LARGE SCALE SLOPE STABILITY IN OPEN PIT MINING — A REVIEW

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by

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PREFACE

This literature review is part of a joint research project between Boliden Mineral and Luleå University of Technology concerning large scale slope stability in open pit mining. The research project is aimed at increasing the knowledge regarding large scale slope stability and to develop a design methodology for large scale open pit slopes, with special application to the Aitik Mine in northern Sweden. Financial support for the project is being provided by Boliden Mineral AB.

The work presented in this report is the result of a comprehensive literature review and a number of discussion meetings with the project reference group. This group consists of Mr. Norbert Krauland, head of Rock Mechanics at Boliden Mineral AB, Dr. William Hustrulid, head of Mining Research and Development at LKAB, and Dr. Erling Nordlund, Associate Professor at the Division of Rock Mechanics, Luleå University of Technology. They are all acknowledged for their support and many suggestions as to how to improve upon the manuscript. Furthermore, special thanks must be given to Tech. Lic. Gunnar Rådberg, Laisvall Mine, Boliden Mineral AB, formerly at the Division of Rock Mechanics, Luleå University of Technology, for his critical review of the report. I would also like to thank Mrs. Claudia Hustrulid for help in correcting the English.

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Jonny Sjöberg
SUMMARY

Design of open pit slope angles is becoming more and more important as the mining depths of
open pits continuously increase. Small changes in the overall pit slope angle have large
consequences on the overall economy of the mining operation. A case in particular is the Aitik
open pit mine in northern Sweden, which currently faces the design of the overall slope angles
for continued mining toward a depth of around 500 meters. This report constitutes the first
phase in a research project aimed at developing design methods for large scale pit slopes. In
this report, the stability and design of large scale pit slopes in open pit mining is reviewed, with
special reference to slopes in hard, jointed, rocks, similar to the rock types found at the Aitik
mine. The review covers the mechanics of pit slopes, existing design methods for large scale
slopes, remedial measures and mining strategy to cope with slope failures, and a compilation
of case studies from open pits worldwide. Finally, suggestions for future research in this area
are presented.

The factors governing large scale slope stability are primarily: (1) the stress conditions in the
pit slopes, including the effects of groundwater, (2) the geological structure, in particular the
presence of large scale features, (3) the pit geometry, and (4) the rock mass strength.
Observed failure modes in rock slopes are of a wide variety. On a bench scale, structurally
controlled failures such as plane shear and wedge failures are common. However, as the scale
increases, simple structurally controlled failures are less dominate, and more complex failures
such as step-path failures start to develop. From observations, it appears that for large scale
slopes, two failure modes are especially important to consider. These are (1) rotational shear
failure, and (2) large scale toppling failure. Rotational shear failure in a large scale slope
involves failure both along pre-existing discontinuities and through intact rock bridges, but
where the overall failure surface follows a curved path. Large scale (or deep seated) toppling
failures have been observed in several large scale natural slopes and high open pit slopes.

The mechanisms behind large scale failures are, however, not well known, in particular for
hard, strong rocks. Criteria for the shape and location of the failure surface are lacking, as is
detailed knowledge regarding failure through intact rock versus failure along discontinuities.
Knowledge of the kinetic behavior of failing rock slopes is mostly empirical and requires more
studies, in particular for hard and brittle rock masses in which rapid failures can be expected.

This review has shown that the strength of a large scale rock mass is very difficult to assess.
At the same time, the required accuracy for the strength parameters which are needed for the
design is very high. For large scale rock masses, back-analysis of previous failures proves to
be the only practical means of obtaining relevant strength parameters. However, the
interpretation and translation of such data from one geological environment to another, is very cumbersome and lined with problems.

Design methods for rock slopes can divided into mainly four categories, namely: (1) limit equilibrium methods, (2) numerical modeling, (3) empirical methods, and (4) probabilistic methods. The advantages and disadvantages of each of these methods are discussed in the review. For the design of large scale slopes, it appears that the choice of design method is less important than the choice of input parameters to the design, in particular the rock mass strength parameters.

Remedial measures for controlling the stability of slope include support and drainage. While support can work for small scale slopes, only drainage is feasible for increasing the stability of large scale slopes. Monitoring of displacements, preferably using survey networks, should be carried out routinely in all open pit mining. Provided that the failure is slow and stable, it is also possible to continue to mine a failing slope. This requires that contingency plans are being made at an early stage in the mine planning process.

From the collection of a number of case studies from North and South America, Africa, Asia and Europe, several examples of large scale failures were found, although mostly occurring in weak rocks. There are much fewer examples of slope failures in hard, brittle rocks. The few cases found indicate that failures in this type of rock is more uncontrollable. A compilation of slope height, slope angles, rock strength, and stability conditions for the studied cases concludes this chapter.

Future research in the field of large scale slope design must be focused on quantifying the mechanisms for large scale slope failures. Once the mechanisms are better known, design methods based on the actual slope mechanics can be employed. Also, better and more reliable methods for determining the strength of large scale rock masses are important to develop.

Keywords: Slope stability, open pit mining, design.
SAMMANFATTNING


Storskaliga slänters stabilitet styrs av främst (1) spänningssituationen i dagbrottsslänterna, inklusive effekterna av grundvatten, (2) geologiska strukturer, särskilt storskaliga sådana, (3) dagbrottsgeometrin, och (4) bergmassans hållfasthet. Observerade brottformer i bergsslänter varierar från enkla strukturstyrda brott typ plant skjuvbrott och kilbrott, till mer komplexa brottformer som ”trappstegs-brott”. För slänter av mättlig storlek, typ enstaka pallar, dominerar strukturstyrda brott. Med ökad storlek minskar dock inverkan av befintliga strukturer. I storskaliga slänter är det främst två brottformer som visat sig vara viktiga att ta hänsyn till. Dessa är (1) rotationsbrott längs en krökt (cirkulär) brottyta, och (2) storskaligt överstjälpningsbrott. Mekanismen vid rotationsbrott omfattar troligen brott både längs befintliga diskontinuiteter och genom intakta bergbryggor. Den slutliga brottytan är dock krökt. Storskaligt överstjälpningsbrott har observerats i ett flertal storskaliga naturliga slänter samt i höga dagbrottsslänter.

Kunskapen om mekanismerna bakom storskaliga släntbrott är dock bristfällig, särskilt för hårt, höghållfast berg. Det saknas kriterier för form och läge av brottytan, och kunskapen om brott genom intakt berg kontra brott längs diskontinuiteter är bristfällig. Vad beträffar rörelsehastigheter vid brott så är den mesta kunskapen enbart empirisk. För hårda och spröda bergmassor kan man dock förvänta sig snabba och mer plötsliga brottförlopp jämfört med slänter i låghållfast berg.

För tillämpning på slänter där storskaligt brott ej skett, är det dock svårt att översätta värden från släntbrott i dagbrott med annan geologi.

Dimensioneringsmetoder för slänter kan indelas i fyra kategorier: (1) jämviktsanalyser, (2) numeriska modellanalyser, (3) empiriska metoder, och (4) sannolikhetssteoretiska metoder. I rapporten beskrivs nackdelarna och fördelarna med respektive metod. För dimensionering av storskaliga slänter är dock valet av beräkningsmetod underordnat valet av ingångsdata till beräkningarna. Framförallt gäller detta bergmassans hållfasthet.


Framtida forskning måste inriktas på att bättre kvantifiera mekanismerna för storskaligt släntbrott. Dimensioneringsmetoder som baseras på slänters mekaniska beteende kan sedan tillämpas. Det är också av största vikt att utveckla bättre metoder för bestämning av bergmassans storskaliga hållfasthet.
LIST OF SYMBOLS AND ABBREVIATIONS

\( \sigma_1 \) = major principal stress (compressive stresses are taken as positive)  
\( \sigma_2 \) = intermediate principal stress  
\( \sigma_3 \) = minor principal stress  
\( \sigma_v \) = vertical stress  
\( \sigma_h \) = horizontal stress  
\( K \) = ratio between horizontal and vertical stress  
\( \sigma_c \) = uniaxial compressive strength  
\( E \) = Young's modulus  
\( \rho \) = density  
\( g \) = gravitational acceleration  
\( H \) = slope height  
\( c \) = cohesion  
\( \phi \) = friction angle  
\( \lambda_{c0} \) = Janbu's number  
\( \tau \) = shear stress  
\( \sigma_n \) = effective normal stress  
\( \tau_f \) = peak shear strength  
\( \tau_r \) = residual shear strength  
\( I_b \) = brittleness index  
\( i \) = inclination of surface asperities on a discontinuity  
\( \phi_b \) = basic friction angle  
\( m \) = parameter in the Hoek-Brown failure criterion  
\( s \) = parameter in the Hoek-Brown failure criterion  
\( F_s \) = factor of safety  
\( r_u \) = porewater (groundwater) pressure ratio  
\( u \) = groundwater pressure  
\( z \) = depth below ground surface  
\( K_{ic} \) = fracture toughness for Mode I fracturing  
\( K_I \) = stress intensity factor  
\( G \) = strain energy release rate  
\( G_c \) = surface energy required for fracturing  
\( r \) = radius  
\( \omega \) = angular velocity  
\( R \) = resistance  
\( S \) = load  
\( SM \) = safety margin
\( G(X) = \) performance function  
\( X = \) all random input parameters which make up the resistance and load distribution  
\( m_G = \) mean of the performance function  
\( s_G = \) standard deviation of the performance function  
\( m_{SM} = \) mean of the safety margin  
\( s_{SM} = \) standard deviation of the safety margin  
\( P = \) probability  
\( P_f = \) probability of failure  
\( \beta = \) reliability index  
\( \Phi = \) standardized normal distribution  
\( N = \) number of input variables  
\( C_T = \) total cost  
\( C_0 = \) sum of initial costs  
\( C_D = \) cost associated with failure  

Mton = million metric tons  
\( DCF = \) Discounted Cash Flow (economics)  
\( NPV = \) Net Present Value (economics)  
\( EDM = \) Electronic Distance Measurement (monitoring)  
\( RQD = \) Rock Quality Designation (rock mass classification)  
\( REV = \) Representative Elementary Volume  
\( JRC = \) Joint Roughness Coefficient (Barton shear strength criterion)  
\( JCS = \) Joint wall Compressive Strength (Barton shear strength criterion)  
\( BEM = \) Boundary Element Method (numerical modeling)  
\( FEM = \) Finite Element Method (numerical modeling)  
\( FDM = \) Finite Difference Method (numerical modeling)  
\( DEM = \) Distinct Element Method (numerical modeling)  
\( DDA = \) Discontinuous Deformation Analysis (numerical modeling)  
\( FLAC = \) Fast Lagrangian Analysis of Continua (numerical modeling)  
\( UDEC = \) Universal Distinct Element Code (numerical modeling)  
\( 3DEC = \) Three-Dimensional Distinct Element Code (numerical modeling)  
\( PFC = \) Particle Flow Code (numerical modeling)  
\( RMR = \) Rock Mass Rating (rock mass classification)  
\( MRMR = \) Mining Rock Mass Rating (rock mass classification)  
\( SMR = \) Slope Mass Rating (rock mass classification)  
\( REMIT = \) Rock Engineering Mechanisms Information Technology (empirical design)  
\( FOSM = \) First-Order-Second-Moment method (probability theory)  
\( PEM = \) Point Estimate Method (probability theory)
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1 INTRODUCTION

1.1 Background

Open pit mining is a very cost-effective mining method allowing a high grade of mechanization and large production volumes. It is therefore possible to mine mineral deposits of a very low grade which could not be mined economically using underground methods. Mining depths in open pits have increased steadily during the last decades, and open pits with mining depths of up to and exceeding 500 meters are not unusual. Since underground mining is still significantly more expensive than open pit mining, it is not likely that such mines will switch to an underground mining operation for future mining of existing mineral deposits. One can instead expect open pits with even larger depths, provided production costs continue to decrease and metal prices remain at a fairly constant level.

A major complication with increasing mining depths is the increased risk of large scale stability problems. Large scale failure could potentially involve the entire height of the final pit slope. Maintaining pit slope angles that are as steep as possible is of vital importance to the reduction of stripping (mining of waste rock), which will in turn have direct consequences on the economy of the mining operation. Design of the final pit limit is thus governed not only by the ore grade distribution and the production costs, but also by the overall rock mass strength and stability. The potential for failure must be assessed for given mining layouts and incorporated into the design of the ultimate pit.

For a typical open pit mine, several different mining units can be identified, each of which has an associated slope angle, as shown in the cross-section in Figure 1.1. The bench face, interramp and overall slope angles must be chosen based upon an assessment of the stability of each slope unit. In general, bench instabilities are relatively common occurrences which affect smaller portions of the pit. Single benches and interramp areas of the pit could also be stable at the same time as the overall pit slope may be potentially unstable. The Aitik mine in the northern Swedish city of Gällivare is currently facing the problem of designing the overall slope angle as the pit is being deepened. The relatively limited experience of large scale slope failures and existing design methods for ultimate pit slopes prompted this literature review. This review also constitutes the first phase of a research project concerning large scale slope stability at Aitik. The project is a joint effort between Boliden Mineral and the Luleå University of Technology and is aimed at arriving at practical and useful design rules for the Aitik mine. It also serves as a thesis project, the purpose of which is to satisfy the academic requirements for a Ph.D. degree. This literature review is thus a first step toward being able to define the thesis project in detail.
Figure 1.1 Definition of slope angles according to the current practice at the Aitik Mine.

In order to clearly define the problem at hand, a short description of the Aitik mine is given in the next section along with a review of the current status of slope design at Aitik. The mining terms used in the next section are not defined in detail at this point, but will be discussed in Chapter 2 of this report.

1.2 Slope Stability at the Aitik Mine

The Aitik open pit is located approximately 20 km east of Gällivare in northern Sweden. The mine is owned and operated by Boliden Mineral AB. Ore grades are generally very low with an average 0.38% Cu, 4 ppm Ag and 0.2 ppm Au, and a cutoff grade of 0.26% Cu. Mining started in 1968 with a relatively low annual production. Production rates have increased steadily ever since, and the current annual production rate is approximately 16 Mton (million metric tons) of ore. In addition to this, some 15 Mton of waste rock is stripped annually. The pit is approximately 2500 meters long, 500 meters wide, and has a current depth of 225 to 270 meters (Lindström, 1989; Forsman et al., 1992; Rönnkvist, 1992; Hardell et al., 1993), (Figure 1.3).
The Aitik orebody strikes nearly north-south and dips approximately 45° to the west. The hangingwall contact is relatively sharp (thrust contact) while the footwall contact is more diffuse. Footwall host rocks are diorite, amphibolite gneiss and biotite gneiss. The ore zone consists of biotite gneiss, biotite schist and muscovite schists. The hangingwall host rocks are mainly biotite gneiss and amphibolite gneiss. Pegmatite dykes occur both in the hangingwall and in the main ore zone (Abrahamsson, 1995; Welin, 1995). These rocks are generally of relatively high strength and stiffness. For the intact rock, the uniaxial compressive strength varies from around 70 to 140 MPa, corresponding to a friction angle of around 43° to 49° and a cohesion of around 12 to 20 MPa. Young’s modulus varies substantially with mean values in the range 50 to 90 GPa. Shear tests on pre-existing rock joints gave friction angles in the range of 19° to 34° for different rock and joint types (Call et al. 1976; West et al., 1985).

The ore zone can be divided into two portions. In the northern end, the ore grades are relatively constant with depth. In the southern end they decrease with depth. Currently, the ore reserves in southern portion of the pit extend to a depth of 330 meters (below ground surface), while the northern portion of the ore is known to be more than 800 meters deep. This is reflected in the long-term mining plans which call for different final pit depths for the two portions. The south portion has a planned final depth of 330 meters, while it is planned to mine the northern portion to a final depth of approximately 435 meters.

The long-term mining plans require that the slope angles necessary to maintain large scale stability be determined. This would provide an upper bound to the overall slope angle. The footwall is especially crucial since the mill is located on the footwall side of the orebody and in the middle portion (Figure 1.3). Slope instabilities of a larger scale have not been observed at Aitik, thus far; however, several smaller scale bench instabilities have been noted.

Two very comprehensive slope stability analyses have been conducted at Aitik. The first study (Call et al., 1976; Call et al., 1977; Eriksson and Krauland, 1976) was focused on characterization of the geomechanical conditions at the mine. Based on this, potential stability problems were outlined and analyzed. It was found that pre-existing fractures in the rock mass and their characteristics were the most important factors contributing to potential slope failures, along with the hydrologic conditions at the mine. A probabilistic approach was used to calculate the possibility of slope failure and to determine the optimum slope angles. Optimum slope angles imply some instability of the pit slopes since completely stable slopes would be too conservative and too costly. The second study (West et al., 1985) included more detailed structure mapping, a review of the hydrologic data, re-analysis of earlier tests, a more detailed stability analysis, and design recommendations.
In these studies, a number of failure modes were considered and analyzed. These included rotational shear, plane shear, step path, simple wedge, slab and block flow failures. These and other failure modes will be described in more detail in Section 3.3 of this report. Overall stability, interramp stability and bench stability were analyzed. It was found that large scale failure was not likely (for the conditions at that time) since the presence of larger structures going from toe to crest of the pit could not be confirmed. Furthermore, rotational shear and step path failure had very low probability for large slope heights since most fractures were very short (typically less than 10 meters) and the intact rock strength was high. For the interramp stability, expected failure volumes were calculated from the probability of failure for the various failure modes. These were then compared to the costs associated with the actual mining. This cost-benefit analysis was not completed due to the difficulties involved in assessing all costs. It was, however, concluded that the failure volumes were relatively moderate, and the design slope angles were adjusted manually. The largest probability of failure was found for the benches. Bench face angles and bench geometry also control the maximum achievable interramp angle (Figure 1.1). The recommendations for bench geometry involved designing a catch bench to "catch" falling rock from upper levels. With this design some local failures are allowed provided that the failure volumes are moderate.

The recommended catch bench design for double benching which gives the steepest slope angles is shown in Figure 1.2. The term double benching is used for describing a mining geometry where catch benches are left only at every second bench. Individual benches have a height of 15 meters. These are first created and then the crest of the lower bench in each set of two (which form the future double bench) is mined back to the position of the upper bench. A double bench, 30 meters high, is thereby created. The actual bench geometry is thus slightly more complicated than shown in Figure 1.2. Furthermore, double benching requires an offset of around 5.5 meters at mid-height of the double bench to be able to drill the blast holes (Figure 1.4). This reduces the maximum achievable bench face angles; however, decreasing the offset significantly is not possible from an operational standpoint. Adopting the recommended criteria for bench and slope design resulted in interramp angles of 51° to 56° for the hangingwall, 47° for the north footwall, 42° for the middle portion of the footwall and 49° for the south footwall.

An important note to be made is that the above analyses were carried out assuming depressurized conditions within the pit. If groundwater pressures develop, significantly higher probabilities of failure would follow. This was in fact addressed by West et al. (1985) who proposed that monitoring of water conditions should be conducted and if necessary, drainage holes should be drilled.
Figure 1.2  Recommended catch bench design for Aitik (from Call et al., 1976; West et al., 1985).

The location of the phreatic surface according to West et al (1985) was that of a slightly inclined plane. A more detailed groundwater study, including test pumping, had been made by VIAK (1981) in between the two stability analyses. In this study, the groundwater conditions were interpreted quite differently. The south portion of the footwall was believed to be relatively impermeable, whereas the northern portion was considered much more permeable. Mapping and investigations also indicated the presence of large scale conductive zones around the pit (Figure 1.3). This was confirmed in later studies by VIAK (1990, 1991), which also indicated that the water table was fairly close to the slope face. The orientation of the most conductive zones is sub parallel to the orebody and dipping at 55 to 60° toward the west, and with an estimated spacing of around 100 meters (VIAK, 1990, 1991). Recent measurements have indicated that the groundwater level has remained high despite the increase in pit depth.

A drainage plan was also suggested to penetrate these conductive zones (VIAK, 1990, 1991). However, drilling of the first drainage hole resulted in virtually no water inflow at all, and further drilling was suspended. The suggested drainage plan has therefore not yet been completed. The lack of water inflow from the first drainage hole could be due to local variations and shows that the geohydrological conditions at Aitik appear to be relatively complex and deserve more attention.
Moving back to the issue of bench face angles, another important factor is the fact that the rock mass at Aitik has a very predominate schistosity from the orientation of mica minerals (biotite and muscovite) in the metamorphic rocks. This schistosity, more commonly referred to as foliation, is very prominent in the footwall and dips at approximately 67° toward the west (sub parallel to the footwall). Observed bench failures are in many cases associated with this foliation which implies that steeper bench face angles can be difficult to obtain. In fact, the drilling offset of 5.5 meters between each single bench is partly governed by the dip of the foliation. An offset of 5.5 meters was chosen so that a foliation plane cutting through the toe of the upper bench would also go through the toe of the lower bench. A thorough follow-up of the achieved bench face and interramp angles in the pit was conducted on several occasions using aerial photography (Blomberg, 1986; Petersson, 1988, 1990). These studies revealed that the measured interramp angles in general were lower than those recommended by West et al. (1985). Furthermore, the actual catch benches were narrower than the recommended bench widths, thus reducing their efficiency. The measured bench face angles for the footwall were on average 60° (for smooth blasting).

A major reason why the measured actual bench face angles are lower than those recommended by West et al. (1985) is the current blasting techniques. The blasting design has a large impact on the amount of backbreak at the crest of the bench, which may reduce the effective bench face angle (Figure 1.4). Also, in the biotite schist, failures tend to concentrate along the foliation planes, thus limiting the bench face angle to around 67° or less. Recent observations of bench failures also indicated that a zone of epidote and feldspar (associated with an altered old fault structure) could play a significant role in the initiation of bench failures (Krauland and Sandström, 1994). This zone is very continuous and can be up to 20 meters wide, and to some extent defines the ore boundary in the footwall.
The hypothesis that steeper bench face angles could be achieved if blasting could be kept under control was investigated in a recently concluded research project (Krauland and Sandström, 1994). Steeper bench face angles can actually be obtained by using various smooth blasting techniques. The blasting pattern must also be adjusted for different rock types. The biotite schist requires more smooth blasting (more rows of small-diameter boreholes) to limit foliation failures.

The most important conclusion from this study is that bench face angles probably can be increased from today's values, using controlled blasting (Krauland, 1996). The interesting possibility now arises as to whether it is possible to increase both the interramp and the overall slope angle. It then becomes an issue as to whether the overall large scale slope stability can be guaranteed? This has only been touched upon briefly by Call et al. (1976, 1977) and West et al. (1985) who concluded that the probability of large scale failures is very low. As mentioned earlier, this was based upon the fact that larger structures are yet to be confirmed. Furthermore, rotational shear failure involving both failure along existing fractures as well as failure through intact rock had a very low probability. This is due to the high strength for the intact rock, which implies that only 10 percent of the failure surface need to go through intact rock to maintain stability. West et al. (1985) estimated that a potential rotational shear failure surface would pass through at least 15 to 30 percent intact rock, thereby virtually eliminating the risk of failure. These figures are for the hangingwall. An even lower percentage of intact
rock is required to maintain the footwall stability. Step path failure, i.e., failure along several joint planes which are interconnected was, based on the mapped structural features, judged to be of importance only if overall slope angles on the footwall exceeded 51°. Analysis of slab failure and block flow instability lead to similar conclusions.

