

Rock slopes in Civil and Mining Engineering

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ROCK SLOPES IN CIVIL AND MINING ENGINEERING

Evert Hoek¹, John Read², Antonio Karzulovic³ and Zu Yu Chen⁴

ABSTRACT

Rock slopes with heights approaching 1000 m are now being designed and excavated in various open pit mining and civil engineering projects around the world. The economic impact of excessively conservative designs or of failures in these slopes can be very large and every effort has to be made to optimise their design. Their size means that they will almost always contain a number of significant structural features and a variety of geological materials.

The input required for these models includes a comprehensive geological data base that contains both structural geology and lithological information, an hydrogeological model that permits water pressures to be estimated throughout the slope, estimates of rock mass and discontinuity strength and deformation properties and an understanding of external forces, such as those due to earthquakes, that may be imposed on the slope. Site investigation techniques that can be used to obtain this information and the methods, that can be used to estimate rock discontinuity and mass properties, are reviewed in this paper.

Developments in limit equilibrium and numerical modelling techniques are reviewed and their applicability to the design of these large slopes is presented. The importance of blasting control in the excavation of the slope is discussed. The use of drainage to improve the stability of the slopes is also considered. Monitoring the behaviour of the rock mass and the subsurface groundwater during construction and subsequent operation of the slope is an important component of rock slope engineering and these techniques are reviewed. Practical examples from large projects around the world are presented.

THE GEOLOGICAL MODEL

A comprehensive geological model is absolutely fundamental to any slope design. Without such a model, slope designers have to resort to crude empiricism and the usefulness of such designs, except possibly for very simple pre-feasibility evaluations, is highly questionable.

In open pit mining a great deal of effort is devoted to defining a reliable geological model in order to quantify ore reserves. Therefore a reliable geological model exists and, provided that there is good co-operation between the geological and geotechnical departments of the mine, a small amount of additional effort will produce a sound geotechnical model.

Routinely, these models are stored and manipulated electronically and the availability of a number of computer geological modelling tools is an important aspect of modern rock slope design. Packages such as the Minex-Horizon, Vulcan and Minescape three-dimensional solid modelling systems permit the construction and visualisation 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 by the model. These modelling systems are already fully operational and are used routinely by most large open pit mining organisations. However, they are very seldom used in civil engineering projects and this is a serious deficiency which the authors hope to see remedied in the years to come. Under development are interfaces that will allow a direct transfer of data from these three-dimensional models to other types of models used for slope design. For example, it is anticipated that, within a few years, it will be possible to transfer

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geometrical and geotechnical information directly from a three-dimensional geological model into a limit equilibrium or numerical model for slope stability analysis.

GEOLOGICAL AND GEOTECHNICAL DATA COLLECTION

The tools and techniques for geological data collection are well developed. Most of these tools have been available for the past 20 years and have been widely used by the mining and civil engineering industries. Consequently, there does not appear to be any need for the development of new tools and techniques. The definition of what information to collect and the efficient use of the collected data are another matter. There is a fundamental need to collect good quality basic geological data in the form of regional and site lithological, structural and hydrogeological information, detailed core logs and photographs that can be interpreted by an engineering geologist at any time. Currently, there are few standards for data collection and the type and quantity of information gathered depends very much upon the personal opinions and preferences of the individuals or organisations 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 organisational changes occur.

While there does not appear to be any immediate prospect for the adoption of international or even industry wide standards for geological data collection, there are examples of organisations that have established company-wide standards. These generally comprise a manual of procedures for geological data collection, rock mechanics laboratory testing procedures and the interpretation and presentation of results. The development of such company manuals is considered to be a useful process provided that it does not impose rigid boundaries on the data collection and interpretation process.

The current state of practice includes the use of 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 is highly questionable since most of them were developed for the confined conditions that apply in the rock masses surrounding underground excavations and they 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 characterised 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 and these ranges may be used to delineate zones that can be used to complement all of the other geological, hydrogeological and geotechnical information. Where classification systems are used, the need to understand their limiting assumptions and to calibrate them to the regional and local site conditions is emphasised.

