength reduction factor’. The displacement of a target on the slope is monitored during this strength reduction process and, as

[2] Details available from www.itascacg.com

shown in the plot in Figure 13, a sudden increase in displacement indicates that failure of the slope has started. The factor of safety of the slope is equivalent to the strength reduction factor at which failure starts. Factors of safety calculated in this way have been found to coincide very closely with those determined by limit equilibrium analyses in cases where limit equilibrium analyses are known to give reliable results.

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