Without going into too much detail it can be concluded that these analyses are very sensitive to the input data. Although a very large number of joints were mapped on several occasions, it is not certain whether these are fully representative for the entire rock mass. The mapping techniques used tend to underestimate the joint length, something which may have a large impact on the potential of large scale failures. Furthermore, the strength parameters used for the intact rock bridges between the joints were those obtained from laboratory tests on drill cores of intact rock. These values may not be representative of the material strength on a larger scale of up to several meters and should thus be reduced. A reduction of the intact rock strength could result in higher probability of large scale rotational shear and step path failure. The data on rock mass strengths is also relatively limited, in particular for the footwall rocks, compared to the data on joint distributions (Call, 1994). It is also important to note that the analysis of rotational shear failure was carried out for a pit slope with a height of 360 meters. With the northern portion of the pit being planned for mining to at least 435 meters in depth, the validity of the conclusions regarding the low risk for large scale failures can be questioned. Finally, the effects of groundwater were not, as mentioned earlier, considered in these studies, nor were the virgin stress states in the rock mass.

A new study was undertaken by Call in the fall of 1994 (Call, 1994). The objective of this study was to review and update current bench design criteria and also to review the overall slope stability, given the new mining plans. From this study, a slightly modified catch bench criterion was proposed which would result in slightly narrower catch benches. A final report is not yet available from this study.

Other rock mechanics activities at Aitik have included additional structural mapping carried out by the Aitik staff. Mapping was conducted in two campaigns, the latest being line mapping of large structures which was done during the summer of 1994. Large structures were defined as structures which continued over at least two benches, i.e., more than 30 meters in length. Mapping was conducted in the northern portion of the footwall (Abrahamsson, 1995). Monitoring of slope displacements at Aitik has been carried out on a fairly regular basis since the mid 1980's. Total stations have been used to measure the displacement of prisms installed at various locations in the pit. The total station (GEODIMETER) was located on the slope crest at the south end of the pit. Some of the prisms have been damaged by blasting which has caused an interruption of the measurements
during the last few years. New prisms have been installed and a new measurement campaign has been initiated during the winter of 1996. The total station has been moved to the middle portion of the hangingwall.

Recent efforts by Boliden (Krauland and Sandström, 1994; Hustrulid, 1995) have included a preliminary analysis of large scale slope stability assuming a circular failure surface (rotational shear). Simple limit equilibrium methods were used together with assumed rock mass strengths to study the implications of deepening the pit. Different groundwater conditions were also considered. The results showed, as expected, that the overall pit slope angle is extremely sensitive to the choice of friction angle and cohesion for the rock mass. With an assumed friction angle of 30° and a final pit depth of 500 meters for a fully drained slope, a cohesion of 0.8 MPa resulted in a stable slope angle of 60°, whereas a cohesion of 0.5 MPa resulted in a stable slope angle of 50° (also see Section 3.5 of this report).

To summarize, it is clear that further studies of the aspects of large scale failures at Aitik are very much warranted. The problem consists of (1) determining the probable mechanism and mode of failure for large scale instabilities, (2) identifying the geomechanical parameters that govern failure, and (3) determining the values for these parameters. Furthermore, identifying how a potential failure progresses (rapidly or slowly) is of vital importance from a mine planning and production perspective. Development of a design methodology for the overall slope angle could also include strategies for mining given certain probabilities or risks of failure.

In the current mining plans, both the south hangingwall and the south footwall will be mined during 1996. The middle portion of the hangingwall will be completed during the period 1999-2003. In the northern portion, the footwall will be mined out completely (final pushback) during 1996 to 1997. The hangingwall will be mined in three more pushbacks during the periods 1996-1999, 1999-2001 and 2002-2005. According to the current production plan, the pit will be mined out completely by the year of 2005. If metal prices remain high and slope stability can be guaranteed, it may be possible to deepen the pit further in the north and mine additional pushbacks in this portion. There is no doubt that there is an immediate need for a better method of designing the overall slope angle, in particular for the middle portion of the footwall, which is the region where failure may affect the mill. The final pushback is already in production in the north footwall and efforts should be focused on the design of the overall final pit angle. The hangingwall situation is slightly different in that, at least, three pushbacks will be mined before the ore production is completed. Here, both the interim slopes in the pushback areas and the final pit slopes are important to the design.
Furthermore, several pushbacks may be mined at the same time which can cause additional problems that must be considered.

1.3 Objective and Scope of Report

Based on the above description of the problems at Aitik, the objective of this study can now be outlined as:

1. Review the rock mechanics state-of-the-art of final (ultimate) pit slope angles for open pit mines. The review should focus on large scale rock slopes and associated large scale failures. The review should be critical and point out advantages and disadvantages of the current design methods, and if and how these methods can be applied to the Aitik problem.

2. Based on this, propose a thesis project focusing on large scale slope stability with special application to the Aitik mine. The guiding principle for a thesis project is that it should be a significant and original contribution to the technical and/or scientific literature. Furthermore, the entire project should focus on achieving practically useful results which can be applied at Aitik.

3. Outline a preliminary work plan for the research project. This would include a list of anticipated activities and an approximate estimate of the time and cost requirements. The project will require efforts from both the Luleå University of Technology and the staff at Boliden Mineral and the Aitik mine.

The review is focused on the rock mechanics aspects of designing final pit slopes, but it is not within the current scope to fully describe all aspects of the design of final pit slopes. The aim is merely to provide enough background to propose a thesis plan and establish a platform for the entire research project. Further literature studies are probably warranted as the project is continued. This report deals with large scale stability of pit slopes, i.e., cases where failure involves large portions of a slope or entire slopes of several hundreds of meters in height. Bench scale failures are not covered in detail, nor is the design of safety berms and catch benches. Furthermore, location of ramps/haulage roads are not covered. The literature review also concentrates on experiences and design involving open pit slopes in hard rock. It has been necessary, however, to also review some of the knowledge regarding large scale slopes in weaker rock masses, since many of these slopes have experienced substantial failures. Also, experiences from soil mechanics regarding slope stability and some of the available
information concerning natural rock landslides have been briefly reviewed in order to provide a larger knowledge base.

1.4 Outline of Report

Following this introductory chapter, a general description of the economics of open pit mining and the role of pit slopes as construction elements is given in Chapter 2. Some basic terminology is also defined in Chapter 2. Chapter 3 deals with the mechanics of pit slopes, starting with stress conditions and then moving over to failure modes and failure mechanisms which have been observed. The specific characteristics of large scale slopes, as compared to slopes of small or moderate scale, are described, as well as the kinetics of rock slopes. Finally, in Chapter 3, the strength of large scale rock masses are discussed.

In Chapter 4, the design of rock slopes are described, starting with analytical techniques and moving over to numerical analysis, empirical design, physical model tests and finally probabilistic design methods. Remedial measures including support, drainage, and mining strategies for avoiding and dealing with failures are discussed in Chapter 5. A selection of case studies has been compiled in Chapter 6. In Chapter 7, recommendations for future research are given along with a preliminary thesis project plan. A comprehensive list of references concludes the report.
2 OPEN PIT SLOPES — AN INTRODUCTION

2.1 General

In open pit mining, mineral deposits are mined from the ground surface and downward. Consequently, pit slopes are formed as the ore is being extracted. It is seldom, not to say never, possible to maintain stable vertical slopes, or pit walls, of substantial height even in very hard and strong rock. The pit slopes must thus be inclined at some angle to prevent failure of the rock mass. This angle is governed by the geomechanical conditions at the specific mine and represent an upper bound to the overall slope angle. The actual slope angles used in the mine depend upon (i) the presence of haulage roads, or ramps, necessary for the transportation of the blasted ore from the pit, (ii) possible blast damage, (iii) ore grades, and (iv) economical constraints. In this chapter, the basic economics of open pit mining are discussed to put the problem of designing large scale slopes into the right context. The terminology used throughout this report will also be defined in this chapter.

2.2 Economics of Open Pit Mining

Aside from the stability aspects, the planning of an open pit is a matter of determining the most economic pit limit and the most economical mining program for a given mineralization (Steffen et al., 1970). The approach to open pit mine planning can be divided into two major phases; pit optimization and pit design and production planning. Pit optimization includes defining the final pit outline and the total mineable reserves and setting guidelines for the location of the initial pit and the optimal pit expansion (Boliden, 1993). Computer assisted hand methods or fully computerized methods are normally used for this. Several techniques are available for determining the ultimate pit limit, for example, the floating cone method and the Lerch-Grossman method (Armstrong, 1990b; Whittle, 1990; Hustrulid and Kuchta, 1995).

The second phase, pit design and production planning, involves determining mining sequence, cutoff grades and final layout and production plans. A completely automatic optimization of these factors is seldom possible. Instead, optimization is often done manually by trial and error and using computerized planning tools (Boliden, 1993). A more detailed description of the process of design and planning can be found in e.g., Hustrulid and Kuchta (1995). As the pit is being mined, continuous refinements to the design of the pit will be undertaken. More information will be available, for example, better knowledge of grade distribution, more structural geology data and data on production rates. A continuous update of the pit limit design is thus necessary to achieve optimal open pit mining.
The concept of waste rock mining and how this affects the choice of slope angles for an open pit will now be discussed. The removal of waste rock, or stripping, is intrinsic to almost all open pits, since the limits of the orebody seldom coincide with the practically achievable slope outline. The stripping ratio is defined as the ratio between waste removed and ore mined in the pit down to a certain depth. The stripping ratio could also be defined in economic terms, see e.g., Steffen et al. (1970), Armstrong (1990a), Stewart and Kennedy (1971), and Coates (1977). The overall stripping ratio refers to the total amount of waste in relation to the total amount of ore. On the other end of the scale is the instantaneous stripping ratio (also termed the current stripping ratio) which only applies to a limited period of time. Closely associated to the stripping ratio is the cutoff grade, defined as the minimum ore grade which will be mined. At economic breakeven, the cutoff grade is defined as the grade of ore that can pay for the costs of mining, processing and marketing but not stripping (Armstrong, 1990a). The corresponding stripping ratio is thus termed the cutoff stripping ratio (Coates, 1977). These definitions will do for our purposes, but as Halls (1970) pointed out, there are several other aspects which need to be considered for a precise determination of the cutoff grade. An economic optimization of the ultimate pit limit must also account for the time value of money. Several techniques are available such as Discounted Cash Flow (DCF) and Net Present Value (NPV), see e.g., Stewart and Kennedy (1971). Common for these is that they in various manners try to account for the fact that because of interest rates and inflation, future revenues are worth less than revenues that are closer in time. Also, waste rock which is mined ahead of the ore is charged with interest. Some of these techniques have also been applied to the issue of slope stability which will be discussed in subsequent sections of the report.

These concepts refer to the ultimate, or final, pit limit. The ultimate pit limits are not, however, achieved instantaneously and the working pit will thus differ from the final pit. The working slope, or operating pit, is a segment of the pit where active mining on several benches is conducted. The smallest units of a pit are the mining benches. The relation between bench face angle, interramp angle and overall pit slope angle was shown in Figure 1.1. Several benches can be mined as one unit, for example double benching where two benches are mined between each catch bench (Figure 1.2). This is sometimes also referred to as berms (Coates, 1977, 1981). When working the pit, the normal procedure is to mine a series of expanding shells, or pushbacks (Coates, 1977, 1981), (Figure 2.1).
Figure 2.1  Interim and ultimate pit slopes along with pushbacks and working benches (after Coates, 1981; Kim et al., 1977a).

Normally, the interim, or working, slope will be much flatter than the ultimate pit slope. Increasing the angle of the working slope can be advantageous from an economical standpoint since this will increase early revenues, i.e., delay mining of waste rock (Halls, 1970). Interim slopes could also be argued to be less important financially because they can be located solely in ore, and may not be as high as the ultimate slopes. In many cases, it may be worthwhile to conduct an optimization study also of the interim slopes (Coates, 1981), but establishing the mining sequence in an open pit is in itself a formidable task (see e.g., Hustrulid and Kuchta, 1995). A major reason for keeping flatter working slope angles flatter than those planned for the final slopes is that the risk of failure is substantially reduced. Massive slope failures at an early stage of mining could prove disastrous for the entire operation. It may also be necessary to mine at several pushbacks simultaneously to achieve high productivity, in which case stable interim slopes are a necessity.

Another aspect of the pit geometry which needs to be taken into account is the necessary operating width of the pit. Using the existing equipment in a certain mine, there is a minimum operating width for efficient ore extraction. This width becomes especially important during the final mining of the pit, when the pit bottom can be quite narrow. Sequencing alternatives that can be considered during the final mining stages are thus greatly limited.

The main benefit of steep slope angles thus comes from a minimum of waste excavation for ultimate walls and delayed waste excavation for interim slopes, whereas the principal cost of having a steeper slope comes from increasing instability and costs associated with this (Coates, 1981). In some cases, there may be a limit as to how steep the interim pit slopes can be made.
to allow for efficient operation of the pit. This could be regarded as an additional cost of steepening pit slopes. This means that both the final and the interim slopes are extremely important to design so that costs are minimized. The time factor must, however, also be considered. Open pit slopes need not be permanently stable. Generally speaking, the final pit is only required to be stable for as long as it takes to mine the last portion of the ore and get all personal and equipment safely out of the pit. Interim slopes on the other hand, need to be stable for as long as mining continues in that portion of the slope. The conventional definition of failure in rock mechanics is thus replaced by an economic definition of failure. In fact, optimal slopes often imply that some instability occurs as a completely stable slope would be too conservative and financially sub-optimal.

To summarize, it can be stated that financial considerations become of vital importance when setting out to design large scale slopes in open pit mining. It has also been demonstrated how important it is to assess slope stability at an early stage in the life of a mine. Mine planning activities must to some extent be based on an accurate estimate of the maximum achievable stable slope angle. There can also be situations in which completely stable slopes are required because costs inflicted by massive failure are simply unrealistically high. A total production stop for longer time periods for the relocation of the mill and plant are examples of this. For these cases, a comprehensive economic analysis might not be necessary. Whatever the implications of failure are, a thorough understanding of the mechanisms of slope behavior is fundamental, since this is something that all subsequent analysis and decisions must be based upon. It is thus necessary, before embarking on any statistical or financial analyses, to first study the mechanical behavior of rock slopes.

2.3 Pit Slopes as Construction Elements in Mining

Rock slopes are found both in civil and mining applications. Open pit rock slopes are, however, in many ways different from rock slopes in civil engineering projects. The location of the slope is often more fixed in an open pit mine, whereas, for example roads can be re-routed if difficulties are encountered when constructing a road cut. The only design parameter which can be varied is the shape of the slope and the slope angle, and to some extent the slope height. On the other hand, the life of an open pit slope is usually far shorter than the required life of slopes in civil engineering constructions (e.g., roadcuts). The economics of the pit operation are, to a much larger extent than in civil engineering applications, closely linked to the slope geometry. The consequences of failure are also vastly different and require different approaches, for example, very safe (over-designed) slopes in civil projects in urban areas compared to pit slopes with an often accepted risk of instability (Ross-Brown, 1972).
Open pit slopes share several characteristics with other construction elements in underground mining. The mechanical stability of the pit slopes determines the maximum achievable overall pit slope angle, and thus the maximum ore recovery and minimum stripping ratio. Compare this with pillars in an underground mine, defined as a portion of the mineable orebody which is left only to maintain stability in the mine. Consequently, in pillar design one strives to minimize the pillar area, whereas in pit slope design one strives to increase pit slope angles. The approach to design of both these construction elements is thus in many ways very similar, although the mechanics of underground construction elements differ significantly from that of pit slopes.

From the above discussion, one realizes that the manner in which a slope is designed to some extent depends on how failure is defined. For an open pit mine slope, a general definition of failure should reflect the function of the slope. Hoek and Pentz (1968) suggested the following: "Slope failure in an open pit mine may be defined as that rate of displacement of the rock mass surrounding the open pit which would render the recovery of ore uneconomic if the pit was being actively mined". This definition emphasizes the rate of displacement and the time of failure as design considerations for an open pit slope. One needs, however, to distinguish between this type of failure and mechanical failure of the slope. In this report, the following terminology is being used (the various aspects of slope failures and slope collapses will be discussed in detail in the Chapter 3):

**Failure**

Failure occurs when the loads or stresses acting on the rock material (intact rock or discontinuity) exceed the strength (compressive or tensile) of the rock. Failure could also occur through destressing of the rock. The term *failure* is used to describe failure on a small (but not microscopic) scale involving failure of the intact rock material, but the same term is also used to describe failure of the entire construction element. For this case, failure occurs when the construction element (in this case the pit slope) has exceeded its capacity to carry more of the loads and forces that are acting upon the slope. This means that the construction element still can carry some load after failure, but less than before failure.

**Slope collapse**

Slope collapse corresponds to economic failure of the slope, meaning that the consequences of the failure developing in the slope are so serious that they render the slope impossible to mine. A slope collapse could thus be both of local scale, involving one or a few benches, or of
global scale involving the entire slope, but in both cases causing interruptions of the mining production in that area.

**Failure mode** Failure mode is a macroscopic description of the manner in which failure occurs, for example the shape and appearance of the resulting failure surface. The failure mode can be regarded as a geometric description of the failure development.

**Failure mechanism** Failure mechanism is a description of the physical process that takes place in the rock mass as the load increases and failure initiates and propagates through the rock.

**Failure kinematics** Failure kinematics is simply a geometrical description of the motion or movements which results from a failure (Meriam, 1980).

**Failure kinetics** Failure kinetics relates the action of forces and loads on the slope to the resulting motions, and is thus closely linked to the failure mechanisms (Meriam, 1980).

Now that some of the basic terminology has been defined, it will be possible to consider the different components of the construction element — the open pit slope. The overall pit slope in an open pit mine, going from the pit bottom (toe of slope) to the crest of the pit, is made up of benches, interramp areas and haulage roads (Figure 1.1). Consequently, the maximum interramp angle is a function of the bench face angle, the bench height, and the width of the bench. The maximum achievable overall slope angle is in turn a function of the interramp angle and the width and number of haulage roads. It is important to realize that different failure modes can prevail at different scales and thus affect different portions of the pit. Separate design for different slope units and slope heights are therefore necessary. The geomechanical environment may also vary in horizontal direction, requiring different slope angles in different regions of the pit.

An interesting aspect of the definition of slope angles in Figure 1.1 is that both the bench face angle and the overall slope angle are defined from the toe to the crest of the slope face, whereas the interramp angle is not. This has a very practical advantage since interramp areas then have the same interramp angle as the height is being increased, which facilitates mine planning considerably. From a stability perspective, however, the slope is likely to become more prone to failure as the height is increased, which should be reflected by a decreasing slope angle. This could be accomplished if the angle was defined from toe to crest (Hustrulid
and Kuchta, 1995), as shown in Figure 2.2. Since the previous definition (Figure 1.1) is currently in use at Aitik, this definition of slope angles will also be used through this report. The possible differences in stability of different interramp heights must be considered separately in the design process.

Figure 2.2 Different definitions of interramp slope angle.

Finally, considerations must be given as to what governs the stability of an open pit slope. Stacey (1968) provided the following list of important factors:

- Geological structure
- Rock stresses and ground water conditions
- Strength of discontinuities and intact rock
- Pit geometry including both slope angles and slope curvature
- Vibrations from blasting or seismic events
- Climatic conditions
- Time

The above list is probably not complete; however it serves to show the difficulty in assessing rock slope stability. These factors also determine the mode of failure of a pit slope. It is only logical that the design of pit slopes should be based on the failure modes expected to occur and the governing failure mechanisms. This is, as will be shown, not always the case, and it is therefore necessary before the design methods are presented to describe the mechanics of rock slopes.
3 MECHANICS OF OPEN PIT SLOPES

3.1 Stress Conditions in Open Pit Rock Slopes

3.1.1 Virgin Stress and Induced Stress

Knowledge of the stress state in a slope is essential in order to understand the mechanics of slope behavior. The stresses acting upon a structure in comparison to the strength of the structure govern the stability of that structure. The virgin stresses (before excavation) in the rock mass are, in almost all cases, compressive, and are primarily a combination of:

- Gravitational stresses arising from the weight of the overlying rock
- Tectonic stresses which stem from external tectonic forces
- Stresses caused by previous glaciation
- Residual stresses

The gravitational and tectonic components are in most cases the major contributors to the overall virgin stress state. The virgin vertical stress can usually be assumed to be solely due to the weight of the overlying rock mass. The virgin horizontal stress, on the other hand, is more difficult to quantify due to the tectonic component normally present. The tectonic stresses vary significantly in different regions of the world, and this only further emphasizes the importance of conducting stress measurements to determine the virgin stress state before mining. Stress measurement techniques are not discussed here, but can be found in, for example, Brady and Brown (1985).

In general, an excess of horizontal stresses compared to vertical stresses are found at shallower depths, in particular in old shield regions such as Scandinavia. At deeper depths (>1000-2000 meters) the horizontal stresses decrease and at very deep depths the horizontal stresses are smaller than the vertical stresses (see e.g., Hoek et al., 1995). For an open pit mining operation in Scandinavia and other old shield regions, one will in most cases have a virgin stress state in which the horizontal stress is higher than the vertical stress, whereas in younger, sedimentary rock strata, the opposite can be true.

The virgin stress state is altered as the open pit is being excavated. The void created forces the stress to redistribute around the open pit. To understand this, consider a two-dimensional section of a very long open pit, in which the virgin stress state is characterized by a vertical, gravitational stress, and a horizontal stress which is higher than the vertical stress. Theoretically, the horizontal stress is forced under the bottom of the pit, resulting in stress
concentrations at the toe of the slope, and destressing of the pit walls, as is illustrated in Figure 3.1. The vertical stress after excavation will still mainly be due to the gravitational load. The resulting principal stresses around the open pit will thus have reoriented themselves compared to the virgin stress state. For a very long open pit, the principal stresses will be oriented parallel and perpendicular to the slope outline (Figure 3.1).

![Figure 3.1](image)

**Figure 3.1** Two-dimensional representation of the redistribution of horizontal stress around a long open pit (large dimension perpendicular to this plane).

Compressive stress concentrations at the toe of the slope promote stress-induced failure in this region. A zone of increased shear stresses also develops under the toe of the slope. Close to the pit wall, the principal stresses will be lower than the virgin stresses (destressing) which promotes joint opening and shear failure along pre-existing discontinuities in the rock mass, due to low normal stress.