Rock slope failures are geological events controlled by natural physical processes. Geological-geotechnical models that can be used to understand and to 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 joints. The most important information is the structural data and this should include information on major structures and on structural domains. This information must be sufficiently detailed that meaningful statistical trends can be derived and used as input for analytical models.

As discussed above, the input of the results of the geological data collection process into a geological model, preferably by the engineering geologists themselves, is an important part of the development of the understanding of the geological processes that control the rock mass behaviour. Deficiencies and anomalies in the data become obvious during this model construction process and these provide useful guidance to the geologists in the development of further 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 recognised but, because of the large

number of such features, the 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 analysed using limit 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 an analysis of simple wedge failure.

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

State-of-the-art specialists in slope stability recognise that the two categories described above are inadequate. Figures 1, and 2 illustrate the difficulties.



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 lower part of the failure are unclear.

Figure 1 shows a 165 m high wedge-shaped slope failure in a mine slope. There are clearly defined faults at the back and on each side of the failure in the uppermost five to seven benches, but elsewhere there is no unique structural definition of the failure. Different faults control the shape of the failure in the lower benches, there is differential movement on faults inside the failure, and it is not clear whether the lower three benches moved out of the slope on (i), a low-angle fault (ii), through dilation of a closely jointed and highly altered and weakened rock mass or (iii), simply through failure of the weakened rock mass. How can the complex shape of the failure be modelled and how can it be analysed? More importantly, can it be predicted that the failure will not happen again when the slope is pushed back?



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

Figure 2 is an approximately 350 m high mine slope failure. The rock mass within the failed mass is altered and closely jointed. There is evidence of faults at the back of the failure, but what triggered the failure? Was it the faults at the back of the failure or was it a non-structurally controlled 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 analysed and the slope re-designed?

There is extensive discussion amongst practitioners and researchers on how the more complicated failure geometries illustrated in Figures 1 and 2 can be addressed. One of the more promising methods is that being undertaken in a research project being carried out by the CSIRO Division of Exploration & Mining in Australia with funding from a number of international mining companies. The initial goal is to develop a tool which will produce a three-dimensional geological model that accounts for and can visualise (i) the orientation, spacing and continuity of the joints or other second order structures that appear in the closely fractured rock mass (ii) the location and orientation of the larger faults or structures that may subdivide the closely fractured rock mass into separate zones or domains and (iii), the quality of the rock mass in each domain. The model will then be used to (a), determine whether or not three-dimensional failure surfaces can propagate through the rock mass (b), which of those candidates exhibits the path of least resistance (c), how much of that candidate surface is along either primary and/or second order structures or through a combination of structures and rock bridges between the structures and (d), if rock bridges are encountered, where they must break to in order for the surface to continue propagating.

Fundamental to this work is the belief, based on observation at many different mine sites, that the low stress environment in slopes virtually guarantees that existing or incipient structures in the fractured rock mass will be exploited before a through-going failure develops. When the modelling outlined above has been completed, attention will be focused on (i), describing and setting criteria for the different failure mechanism that may be associated with or attached to the predicted candidate surfaces and (ii), developing

methods of analysis that examine the reliability of each of the surfaces. A major goal of this phase of the research will be to determine a means of avoiding the need to resort to the simplified 'smeared' rock strength models referred to earlier.

Parallel work is being carried out in China (Wang, Chen and Jia, 1998) and Figure 3 illustrates a joint map of the foundation for the Three Gorges Dam. 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.