In the literature on stability of rock slopes, it is often stated that stresses in slopes are low. Such a general statement is very dangerous since it only applies to the region close to the pit walls, and at the toe of small scale slopes. For large scale slopes, the stress state is much more complex with zones of both low and high stresses. There are, unfortunately, very few studies on the stress state in open pit slopes. Most of what has been learned is through photoelastic analysis, see e.g., Hoek and Pentz (1968), and from numerical analysis, see e.g., Stacey (1970, 1973). In the two-dimensional analyses carried out by Stacey, the slope geometry and the
virgin stress state were varied. It was found that destressing from horizontal stresses became more pronounced for steeper rock slopes. The width of the pit bottom influenced the toe stresses but not the stress state higher up in the slope. Varying the lateral (horizontal) virgin stress had a very large effect on both magnitudes and orientations of the resulting principal stresses. This suggests that the horizontal virgin stress is important to determine accurately. Later studies, for example, by Hustrulid and Kuchta (1995) and by the author, have shown that varying the horizontal virgin stress only affect the stress state in the toe region, whereas the region close to the slope face still will only be subjected to gravitational loads (the destressed region in Figure 3.1). Stacey's study also showed that tensile stresses developed at the crest of the slope. The tensile zone increased in extent with increasing virgin horizontal stress and steeper slope angles. This phenomenon was also found in analyses by Nilsen (1979) and Coulthard et al. (1992), but is yet to be confirmed by measurements.

Comprehensive evaluations and verifications of the overall stress state in and around open pits are relatively scarce. A comparison between measured and calculated displacements was presented by Blake (1968), in which good agreement was found, thus indicating indirectly that the calculated stresses were reasonable. Stacey (1970, 1973) compared calculated stresses with measured stresses at the Kimbley pit (Blake, 1968) and found relatively good agreement between the two. Nilsen (1979) and Broch and Nilsen (1982) compared calculated stresses around the Ørtfjell open pit, with measured stresses in a drift close to the pit. A qualitative agreement was found, indicating the same orientation and the same order of magnitude of the principal stresses.

In the above analyses of the stress state around open pits, homogeneous, isotropic and elastic material behavior was assumed. In reality, one can also expect some local stress redistributions around pre-existing discontinuities, as well as stress redistributions due to local failure and yielding of the rock material. A more serious assumption is that the pit geometry was simplified to enable a two-dimensional representation. The actual curvature of an open pit will change the way in which the virgin stresses are redistributed around the pit. In the two-dimensional case considered in Figure 3.1, the horizontal stress acting perpendicular to the cross-section is almost unaffected by the pit. For an open pit with finite dimensions, the intermediate principal stress will no longer be oriented parallel to the slope face, and its magnitude will also change significantly.

Some of the three-dimensional stress effects were quantified by Stacey (1973) using a three-dimensional numerical model. In this model two different slope curvatures (different radii) were analyzed with different virgin horizontal stresses. The slope angle was kept constant at 45°. The results showed that the stress redistribution in a vertical cross-section was of the
same pattern as for the two-dimensional case. However, due to the added confinement from the three-dimensional structure, the stress magnitudes at the toe of the slope were markedly lower. Furthermore, when the magnitude of the two virgin horizontal stresses were equal, no tensile zones were detected in the models. This is also an effect of the added confinement in a three-dimensional geometry. However, when the virgin horizontal stresses were unequal, tensile zones developed at the slope crest and behind the face of the slope, with more extensive tensile zones with larger differences between the two horizontal stress components.

The extent of low-stress zones in the pit walls also depends on the orientation of the pit in relation to the virgin stress orientations. In general, for an elliptical or elongated pit, the three-dimensional effects will be most pronounced on the short-axis of the pit. In these concave regions, added confinement by the intermediate principal stress will result in less destressing of the pit walls. The principal stresses will all be compressive, except perhaps for the minor principal stress (Long, 1964; Hoek and Pentz, 1968), (Figure 3.2).

These effects have been confirmed by field observations (Piteau, 1970; Hoek and Bray, 1981). These observations indicated an increase in stable slope angles of a little more than 10° when the radius of slope curvature decreased from 1000 feet (300 meters) to 200 feet (60 meters), for a slope height of 320 feet (100 meters). This was for a concave slope, but there could also be smaller portions of an open pit with a convex slope curvature. For this case, the stress state will be much more unfavorable with the minor principal stress, $\sigma_3$, being tensile and oriented tangential to the slope face (Figure 3.2). The intermediate principal stress can also be very low or tensile in this case. For a convex slope with a radius of curvature less than the slope height, Hoek and Bray (1981) suggested a flattening of the slope angle by 10° compared to a stability analysis in which only a two-dimensional geometry is considered. Bear in mind though, that such recommendations are only rough guidelines based on very few observations.

The non-uniformity of the stress state in a slope also affects the strength of the rock mass. The shape and location of the failure surface will be described in detail later, but for now assume that failure in a high rock slope occurs either along a curved failure surface which is deep seated in the slope, or along a planar, but more shallow, failure surface. The stress state at different points in the slope is illustrated in Figure 3.3, along with an assumed curved failure envelope. Low compressive, or even tensile, normal stresses are found at the crest of the slope. At the toe, the normal stress is moderate whereas the shear stress is relatively high. In the interior of the slope, the normal stress is high which implies that the resulting strength of the curved failure surface is high in this portion. Similar conclusions were drawn by Jennings and Steffen (1967).
Figure 3.2  Principal stress state in different portions of an open pit mine with varying slope curvature (partly after Hoek and Pentz, 1968 and Long, 1964).

The different stress states and the different strengths along various portions of the failure surface indicate that the mechanism of failure can be significantly different along the failure surface. In Figure 3.3, only a two-dimensional stress state is considered with the major and minor principal stresses oriented parallel to the cross-section of the slope. As was discussed earlier for a long open pit, the horizontal stress parallel to the long axis of the pit is less affected by the excavation of the pit slopes. For some cases it can even exceed the other two stress components in magnitude. This leads to the possibility of compressive failure in the direction perpendicular to the cross-section shown in Figure 3.3.
3.1.2 Groundwater and Effective Stress

The stress state in a slope is also dependent upon the groundwater conditions. Groundwater is defined as water below the water table, which is also known as the zone of saturation. Factors influencing groundwater pressure will be discussed later; however, for now it will be assumed that groundwater is present in a certain rock slope, thus resulting in varying groundwater pressure distribution throughout the slope.

The primary effect of groundwater pressure is through the principle of effective stress. The stress state at a point in the rock mass is governed by the principal stresses and the acting water pressure. Since the water pressure is equal in all directions, it acts to reduce the effective stress at a given point in the rock mass. This principle applies to all permeable
materials and both soils and rocks. A reduction in the effective stress state has a bearing on the shear strength of the rock mass. The shear strength of a discontinuity is directly proportional to the applied normal stress. A reduction of normal stress consequently leads to a reduction of shear strength of the failure surface. Furthermore, existing groundwater pressures can act as additional driving forces on failure surfaces for certain failure modes. Secondary effects of having water present is that some minerals react unfavorably with water thus reducing the material strength of a filled discontinuity for certain rock types. Erosion brought about by flowing water could also result in reduced strength (Morgenstern, 1971; Sage, 1976; Sharp et al., 1977; Hoek and Bray, 1981).

The sources of groundwater and the groundwater pressure distribution in slopes also need discussion. The undisturbed water table (before excavation) in a rock mass depends on (1) the infiltration of rainfall and melting snow, (2) the surrounding topography, (3) nearby rivers and lakes, and (4) the geohydrological characteristics of the rock mass. It follows from this that the water table changes with time, for example, during spring runoff or heavy rainfalls. As an open pit is being mined, the initial water table will change (be drawn down) due to inflow of water into the excavation. The water table, or the phreatic surface, will thus change constantly depending upon the development of the excavation (Morgenstern, 1971; Sharp et al., 1977). This drawdown will result in a water table as illustrated schematically in Figure 3.4 for a homogeneous rock mass. The actual shape and location of the water table will depend upon the slope geometry, the permeability characteristics, and recharge from the surrounding rock mass. Furthermore, freezing of water during wintertime can prevent flow into the pit and thus increase groundwater pressures in the slope. There may also be zones with contained, or perched water, in a rock mass which further complicate the groundwater pressure distribution.

![Figure 3.4](image)

**Figure 3.4** Drawdown due to mining of an open pit and the resulting phreatic surface (partly after Sharp et al., 1977).
Points in the rock mass below the water table are subjected to groundwater pressure. Consequently, these points are also subjected to a lower effective stress than points which are located above the water table, as shown in Figure 3.5.

![Diagram of slope with water pressure and stress states](image)

**Figure 3.5** Comparison of the effective stress state in a partly saturated slope (top figure) and an almost drained slope (bottom). Stress states are shown for the same point in the slopes and for two different phreatic surfaces.

The groundwater pressures along a surface in the saturated zone represents the piezometric surface, which is a profile of water pressure on a given surface. For static conditions, i.e., no flow or uniform horizontal water flow, the piezometric surfaces all coincide with the water table, but in all other cases there will be a different piezometric surface for each physical surface through the zone of flow (Sharp et al., 1977). It follows that in order to correctly assess the groundwater pressure on an actual or potential failure surface, the water pressures cannot be calculated from the location of the water table but must instead be measured. It is
often difficult to precisely determine water pressure distributions along failure surfaces and in practice, an assumption of static conditions is often made and the groundwater pressures are thus calculated from the vertical distance to the water table.

Measurement of groundwater pressure can be conducted using various types of piezometers, ranging from simple standpipe tubes to electrical piezometers. Open holes and standpipes can be used for rocks with high permeability, whereas a closed system, and preferably an electrical piezometer is required for less permeable rocks (Hoek and Bray, 1981). Geophysical techniques, preferably resistivity measurements, have also been used to quantify flow paths and groundwater levels in the rock mass (Kusumi, Taniguchi and Nakamura, 1993). The application of these techniques is new and promising but not yet fully tested and verified.

The permeability, or hydraulic conductivity, of the rock determines its ability to transmit a fluid. Compared to a granular material such as soil, intact rock has a relatively low permeability. This applies in particular to hard, igneous rocks. However, since the rock mass also contains numerous discontinuities, the mass, or bulk, permeability can be significantly higher, as the flow of water concentrates along pre-existing discontinuities (Louis, 1969; Londe, 1973b). The combined rock mass permeability can, depending upon the degree of fracturing, be several magnitudes of orders higher than the permeability of intact rock. By only considering an equivalent permeability for the rock mass, the actual flow pattern in the rock mass is grossly simplified. If larger and more dominate structural features exist, this simplification cannot be justified.

The permeability of individual discontinuities is very sensitive to changes in the joint aperture (opening) which in turn depends upon the normal stress acting on the joint surfaces. Thus, a coupled behavior between stresses and groundwater flow arises. For instance, a slope which is being destressed close to the face will permit a higher flow of water in this portion, thus changing the location of the phreatic surface which in turn changes the effective stress. The same could occur in a rock mass which is strongly fractured during failure. On the other hand, a decrease in the overall permeability can be expected in regions of high compressive stress, for example, at the toe of the slope (Sharp et al., 1977).

The coupled effects discussed above are difficult to quantify, and illustrate the difficulty in accurately determining the groundwater pressure in a rock slope. In most cases, it is not economically or practically feasible to measure the actual groundwater pressure distribution, other than at a few selected points. To be able to assess the groundwater pressures at other locations in the rock mass surrounding the pit, an analysis tool must be utilized. Previously, graphical flow nets have been used extensively but this technique is very time-consuming for
more complex problems, involving anisotropic and heterogeneous materials. Today, numerical analyses are commonly used for assessing the resulting groundwater pressures, thereby also accounting for coupled effects and flow in discontinuities.

In the presentation so far, only the two-dimensional case of groundwater flow has been discussed. In reality, the groundwater pressure will vary in all three dimensions as well as around the slope outline, just as was discussed with respect to the stress state. The three-dimensional flow of water is difficult to analyze theoretically (Louis, 1969), in particular for jointed rock masses and accounting for coupled effects between stress and water pressure.

To summarize, the stress state in the rock mass surrounding an open pit depends on (1) the virgin stress state before excavation, (2) the slope geometry in three dimensions, (3) the groundwater conditions, and (4) to some extent the local geology of the rock mass. An assessment of the true three-dimensional stress state around an open pit requires good measurements of both the virgin stress state and the groundwater pressures, and a good analysis tool for calculating stress redistributions due to mining. The effective stress state will obviously influence both the failure modes and the resulting strength parameters for a rock slope. Preferably, the analysis tool should be able to handle groundwater pressures and coupled effects but this is seldom practically feasible. In many cases, it might be necessary to assess the effects of existing groundwater pressures separately or in a simplified manner. The accuracy required obviously depends on the accuracy with which other parameters which govern the stability of a slope can be determined.

### 3.2 Geological Structure

The geological structure is obviously one of the most important factors governing the stability of slopes. The term geological structure includes both rock type and discontinuities which together form the rock mass. Discontinuities are present on virtually all scales, from microcracks which have lengths less than 10 mm, to the plate boundaries of the earth crust with lengths of thousands of kilometers. The strength and stiffness of small scale (laboratory) samples of rock depend upon the presence of microcracks. For a rock slope, however, microcracks are of little importance. Different rock types can, however, also have a more or less developed structure, for example, parallel bedding in sedimentary rocks and schistosity, or foliation, in metamorphic rocks. Also, boundaries between different rock types and dykes can be viewed as discontinuities. Bedding and schistosity are of much larger scale than microcracks. Bedding planes can extend for kilometers and a well developed schistosity can be several tens of meters long. Other geological discontinuities include joints which vary in
size from less than 1 meter to over 100 meters and faults which can be anywhere from a few tens of meters to several hundreds of kilometers in length. Faults are distinguished from joints in that faults have undergone shear movements. It is also common that several parallel joints or faults form a shear zone (Herget, 1977; Nordlund and Rådberg, 1995), (Figure 3.6).

![Figure 3.6](attachment:Figure3.6.png)  
Figure 3.6  Different types of joints and faults (partly after Nordlund and Rådberg, 1995).

In the above discussion, all discontinuities were treated as one-dimensional elements only having a length. In reality all discontinuities are two-dimensional planes with an area and an orientation. The actual shape of a discontinuity is not well understood. Often they are assumed to be circular disks but this assumption is often made for mathematical reasons since it simplifies three-dimensional calculations significantly, and is not based on actual knowledge of discontinuity geometry.

One realizes that the geometry and orientation of pre-existing discontinuities can have a large impact on the behavior of a slope. Discontinuities represent weak links in the rock mass. For cases in which the normal stress is low, pre-existing discontinuities in the rock mass can more easily open up or be subjected to slip. Under the assumption of low normal stress, the mode of failure in an open pit slope to a large extent depends upon the orientation of discontinuities in the rock mass in relation to the orientation of the slope and the slope angle. Obviously, the frequency and the continuity of discontinuities are also important in this aspect (Hoek, 1971a). In analyzing the stability of a rock slope, the following assumptions are often made (Piteau, 1970):

i. Structural discontinuities are detectable features and can be described quantitatively.
ii. Structural regions exist in a rock mass.
iii. A reliable model representing a jointed rock mass can be constructed.
iv. Failure surfaces will be essentially plane or combinations of planes.

Discussion relating to each these propositions will now follow. To identify structural patterns in the rock mass, mapping of discontinuities is normally conducted. Current mapping techniques are in most cases based upon surface mapping, in the form of scanline mapping and cell mapping. In scanline mapping, discontinuities which intersect an imaginary or physical line are recorded. Line mapping is normally conducted along benches in an open pit. Using cell mapping, a "window" is defined on the bench face and discontinuities which lie fully or partially in that cell are included in the joint survey (Mathis, 1988). Evaluation of mapped joints is commonly done through the use of stereographic projection, in which discontinuities with similar orientation are grouped together in joint sets using various statistical techniques. Stereographic, or hemispherical, projection is a very versatile tool for visualizing structural data and identifying potential modes of failure (see e.g., Hoek and Bray, 1981; Priest, 1993a). The various combinations of these joint sets are then checked for kinematic admissibility.

It is important to realize that all surface mapping automatically introduces a mapping bias concerning discontinuities located farther into the rock mass. Cell mapping overcomes part of the sampling bias but a completely unbiased mapping cannot be guaranteed. Joint set orientations are relatively reliable but, for example, joint trace lengths are notoriously difficult to determine (see e.g., Call, Savely and Nicholas, 1977; Grossman, 1995). Mapping of borehole cores can lead to better knowledge of the location of weak zones farther into the rock mass, but the same areal coverage as in surface mapping cannot be achieved. The assumption that structural regions exist is also difficult to validate. Perhaps the biggest problem is associated with constructing a reliable model of the rock mass. Although advances have been made in representing and delineating different joint sets (Priest, 1993b), there are still problems concerning the true three-dimensional shape of a discontinuity. Some of these aspects will be discussed in the section on probabilistic design methods (Section 4.6) since most of those methods also rely heavily on statistical sampling of pre-existing discontinuities. Constructing a representative model of the rock mass becomes even more difficult for large scale slopes due to the large volume involved. These complications should not, however, be used as excuses to avoid extensive geological mapping. Knowledge, albeit somewhat biased, of the geological structure is very important to have, but the above discussion illustrates how difficult interpretation of the geological structure can be.

One must also examine the assumption regarding plane failure surfaces. The traditional approach to rock slope stability analysis has focused on failure modes which are directly
governed by structural discontinuities such as plane shear and wedge failures (these and other failure modes will be described in Section 3.3). This approach appears to work reasonably well for small scale slopes, but may not be valid for large scale slopes. Using the Aitik case as an example, mapping of structural features and discontinuities in the rock mass indicated that the majority of the pre-existing discontinuities in the rock mass were of limited length (less than 10 meters), (Section 1.2). Excluding major fault and shear zones, this could be the case for many rocks of a similar type. For a high slope, the rock mass will appear very fractured as the length of individual discontinuities is small compared to the dimensions of the slope. This difference in scale is illustrated in Figures 3.7, 3.8 and 3.9 for (1) a bench 30 meters high with a 70° face angle, (2) an interramp slope of 90 meters height and 50° interramp angle, and (3) an overall slope with a height of 500 meters and a 50° overall slope angle, respectively. In these figures, a two-dimensional cross section of the slope is shown to contain two hypothetical joint sets with the following characteristics:

<table>
<thead>
<tr>
<th>Joint set A</th>
<th>Joint Set B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dip:</td>
<td>60°</td>
</tr>
<tr>
<td>Joint length:</td>
<td>10 ± 1 meter</td>
</tr>
<tr>
<td>Rock bridge:</td>
<td>3 ± 1 meter</td>
</tr>
<tr>
<td>Spacing:</td>
<td>3 meter</td>
</tr>
</tbody>
</table>

Figure 3.7  Discontinuity pattern for a bench slope with a slope height of 30 meters and a slope angle of 70° and with two different pre-existing joint sets.
Figure 3.8  Discontinuity pattern for an interramp rock slope with a slope height of 90 meters and a slope angle of 50° and with two different pre-existing joint sets.

Figure 3.9  Discontinuity pattern for a large scale rock slope with a slope height of 500 meters and a slope angle of 50° and with two different pre-existing joint sets.

The two joint sets used here are only a crude approximation of a jointed rock mass. In reality, there are probably more than two joint sets present and the joint spacings and joint lengths are, in general, much smaller than as shown in Figures 3.7 to 3.9. The discontinuity pattern used
here has a $RQD$ of 100%, indicating very good and solid rock. Nevertheless, these three figures clearly illustrate the dramatic difference between a slope of moderate height and a large scale slope.

The actual situation in a large scale slope is thus that of a heavily fractured rock mass in relation to the height of the slope, and with block sizes being small relative to the dimensions of the slope. A rock mass can thus be described as closely jointed when the joint spacing is small in relation to the scale of the entire slope (Pender and Free, 1993). From this, one must also realize that the failure mode of a large scale slope can differ significantly from that of a small scale slope. Although it appears reasonable that single discontinuities are much more likely to be dominant on a small scale than on a large scale, discontinuities of a larger scale cannot be neglected. Discontinuities of the same size as the pit slope must still be accounted for and kept track of, since these can drastically alter the situation pictured in Figure 3.9.

3.3 Failure Modes and Failure Mechanisms

3.3.1 Introduction

Based on the geological structure and the stress state in the rock mass, certain failure modes appear to be more likely than others in large scale slopes. The belief that the stress state in general is low in rock slopes (Section 3.1) has had a major influence on what has been reported earlier in terms of failure modes. Structurally controlled failures dominate and there is a chance that more plausible failure modes for large scale slopes are yet to be discovered. This is not unlikely, considering that today's deep open pits will be superseded by much deeper pits in the future, particularly in hard rocks.

In Section 3.2, the difference between slopes of different scale but in the same geological environment was discussed. This discussion can be extended to the stress state in rock slopes. The assumption of low normal stress is not generally valid since very high compressive stresses can also be found in rock slopes, (see Figure 3.3). In particular for slopes of a large height, the stress concentrations at the slope toe cannot be neglected when considering the potential failure modes. Consider, for example, a 500 meter high slope in hard rock with a density of 2700 kg/m$^3$ and a uniaxial compressive strength ($\sigma_c$) of 200 MPa for the intact rock material. At the slope toe, the virgin vertical stress ($\sigma_v$) resulting from the overburden weight would be 13.5 MPa. Moreover, assume that the strength of the large scale rock mass is one-tenth of the intact rock strength, i.e., 20 MPa. Without considering the horizontal stress or the stress redistribution at the toe, this gives a ratio between the virgin vertical stress and the
compressive strength \((\sigma_v/\sigma_c)\) of 0.67. For comparison, a slope height of 90 meters (interramp slope) in the same material, gives a stress/strength-ratio \((\sigma_v/\sigma_c)\) of only 0.12. This clearly illustrates another difference between small scale and large scale slopes.

There are also some factors which are very important for bench slopes, but have less bearing on high overall pit slopes. Blasting was mentioned in Section 1.2 as a primary factor governing the maximum achievable bench face angles at Aitik. The effects of careless or poorly designed blasting can be very significant for bench stability, as noted by Sage (1976) and Bauer and Calder (1971). Besides blast damage and backbreak which both reduce the bench face angle, vibrations from blasting could potentially cause failure of the rock mass. For small scale slopes, various types of smooth blasting have been proposed to reduce these effects and the experiences are quite good (e.g., Hoek and Bray, 1981; Sage, 1976). For large scale slopes, however, blasting becomes less of a problem since backbreak and blast damage of benches have negligible effects on the stable overall slope angle. Furthermore, the high frequency of the blast acceleration waves prohibit them from displacing large rock masses uniformly, as pointed out by Bauer and Calder (1971). Blasting-induced failures are thus a marginal problem for large scale slopes. Seismic events, i.e., low frequency vibrations, could be more dangerous for large scale slopes and several seismic-induced failures of natural slopes have been observed in mountainous areas (Voight, 1978). Since the Aitik mine is located in a seismically stable region, it is outside the scope of this review to further discuss this issue.

Observed failure modes in rock slopes will be described in the following section. Emphasis is put on the most likely failure modes for large scale slopes, but typical failures in small scale slopes are also briefly described. Special attention is paid to geomechanical environments similar to those encountered at the Aitik mine, i.e., relatively stiff and strong, jointed rock masses. A more detailed description of the governing failure mechanisms for large scale failures is given in subsequent sections, based on field observations, physical model tests in the laboratory, and numerical analysis.

### 3.3.2 Observed Failure Modes

**Plane Shear Failure**

The first class of failure modes refers to various geometrical combinations of discontinuities which form blocks or wedges of rock which are kinematically free to move. Whether failure occurs or not depends upon (a) the shear strength of the discontinuities, or (b) the strength of discontinuity intersections. In a simplified classification of failure modes in rock slopes,
Coates (1977, 1981) gave this type of failure the name "plane shear" failure, (Figure 3.10). The failure surface, defined as the surface along which failure develops, could be a single discontinuity (plane failure), two discontinuities intersecting each other (wedge failure) or a combination of several discontinuities connecting together (step path and step wedge failures). A common feature of most failure modes is the formation of a tension crack at the slope crest.

![Diagram of failure modes](image)

**Figure 3.10** Combinations of discontinuities forming a failure surface (after Coates, 1977; West et al., 1985; Call and Savely, 1990).

The failure modes depicted in Figure 3.10 are, with the exception of wedge failure, two-dimensional representations. For failure to occur, release surfaces must be present to define the rock block in the lateral direction. Alternatively, step path failure may be a true three-dimensional failure in which combinations of discontinuities define the failure surface in all three dimensions.
No scale is given in Figure 3.10 since the figures in this section are only intended to illustrate the principal failure modes. A discussion regarding the probable occurrence of these failure modes is therefore necessary. Referring to Figures 3.7 to 3.9, it is clear that plane failure and wedge failure along single discontinuities is most likely on a bench scale in an open pit. A good example of plane shear failure on a single, prominent discontinuity is failure along the foliation planes at Aitik.

For large scale slopes, it is obvious that plane failure and wedge failure are unlikely, unless major structural features of the size of the entire slope are present. Although this cannot be ruled out, a much more likely event would be where several shorter and less prominent discontinuities connect to form a failure surface as shown in Figure 3.10 for step path and step wedge failures. It may also be possible that failure through intact rock is involved in complex step path failures.

Rotational Shear Failure

Following the classification by Coates (1977, 1981), the next group of failure modes are rotational shear failures (Figure 3.11). These are sometimes referred to as circular failures (Hoek and Bray, 1981) which implies that failure takes place along a circular arc. Rotational shear failure occurs in slopes without critically oriented discontinuities or planes of weakness, and this is the typical mode of failure in soils. As has been pointed out by Hoek (1970) and Hoek and Bray (1981), rotational shear failure could also occur in rock slopes if there are no strong structural patterns in the slope, for example, a heavily fractured slope with no predominante orientations of discontinuities.

The condition for a rotational shear failure is thus that the individual particles in a soil or rock mass should be very small compared to the size of the slope, and that these particles are not interlocked as a result of their shape (Hoek and Bray, 1981). Comparing these conditions with those of Figure 3.9, it appears that for large scale slopes, rotational shear failure is a very plausible failure mode. The issue of whether a rock mass can be considered as heavily fractured is thus mostly a matter of scale. Rotational shear failure in a large scale slope, would probably primarily involve failure along pre-existing discontinuities with perhaps some portions of the failure surface going through intact rock. Also, rotation and translation of individual blocks in the rock mass would help to create a failure surface. The resulting failure surface would follow a curved path. This is illustrated schematically in Figure 3.11, but with the individual discontinuities drawn with very large lengths. In practice, a fracture pattern similar to that in Figure 3.9 is more likely and thus it is relatively easy to visualize the
development of such a failure surface. The resulting failure surface need not, however, be
circular in this case. In Figure 3.11, the failure surfaces are drawn to be relatively deep, but
they could also be more shallow. There may also be combinations of plane failure, step path
failures and rotational shear failures, with or without tension cracks at the slope crest.

![Diagram of failure surfaces](image)

**Figure 3.11** Rotational shear failures and combinations of rotational shear and plane

In Figure 3.11, only two-dimensional representations of the failure surface are illustrated. In
reality, rotational shear failure is a three-dimensional phenomenon and the resulting failure
surface of a rotational shear surface will be bowl- or spoon-shaped (Figure 3.12). The factors
determining the shape and location of the failure surface will be discussed in more detail in
Section 3.3.3.
Several large scale failures with curved failure surfaces have been observed in open pits with relatively high slopes, in particular in weak rocks such as shales (e.g., Brawner, 1977, Franklin and Dusseault, 1991, and Chapter 6 of this report). Consequently, rotational shear and other associated failure modes where several sets of discontinuities together form the failure surface and where failure through the intact material cannot be ruled out, are important to consider for large rock slopes.

**Block Flow and Toppling Failure**

The third group of failure modes is named block flow failures (Coates, 1977, 1981). Characteristic for these type of failures is that a successive breakdown of the rock slope occurs. Failure can initiate by crushing of the slope toe, which in turn causes load transfer to adjacent areas which may fail (Figure 3.13). Obviously, *in situ* stresses in relation to the rock strength are important factors governing this failure mode. As discussed earlier, the stress concentration at the slope toe increases with the slope height. These types of failure are therefore more plausible in large scale slopes.

The presence of discontinuities in the rock mass can result in several secondary modes of failure once crushing of the toe has occurred. Large blocks and wedges, or assemblages of many smaller blocks can be relieved and a combination of block flow and plane shear failure can develop. An associated form of failure is toppling failure (de Freitas and Watters, 1973; Goodman and Bray, 1976; Hoek and Bray, 1981). Toppling refers to overturning of columns of rock formed by steeply dipping discontinuities and sub horizontal cross joints (primary toppling). It could also be initiated by crushing of the slope toe, which is termed secondary
toppling (Hoek & Bray, 1981; Glawe, 1991). Pure flexural toppling (Figure 3.13) is more likely on a bench scale since it requires very continuous pre-existing discontinuities.

![Figure 3.13 Block flow and toppling failures (after Coates, 1977; Hoek and Bray, 1981).]

Toppling failure can also be an important secondary mode of failure, initiated by other failures as shown in Figure 3.13. Secondary toppling could also occur even if the discontinuities dip in the same direction as the slope dips, but are then a result of more deep seated movements in the slope. Toppling of such "underdip" slopes have been observed in natural rock slopes (Cruden, 1989; Hu and Cruden, 1993; Cruden and Hu, 1994).

In several large open pit mines, deep seated toppling has been observed to occur even in very high slopes (Daly, Munro and Stacey, 1988; Martin, 1990; Pritchard and Savigny, 1990; Orr, Swindells and Windsor, 1991; Board et al., 1996), see also Chapter 6. In this case, rotation
and shearing along steeply dipping joints occurred deep into the pit walls, resulting in a "base" shear surface along which a larger portion of the slope could slide. The failure surface sometimes develops almost parallel to the slope face and sometimes in a curved, near-circular shape (Figure 3.14). The formation of a basal failure strip due to toppling has also been observed in high natural rock slopes (Giraud et al., 1990). This process eventually leads to planar slip along the base failure surface. Obviously, relatively long and continuous joints must be present for these failure modes to develop. There are, however, some indications that large scale toppling can develop as a secondary phenomenon where steeply dipping discontinuities are formed as part of the failure process.

Figure 3.14  Large scale toppling failure (after Daly, Munro and Stacey, 1988; Martin, 1990; Board et al., 1996).

**Slab and Buckling Failure**

There are also a number of other failure modes which have been observed in open pits over the world and which do not easily group into the above categories. Among these one finds the thin slab failures, including buckling failures (Piteau and Martin, 1982; Nilsen, 1987; Hu and Cruden, 1993; Cruden and Hu, 1994), shown in Figure 3.15. These types of failures could develop in slopes with long, continuous bedding planes or joints oriented parallel to the slope.
face. Crushing failure of the toe or plane shear failure along cross joints help to initiate slab failure, but slab failure could also be initiated by hydrostatic uplift due to high groundwater levels. Buckling could develop if the axial stresses on the rock slab are high and the slab is very thin in relation to its length.

It is difficult to picture a high rock slope with continuous joints running from the toe to the crest of the slope, and bucking and slab failure are therefore probably more of a local nature. To conclude, the conditions for these types of failures are rather unusual and there are few cases where they have been observed in the field. Both the toppling failure modes in Figures 3.13 and 3.14 and the slab failures in Figure 3.15 require release surfaces of some sort in the third dimension to develop.

**Summary**

- The most important failure modes to consider in large scale slopes are rotational shear failures and the secondary failure modes associated with these. Large scale toppling failures have also recently been observed in high pit slopes, and need to be considered.

- Plane failure or wedge failure could develop if major faults of very large dimensions are present in the rock mass.
Judging from the relative dimension of high slopes and most discontinuities, the actual failure surface for a rotational shear failure is probably to a large extent composed of pre-existing discontinuities, with only smaller portions of intact rock failure.

It is difficult to determine in advance what modes of failure that could occur in a certain open pit and how discontinuities interact to form a failure surface. These aspects will be discussed in more detail in the next section on failure mechanisms.

3.3.3 Failure Mechanisms Inferred from Field Observations

It is clear that the above failure modes are only text book models. The exact failure surface is very difficult to predict in advance. Even when failure has occurred, it is hard to identify the actual failure surface, with the exception of, for example, plane shear along a well defined pre-existing discontinuity. Description of the mechanism behind observed failure modes also presents some difficulties. Several issues are of interest:

- What is the actual shape and location of the failure surface? Does the failure surface pass through the toe of the slope? Is the failure surface curved, circular or more planar? What are the conditions for a deep seated versus a shallow failure surface?

- Where does failure initiate and how does failure progress in the slope?

- Does failure go through portions of intact rock or is it merely a combination of failure along pre-existing discontinuities?

- What are the conditions for failure initiation? At what stress level does failure initiate?

In the following section, these issues will be addressed and discussed based on observations from both soil and rock slopes.

Shape and Location of Failure Surface

For plane shear failures, the mechanism of failure is relatively simple to explain. Failure will initiate when the supporting rock at the location where the discontinuity daylights the slope is removed by excavation. The shape and location of the failure surface is determined solely by
the location and orientation of discontinuities. The dominate failure mechanism is shear failure as the shear strength of the discontinuity is exceeded. Once a slope geometry with cracks daylighting the slope face is created, the strength characteristics of the discontinuities and the intact material, together with the stress state in the slope, including the groundwater conditions, will govern whether failure will initiate or not. This also applies to simple wedge failures. The driving forces for plane failures and wedge failures are normally attributed to the self-weight of the separated rock block. For a saturated or partly saturated slope, the groundwater pressure could also act as a driving force, and to reduce the effective normal stress on the slip plane. Although the driving forces behind more complex failure modes such as step path and rotational shear failures in principal are the same, the actual mechanism of failure is more difficult to quantify.

The mechanisms of rotational shear failure are especially interesting to consider for large scale slopes. Considerable amount of work on this failure mode has been done in the field of soil mechanics. It is thus natural to review some of this work and see how this applies to rock slopes. As was discussed earlier, a very high slope can almost be considered as a granular material, thus having several similarities with a typical soil. An important difference, though, is that sandy soils are mostly frictional materials and clayey soils are mostly cohesive materials, whereas a rock mass probably exhibits both an effective friction angle and an effective cohesion. One must therefore be careful when translating the experience gained on the behavior of soil slopes to the behavior of open pit rock slopes.

First, the shape of a rotational shear surface will be discussed. One of the very early (perhaps the first) observation of a curved failure surface was that by the French engineer Alexandre Collin in the mid 19th century. Collin observed clay failures along a canal in France and concluded that the failure surface approximated a cycloidal arc (Skempton, 1949). An important finding from this work was the fact that the failure surface almost always cuts across existing clay strata. This discovery was followed by observations of failures of quay walls in Gothenburg in the beginning of this century (Nash, 1987). It has later been found that for a homogeneous slope in soil, the failure surface will be nearly circular. For soils which are more heterogeneous, a non-circular, but still curved, failure surface is more common (Skempton and Hutchinson, 1969).

The location of the failure surface (shallow or deep seated) is determined by the relation between the friction angle and cohesion (Spencer 1967; Johansson and Axelsson, 1991; Bromhead, 1992). In a purely frictional material, such as sand, the failure surface is more shallow and daylightts at the toe of the slope, whereas in a purely cohesive soil, the failure surface always tend to be more deep seated. This can be explained by the fact that cohesion is
constant with increasing normal stress, whereas the frictional component of the shear strength increases with increased depth and increased normal stress. The higher normal stress in the interior of the slope (see Figure 3.3) then results in high shear strengths being developed in the interior of a frictional slope, thus promoting more shallow failure surfaces. The depth of the failure surface also depends upon the difference between the slope angle and the friction angle; larger differences result in a more deep seated failure. For cohesive soil slopes, observations have also shown that it is more common that the failure surface daylights below the toe of the slope. The conditions for the location of the failure surface can be summarized using Janbu’s number $\lambda_{c\phi}$ (Figure 3.16). Janbu’s number is defined as (Spencer, 1967; Janbu, 1973; Johansson and Axelsson, 1991):

$$\lambda_{c\phi} = \frac{\rho g H \cdot \tan \phi}{c}$$

(3.1)

where $\rho g$ is the unit weight of the material, $H$ is the slope height, $c$ the cohesion and $\phi$ the friction angle for the material.

This basic relation has been confirmed by observations in soil slopes, but probably also applies to rock slopes, as was noted by Piteau and Martin (1982). Another factor which can influence the shape of the failure surface is the failure envelope. Charles (1982) showed that a more curved failure envelope would result in a more deep seated failure surface and a smaller radius of the slip circle. However, this has not been verified by field observations.
There is currently a lack of knowledge regarding the true three-dimensional shape of the failure surface, and what parameters determine this shape. This is due to the limited amount of good field observations and measurements of the three-dimensional failure behavior.

**Failure Initiation and Propagation**

Common for both soil and rock slopes is the fact that the failure surface cannot develop at the same instant throughout the slope. There must be a progressive mechanism of failure development eventually leading to the full collapse of the slope. The failure development has been difficult to quantify even for homogeneous soils. Progressive failure is defined here as the successive development of a failure surface in a slope through stress redistribution and loss of shear strength of the material. Although the process of progressive failure could take some time, the term does not refer to time dependent properties of the soil (or rock). Failure caused by a decrease in the strength properties with time and associated creep movements could instead be termed delayed failures (Skempton and Hutchinson, 1969), and are of less importance for open pit rock slopes which have a limited life.

The mechanisms for progressive failure in soils were addressed by Bishop (1967, 1971). In soils, progressive failure is more prone to occur if the soils are very brittle, i.e., if there is a large difference between peak and residual shear strengths (Veder, 1981; Bromhead, 1992), as shown in Figure 3.17. Also, progressive failure is more likely if the drop from peak to residual strength is along a steep curve, i.e., little additional straining of the material to reach the residual strength (Bjerrum, 1966).

![Shear Stress vs Shear Displacement](image)

**Figure 3.17  Peak and residual shear strengths.**

In soil mechanics, the brittleness index, $I_b$, is often used to quantify this material behavior. The brittleness index is expressed as (Bishop, 1967):
\[ l_b = \frac{\tau_f - \tau_r}{\tau_f} \]  \hspace{1cm} (3.2)

where \( \tau_f \) and \( \tau_r \) are the peak and residual strengths, respectively.

The mechanism for progressive failure in slopes is that the peak shear strength is exceeded at one point in the slope, resulting in a stress redistribution due to the lower residual strength of the material. This stress redistribution causes nearby points to yield which results in further stress redistribution and so the process continues. Failure can therefore develop for slopes which would appear to be stable when considering only the peak strength, but where local failure can occur. Brittle and progressive failure behavior of soils is most common for strong overconsolidated clays, but there are also indications that it could occur in normally consolidated clays in which the difference between the peak and residual strength is smaller (Bernander, 1978, 1983; Bernander and Olofsson, 1983). A progressive failure behavior similar to that of soils, could also be envisioned for rock slopes which exhibit brittle and strain-softening behavior, but there are much fewer observations to substantiate this.

The interesting question now is where such a failure is initiated? In the early work by Collin (Skempton, 1949), it was found that the first visible signs of failure was the opening of a tension crack at the crest of the slope, in some instances followed by heaving of the slope toe. The same conclusion was drawn by Terzaghi (1944). This does not, however, prove that failure is actually initiating at the slope crest. An alternative would be that failure initiated at the toe of the slope, where the highest shear stresses are found (Veder, 1981; Záruba and Mencl, 1982). Záruba and Mencl (1982) also stated that shear failure in a soil slope can begin to develop already when the local shear stress is about 60% of the shear strength. The resulting failure can be of two types; (1) through an increase in volume due to shear strain which results in a brittle failure and a fairly thin slip surface, or (2) through a decrease in volume leading to a ductile failure and a thicker shear zone. The thicker shear zones would be more prone to develop under high stresses. The interesting phenomenon then develops in which a thick shear surface is formed in the interior of the slope, and thinner shear surfaces form at the toe and the crest of the slope (Figure 3.18). The failure surface forming at the crest will in most cases be a tension crack.

Failure initiation in clayey soils is also dependent upon the groundwater conditions. If undrained conditions prevail, failure can be expected to initiate within the slope and progress outward and toward the toe and the crest, according to Figure 3.18 (Bishop, 1967;
Recent research on clay slopes seems to confirm this behavior (Johansson, 1995). If on the other hand, the slope is drained (or the effective stress state is considered in the analysis), failure is more likely to initiate at the toe or the crest (or both) and propagate inward into the slope (Bishop, 1967; Chowdhury, 1978).

Figure 3.18 Development of thin (brittle) and thick (ductile) shear surfaces (after Záruba and Mencl, 1982).

The conditions for failure initiation at the slope crest have been discussed by Romani, et al. (1972) and Chowdhury (1978). For such a failure (under drained conditions), tension cracks will be the first sign of failure. Once tension cracks have initiated at the crest, this portion of the slope (the active zone) is free to move, and thus act to increase the load on the lower portion of the slope (the passive zone). Overall failure occurs when the loads on the middle portion of the failure surface exceed the shear strength. This will be after substantial displacements have occurred within the failing volume. This complex failure mechanism has been observed in clayey slopes in Sweden, where the clay is relatively ductile (Johansson, 1995). The failure surface can be relatively thick in these slopes, even near final collapse. Thick shear zones have also been found elsewhere in clay slopes where several slip surfaces were overlayed on each other, thus forming a thicker shear band, and when failure occurred across existing bedding structures in the clay (Bromhead, 1992).

Development of active and passive earth pressures in the slope also affect the geometry of the slip surface. Using classic earth pressure theory, the angles at which failure occurs in the active and passive zones can be calculated. The upper portion of the failure surface would thus make an angle of $45^\circ + \phi/2$ with the horizontal, and the lower portion would make an angle of $45^\circ - \phi/2$ with the horizontal, as shown in Figure 3.19 (Terzaghi, 1944; Johansson and Axelsson, 1991). It is not known whether this has actually been confirmed by field
observations; however, model tests by Ohshima et al. (1991) show that failure is more brittle in the active pressure zone than in the passive zone.

![Diagram](image)

**Figure 3.19** Failure angles calculated from classic earth pressure theory (from Johansson and Axelsson, 1991).

**Progressive Failure in Rock Slopes**

How does this then translate into failure mechanisms for rock slopes? Rock is a much more brittle material than clay which implies that the above mechanism is not entirely applicable to rock slopes. Coates (1977) stated that rock must be ductile for rotational shear to develop. Coates defined a ductile rock as having a ratio between the maximum and residual compressive strengths greater than 0.6, but it is difficult to judge the validity of such a general statement. It is true that rotational shear failure would be more likely the weaker and more ductile the rock is; however, there is no evidence that it could not develop in rocks which are of high strength and stiffness.

Müller (1966) discussed some aspects of progressive failure in rock slopes and concluded that progressive failure in rock would first involve failure along pre-existing discontinuities but that failure through the intact rock bridged between pre-existing joints would also contribute to the failure development. Larger deformations would be required to cause these rock bridges to fail; however, according to Müller (1966), this could occur progressively as the stress distribution changes due to failure along the discontinuities. Such a mechanism appear very
plausible, and the resulting failure surface would thus be composed mainly of pre-existing discontinuities but with some portions of initially intact rock. It is difficult to visualize that a kinematically possible slip volume can be bounded only by pre-existing discontinuities, which implies that some fracturing through portions of intact rock is necessary. The shape and location of the failure surface would be determined by the stresses acting on the slope, the slope geometry, and the overall rock mass strength. Failure would then occur along those discontinuities that lie in this favorable "shear band".

There is unfortunately very little in terms of field observations to verify this hypothesis. One example from underground mining is the Långsele hangingwall failure (Kolsrud and Krauland, 1979; Krauland 1975). The hangingwall of an open stope failed suddenly and the failure extended up to the ground surface. Observations from this event indicated that failure went through both intact rock and along pre-existing discontinuities. The type of failure (shear or tensile) in the portions of intact rock at the Långsele failure could not be determined. The geometry of the Långsele hangingwall failure was truly three-dimensional. In a vertical cross-section, failure appeared to follow a more or less straight line (planar failure) but in a horizontal cross-section, the failure surface was curved and shaped like a bowl. The issue of failure through intact rock has also been investigated in model tests as will be discussed in the next section.

The thickness of the developed shear band in a rock slope is also interesting to consider. From experiences and measurements at the Jeffrey mine (Chapter 6 of this report), the shear zone was up to 25 meters thick, with more concentrated shear displacements in a 4-5 meter thick zone. Experiences from the Aznalcollar mine (Chapter 6) also indicated that the shear zone has some thickness. In these two cases, the failure surface developed across the dominate joint sets, which probably also contributed to a thicker shear zone, in analogy with the existing experiences from clay slopes (Bromhead, 1992). This perhaps suggests that several discontinuities and fractures act together to form the failure surface.

The question of where failure initiates is also of importance. Although the first signs of instability in an open pit often are tension cracks at the slope crest, this does not imply that failure must initiate at this point. For a material in which a substantial part of the shear strength stems from friction (such as rocks) the failure surface will pass through the toe (Piteau and Martin, 1982). It is then more plausible that the failure progresses from this point toward the crest of the slope (Bishop, 1971). This has also been argued by Harr (1977) who concluded that for a failure going through the toe, it would be very unlikely to have a rupture initiation at any other point than at the toe of the slope. In open pit mining, where the toe of the slope is excavated continuously as the pit is being deepened, it is also more likely that
successive failures initiate at the toe. The failure surface can be almost fully developed before any tension cracks occur at the slope crest. Movements at the toe of the slope could be small due to the acting confinement. The slope near the toe could also move more or less as a rigid block in the early stages of failure, thus making it difficult to visually observe such displacements (Chowdhury, 1995).

Similar to soils, active and passive zones of pressure have also been observed in rock slopes. The cases at the Chuquicamata and Aznalcollar mines (Kvapil and Clews, 1979; Call et al., 1993, Board et al., 1996) are described in Chapter 6. In these cases, the upper section of the slope acted as an active wedge and slid downward, while the lower section of the slope acted as a passive stabilizing wedge; however, it is not known what the overall conditions were for such a failure mechanism to develop.

An important issue which remains to be resolved is what the conditions are for a curved failure surface in favor of a planar failure surface. One could imagine a step path failure along pre-existing discontinuities following a more or less straight line from the toe to the crest of the slope, as well as following a more curved failure surface (Figure 3.20).