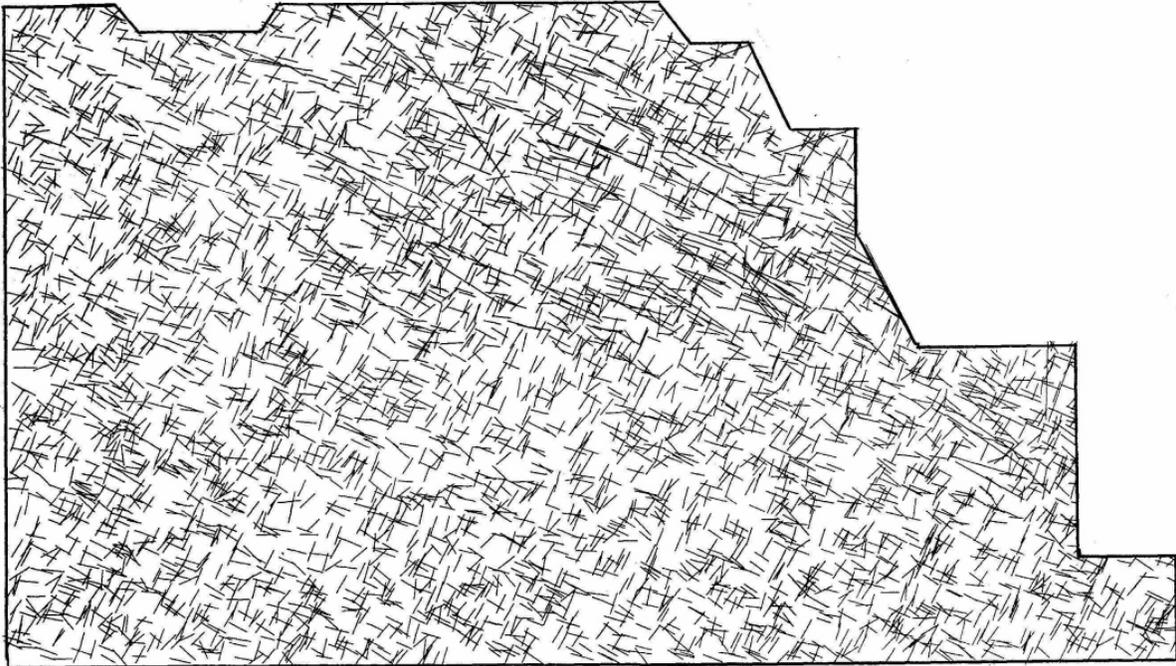


Figure 3: Joint map for the rock mass forming the foundation of the Three Gorges Dam in China.

DETERMINATION OF ROCK MASS PROPERTIES

The determination of rock mass strength is a major deficiency in current rock slope design practice. Even in terms of the state-of-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,

In terms of the state of practice, as mentioned above, 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 a rock mass shear strength value from the component parts along a given candidate failure surface (Figure 4). For the failure surface illustrated in Figure 4, 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.

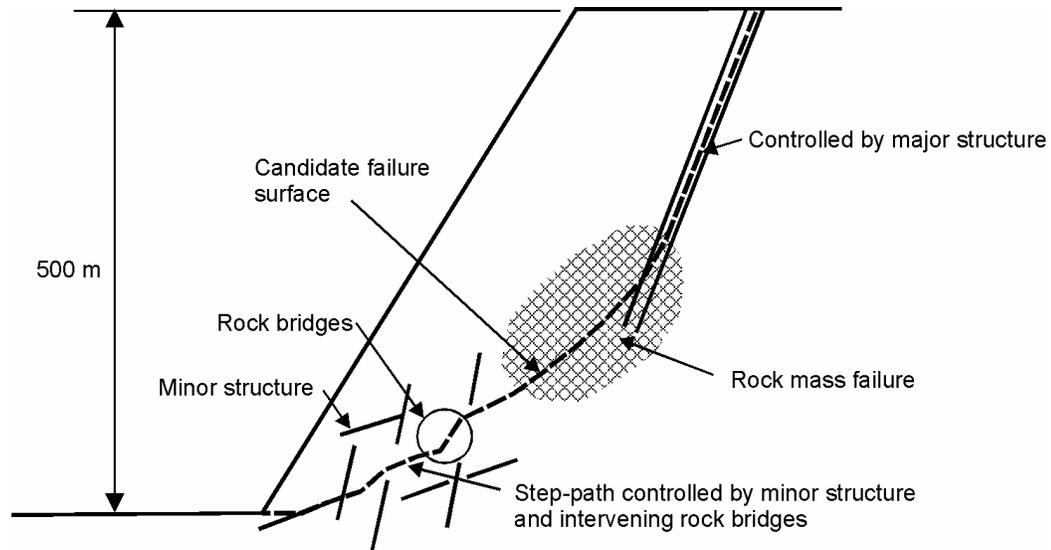


Figure 4: Candidate failure surface involving a number of different shear failure mechanisms.