![Figure 3.20 Planar versus curved failure surface for a large scale slope.](image-url)
A difference between a curved and a planar failure surface is that a curved failure surface bounds a large volume of rock in relation to the length of the failure surface, thus resulting in a larger ratio between the self weight of the sliding volume and the total resisting shear force of the failure surface. This was investigated by Chen (1975) who compared plane, circular and logarithmic spiral slip surfaces, and found that the most critical shape was that of a logarithmic spiral. This may not be universally applicable, but it does indicate that plane failure is less likely for homogeneous materials. For a small scale slope, plane shear failures along discontinuities are dominate failure modes. As the height of the slope increases, the significance of individual discontinuities decrease (with the exception of large scale structures). This implies that there is a "transition point" at which curved failure surfaces become more critical. This remains to be verified; however, it is important to do so, since it may have tremendous implications as regards the affected failure volume.

Another failure mechanism which could develop in rock slopes is where several failure surfaces develop on top of each other. These types of failures will be referred to as multiple failures. In open pit mining where the pit is deepened successively, such a behavior is not uncommon (Chapter 6). The mechanisms for this are not yet quantified, and thus deserve more attention in the future. The same can be said for local failures which, under certain conditions, can initiate a much larger failure.

The large scale, deep seated, toppling mechanism described in Section 3.3.2 (Figure 3.14), is one example of where one, seemingly unimportant failure mechanism causes a large scale slope failure. In addition to observations from open pits, deep seated toppling failure has also been observed in natural slopes in mountainous areas (Stini, 1941, 1942; Zischinsky, 1966; Tabor, 1971; Holmes and Jarvis, 1985; Evans, 1987; Bovis, 1982, 1990). The phenomenon is known under names such as ridge-top depression, antislope scarp,s and Talzuschub (in German), and is characterized by slowly moving (time-dependent), very large rock masses, exhibiting signs of toppling failure near the surface but with a more deep seated shear zone along which sliding occurs. A common denominator for these failures appear to be that the rock mass exhibits bedding or strong foliation. As an example of typical volumes and movement rates, an estimated $3 \times 10^7$ m$^3$ of rock moved at an rate of less than 40 mm per year in a slope at Affliction Creek in British Columbia (Bovis, 1982, 1990). This slope has been moving for an estimated 4000 years. Here, flexural toppling near the slope surface was a precursor to more deep seated sliding. Toppling was induced as lateral support provided by a glacier was removed as the glacier retreated (Figure 3.21).
Another very large toppling failure, with an estimated volume of $140 \times 10^7$ m$^3$, was observed by de Freitas and Watters (1973). In addition to the common, slowly moving, failures, de Freitas and Watters (1973) also found evidence of brittle failure for some observed toppling failures.

An interesting fact is that large scale toppling has also been observed in strong, metamorphic rocks. Furthermore, the resulting "base" failure surface has in several cases been curved. Although the initiating mechanism is quite different, the resulting failure surface can thus be very similar to that of a homogeneous continuum. There are also cases when the resulting failure surface is more or less parallel to the slope outline (Giraud et al, 1990 and Chapter 6 of this report).

Although the movement rates for this type of failure in general are very low, the failure mechanism as such could very well apply to open pit slopes. The constant removal of lateral support at the slope toe could induce toppling failure, provided of course that steeply dipping discontinuities exist in the rock mass. It clearly shows what the effect of large scale geological structures can be. Provided that such discontinuities exist, the mechanism of failure could be dramatically different than expected. The above example also shows that much more can be learned from experiences regarding natural slope failures.
3.3.4 Failure Mechanisms Indicated by Model Tests

The relative lack of good field observations of large scale slope failures in hard rock has lead to the use of physical model tests to explain failure behavior. A big advantage of model tests conducted in the laboratory is that the conditions believed to influence failure can be varied relatively easy, thereby allowing various possible failure mechanisms to be studied. The different test techniques will be described in more detail in Section 4.6.

Examples of simple model tests are the base friction tests described by Hoek and Bray (1981) and used by among others, Soto (1974), to verify the mechanism of toppling failure. Primary toppling was also studied by Barla et al. (1995) who used a tilting table frame. Some of the most interesting model tests with respect to possible failure mechanisms in large scale slopes are those carried out by Ladanyi and Archambault (1970, 1972, 1980), which were also reported and discussed by Hoek and Bray (1981) and Hoek (1983). In these tests, compressed concrete blocks were sawn to different sizes and stacked together to simulate a rock mass intersected by two joint sets. The total size of the models used varied from 0.4 to 0.5 meters in width and 0.4 to 1.0 meter in height, with a thickness of 0.10 to 0.15 meter. The concrete blocks had a uniaxial compressive strength of around 25 MPa and a Young's modulus of 11 GPa, i.e., fairly similar to rock mass of medium quality. Biaxial loading was applied to the model, and the loading direction was varied in relation to the orientation of the simulated joint sets. Two different test setups were used; one where all simulated joints were continuous (i.e., going through the entire model), and one in which the joints were discontinuous so that failure along a single shear surface (plane shear failure) could not develop. The actual composition of a large scale rock mass is probably somewhere between these two extremes of joint configurations.

In the test with continuous joints, three distinctly different types of failure were observed, depending upon joint orientation in relation to the loading direction. These were:

1. Shear failure along a well defined failure surface which was inclined to both joint sets. Failure was thus not along one single joint set.
2. Formation of a narrow failure zone in which block rotation and sliding occurred simultaneously.
3. Formation of a "kink band" of rotated and separated columns of blocks, each column consisting of three to five individual blocks.
It is interesting to note that plane shear failure was not predominant in these tests. The actual mechanisms of failure were instead much more complex. A large degree of interlocking between the individual blocks added to this behavior.

In the test with discontinuous joints, two types of failures were observed. The first type was shear failure along a well defined failure surface, and the second type of formation was of a shear zone, much like the first and the second failure types for the continuous joints. The big difference, however, was that shear failure did not occur along the interfaces of the concrete bricks, but instead as shear failure through the intact material. The failure was progressive in the sense that the intact blocks were sheared off one after another as the failure plane developed. The conclusion drawn by Ladanyi and Archambault (1980) was that cohesion and friction of the rock mass are unlikely to be mobilized simultaneously.

The formation of "kink bands" was investigated in more detail in a later study (Archambault and Ladanyi, 1993). It was found that kink band formation reduced the strength of the rock mass considerably. Strength values lower than the shear strength of a polished discontinuity surface were obtained. A conclusion drawn by the authors was the kink band formation could be a very dangerous mode of failure. Even when interlocking was observed in the kink zone, movement inertia stored in the upper part of a slope can be sufficient to cause a rapid failure (Archambault and Ladanyi 1993).

Somewhat similar tests were conducted by Brown and Trollope (1970a, 1970b), but using plaster as model material. The observed failure modes were of a wide variety, ranging from simple plane shear failure to shearing through intact blocks. However, relatively few blocks were used in each model, which makes these tests less applicable for predicting failure modes in a slope of large dimensions. Einstein et al. (1970) carried out model tests using a mix of gypsum plaster, water and celite. The intention was to simulate a brittle rock of relatively high strength, such as granite and quartzite. The uniaxial compressive strength of the model material was around 25 MPa, and the Young's modulus was 11 GPa, i.e., similar to the material used by Ladanyi and Archambault (1970, 1972, 1980). The size of the model samples was 5 by 10 by 20 centimeters. Joint configurations ranged from a single joint at different angles to two perpendicular joint sets with various spacings. The samples were loaded under triaxial stress conditions but with the intermediate principal stress equal to the minor principal stress. An interesting conclusion which could be drawn from these tests was that the confining stress strongly affected the failure mechanisms in the samples. For low confining stress (less than 10 MPa) failure occurred along the pre-existing joints, but for higher confining stress, failure occurred mainly through the intact material. The test results also indicated that
although failure occurred through the intact material, the overall strength of the jointed samples was lower than the intact strength of the model material.

A set of model tests which were not primarily intended for slope design were those conducted by Horii and Nemat-Nasser (1985, 1986). The results from these tests showed, however, some interesting aspects of the fracturing process through intact materials. A thin (6 mm) sample of relatively brittle resin was used, in which a number of "flaws", or "fracture seeds", were introduced by sawing thin (0.4 mm) slits into the sample. The samples were loaded in uniaxial compression, whereby the fracture propagation, initiating at these flaws, could be observed. Cracks first propagated from the larger flaws in the sample, but the final failure surface developed from the flaws which were oriented most favorably in relation to the applied load and the stress state in the sample. Although these results cannot be directly translated to rock slopes, it is interesting to note that the failure surface was governed by the loading conditions, i.e., the stresses acting in the sample, and not by the length and orientation of the fracture seeds. It could be envisioned that failure in a large scale rock slope displays a similar behavior, in which the curved overall failure surface develops as a function of the applied loads and the slope geometry, and where the joints which are most favorably oriented in this failure zone combine to form the actual failure surface.

In neither of the tests discussed above, was the actual geometry of an open pit slope simulated. An interesting model test of a real slope geometry, including both the hangingwall and the footwall of an open pit, was conducted by Barton (1971, 1972, 1974). In these tests, a specially developed model material was used to simulate down-scaled rock mass properties. A mix of sand, ballotini (small glass spheres), red lead, plaster, and water, was used to simulate a low strength brittle material, which could be split into uniformly oriented fractures. In Barton's model, approximately 40 000 discrete blocks were used, simulating three different joint sets. Only one of these was a continuous joint set. The model scale was 1:500 and the final two-dimensional model measured 2.4 by 1.2 meters, with a thickness of about 2.5 cm. The model was gravitationally loaded by its own weight and laterally loaded by stiff springs, which permitted stress relaxation in the model as the slope was excavated.

The results from these tests showed that failure occurred only along pre-existing discontinuities. The failure surface was, in most cases, very steep. The development of the failure surface initiated at the toe with small shear movements along the plane which eventually became the overall slide surface. The cumulative effect of these shear movements resulted in the formation of a tension crack at the crest. The separated material initially moved as an intact block, but then started to break up from the toe. Lateral stresses in the model were relieved by opening of the joints. The recorded displacements were also independent of
the magnitude of the horizontal stresses, and little evidence of overstressing of the toe was found (Barton, 1974; Hoek, 1971b). This indicates that the horizontal stresses are less important in jointed rock slopes of this kind. More importantly is perhaps that these tests seem to verify the hypothesis that slope failure must initiate at the toe.

In soil mechanics, centrifugal tests (Schofield, 1988) have been used to model slope stability, see e.g., Ohshima et al. (1991). In these tests, centrifuges are used to increase the body forces in a model, thus enabling smaller models with stronger materials to be tested (see also Section 4.6). Centrifugal tests have also been used to model rock slopes. Goldstein et al. (1966) used a plaster-sand mix and modeled joints using oiled sheets of paper. Their results showed that the model slope could fail in (1) plane shear, (2) rotational shear, or (3) a combination of both. Stacey (1973) performed both two- and three-dimensional tests. The model material was a mixture of water, silica sand, and high alumina cement, simulating the behavior of a typical hard rock. Several different joint configurations were tested, and the results showed that failure occurred as sliding along pre-existing joints. Failure through the intact model material was not observed in the tests by Stacey (1973). The tests also revealed that the failure mechanism was progressive and coherent sliding of one large block could not be observed. Instead, displacements were recorded throughout the slope. The three-dimensional tests gave significantly different results as compared with the two-dimensional tests, implying that the three-dimensional aspects of failure deserve more attention.

To summarize, there appears to be some difference in the results from different model tests. The tests by Barton (1971, 1972, 1974) and Stacey (1973) showed that failure only occurred along pre-existing discontinuities in the model slope, while the tests of Ladanyi and Archambault (1970, 1972, 1980) and Einstein et al. (1970) showed that failure could occur through the intact material as well. These differences can probably to some extent be explained by the differences in model material and loading conditions. To compare the different model tests, one can calculate the ratio of the uniaxial compressive strength of the model material and the applied vertical stress in the model. This should provide a rough estimate of the potential for failure of the intact material (compare Section 3.3.1). The resulting stress/strength-ratios are summarized in Table 3.1.

A strict comparison among the different tests is not possible, since the loading conditions vary from gravitational loading alone to biaxial and triaxial loading. Furthermore, the reported vertical stress is only estimated for the tests by Barton (1971, 1972, 1974) and Stacey (1973), but is actually measured in the tests by Ladanyi and Archambault (1970, 1972, 1980) and Einstein et al. (1970). Nevertheless, Table 3.1 shows that it is not surprising that failure through the intact material was found in the tests by Ladanyi and Archambault (1980) and
Einstein et al. (1970) since the stress-strength-ratio is relatively high for these tests compared
with the other tests. Table 3.1 also illustrates the basic problem with all model tests. Despite
very carefully engineered model materials in each test, large differences in test results between
different test setups are observed, and these differences are not always easily explained. It is
therefore difficult to determine which tests best replicate actual slope behavior.

Table 3.1  Comparison of different model tests.

<table>
<thead>
<tr>
<th>Material</th>
<th>( \sigma_c ) [MPa]</th>
<th>( \sigma_v ) [MPa]</th>
<th>( \sigma_v/\sigma_c )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete bricks (Ladanyi and Archambault, 1970, 1972, 1980)</td>
<td>25</td>
<td>3.8 - 22</td>
<td>0.15 - 0.88</td>
</tr>
<tr>
<td>Plaster mix (Einstein et al., 1970)</td>
<td>25</td>
<td>31 - 43</td>
<td>1.24 - 1.72</td>
</tr>
<tr>
<td>Plaster mix (Barton, 1971, 1972, 1974)</td>
<td>0.14</td>
<td>0.023</td>
<td>0.16</td>
</tr>
<tr>
<td>Sand and cement (Stacey, 1973)</td>
<td>3.9</td>
<td>0.8</td>
<td>0.20</td>
</tr>
<tr>
<td>Actual rock in a 500 m high slope (see Section 3.3.1)</td>
<td>20</td>
<td>13.5</td>
<td>0.67</td>
</tr>
</tbody>
</table>

\( \sigma_c \) = average uniaxial compressive strength of model material
\( \sigma_v \) = applied vertical stress in the model, or virgin vertical stress at the slope toe

Despite the obstacles associated with physical model tests, the reported tests indicate some of
the possible mechanisms at work in large scale slopes. It is for instance, very likely that slope
failures initiate at the toe of the slope. Furthermore, the failure surface is probably not a single
shear surface but rather is composed of several discontinuities linking together. Block rotation
is also involved in the failure. Failure of the intact bridges between pre-existing joints is also
possible under the right loading conditions. However, these general conclusions and indices
need to be verified through field observations of slope behavior and a better link between
observed and modeled failure mechanisms must be established.

3.3.5  Failure Mechanisms From Numerical Analysis

Numerical analysis has in recent time, to a large extent replaced physical model tests for
parameter- and sensitivity studies of slope behavior. Today, methods exist which can simulate
both continuum (intact rock) and discontinuum (pre-existing discontinuities included)
behavior. Fracturing and the formation of discontinuities in a material is more difficult to simulate though, in particular when modeling an actual slope geometry. This to some extent limits the applicability of numerical models but they are still considered useful in that they can, when properly used, increase the understanding of the fundamental failure mechanisms. A detailed description of existing numerical methods will be given in Section 4.4.

The issue of large scale (or deep seated) toppling has been investigated using numerical modeling. Examples of this can be found in Pritchard and Savigny (1990, 1991), Martin (1990), Orr, Swindells and Windsor (1991), Coulthard et al. (1992), and Martin and Mehr (1993). Common for these cases is that an observed slope behavior (in an open pit or a natural slope) is being replicated using a numerical model. Field data, such as the orientations of pre-existing discontinuities, have been used as input to the models. It is thus not surprising that the modeling results, in general, confirmed the observed behavior.

Despite this, some interesting results have indeed been reported. As an example, consider the study by Pritchard and Savigny (1991). Different parameters such as joint spacing, joint strength and intact rock strength were varied in the model until good agreement was found with the observed failure geometry (a large landslide in a natural slope). Despite the need for calibration of the model, the results confirmed some hypotheses not explicitly observed in the field. It was, for example, shown that a deep seated, curved, failure surface could in fact develop as a result of toppling along steeply dipping joints. Curved deep seated failure surfaces as a result of toppling were also reported by Orr, Swindells and Windsor (1991). From these studies, it can be concluded that it appears that large scale toppling is a feasible failure mechanism in large scale slopes with extensive, steeply dipping joints. This is supported both by observations and by analysis of actual cases, mostly natural slopes.

### 3.4 Kinetics of Rock Slopes

The formation of a failure surface in a slope does not automatically mean that the pit slope is impossible to mine. As was discussed in Section 2.3, a slope collapse is defined as a failure resulting in loss of production from the mine. To avoid a slope collapse, one of two conditions must be satisfied:

i. The consequences of failure are so small that they do not interfere with the production,

ii. The kinetic behavior of the slope can be predicted with some precision, thus enabling remedial measures to be taken.
The first condition concerns the failure mechanisms and has been discussed previously. In this section the kinetic and kinematic behavior of slopes and how this can be predicted, will be discussed. The implications of being able to determine the type of failure and predict the failure behavior are enormous. Mining could continue despite large scale displacements if the failure is stable, the failure mechanism well-defined, and monitoring is performed continuously (Call et al., 1993). Most open pit mines have some operational flexibility, at least in the early stages of mining, and this can be used fully provided that the above conditions are satisfied.

This recognition has lead to the introduction of comprehensive monitoring programs in most large scale open pits. Probably the most common system is the monitoring of displacements through electronic distance measurement (EDM) techniques (see also Section 5.2). In addition to displacement magnitudes, other parameters such as displacement rates, acceleration or strain which can be more sensitive to changes in slope behavior are measurable (Voight et al., 1992; Call et al., 1993). Slope monitoring is especially important for potential large scale failures since these have such huge economic and safety impact. These efforts have also increased the ability to detect slope failures at early stages. A classical example of how long-term displacement monitoring was used to reduce the risk from a slope collapse and limit interference with production is the relatively large scale failure at Chuquicamata in 1969 (Kennedy and Niermeyer, 1970). Alternative mining strategies and operational procedures will be discussed in more detail in Section 5.3 of this report.

An attempt to describe the kinetic characteristics of large slope failures based on displacement measurements was presented by Zavodni and Broadbent (1978). They concluded that almost all large scale failures occurred gradually, with the exception of slides initiated by seismic loading (earthquakes). Serious slope instabilities were almost always accompanied by the gradual development of tension cracks behind the slope crest and measurable displacements. As the failure continued to develop, increasing displacement could be recorded until the point of slope collapse. Based on empirical data from several open pit mines, Zavodni and Broadbent defined two failure stages for large scale failures; a "regressive" stage and a "progressive" stage. The unfortunate choice of the term "progressive" may lead to some confusion since this also is used to describe the development of failure surfaces in the slope. A better definition of these two phenomena would be stable and unstable failure (Figure 3.22). These definitions will be used throughout this report.

Stable failure is defined as a failure which has started to exhibit displacement, but where the displacement rate is almost constant and relatively low. An unstable failure, on the other hand, exhibits an increasing displacement rate eventually leading to a slope collapse. A stable failure will exhibit a decreasing displacement rate if external disturbances are removed. "Low" and
"high" displacement rates are subjective terms and must be judged individually for a specific slope. Here, it is important to distinguish between stable failure and creep behavior. Creep failure implies increasing displacement without any change in the loading conditions.

![Diagram](image)

**Figure 3.22** Typical record of displacement rate versus time for large scale slope failure (after Zavodni and Broadbent, 1978).

The displacement records reported by Zavodni and Broadbent (1978), appeared to be of exponential form for both failure stages, thus forming straight lines in a logarithmic-linear diagram of the displacement rate versus time. Separating the stable and unstable failure stages is an obvious "knee" in the curve, which defines as the "onset of failure point" (Figure 3.22). The term "onset of failure" does not refer to the initiation of failure in the rock mass, since this has occurred long before the transition between stable and unstable failure. The term *transition point* will be used in the following rather than the term "onset of failure". Zavodni and Broadbent (1978) also suggested an empirical relation for predicting the time of total collapse of a slope. They stated that if the transition point could be determined together with a mine-specific constant, the time of slope collapse could be predicted. Comparisons with actual field data showed that this relatively simple formula yielded results in close agreement with actual failure occurrences.

The relevance of these results must, however, be questioned. There are uncertainties regarding where measurements were taken for the different slopes. Movements cannot be expected to be uniform throughout the slope. Heaving of a zone close to the toe of the slope is common, which could seriously affect the conclusions drawn in the above studies and the
derived empirical formulas. Furthermore, the measured displacements in these cases were caused by changes in the external forces on the slope, and are thus not strictly time-dependent. External load changes include not only changes of the slope geometry, but also changes in groundwater pressure as a result of rainfall or spring melt-off of snow. Increased groundwater pressure causes a reduction in the effective normal stress, thus decreasing the strength of, for example, a pre-existing discontinuity (compare Section 3.1.2). It is thus difficult to interpret Figure 3.22, since it shows slope behavior under changing loading conditions and strengths, and for different slope geometries.

The ideas introduced by Zavodni and Broadbent (1978, 1982) are still interesting from a conceptual perspective. It seems likely that there could be a point in time after which slope failures cannot be prevented regardless of what remedial measures are taken. The terms stable ("regressive") and unstable ("progressive") failure have been expanded to define the entire failure process of large scale slopes (Ryan and Call, 1992; Call et al., 1993). An intermediate condition defined as steady state was also introduced. In a steady state condition, the slope displays neither unstable nor stable failure behavior but is still moving at a constant rate. This would probably correspond to the region around the transition point in Figure 3.22.

Zavodni and Broadbent (1978), Broadbent and Zavodni (1982), and Ryan and Call (1992) all noticed that displacements often occurred in well defined cycles of varying duration (Figure 3.23). These cycles could be attributed to some external disturbance to the system. This included, as mentioned above, heavy rainfall, spring melt-off and excavation of the slope. Production blasting could potentially trigger small scale failures which then could propagate and result in larger failures. For a stable failure, these disturbances lead to a temporary increase in velocity which is then followed by a re-stabilization of the slope. However, these events may also trigger unstable slope failure thus ultimately leading to a slope collapse. It appears to be more difficult to quantitatively predict the duration and the occurrence of the displacement cycles than to describe and predict the long-term behavior of the slope.

The displacement magnitudes varied within a wide range for the above case studies. Total displacements varied from as little as 70 mm close to the point of total collapse of an unstable failure, to 2.4 meters for a slope exhibiting stable failure. A better indicator of incipient failure was the displacement velocity, which exhibited smaller variations. The critical velocity, i.e., the velocity defining the transition point, was on average 12 mm/day (Ryan and Call, 1992). The empirical data also suggested that when the velocity was 50 mm/day there was at least a time span of 48 hours before slope collapse. It is, however, not likely that these magnitudes of displacements and velocities are globally applicable. This is verified by the experience from the Aznalcollar Mine in Spain (see Chapter 6 of this report) where movement rates have
exceeded 1600 mm/day and total slope displacements are of the order of 10 meters. Even with these extremely high movement rates, the pit slope is still in place and the mine is fully operational. Guidelines such as those proposed by Ryan and Call (1992) can only be used for very crude estimates and when more site-specific data are unavailable.