For the future, there appears to be great merit in having some suitably qualified researcher carry out a systematic study of rock mass strength using existing numerical modelling tools.

A major goal of the second phase of the CSIRO structural modelling project described earlier will be to determine a way of avoiding the simplified 'smeared' rock strength model. One of the avenues being pursued will attempt to simulate continuum processes using a particle-in-cell finite element code. The code can be applied to high strains and history dependent material properties and has been used successfully to model lithospheric instabilities such as plate buckling and folding. Alternatively, FLAC, UDEC and PHASE2 and other numerical models contain interface elements that can be used to simulate some aspects of the problem.

A second research direction in the CSIRO project involves the use of probabilistic techniques. As the low stress environment of an excavated slope is created the closely jointed rock mass will dilate and the rock mass strength may be controlled by particle interlocking rather than the frictional strength of potential sliding surfaces. The probabilistic analysis will seek to account for the interlocking strength effect of the rock mass as it dilates. Rather than predicting the location of a specific candidate failure surface, it will assess the probability that a specific zone or three-dimensional volume of rock behind the slope will be distressed and start to move out if, say, the slope is over-steepened or undercut, or if a 'key block' is removed from the face.

One of the problems that will impede progress in this work is the lack of current research in rock slope stability. Changes in priorities of university research and research funding means that there are very few research centres or individual researchers working in this area. Other than the CSIRO project described above, it is not clear where suitably qualified researchers could be found and how such research would be funded.

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 it 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 and the impact on rock mass strength, and hence 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.

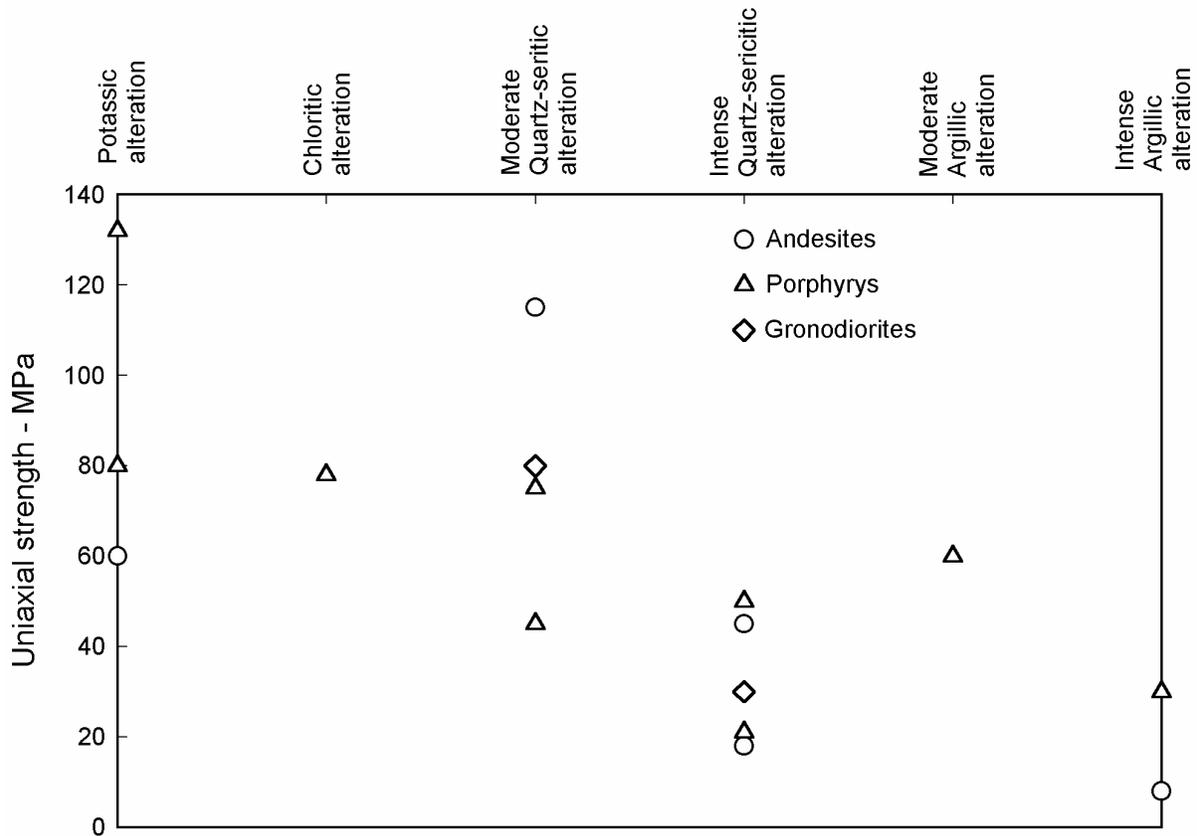


Figure 5: Relationship between intact rock strength and degree of alteration.

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 and flow evaluation and control are well developed and 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 and it 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 and this 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 about 6% by the depressurisation programme, measurement of displacements indicated that these had been reduced to negligible levels and the slope stabilisation programme was judged to have been successful.

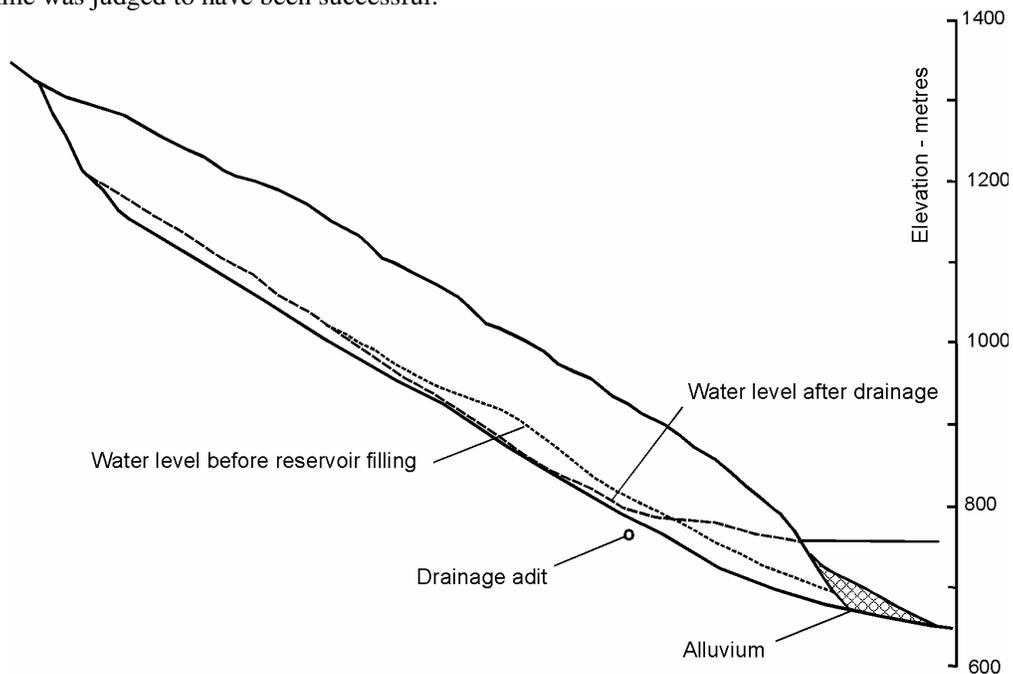


Figure 6: Section through Dutchman's Ridge slide showing the location of the drainage gallery and the phreatic surfaces before impoundment of the reservoir and after impoundment and drainage.