![Diagram of displacement cycles](after Broadbent and Ko, 1972)

![Diagram of velocity cycles](after Ryan and Call, 1992)

Figure 3.23 Slope displacement cycles (after Broadbent and Ko, 1972; Ryan and Call, 1992).

An attempt to model the apparent cyclic behavior of slope displacements was made by Broadbent and Ko (1972). They developed a simple rheological model based on empirical data. The rheological model consisted of spring connected in parallel with a dashpot (often referred to as a Kelvin or Voigt model). Surprisingly good results were achieved for predicting cyclic slope displacement using this simple model. However, displacement data must be available to calibrate the model for a specific mine. Broadbent and Ko (1972) suggested that this could be done for the first cycles of a failure and then used for future predictions. The question of predicting failure time was also discussed by Voight et al. (1989). They proposed a general relation for failure-time prediction using a differential equation, which can be applied to, for example, measured displacements and slope stability but also to other types of failures and measured parameters. Estimation of the two constants included in this relation could be done by back-analysis or by evaluating displacement records from occurring failures. As Voight et al. (1989) pointed out, constants determined in this manner apply strictly only to the time intervals observed. They may not necessarily be the same for other time periods since external conditions may change. Slope collapse prediction
from a stable failure could thus be difficult. Voight et al. (1992) also agreed to the fact that short-term predictions of displacement cycles could prove more difficult than prediction of the gross movement of the slope.

Although it is not clear whether all slope failures of large scale have identifiable stages in displacement-time records, it is interesting to note that the kinetic failure characteristics appear to be principally the same for a large variety of mines and geomechanical environments (Ryan and Call, 1993). Failure modes were also different for these cases, ranging from structurally controlled shear and wedge failures to rotational shear failures. It is still very premature, however, to state that this could also apply to other mines.

The inherent assumption so far has been that slope movements can be detected in advance for all failure types. Failures could, however, develop so rapidly that there are virtually no warning signs in terms of displacements prior to a total collapse of the slope. A failure exhibiting no, or very little, warning before a slope collapse is defined as an uncontrollable failure. These uncontrollable failures are extremely dangerous for open pit mining situations since even if there is operational flexibility, there is no time to adjust mining, or even evacuate the pit. There are only a few examples of uncontrollable failures, among which can be mentioned the 1990 failure at the Brenda Mine in Canada (see Chapter 6) which occurred very rapidly and without prior warning. Once the rock mass was moving, the velocity of the failure was estimated as 175 km/h (Sharp, 1995). Another example of a rapid failure was in a Brazilian iron ore mine. A 240 meter high pit slope failed and the debris moved 500 meters across the pit floor to the opposite pit wall. Failure was along a weak bedding plane dipping parallel to the slope and only 15 cm of movement was detected before the uncontrollable failure (Stacey, 1993).

Extremely high failure speeds have also been recorded for rockslides, or rock avalanches, in natural slopes, for example, in the Andes in South America where velocities in excess of 300 km/h were recorded (Voight, 1978). Many of these rockslides were, however, triggered by earth-quakes and seismic loading. The displacement history up to the point of collapse is poorly quantified for most of these natural rockslides. What can be inferred from these slides is that failure can be very violent under certain conditions, but that these conditions are not always easily quantified. While it may be difficult to apply the experiences from these phenomena to open pit rock slopes, this is an area which deserves more attention since the database of case studies is relatively large. Another example of an uncontrollable failure, but this time in an underground mine, is the hangingwall failure of the Långsele mine in northern Sweden (Kolsrud and Krauland, 1979; Krauland, 1975). Although this failure occurred underground, it bears many similarities to large scale failures in slopes, since the failure
daylighted the ground surface. Very little warning in the form of cracking and noticeable displacements were recorded before failure, and the failure development was very rapid.

When comparing open pits which have and have not experienced uncontrollable failures, it appears that rapid and uncontrollable failures are more prominent in (1) steep rock slopes, and (2) slopes in brittle and high strength rock masses. The difference between peak and residual strengths appears to be a key factor. A large difference means that there is more energy which can be released in the event of failure. This is confirmed by the current knowledge in soil mechanics, where it is known that higher brittleness (compare the brittleness ratio in Equation 3.2) of a material results in both larger displacements and higher displacement rates. A steep stress-strain curve in the post-failure stage also induces more rapid displacements (Bromhead, 1992). Skempton and Hutchinson (1969) also noted that if the soil exhibits a very small difference between the peak and the residual strengths, there will be no tendency for an unstable failure behavior. The same behavior could be expected for rock masses, as stated by Hoek and Pentz (1969). It has not yet, however, been possible to propose a quantitative relation describing this expected behavior even for soils, not to say for rock masses.

To conclude, it appears that the current knowledge of the kinetic behavior of rock slopes, up to, and after, total collapse is mostly empirical. In open pit mining, it is crucial that the failure stages of a slope be determined as quantitatively as possible. In particular, this applies when setting out to design failing slopes. This requires methods whereby the deformation characteristics can be assessed, which may limit the choice of an appropriate design method. The proposed criteria for failure prediction and displacement behavior discussed earlier in this section are along the right lines, but they need to be more firmly based, which implies a better "connection" with the governing failure mechanism.

3.5 Rock Mass Strength

3.5.1 Sensitivity and Scale

The strength of a large scale rock mass will ultimately determine whether slope failure will occur or not. It is thus of utmost importance to be able to quantify the rock mass strength for design purposes. To illustrate the problem consider a slope in a homogeneous and isotropic rock mass, and assume that failure occurs as rotational shear failure. Let the strength of the rock mass be characterized by a friction angle, $\phi$, and a cohesion, $c$, representative for the composite rock mass as a whole, i.e., both intact rock and discontinuities. For this case, how sensitive is the calculated slope stability to changes in the strength properties? Assuming a
friction angle of 30° and a density for the rock mass of 2700 kg/m³, the required cohesion for maintaining the stability of the slope was calculated using the charts for circular failure given by Hoek and Bray (1981). (These and other design methods for rotational shear failure will be described in more detail in Section 4.3.5). The calculated cohesions are shown in Figure 3.24, for slope heights of 100 to 500 meters. The slope was also assumed to be fully drained (no groundwater pressure).

![Figure 3.24](image)

**Figure 3.24** Required cohesion for the stability of slopes of various heights in a fully drained rock mass with a friction angle of 30°, and subject to rotational shear failure.

One finds that the required cohesion increases strongly with increasing slope height (or pit depth). Furthermore, the cohesions required to maintain slope stability vary markedly with slope angles. For a 500 meter high slope, an increase in cohesion of only 0.3 MPa corresponds to an increase in stable overall slope angle from 40° to 50°. Small changes in the strength parameters thus correspond to relatively large geometrical changes of the slope geometry which has a large economic impact. Consequently, the accuracy by which the strength properties need to be determined for the design of large scale slopes is thus very high. This effect is even larger for a slope with high groundwater pressure (fully saturated), as shown in Figure 3.25.
The strength of the rock mass depends upon the strength of (1) the intact rock, and (2) the discontinuities present in the rock mass. Since the discontinuities in general have a much lower strength than the intact rock, a relatively low percentage of intact rock along the failure surface increases the composite rock mass strength dramatically. Nilsen (1979), showed that an increase from 0% to 10% of intact rock along the failure surface resulted in an increase of the safety factor from around 0.3 to 1.7, i.e., more than five times. It follows that a difference in the percentage of intact rock by only one or two percents may be decisive for the stability of the slope. However, in practice it is almost impossible to determine the ratio of intact rock along a failure surface with this accuracy.

To illustrate the effect of scale, take the 500 meter high slope as an example. The required cohesions for slope angles of 40° and 50° are 0.3 and 0.6 MPa, respectively. From the friction angle and the cohesion, the corresponding uniaxial compressive strength can be calculated as:

\[
\sigma_c = \frac{2 \cdot c \cdot \cos \phi}{1 - \sin \phi} \tag{3.3}
\]
This yields a rock mass compressive strength of 1.0 to 2.1 MPa. For a hard rock, the intact uniaxial compressive strength determined from laboratory tests can be around 200 MPa. The required strengths for a 500 meter high rock slope thus represent a reduction of the intact rock strength by a factor of 100 to 200. The effect of increasing scale on the rock mass strength is illustrated by the curve in Figure 3.26. The decrease in strength with increasing volume is primarily due the increased number of pre-existing discontinuities that are included into the rock mass, from small scale joints to larger faults, see e.g., Krauland, Söder and Agmalm (1986) and Pinto da Cunha (1990, 1993a, 1993b).

Figure 3.26 Schematic illustration of the relation between strength and volume of a rock mass.

Measured and back-calculated strengths from several Swedish hard rock mines are summarized in Table 3.2. Strength values for hangingwalls are exclusively from hangingwall failures in sublevel caving mines (see also Section 3.5.4). The relative decrease in strength becomes less prominent for large volumes. However, even on a very large scale it is not certain that the rock mass strength tends toward a constant value. This is obvious from Table 3.2, where even for volumes larger than $15 \cdot 10^6$ m$^3$, the scale factors vary from 37 to 220.

The scale effect according to Figure 3.26 is prominent in many materials (Weibull, 1939a, 1939b). There is, however, evidence that the strength may become constant for very large volumes. From studies on coal pillars, Bieniawski (1968) concluded that for samples larger than a certain volume the strength remain constant. Samples smaller than the average joint spacing also exhibited constant strength. Moreover, the scatter in strength decreased with
increasing pillar size in the tests by Bieniawski (1968). The same behavior could potentially be expected also for large scale slopes, but this remains to be verified.

The volume above which scale-free property values can be obtained, is commonly referred to as the Representative Elementary Volume (REV), (see e.g. Pinto da Cunha, 1993a). The REV is the smallest volume for which there is equivalence between the real rock mass and an ideal continuum material. However, this is under the assumption that the rock mass on a large scale actually behaves as a continuum. The REV could be different for different rock masses though, and different for different properties. Although much has been written on REV, the focus has been toward theoretical and laboratory studies of relatively small scale samples. Practical applications to large scale slopes are yet to be found.

Table 3.2  Large scale rock mass strength and scale factor for hard rocks. Values obtained through stress measurements (pillars) and back-analysis of failures (stope roofs, sill pillars and hangingwalls) from Swedish mines (from Krauland, Söder and Agmalm, 1986; Herdocia, 1991; Sjöberg et al., 1996).

<table>
<thead>
<tr>
<th>Construction element</th>
<th>Rock type</th>
<th>Uniaxial compressive strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Laboratory [MPa]</td>
<td>Large scale [MPa]</td>
</tr>
<tr>
<td>Pillar</td>
<td>Sandstone</td>
<td>180</td>
</tr>
<tr>
<td>Pillar</td>
<td>Sandstone</td>
<td>160</td>
</tr>
<tr>
<td>Pillar</td>
<td>Sandstone</td>
<td>340</td>
</tr>
<tr>
<td>Pillar</td>
<td>Sulfide ore</td>
<td>210</td>
</tr>
<tr>
<td>Pillar</td>
<td>Keratophyre</td>
<td>130</td>
</tr>
<tr>
<td>Stope roof</td>
<td>Sulfide ore</td>
<td>250</td>
</tr>
<tr>
<td>Sill pillar</td>
<td>Sulfide ore</td>
<td>220</td>
</tr>
<tr>
<td>Sill pillar</td>
<td>Sulfide ore</td>
<td>260</td>
</tr>
<tr>
<td>Sill pillar</td>
<td>Sulfide ore</td>
<td>225</td>
</tr>
<tr>
<td>Hangingwall</td>
<td>Quartzite</td>
<td>180</td>
</tr>
<tr>
<td>Hangingwall</td>
<td>Leptite</td>
<td>100</td>
</tr>
<tr>
<td>Hangingwall</td>
<td>Leptite</td>
<td>108</td>
</tr>
<tr>
<td>Hangingwall</td>
<td>Quartz porphyry</td>
<td>240</td>
</tr>
<tr>
<td>Hangingwall</td>
<td>Granite</td>
<td>186</td>
</tr>
</tbody>
</table>
Taken together, these findings have some very practical significance for open pit slopes. One cannot, for instance, safely extrapolate derived strengths from bench or interramp slope failures to the design of overall pit slope angles. Furthermore, it can be questioned as to whether the required high accuracy in rock mass strength determination for the design of high slopes really can be achieved? If, however, the scatter in strength actually decreases with increasing scale, the possibility arises of extrapolating strength values from one slope to another, provided that the failure mechanisms and the geomechanical conditions are similar.

It is necessary to first consider the different factors contributing to the rock mass strength. The strength of intact rock and pre-existing discontinuities is briefly reviewed in the following sections. Attempts to describe the rock mass strength are then presented, and finally, approaches for determining the rock mass strength for large scale rock slopes are discussed.

### 3.5.2 Strength of Intact Rock and Discontinuities

The strength of the rock mass is obviously a function of the strength of both the discontinuities and the rock bridges separating the discontinuities. The strength depends on the mechanism of failure, which in turn depends on the stress state in the slope (Figure 3.3). Tensile failure is possible when tensile stresses exist. This is relatively uncomplicated although determination of the tensile strength can be difficult. However, the tensile strength for a large scale rock mass is very small and can in most cases be assumed to be zero. Pure uniaxial compressive failure (no confining stress) is relatively uncommon and deserves less attention in slope applications. More important are different types of shear failure, where a significant normal stress acts on the rock mass. Most failure modes are believed to involve some shear failure, in particular along discontinuities.

For a completely planar discontinuity, the shear strength is normally a linear function of the normal stress acting on the discontinuity. The Coulomb shear strength criteria states that:

\[
\tau = c + \sigma_n \tan \phi
\]

(3.4)

where \(\tau\) is the (peak) shear strength, \(\sigma_n\) the effective normal stress, and \(c\) and \(\phi\) the cohesion and friction angle of the discontinuity, respectively.

The Coulomb slip criterion according to Equation 3.4 is a very simplified representation of the physical processes that take place during shearing of a discontinuity. True cohesion normally
only exists for discontinuities with infillings. As will be shown later, the cohesion could also represent fracturing through intact rock. The frictional resistance is highly dependent on the normal stress. There is, however, no strong dependence of friction on rock type or lithology (Byerlee, 1978). Instead, the frictional component is mostly a function of the surface geometry of the discontinuity.

The factors believed to contribute to the shear strength of a discontinuity are as follows (Hencher, 1995), (Figure 3.27):

1. Adhesion bonding
2. Interlocking of surface asperities and ploughing through asperities
3. Overriding of surface asperities
4. Shearing of rock bridges and locked asperities

![Diagram](image)

**Figure 3.27** Factors contributing to the shear strength of discontinuities (top) and definition of inclination angle of asperities (bottom). After Hencher (1995) and Patton (1966).

Adhesion bonding (chemical bonding) is significant for metals but in general quite small for rocks. Interlocking of minor asperities and damage to these add to the basic friction angle for the discontinuity. These minor asperities were referred to as second order projections by Patton (1966). The third factor, overriding of asperities, occur on a larger scale of the discontinuity surface. These asperities were termed first order projections by Patton (1966). Overriding of the asperities adds significant dilation to the shear behavior. Patton (1966) formulated a shear strength criterion to account for this effect as follows:
where \( \tau_f \) is the peak shear strength, \( i \) is the inclination (angle) of the surface asperities (Figure 3.27), and \( \phi \) is the friction angle for a flat (planar) surface. This simple extension of the Coulomb slip criterion can explain several of the effects which have been observed in shear tests of natural rock discontinuities. The resulting shear stress-normal stress curve from such tests is often non-linear, reflecting changes in the failure mechanism as the normal stress increases. Second order asperities have more influence on the shear behavior at low normal stress. As the normal stress increases, smaller asperities are sheared off and first order asperities become more important. At even higher normal stress, also these asperities will be sheared off and eventually failure of the intact rock material (or the rock bridges between the joints) will occur. For high normal stress, Patton (1966) suggested the use of the Coulomb slip criterion with an residual (and lower) angle of friction and an apparent cohesion. The complete Patton criterion is thus bilinear.

The definition of first and second order projections is of course a function of scale. What may seem like a major asperity on laboratory samples can be an insignificant undulation on, for example, bench scale. The same basic mechanism of shear failures are, however, believed to exist even for large faults (Scholz, 1990) but the transfer from one physical scale to another is not well understood.

In addition to Patton's failure criterion several other shear strength criteria have been formulated. Jaeger (1971) proposed a power law criteria which better agreed with the curved failure envelope observed from shear tests but this did not explain the mechanisms of the shearing process. Ladanyi and Archambault (1970, 1972) proposed a criterion which accounted for the dilation rate and the ratio of the actual shear area to the complete sample area. Barton (1976), Barton and Bandis (1990), and Bandis (1992) developed an empirical shear failure criterion which included terms for the roughness (asperities) of the discontinuity surface and the compressive strength of the wall rock. The Barton shear strength criterion is as follows:

\[
\tau_f = \sigma_n \tan \left( JRC \log_{10} \left( \frac{JCS}{\sigma_n} \right) + \phi_b \right)
\]  

(3.6)

where \( JRC \) is the Joint Roughness Coefficient, \( JCS \) the Joint Wall Compressive Strength and \( \phi_b \) the basic friction angle for the discontinuity. The basic friction angle corresponds to the friction angle of a flat, unweathered rock surface.
Application of the Barton failure criterion is not always straightforward, in particular the determination of the \( JRC \) value is difficult. \( JRC \) can by back-calculated from tilt tests of the actual discontinuity (Barton and Choubey, 1977). However, if samples of the actual joint are not available, \( JRC \) must be determined by visual comparison with typical joint profiles (Barton and Choubey, 1977). It is notoriously difficult to judge how representative the standard profiles of joint surfaces are. However, determining the value for the inclination of the asperities, \( i \), in Patton's criterion does not come any easier. Quantifying the roughness of a discontinuity surface thus remains a major problem in assessing the shear strength (Lindfors, 1996).

An appealing feature of the Barton criterion is the scale effect. An empirical formula which adjusted the \( JRC \) and \( JCS \) values with increasing size has been derived (Bandis, 1992). Both \( JRC \) and \( JCS \) decrease as the physical dimensions increase which means that the effects of overriding and failure of the asperities decrease with increasing scale. The basic friction angle is, however, believed to remain unchanged as the scale increases. Practical application and verification of these scaling laws is still lacking.

As was mentioned above, for a discontinuity with no joint infilling the true cohesion is normally zero. True cohesion is an effect of either the presence of joint filling such as clay gouge, or fracturing through intact rock bridges and locked asperities on the discontinuity surface (Hencher, 1995). Failure of the rock bridges between discontinuities have been studied much less than the shear behavior of the discontinuities themselves. There seems to be a general consensus that rock bridges between discontinuities primarily fail in Mode I, i.e., tensile failure (Einstein et al., 1983; Einstein, 1993; Shen, 1993). Mode II (shear) fracturing would occur as a secondary phenomenon thus forming shear fractures eventually linking separate discontinuities together (see also Figure 4.6). Model tests and numerical modeling seem to confirm this belief (Savilähti, Nordlund and Stephansson, 1990). Strength criteria for these failure mechanisms are not completely developed and deserves more attention in the future.

### 3.5.3 Strength of Jointed Rock Masses

For a rock mass, not only the presence of discontinuities but also their orientation in relation to the loading direction, contributes to the overall strength. Consider a uniaxially loaded rock sample which contains a weak discontinuity, as shown in Figure 3.28. Depending on the inclination of the discontinuity relative to the principal loading direction, the rock sample will exhibit different strength (Figure 3.28). The strength of the sample is much lower for those
cases when slip on the discontinuity is possible. This has also been confirmed by laboratory tests. For multiple inclined discontinuities, several curves of the type shown in Figure 3.28 are superimposed on each other, thus further reducing the areas where the intact strength governs the total strength of the sample (Hoek and Brown, 1980). Also, when the number of discontinuities increase and many strength curves are superimposed, the rock sample will behave more and more as an isotropic material (having no preferential direction of weakness).

![Inclined Discontinuity Plane](image)

**Figure 3.28** Strength of a rock sample containing an inclined discontinuity (from Hoek and Brown, 1980).

It is realized that application of the shear strength criteria for rock joints described in Section 3.5.2 to a large scale rock mass is very difficult. Both the orientation and the location of each discontinuity must be quantified to be able to derive a criterion which describes the strength of the rock mass. There are also very few examples of criteria which describe the strength of jointed rock masses in this manner. Ladanyi and Archambault (1970, 1972, 1980), (see also later Hoek and Bray, 1981), suggested a modification of their failure criterion for single discontinuities, making the criterion applicable to rock masses. This failure criterion include terms for the dilation rate, the ratio of the actual shear area to the complete surface area, the uniaxial compressive strength of the intact rock, and the degree of interlocking between the blocks in a rock mass. While this approach may seem attractive because it involves some consideration of the mechanics of block movement and intact rock failure, it is difficult to use in practice. Because of the large number of input parameters (a total of seven) and the difficulty in describing a large scale rock mass accurately, it usually takes considerable guesswork to come up with the input data.
The difficulty associated with explicitly describing the rock mass strength based on the actual mechanisms of failure has lead to the development of strength criteria which treat the rock mass as an equivalent continuum. A relatively simple and completely empirical failure criterion for jointed rock masses is the Hoek-Brown failure criterion (Hoek and Brown, 1980; Hoek and Bray, 1981; Priest and Brown, 1983; Hoek, 1983; Hoek, Kaiser and Bawden, 1995), defined as:

\[ \sigma_f = \sigma_j + \sqrt{m \sigma_c \sigma_3 + s \sigma_j^2} \]  

(3.7)

where \( \sigma_j \) and \( \sigma_3 \) are the major and minor principal stress at failure, respectively, \( \sigma_c \) is the uniaxial compressive strength of the intact rock, and \( m \) and \( s \) are parameters which depend upon the type of rock and the shape and degree of interlocking within the rock mass. Values for \( m \) and \( s \) can be determined from rock mass classification, using the RMR-system (Bieniawski, 1976, 1989). The Hoek-Brown failure criterion is widely used in practical rock mechanics, both for underground and slope applications. However, since the Hoek-Brown failure criterion represent a curved failure envelope, a transition to the linear Coulomb criterion (Equation 3.4) is often conducted. This is because a linear failure envelope is easier to handle both analytically and in numerical design methods. A difficulty with this approach is that the strength envelope in reality probably is curved, which means that the "equivalent" cohesion and friction angle are dependent on the normal stress.

Using the linear Coulomb failure criterion for defining the rock mass strength does have other practical advantages, since only two strength parameters need to be determined. Common for both the Coulomb and the Hoek-Brown criterion is that they do not provide a true description of the physical processes that occurs during failure in a large scale rock mass. The cohesion term not only represents the true cohesion due to fracturing of intact rock bridges but also the effects of crushing of asperities and rotation and separation of rock blocks. An "effective cohesion" is thus used to account for several of the mechanisms that take place during rock mass failure. Such an "effective cohesion" could also include the effects of confinement and reinforcement on the rock slope. Although attractive, there are only a few examples (see e.g., Jennings, 1970) available in the literature of how to choose "effective" strength parameters based on various rock mass and failure characteristics. The assumption of an equivalent continuous shear failure surface incorporating both discontinuities and intact rock and corresponding equivalent shear strength properties can also result in an overestimate of the strength since rock bridges can fail in tension rather than shear, as was discussed earlier (Franklin and Dusseault, 1991).
3.5.4 *Strength of Large Scale Rock Masses*

From the above discussion it follows that determination of the rock mass strength in practice can be extremely difficult. There are in principle four different ways of determining the strength: (i) mathematical modeling (described above), (ii) rock mass classification, (iii) large scale testing, and (iv) back-analysis of failures (Krauland, Söder and Agmalm, 1986). Rock mass classification is relatively frequently used to assess the rock mass strength, in particular in conjunction with the Hoek-Brown failure criterion, as discussed above. This approach has been used also in slope applications but the reliability of the results can be questioned since there are very few data from large scale slopes in the original set of data used by Hoek and Brown (1980). This approach can, however, at least be used to obtain first estimates of the strength of a rock mass. The strength of the intact rock can be tested in the laboratory. The same is true for the shear strength of small scale discontinuities, see e.g., Gyenge and Herget (1977). However, even seemingly straightforward laboratory tests can result in large scatter and difficult interpretations (Nicholson, 1994). Larger scale discontinuities can be tested in the field using hydraulic jacks. The problem with this is the same as for full scale tests of slope strength, namely the high costs. Large scale testing is therefore seldom economically or practically feasible and is rarely used in slope applications (see Section 4.4).

Remaining is back-analysis of previous failures in a slope. This is an attractive method to obtain relevant strength parameters. It requires that the failure mode is well established and that there is information available on the failure geometry, groundwater conditions and other factors which are believed to have contributed to the failure. Often, limit equilibrium methods (see Section 4.3) are used to back-calculate the strength, assuming equal driving and resisting forces (safety factor = 1.0). Examples of back-calculated strength values for the composite rock mass are shown in Figure 3.29 (Hoek and Bray, 1981).

These values are for slopes of various heights and in various rock types, ranging from very weak and weathered, to relatively hard and strong rocks. The scatter in strength values is also quite large. In Figure 3.29, Hoek and Bray (1981) included the back-calculated strength from the large scale hangingwall failure at the Grängesberg underground sublevel caving mine in Sweden (point number 4 in Figure 3.29). Back-analysis of similar hangingwall failures in other Swedish sublevel caving iron ore mines gave cohesion values of between 0.4 and 1.3 MPa and friction angles from 25° to 50° (Hall and Hult, 1964; Herdocia, 1991; Hustrulid, 1991; Lupo, 1996). Similarly, back-analysis of footwall failures at the Kiirunavaara mine gave cohesion values between 1.0 and 2.0 MPa with friction angles in the range of 30° to 37° (Dahnér-Lindqvist, 1992; Lupo, 1996). The values for the Kiirunavaara mine, both hangingwall and footwall, are summarized in Figure 3.30.
Figure 3.29 Friction angles and cohesions mobilized at failure. Strength parameters back-calculated for different slopes in Canada, England, Hong Kong, Spain, Sweden and United States (from Hoek and Bray, 1981).
Figure 3.30  Friction angles and cohesions mobilized at failures in sublevel caving mines. Strength parameters back-calculated for hangingwall and footwall failures at the Kiirunavaara sublevel caving mine, Sweden.

It must be noted that the caved rock is included implicitly in these strength values, with the exception of the results by Lupo (1996), in which the effects of the caved rock and the surrounding rock mass were separated. The strength values in Figure 3.30 are higher than those shown in Figure 3.29. This can be due to different mining geometries but also reflects the relatively strong rock at these mines. The values in Figure 3.30 are probably more relevant to the problem at hand, i.e., determining representative strength values for the rock mass at the Aitik mine.

To summarize, the knowledge of the shear behavior of single discontinuities is fairly good but the transfer from small scale shear failures along discontinuities to failures involving complex interaction of many discontinuities and rock bridges in a large scale rock mass, is not well known. Furthermore, failure criteria which consider some of the actual failure mechanisms tend to become complex and difficult to apply in practice. More simplified criteria greatly oversimplifies the actual behavior but can be useful when calibrated against field conditions. Back-analysis of similar cases can provide valuable knowledge on some aspects of the large scale strength but requires a very precise description of the prevailing conditions at failure. Figures 3.29 and 3.30 clearly illustrate the problem of quantifying the strength of a large scale rock mass. The scatter is relatively large, probably governed by the different geomechanical
conditions at each site. To be able to transfer these back-calculated strengths to other geomechanical environments, it is necessary to find a way of describing the geomechanical conditions characteristic for each pit slope. This must be based on the actual mechanisms which govern failure. The high accuracy required for the design of overall pit slopes underlines the importance of being able to better quantify the mechanism contributing to the strength of a large scale rock mass.
4 DESIGN OF LARGE SCALE SLOPES IN OPEN PIT MINING

4.1 Design Approaches

In this section, the design methods most applicable to large scale slopes are described and discussed. Before describing the various existing design methods, one must ask the question: What should we design for?

In the ideal case, the design process can be divided into three different phases, namely:

i. Prediction of mechanical failure, i.e., when the loads acting on a slope exceed the strength of the rock mass. This corresponds to the point at which a failure surface is starting to develop through the slope. Predictions of where the failure surface will develop, and under what conditions this will occur as a function of mining, are necessary before continuing to the next step in the design process.

ii. Prediction of the kinematics of the developed slope failure. Once a failure surface has developed fully through a slope, a prediction of how the affected rock mass will displace is required. A slow and stable failure, for instance, will have much less impact on the mine production than a very rapid and unstable failure. The failure rates should therefore be predicted at an early stage.

iii. Final design of pit slopes as the pit is being mined. The final design stage could be termed "interactive slope design" as, in this stage, the pit slope will be modified during mining. Close monitoring and successive revisions of the mine plan are ingredients in this stage of the design process, which requires close cooperation with mine planning.

In most cases it is not acceptable to design the slope to be completely stable, and it might be more advantageous to design a failing slope. This in turn requires that a prognosis of the failure development can be made (phase ii. above). Traditional engineering design is generally only concerned with resolving the issue of stability versus mechanical failure of a slope (phase i. in the above list), and current design methods are generally not well adapted to predictions of failure kinematics. More efforts need to be devoted toward prediction of the actual failure process once a failure surface has formed, but this should be accompanied by the development of a methodology to deal with failures in case very precise failure prognoses cannot be made.

The same point is made by Lambe (1973) who also notes the great need for simple techniques for predicting failures beforehand. Many of the existing design methods originate from the
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back-analysis of existing slope failures. Provided that measurements and observations of the failure behavior are available, such analyses are invaluable for identifying the failure modes and failure. Back-analysis can also serve to quantify strength parameters for the rock mass. Such an approach is still to a large extent empirical, which often means that the results are very site-specific. As a consequence, forward analysis, i.e., prediction of failures before they occur, is difficult for open pits which have not yet experienced failures.

For the design of open pit slopes, various levels of effort are required at different stages in the life of the mine, as was described by Coates (1977). What has been described above only constitutes the first part of pit slope design. The second part of pit slope design as pointed out by Sage (1976) is to incorporate these data into a financial analysis (see also Section 2.2). Pit slope design need not necessarily focus entirely on determining slope angles. Other controllable factors such as milling capacity and mining strategy could be involved in the design process. These are factors which can be changed during the operation of the pit and thus should be subject to economic optimization. It is important to realize, however, that economic optimization is not necessary in all stages of slope design. It is of most importance when forming the final pit slope, i.e., when the end of mine life is in sight.

Common to almost all existing design methods is the concept of design sectors in an open pit. A design sector is a region of the pit where the most important parameters influencing the slope stability are constant (Coates, 1977). This includes lithology, discontinuities, rock mass properties, grade distribution, pit geometry (curvature) and operational factors (e.g., location of important haulage roads and crushers). The definition of design sectors in a large open pit could be very difficult and time-consuming, but would at the same time reduce the amount of work required in the subsequent analysis stage. If conditions are found to be constant over large regions, it is only necessary to carry out one set of analyses for each of these design sectors. To be able to define design sectors correctly, knowledge of the governing parameters for slope stability is required, and a hypothesis regarding the failure mode and the mechanisms of failure must be postulated already at this stage. As more information is retrieved, this hypothesis must be updated and perhaps revised which in turn implies that the design sectors might have to be revised in an iterative process.

In the existing literature, the design of large scale slopes is very rarely treated as a specific case. Often, the same design methods are applied to bench, intramural and overall slope design, and there appears to be no explicit methods specifically aimed at the problem of designing large scale slopes. This could stem from the belief that the basic potential failure modes, as described in Section 3.3, are the same for large scale and small scale slopes, although the likelihood of a certain failure mode occurring is indeed very different at different
scales. It is also more difficult to identify the fundamental mechanisms of large scale slope failures and to obtain relevant strength parameters. The bulk of earlier slope design work has focused on the design of slopes in either strong, jointed rock, or in weak, yielding rock. In the first case, failure is assumed to be governed by the pre-existing discontinuities in the rock mass, and in the second case, failure through intact rock is more predominant. There are much fewer examples from slope design in hard, more sparsely jointed rock, similar to the large scale conditions at Aitik. One reason for this is probably the fact that there have been very few slope failures in this type of rock, which might suggest that slope angles in general are too flat for this type of geomechanical environment.

From the literature, two distinctly different approaches to slope design can be distinguished; deterministic and probabilistic design methods. In a deterministic approach to design, a point estimate of each variable is assumed to represent the variable with certainty (Coates, 1977). This requires that geological features and discontinuities are known explicitly, along with strength parameters, the stress state in the slope including hydraulic conditions, and pit geometry. An hypothesis for failure and a corresponding mathematical model for failure must be postulated. Possible uncertainties are accounted for afterwards by adjusting the results, through the inclusion of a factor of safety. This is the classical engineering approach to a construction problem and it has the advantage that it is relatively easy to comprehend and apply. The focus in rock slope engineering has been to develop fairly simple failure models. The corresponding mathematical models are thus also relatively simple, but in general, this does not pose a limitation to deterministic methods. The difficulty lies in constructing a model which is representative of the slope behavior. As will be shown later, the assumption of a failure surface is also a prerequisite for most probabilistic methods.

In a probabilistic approach, it is realized that the factors governing slope stability all exhibit variations and that representative point estimates are, if not impossible, at least very difficult to obtain. The variability of these properties are instead accounted for in the design process. Parameters are described as distributions of values, each value having a different likelihood, or probability, of occurrence. By combining the probabilities of each parameter value, the probability of failure for the slope can be calculated (Sage, 1976; Coates, 1977). It is important to note that the conventional approach in probabilistic design requires that a deterministic model of failure exists. The uncertainty in the problem is included through the variations of the parameters included in this model (BeFo, 1985). Unlike deterministic methods, this approach recognizes that there may be cases when the slope is unstable although the average value of the parameters suggest that it would be stable. Probabilistic methods emphasize the fact that it may not be possible to completely avoid slope instabilities.
Consequently, a probabilistic approach can predict the risk of failure and be extended to a financial analysis of pit stability, which in turn serves as a basis for decision making.

Among the deterministic approaches one finds limit equilibrium analysis which is relatively simple and easy to use. Furthermore, mathematical and numerical models are, in most cases, deterministic. Numerical models have become very popular in recent years, much due to the ease with which sensitivity analyses and parameter studies can be conducted. Empirical design methods rely on precedent, but could be, and are often, combined with other analysis methods. Physical model tests are seldom used today for design purposes, but deserve to be mentioned since they have contributed to a better understanding of possible failure modes in rock slopes. There are also design methods which combine certain features of both the deterministic and the probabilistic approach to problem solving.

At this point, it is necessary to point out that several simplifications are necessary in all design methods. The assumed failure mechanisms are more or less crude approximations of the actual failure mechanism. Certain assumptions are also made regarding the slope geometry and the loads acting on the slope. Assumptions are a requirement because otherwise the design methods would be overwhelmingly complex and nearly impossible to use rationally. What marks a robust design method is that the necessary assumptions have very little influence on the end result. In the following sections, an overview of current deterministic and probabilistic design methods will be given. Analytical methods, such as limit analysis and the commonly used limit equilibrium methods are first described. This is followed by a presentation of numerical models for slope design. Strictly empirical methods will be dealt with separately and finally probabilistic methods will be described.

4.2 Limit Analysis

It is instructive to first consider what is required for an exact solution to a continuum mechanics problem, in this case the stability of a rock slope. An exact solution requires simultaneously solving the conditions of equilibrium and compatibility in the slope. This includes the differential equations of equilibrium, the strain compatibility equations, the constitutive equations for the material and the boundary conditions of the problem (Chen, 1975; Brown, 1987; Nash, 1987; Chen and Liu, 1990). Only for very simple geometries and simple constitutive models is it possible to solve these equations and obtain a closed form solution for stresses and displacements in the slope. Numerical analysis presents an alternative to closed form solutions, which will be discussed in Section 4.4. For all other cases, the problem must be simplified by some well-chosen assumptions.
This recognition has lead to the development of limit analysis which is a simplified, yet relatively rigorous, method using concepts from the classical theory of plasticity. Limit analysis is concerned with determining the collapse load for a structure. The collapse load is here defined as the load which causes extensive plastic failure of the slope resulting in displacements which increase without limit while the load is held constant. In limit analysis, a perfectly plastic material and an associated flow rule must be assumed. An associated flow rule, or the normality condition, means that the plastic strains can be directly derived from the yield criterion and that the strain direction (the plastic flow) is normal to the yield surface. In practice, this means that the dilation angle and the friction angle are equal. This is normally not the case for rock materials. The collapse load for a structure can then be calculated using the upper and lower bound theorems (Chen, 1975; Chen and Liu, 1990). An upper bound solution is obtained if the conditions of equilibrium are ignored, while a lower bound solution is found when the conditions of compatibility are ignored (Nash, 1987, Johansson and Axelsson, 1991). The limit theorems are valid for perfectly plastic and slightly strain-hardening materials.

A correct application of limit analysis is slip line analysis, which in addition to the assumption of a perfectly plastic material also requires plane strain conditions and a yield criterion according to Mohr-Coulomb or Tresca. In slip line analysis, different slip directions are considered. A combination of the yield criterion and the equations of equilibrium results in Kötter's differential equations which define plastic equilibrium (Chen, 1975; Johansson and Axelsson, 1991). For this case, neither the elastic constants of the material nor the virgin stresses in the rock mass will affect the collapse load for the slope. Slip line theory has found widespread use in soil mechanics for analyzing bearing capacity of footings. In the original formulation, the soil was assumed to be weightless.

A strict application of limit theory to slope stability is more cumbersome and there are only a few examples of where limit analysis has been used for this purpose. One example of the use of limit theory for analyzing the effect of groundwater pressure on slope stability in soils was given by Michalowski (1994). Furthermore, Chen (1975) used limit analysis for slope stability analysis assuming a logarithmic spiral slip surface, and Chen and Liu (1990) developed this concept further by including effects from seismic loading and for applying it for anisotropic materials. Variations of limit analysis were also used by Afrouz, Hassani and Ucar (1989) and Afrouz (1992) for the development of an analysis method for failure along curved failure surfaces in rock slopes. The approach proposed by Afrouz, Hassani and Ucar (1989) is interesting since it is not as complex as other limit analysis methods, but still has the benefit of being able to determine the shape and location of the critical failure surface. Only very steep
rock slopes (near vertical) were considered in this approach, and the failure surface was assumed to pass through the toe of the slope. It was further assumed that the tangent to the failure surface was a function of the stress state in the slope. Using the Hoek-Brown failure criterion, the shape of the failure surface was obtained by solving the resulting differential equation. Since the method relies on the assumed relation regarding the tangent to the failure surface, it is not clear whether this method is generally applicable to all kinds of rock slopes.

4.3 Limit Equilibrium Analysis

4.3.1 Assumptions and Limitations

Limit equilibrium analysis is a simplification of the more rigorous limit theory in continuum mechanics, and has as such become the method of choice for routine slope stability analysis in soil mechanics. In limit equilibrium analysis, an assumption of the slip-line field is made, usually as a geometrically fairly simple failure surface. The shear strength of the material is normally described by the Mohr-Coulomb yield criterion (compare Equation 3.4), which prescribes a linear relation between normal stress and shear stress. A different failure criterion could equally well be applied to several of the existing methods of limit equilibrium analysis.

Strictly speaking, none of the basic equations of continuum mechanics regarding equilibrium, deformation and constitutive behavior are satisfied completely. The deformation of the material is not accounted for at all, and the condition of equilibrium is normally satisfied only for forces (Chen, 1975; Johansson and Axelsson, 1991). Using limit equilibrium analysis, it is not possible to judge whether the solution represents the upper or lower bound of the collapse load. The exception is for a circular failure surface in a cohesive material with a friction angle equal to zero ($\phi=0$) and for a logarithmic-spiral slip surface in a Mohr-Coulomb material, which both give an upper bound solution. For practical purposes it has been shown that the calculated collapse loads using limit equilibrium methods for rotational shear failure are sufficiently close to the theoretical bounds for the collapse load (Brown, 1987; Johansson and Axelsson, 1991).

4.3.2 Factor of Safety

In the simplest form of limit equilibrium analysis, only the equilibrium of forces is satisfied. The sum of the forces acting to induce sliding of parts of the slope is compared with the sum
of the forces available to resist failure. The ratio between these two sums is defined as the factor of safety, $F_s$:

$$F_s = \frac{\Sigma (\text{Resisting Forces})}{\Sigma (\text{Disturbing Forces})} \quad (4.1)$$

This simple definition of the safety factor can be interpreted in many ways. It could be expressed in terms of stresses, whereby there will be a different safety factor at each point on the failure surface, or in terms of loads or forces, which would represent an average factor of safety for a certain area, such as the whole failure surface. The factor of safety could also be formulated as a ratio between the actual cohesion or friction angle of the slope and the cohesion or friction required for the slope to be stable (Stacey, 1968). Equation 4.1 could also be formulated in terms of resisting and driving moments, which is useful for the analysis of rotational shear failure.

According to Equation 4.1, a safety factor of less than 1.0 indicates that failure is possible. If there are several potential failure modes or different failure surfaces which have a calculated safety factor less than 1.0, this indicates that all these can fail. The minimum safety factor does not, however, always correspond to the most critical failure surface (see also Section 4.7.2). It is important to note that the condition of limit equilibrium strictly means that the only admissible factor of safety is 1.0. At this point, the resisting and disturbing forces or moments on the slope balance each other. The shear strength along the failure surface may not be fully developed but the condition of equilibrium states that the resulting resisting force must be equal to the opposing disturbing force. The ultimate shear strength of the failure surface can thus be larger than this value (Jennings and Steffen, 1967). This means that when back-analyzing previous failures in a slope using limit equilibrium methods, the back-calculated shear strength will be a lower limit to the actual strength. However, if the ultimate strength is actually exceeded, for example, due to a large increase in slope height, the back-calculated shear strength will be over-estimated. In practice it is therefore difficult to judge how close to the ultimate shear strength, the back-calculated strength values are.

The assumptions implied by Equation 4.1 are, however, even more serious. It is assumed in all limit equilibrium analysis, that the shear strength is fully mobilized along the entire failure surface at the time of failure. This is not true, perhaps with the exception of simple plane shear or wedge failures. In other cases, and in particular for complex large scale failures of progressive nature, the shear resistance could differ significantly from point to point on the failure surface. This is a function of both varying shear strength and varying stress conditions along the failure surface. It is also assumed that the material in the zone of failure can be
subjected to unlimited deformations without loss of strength and that the displacements within
the sliding body are small compared with the displacements in the failure zone (Bernander and
Olofsson, 1983). This assumption of rigid body movements is acceptable when failure occurs
in the form of massive sliding along pre-existing discontinuities and the rock mass moves more
or less as a coherent mass. It is less acceptable when failure is more progressive in nature and
no clearly pre-defined failure surface exists (Stacey, 1973).

Although the inherent assumptions of rigid body movement and simultaneously mobilized
shear strength cannot be neglected, the factor of safety can still be a very useful concept for
engineering design. In civil engineering, the use of safety factors for various loads imposed on
a structure accounts for the intrinsic uncertainties in these values. The actual safety of a slope
is also influenced by the correctness of the assumptions made for a particular analysis (Hoek,
1991; Franklin and Dusseault, 1991). From the discussion above, it follows that a safety
factor of, for example, 1.3 does not have any physical meaning other than that the strength of
the slope is not yet fully developed. It is not possible to say if this constitutes a high risk of
failure or not. Safety factors could, however, be used as comparative values for the same
slope, thus quantifying if the slope is becoming more or less prone to failure as mining
progresses or other conditions change.

This is not a unique problem for rock slopes, but applies to many problems in rock mechanics.
A study of coal pillars in underground mining revealed that safety factors of 1.3 to 1.6
appeared to guarantee that no pillar failures would occur (Salamon and Munro, 1967;
Salamon, 1968). In this case, the use of a higher safety factor than 1.0 was to account for
variations in input parameters, and also to include the effects of different loading on different
pillars, a situation which bears little resemblance with slopes. For slope design, Jennings and
Stefan (1967) proposed that a safety factor equal to unity ($F_s = 1.0$) should be used and from
this calculate the maximum achievable slope angle. The slope angle would then be made
slightly less steep to create a safety margin for the pit slope. Hoek (1991) proposed different
design safety factors for different types of rock engineering problems, ranging from $F_s > 1.0$
for extreme loading of gravity dams, to $F_s > 1.5$ for permanent rock slopes and $F_s > 2.0$ for
block fall-outs in shallow tunnels. These suggestions should only be viewed as very crude first
estimates and they have little bearing on the design of ultimate pit slopes in mining, since
safety factors much larger than 1.0 probably cannot be allowed without jeopardizing mining
economy. A complicating factor for large scale slope failures is that progressive failures
would result in a variation of the safety factor along the failure surface. In this case, the factor
of safety must also account for model uncertainties, i.e., the fact that the failure model is too
coarse approximation of the actual behavior of the slope.
4.3.3 Assessing Failure Mode

Assessing the failure mode deserves some discussion. As was briefly mentioned in Section 3.2, the normal procedure is to use stereographic projection to identify which combinations of discontinuities that result in kinematically admissible failure volumes (Hoek and Bray, 1981). Input data include dip and dip direction (or strike) of pre-existing joint sets, normally obtained through surface mapping. Using stereographic projection it is only possible to determine the stability of, for example, a rock wedge, under the assumption that there is no cohesion on the failure planes (although different joint planes can have different friction angle). Stereographic projection must therefore be complemented by another design tool to fully evaluate the stability of a slope, and this is where limit equilibrium methods come into play.

Clearly, stereographic projection is limited in that only failure modes in which the failure surface is made up of planes or combinations of planes, can be identified. The failure surface can in reality be very complex, in particular for large scale slopes, and some form of simplification is necessary also here. This can, however, be a major obstacle for applying limit equilibrium methods.