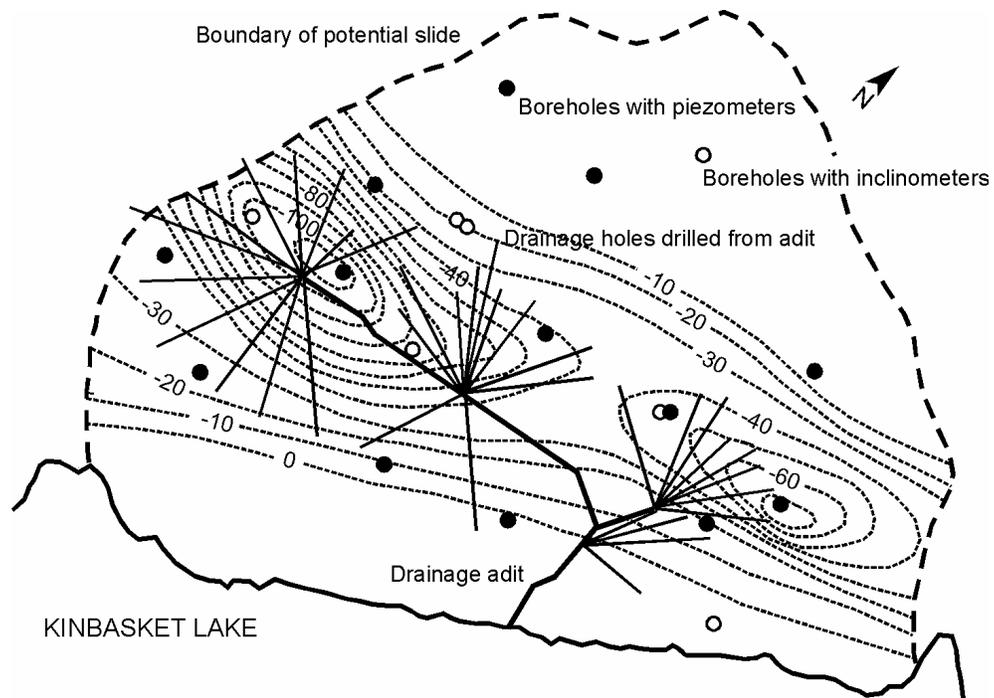


Figure 7: Extent of potential slide with layout of drainage gallery and drainholes drilled from underground. Contours indicate the drawdown of the groundwater levels achieved by the drainage programme.



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 drainholes have continued flowing and have kept the groundwater levels under control for the past 15 to 20 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 and 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.

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.



Figure 9: Bench faces created by normal production blasting.



Figure 10: Bench faces created by pre-split blasting.

SLOPE MANAGEMENT

Slope failures such as that of Mount Toc in the reservoir impounded by the Vaiont 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 economic open pit mine, a variety of slope instabilities may be present at various locations in the mine at any time. The successful management of these features is the art of good open pit mining. The absence of any failures is a sign of over-conservative slope design, and hence, inefficient mine management. 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 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 Downie slides 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 of all of this 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 centimeter over measuring distances of a kilometer 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 to be used for large-scale slope deformation or failure studies. This picture may be changed if equipment to be installed in the west wall of the Chuquicamata open pit mine proves to be successful. Developed by the CSIRO in Australia, this equipment consists of self-contained battery operated devices that can be located precisely in space by radio signal triangulation. The anticipated accuracy of these devices is of the order of millimeters and the design life of about five years. Installed in boreholes at depths of up to 150 m, these devices will be able to track the movement of the rock mass behind the slope. This information is very important in the development of an understanding of how some of these large slopes deform.

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, 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 gradually be 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 25 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 of 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.

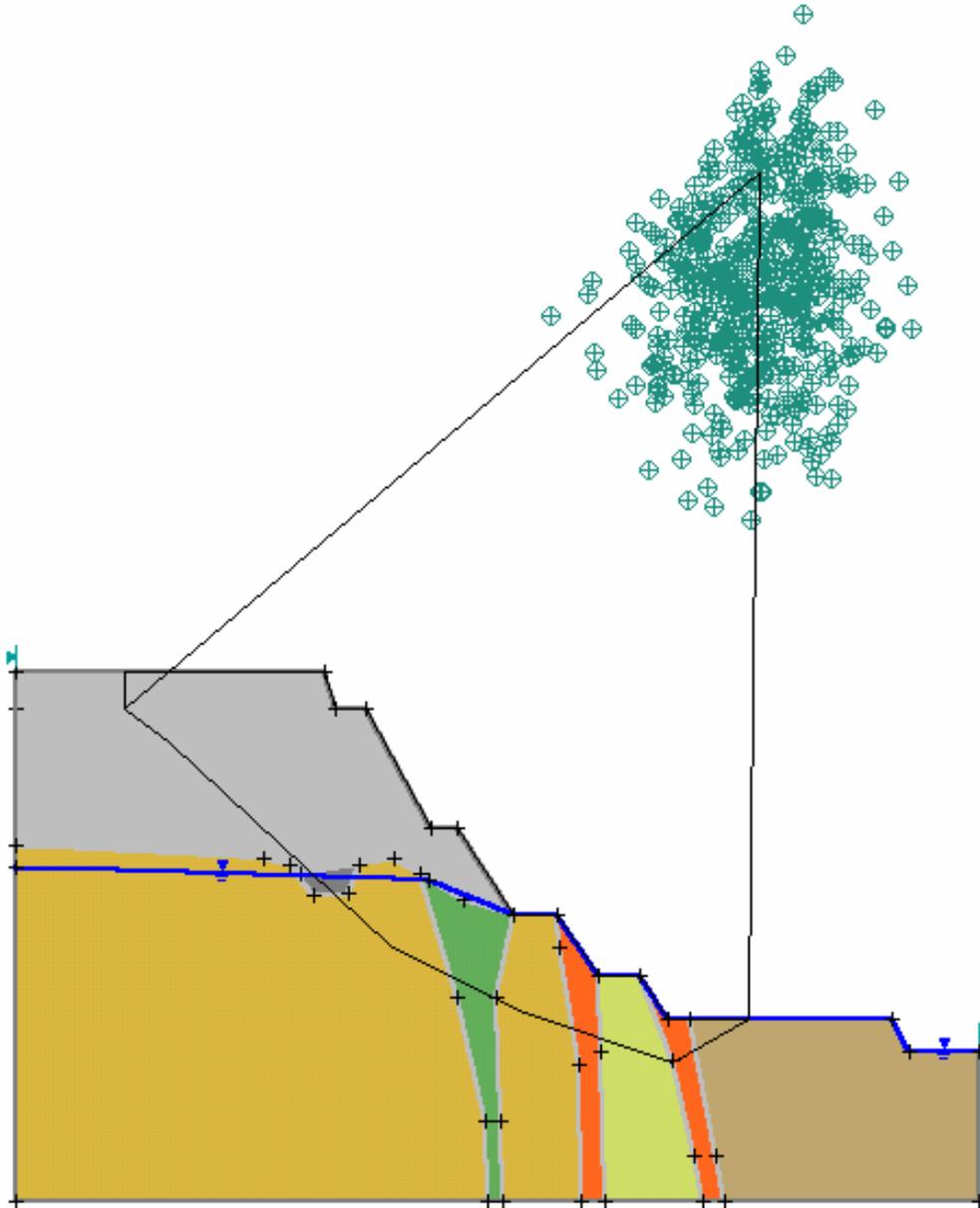


Figure 12: Non-circular failure surface in a rock slope composed of several different rock types separated by fault zones. The factor of safety for the critical failure surface indicated is 2.42. This analysis was performed by means of the program SLIDE⁵.

⁵ 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 UDEC⁶ 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 one to two years, 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 and 14 illustrate this procedure which has been described by Dawson, Roth and Drescher (1999).

The numerical model, in this case UDEC, is set up to incorporate all of the rock types, faults and joint systems as well as groundwater conditions within the slope. The strength of all of 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 shown in Figure 14, 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 has 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.

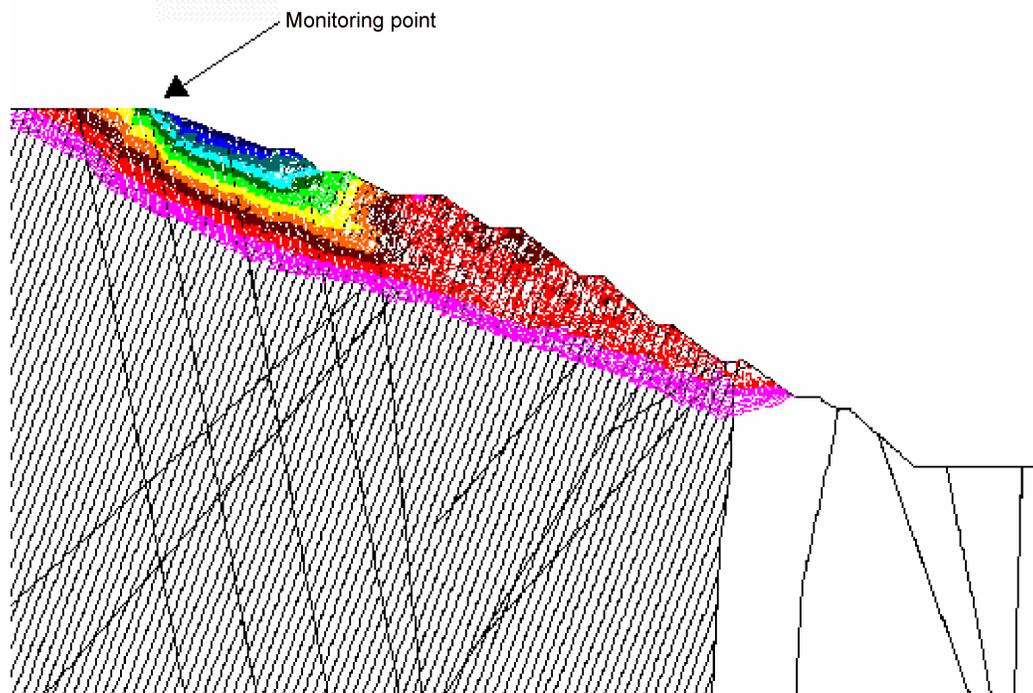


Figure 13: UDEC model of Chuquicamata West Wall showing displacement velocity contours.

⁶ Details available from www.itascacg.com

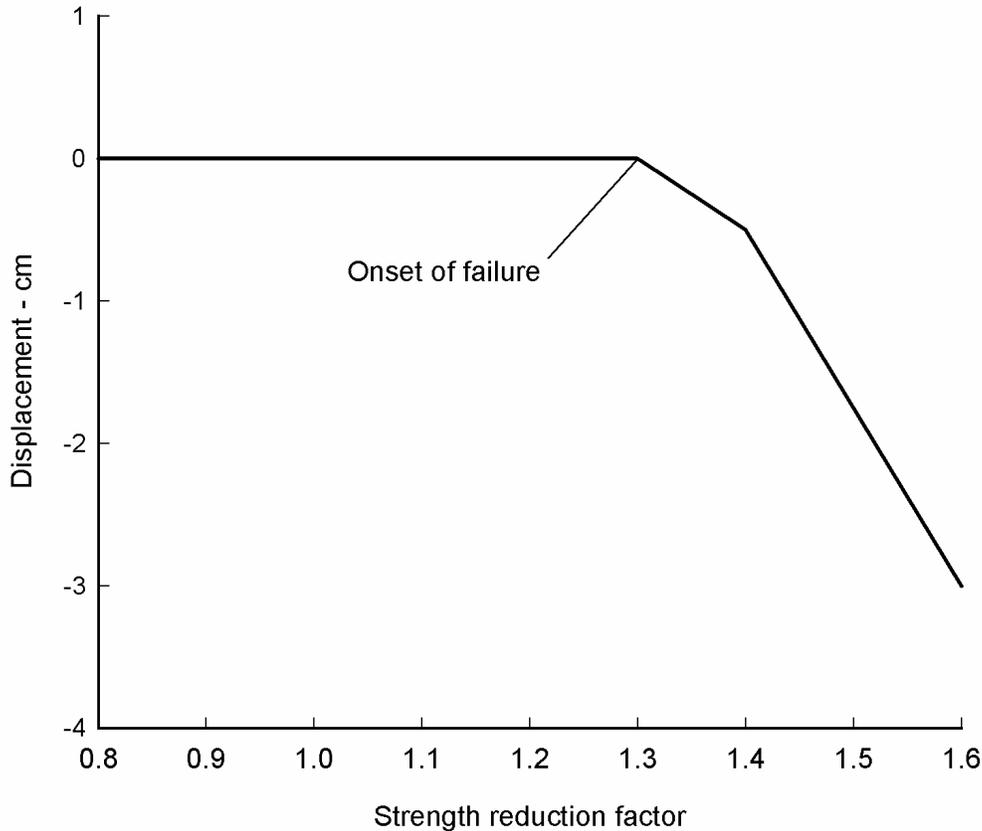


Figure 14: Plot of displacement of monitoring point in Figure 13 against factor of safety.

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