In the following, a few examples are given of how limit equilibrium analysis can be applied to some of the more common failure modes. In this presentation, the failure surface is assumed to be known or have been identified as discussed above. Lengthy equations and derivations are deliberately left out, since they can be found in the original references.

4.3.4 Plane Shear, Wedge and Toppling Failure

Limit equilibrium analysis of plane shear failure are, for most cases, carried out through a two-dimensional simplification of the failure surface. An assumption must be made regarding the failure surface in the third dimension. For both plane shear failures and rotational shear failures (see next section), it is often assumed that the slope is relatively long and that there are discontinuities present which define the failing volume in the third direction (release surfaces). For wedge failures, the three-dimensional geometry must be accounted for directly in the equilibrium analysis equations. In both these cases, the three-dimensional effects arising from the pit geometry are neglected. To account for slope curvature, a numerical model would have to be used. The effect of groundwater is normally accounted for by assuming a pressure distribution along the discontinuities making up the failure surface (usually a linear variation of the groundwater pressure). The groundwater pressure thus acts to reduce the effective normal stress on the slip plane, which results in reduced shear strength. Groundwater in tension
cracks, on the other hand, acts as an additional driving force on the rock wedge, as shown in Figure 4.1.

A comprehensive analysis of both plane shear and wedge failures has been presented by Major, Kim and Ross-Brown (1977), in which even stepped failure surfaces were accounted for. Among the detailed methods, the one by Jennings (1970) also deserves to be mentioned. In this approach, both shear failure along a single discontinuity and shear failure along a combination of several shorter discontinuities were considered (step path failure). To account for the failure surface being a combination of several discontinuities at different orientation, Jennings proposed the use of apparent strength parameters which depend on the continuity of jointing along the failure surface. The resulting equations are relatively complex but comparisons with actual slope failures in the Kimberley area in South Africa proved satisfactory. There are also other examples of these types of very rigorous methods of limiting equilibrium (see e.g., Goode, 1970; Stacey, 1968). An interesting concept was that of Barton (1972) who used a simple slice method (see next section) for analyzing plane shear and step path failures.

A simplified analysis of plane shear failure was presented by Hoek (1970). Under the assumption of a completely homogenous material and a single discontinuity acting as a sliding surface, and for simplified groundwater conditions, Hoek derived a design chart for quick and approximate analysis of plane shear failure. Results obtained from the chart were very similar to analytical solutions for simple failure surfaces. However, and as Hoek (1970) pointed out, the chart was intended for approximate analysis of non-critical slopes, i.e., slopes which are not critical in terms of economics and safety. Most of this does not apply to the design of final pit slopes in mining.
Wedge failures are more difficult to analyze with a computationally simple model. In most cases, only wedges bounded by two discontinuities and the free surfaces of the slope are analytically treatable. Jennings (1970) analyzed also this failure mode using the approach with apparent strength parameters. A true three-dimensional wedge analysis was proposed by Londe (1972, 1973c), using vector algebra. In this approach, the cohesion and tensile strength of the discontinuities were neglected. Furthermore, and similar to analysis of plane failure, only the resultant of the forces on the rock wedge was included in the analysis. The moments were assumed to have a negligible influence on the slope failure. A very comprehensive analysis of wedge failures was also presented by Hoek, Bray and Boyd (1976), in which the cohesion of the discontinuities, the acting water pressure, as well as the existence of a tension crack and external loads on the rock wedge, were included, see also Hoek and Bray (1981). Simplified wedge analyses were given by Hoek (1973), Bray (1975), Bray and Brown (1976), Hocking (1976), and Hoek and Bray (1981). In the latter, the assumption was made that the friction angle was the same for both intersecting planes. This enabled the development of a simple chart for analysis of wedge stability.

Limit equilibrium methods have also been used for analyzing toppling failures (Hoek and Bray, 1981). Brown (1982) applied the principles of energy minimization to this failure mode and derived design charts for toppling failure. Both of these approaches only consider toppling of rigid blocks (primary toppling). Large scale toppling (see Figure 3.14) is much more difficult to analyze. Nieto (1987) made several attempts to analyze this mode of failure, using the concept of an upper active wedge and a lower passive wedge (see also Section 4.3.6) but a rigorous and generally applicable solution could not be derived.

### 4.3.5 Rotational Shear Failure

#### Background and Assumptions

Rotational shear failure is the most common type of failure in soil slopes. It is therefore natural that many of the methods currently being applied to high rock slopes originally were developed within the field of soil mechanics. Basically the same assumptions apply here as for limit equilibrium analysis of plane shear failure, but since the mathematical treatment of the problem differs it is worthwhile to describe this in more detail.

An analysis of slope stability for possible rotational shear failure requires that a failure surface is assumed. Circular slip surfaces are often employed since they are mathematically convenient and show some agreement with commonly observed slope failures, at least in soil.
Once this assumption is made, the equations of equilibrium are postulated for the rigid block bounded by the slope face and the failure surface. From the conditions of static equilibrium, the collapse load or the factor of safety can be determined. One also needs to assume a yield criterion, or failure criterion, for the material in the slope. Normally, failure is assumed to occur when the shear stress on the failure surface exceeds the shear strength of the material. Next, the sliding block is divided into a number of slices (in most cases vertical), as shown in Figure 4.2. This is necessary since the problem is statically indeterminate. Furthermore, the mobilized shear strength varies along the failure surface and to be able to calculate the normal stress and the resulting shear stress along the failure surface it is convenient to use the method of slices. There are also simplified methods in which the slope does not have to be divided into slices, but in general, these yield very conservative safety factors (Nash 1987).

![Figure 4.2](image-url)  
*Figure 4.2 The method of slices for analyzing rotational shear failure.*

**Two-Dimensional Methods**

Only a two-dimensional section of the slope is considered in this approach. Consequently, release surfaces must be present to allow failure to occur. For the two-dimensional section shown in Figure 4.2, the forces acting on each slice are (1) the shear- and normal forces on the failure surface, (2) the normal- and shear forces acting at the boundaries of each slice (these are often referred to as interslice forces), (3) the weight of the slice, and (4) the external loads acting at the top of each slice. Even when assuming that the weight of the material, the
external loads, and the groundwater pressure along the failure surface all are known, the problem is statically indeterminate. Consequently, if the problem is to be solved using only the equilibrium equations, a number of additional assumptions must be made. The assumptions usually employed are (Nash, 1987):

1. Assumptions about the distribution of normal stress along the slip surface.
2. Assumptions about the position of the line of thrust of the interslice forces.
3. Assumptions about the inclination of the interslice forces.

A large number of methods for analyzing rotational shear using the above approach have been developed, see e.g., Bishop (1955), Morgenstern and Price (1965), Spencer (1967), Bell (1968), Janbu (1973) and Sarma (1973, 1979). These methods differ in how well the conditions of equilibrium are satisfied and how the interslice forces are included in the solution. They can be divided into simple, complex and rigorous methods. For simple methods, the effects of interslice forces are neglected, whereas in complex methods, the interslice forces are included in the formulation. Methods where all conditions of static equilibrium are satisfied are named rigorous methods (Johansson and Axelsson, 1991). The various slice methods also differ with respect to the failure surface shape which can be analyzed. For many of the methods, only a failure surface in the form of a circular arc is allowed. There are, however, an increasing number of methods where the failure surface could be non-circular or even partly composed of straight lines (e.g., sliding along an existing discontinuity combined with rotational shear).

Good reviews of existing methods are given by Mostyn and Small (1987), Nash (1987), Johansson and Axelsson (1991), and Bromhead (1992), and will not be repeated here. Nevertheless, two methods deserve to be mentioned specifically. One of the earliest and perhaps the most widespread of these methods is Bishop's routine method for circular slip surfaces (Bishop, 1955). Bishop's routine method is a simple method since interslice shear forces are neglected. Only vertical force and overall moment equilibrium are thus completely satisfied. Using Bishop's routine method, it is feasible to solve the resulting equations manually. The other method considered here is the rigorous method proposed by Morgenstern and Price (1965), in which equilibrium of both vertical and horizontal forces as well as moment equilibrium for each slice and the sliding body as a whole, is satisfied. In the Morgenstern and Price (1965) method, both circular and non-circular slip surfaces may be analyzed. The resulting equations are consequently much more complex and the method is not suitable to hand-calculations.
Nash (1987) and Bromhead (1992) demonstrated that for most cases the differences in calculated safety factors are very small among the different methods. This is especially true when comparing the results obtained using the rigorous methods with Bishop's routine method. Bishop's method gives similarly accurate results as the rigorous methods, except for the case when the failure surface is steeply inclined at the toe of the slope (Nash, 1987). The high relative accuracy of Bishop's routine method can be explained by the fact that the method is based on moment equilibrium and since the moments normally are much larger in magnitude than the forces, force equilibrium is also almost satisfied. If only force equilibrium is considered in the solution, a more inaccurate result will follow. Such methods should therefore be used with caution or avoided completely. For most cases with fairly simple slope geometry, it appears that the application of Bishop's routine method is satisfactory, provided that the general assumptions for limit equilibrium analysis are acceptable.

In the various slice methods, only forces arising from gravitational loading are considered. Seismic loading can be accounted for in Sarma's (1973, 1979) method. Groundwater conditions in the slope are accounted for by assuming a phreatic surface and calculating the resulting water pressures on the failure surface, according to Figure 4.2. Often, a simplified approach is used in which the porewater (groundwater) pressure ratio, $r_u$, is used in the equations. This ratio is defined as:

$$r_u = \frac{u}{\rho g z}$$

where $u$ is the groundwater pressure and $\rho g z$ is the total vertical stress at a point $z$ meters below the ground surface in a soil mass with a density of $\rho$. This approach may be incorrect in many cases and should be used with caution (Bromhead, 1992; Mostyn and Small, 1987). Whenever possible, the actual phreatic surface should be used as input to the analysis instead. There are also some additional difficulties for partly submerged slopes, as discussed by Bromhead (1992). Since submerged slopes do not normally occur in active open pit mining, this issue is not discussed any further in this report.

Tension cracks at the slope crest can be accommodated in many, but not all methods. The rigorous methods are, generally speaking, best suited for including this condition, as well as for the analysis of heterogeneous slopes consisting of different materials. A strict condition arising from the simplifying assumptions of limit equilibrium methods is that the material cannot be strain-hardening or strain-softening. In most slice methods a linear failure criterion, usually the Mohr-Coulomb criterion, is assumed for the shear strength of the slope material.

Early applications of the method of slices for analyzing rotational shear failure in rock slopes can be found in Wilson (1959), Jennings and Steffen (1967) and Stacey (1968, 1973). The applicability of these methods to rock slopes is a matter of discussion. As was mentioned earlier, the method of slices is less suitable for brittle materials or materials which exhibit strain-hardening or strain-softening behavior, like rocks. Experience from soil mechanics has also shown that methods based on the theory of plasticity are less applicable for very long failure surfaces in soils, i.e., in situations when successive development of new failure surfaces can be expected (Sällfors, 1984). There are, nevertheless, some examples of where the concept of a circular slip surface, although a simplification of the actual failure surface, has been used successfully for predictions of multiple large scale rock mass failures, see e.g., Dahnér-Lindqvist (1992).

Three-Dimensional Methods and Progressive Failure

A two-dimensional analysis is only valid for slopes which are long in the third dimension. For shorter slopes, the failure surface will probably be bowl-shaped (see Figure 3.12) and the three-dimensional effects cannot always be neglected. Applying a two-dimensional analysis to a three-dimensional geometry will yield conservative results since the shear resistance of the end surfaces is not included in the formulation. Three-dimensional methods of slices have been developed to overcome these limitations, see e.g., Hungr et al. (1989), and Lam and Fredlund (1993), but three-dimensional methods are, unfortunately, much more complex than two-dimensional analysis. The biggest drawback is not the increased calculation time but the difficulty in defining the three-dimensional failure surface. Currently, there is limited knowledge of how the failure mechanism acts to form a failure surface in rock, and what factors affect the shape in a two-dimensional cross-section of the slip surface. Before three-dimensional limit equilibrium methods can be used routinely, at least in rock slopes, better knowledge regarding the fundamental mechanisms of slope failure must be obtained.

Relatively recently, efforts have been made to analyze progressive slope failure using two-dimensional limit equilibrium methods (Mostyn and Small, 1987; Chang, 1992; Chowdhury, 1995). In these methods, the difference between peak and residual strength is considered. The factor of safety for a given failure surface is first calculated using the peak shear strength.
The shear stress acting along each slice is then calculated (analytically or using numerical methods) and compared with the shear strength. If the shear strength is exceeded, the residual strength is assigned to that slice and a new factor of safety for the failure surface is calculated. Based on the two successive values of the safety factor, the "excess shear stress" is redistributed along the failure surface, and the resulting shear stresses are again compared with the shear strength of each slice. This iterative procedure is repeated until two successive safety factors remain the same. The redistribution of the "excess" stress is not straightforward and the calculation of the shear stress along the failure surface may require numerical methods. This approach can thus be relatively complex and still does not account for the true stress-strain behavior of the slope material.

**Search for the Critical Slip Surface**

The assumption of a certain shape and location of the failure surface in a slope and the application of any of the slice methods to this problem only yields one value of the safety factor or the collapse load. This might not necessarily be the lowest value and thus, the failure surface might not be the most critical one. Consequently, it is necessary to try several potential failure surfaces and go through the calculations for each surface. In many cases, the failure surface is assumed to pass through the toe of the slope, but this is not a necessary condition for failure.

It can be a tricky mathematical problem to find the local minimum of the safety factor or collapse load. A commonly used search technique for circular failure surfaces is to define a grid where each node point represents the center of a circular arc. Different radii of the failure surface are then tested for each node point and the corresponding factor of safety calculated for each of these potential slip circles. When a number of potential slip surfaces have been analyzed, the calculated factors of safety are contoured on the defined grid. If closed contours are obtained, a local minimum of the safety factor has been found. Otherwise, a larger search grid has to be defined. The obtained local minimum does not necessarily have to be the only minimum value of the safety factor. There may be several local minima, but only one corresponds to the critical slip surface. The grid search technique is simple but can be time consuming (Nash, 1987; Johansson and Axelsson, 1991; Bromhead, 1992).

More refined search techniques have been developed to overcome some of the problems with the grid technique described above, and also to be able to apply search techniques for failure surfaces which are non-circular. These more advanced search techniques are often based on dynamic programming (see e.g., Bardet and Kapuskar, 1990; Mostyn and Small, 1987).
this approach, one or several initial estimates of the slip surface are made and the corresponding factors of safety calculated. The estimates are then modified depending upon the relative difference between the calculated safety factors, thus moving the slip circle toward a more critical position. Using this iterative procedure, the result is often a non-circular slip surface. The critical slip surface obtained through dynamic programming often has a lower factor of safety than that obtained from a standard search with circular slip surfaces. These methods are, however, still in an early stage of development. Attempts based on use of the calculus of variations for finding the minimum safety factor have also been made. This requires that the function for the factor of safety is relatively simple and that all variables can be expressed as differentiable functions. The method is thus limited to simple geometries and failure surfaces (Mostyn and Small, 1987). An interesting new approach is the one where a numerical analysis is used to define the critical slip circle, often in combination with dynamic programming (Thompson, 1994; Zou, Williams and Xiong, 1995), (see Section 4.4).

To conclude, it is often necessary to have some knowledge of where the critical failure surface may be located to be able to carry out an effective search technique. Without any information regarding the possible failure development, time-consuming calculations will be required and the results could still be misleading and the resulting critical failure surface invalid. A good portion of engineering judgment is thus necessary in this phase of the analysis.

The relatively large amount of calculations necessary to find the critical slip surface for a slope and the increasingly complex mathematics for the rigorous slice methods has lead to the utilization of computers for conducting slope stability analyses. Computer programs for some of the methods mentioned above are given by Sage et al. (1977), Hoek (1987) and Bromhead (1992). There are also several commercial programs available, mostly developed for soil slopes. Comparative reviews has been published by Johansson and Axelsson (1991), and Oliphant and Horne (1992). The main conclusion from these reviews is that the differences between different programs and different slice methods are small for homogenous slopes and simple groundwater conditions. For more complex cases, the differences can be larger, both due to the fact that rigorous methods are required, and to differences in the algorithms used in different programs.

**Design Charts**

To avoid some of these tedious calculations, simplified graphical methods have been developed. The first of these "chart-methods" was that developed by Taylor (1937). This was followed by, amongst others, Bishop and Morgenstern (1960). The charts by Bishop and
Morgenstern (1960) and several others (see Nash, 1987) were specifically aimed at slopes in soils with slope angles lower than about 45°. Design charts for rock slopes were proposed by Hoek (1970), and developed further by Hoek and Bray (1981), assuming a circular failure surface and simplified groundwater conditions. Five different groundwater conditions were considered ranging from fully drained conditions to a fully saturated slope subjected to surface recharge of water. The failure surface must pass through the toe of the slope but could contain a vertical tension crack at the slope crest.

These charts enable a simple and quick determination of the factor of safety given the unit weight, the cohesion, and the friction angle of the slope material (assuming a linear failure envelope), making use of the inverse of Janbu's number (Equation 3.1). Only homogenous slopes were considered in this approach. The solution does not satisfy the formal requirements for the bound theorems in limit analysis but the authors showed that the results are close to the lower bound of the safety factor, thus proving that the method is a practically applicable tool for analyzing circular slope failure. Hoek and Bray (1981) also presented charts for estimating the location of the critical slip circle and the critical tension crack in the slope, thus substantially reducing the amount of calculation necessary. Design charts provide a rapid assessment of the potential for slope failure under certain assumptions. They should be used to obtain first estimates of slope stability, in particular for non-critical slopes, but are not to be used as a complete replacement for the more comprehensive methods of slices. Hoek's charts have, nevertheless, become a standard tool in rock mechanics slope design, due to their simplicity and the often limited amount of detailed knowledge on strength parameters and groundwater conditions in a rock slope. In these situations, a more detailed method of analysis is not justified.

4.3.6 Energy Methods and Extensions of Limit Equilibrium Methods

Energy methods are not very commonly applied for analyzing slope stability. Brown (1982) used the principle of energy minimization to analyze toppling failures and Asilbekov and Ismagulov (1992) applied the same principle to rotational shear failures in soil slopes. It seems logical that slip and failure should occur along the failure surface which represents the minimum energy dissipation but analytical methods for design based on this principle still remain to be developed.

An alternative approach for analyzing combinations of rotational shear and plane shear failure was proposed by Kvapil and Clews (1979). This approach does not strictly sort under the category of limit equilibrium methods, but it uses concepts from classical soil mechanics and
continuum mechanics. Kvapil and Clews considered a general failure surface in a large scale slope to be composed of an upper active part and a lower resistive part. The two portions of the slope are separated by a transition zone which is necessary for sliding to be kinematically possible as shown in Figure 4.3. The concept of active and passive pressure zones in a slope is consistent with the classical theory of earth pressure in soil mechanics. The idea is not new but was discussed already by Terzaghi (1944). Kvapil and Clews (1979) suggested that the transition zone can be simulated by the use of Prandtl's prism. The principle of Prandtl's prism was adopted from foundation construction. Active pressures developed below a foundation are transferred through the Prandtl prism to the passive zone outside the foundation, see also Terzaghi, 1944). The Prandtl prism changes the direction of the stresses and generates secondary shear fractures in the material (Figure 4.3). Kvapil and Clews (1979) derived the basic equations and stated that an approximate numerical solution has been developed, although it was not presented in the paper.

![Prandtl's prism in large scale slope failures (after Kvapil and Clews, 1979).](image)

The Prandtl mechanism produces three identifiable phases of failure. In the first phase, tensile cracks are initiated at the slope crest and the main shear surface begins developing from these. The second phase occurs when a continuous failure surface is formed in the slope which
results in heaving of the slope above the transition zone. In the third failure phase, heaving is more pronounced as are secondary shearing and crushing. All of this is concentrated in the vicinity of the transition zone. A similar mechanism of failure with the resulting displacements was observed by Call et al. (1993). Kvapil and Clews (1979) also stated that the failure process is very slow in the first and second phase, but could be significantly more rapid in the third phase. Much of this is substantiated through slope monitoring (see Section 3.3) and Kvapil and Clews (1979) have applied the theory of the Prandtl failure mechanism to several open pits.

The concept of active and passive zones in a slope has also been analyzed using standard limit equilibrium methods utilizing both graphical and analytical solutions (see the review by Mostyn and Small, 1987). Jennings and Steffen (1967) discussed the concept of active and passive zones of earth pressure along a curved failure surface. A practical application to rock slopes is given by Hoek (1983) who used the Sarma (1979) non-vertical slice method to analyze failures with active and passive zones.

A review of existing design methods would not be complete without also mentioning block theory (Goodman and Shi, 1985). This is a method to identify the types of blocks that can be formed in a jointed rock mass, and which of these blocks that are potentially moveable. Block theory is mathematically much more convenient than stereographic projection to delineate potentially dangerous blocks. It can handle an unlimited number of joint sets, and the shape and location of removable blocks can easily be determined. The drawback lies in the necessary assumptions which include those that all joint surfaces are perfectly planar and that they are infinitely long. This makes block theory more applicable to small scale slope stability where joint lengths can be of the same order as the slope height. In the original formulation, the only failure mode considered was sliding along joints. More recent work has included, for example, the rotation of blocks (Mauldon, 1992). Similar to other limit equilibrium methods it assumes that the blocks are rigid and ignores changes in slope and block geometry caused by block movements.

4.3.7 **Summary**

- Limit equilibrium methods are very simple to use and are well adapted for making rapid first-estimates of slope stability. There are also a large number methods available, both analytical and graphical, which have been used successfully for rock slope design.
• The disadvantages with limit equilibrium methods are primarily the assumption of rigid body movements, i.e., deformations within the sliding body are completely ignored. Judging from observed failure modes in large scale slopes, this is an oversimplification. Moreover, the failure surface must to some extent be known beforehand. Also, block flow failures and crushing failures at the slope toe (Figure 3.13) cannot be analyzed using limit equilibrium methods.

• Limit analysis requires fewer assumptions to be applied but is mathematically much more complex and there are very few examples of slope applications.

• Despite their relative simplicity, the necessary assumption of failure mode and rigid body movements, somewhat restricts the applicability of these methods. However, if the shape of the failure surface is known and the strength properties can be assessed, experience has shown that limit equilibrium methods are sufficiently accurate for practical slope design.

4.4 Numerical Modeling

4.4.1 Approaches to Numerical Modeling

The terms numerical analysis and numerical modeling are used here to describe analysis methods for the numerical solution of a problem. The boundary conditions of the problem, the differential equations of equilibrium, the constitutive equations for the material, and the strain compatibility equations are thus satisfied. From this one realizes that one of the major benefits of numerical modeling is that both the stress and the displacements in a body subjected to external loads and imposed boundary conditions can be calculated. Furthermore, various constitutive relations can be employed (anisotropic, plastic etc.). There are no restrictions regarding the number of different materials in a model (other than computation time). Numerical models can also handle complex slope geometries better than analytical or limit equilibrium methods. Another interesting feature of many commercially available programs is that they can model groundwater flow and the coupled effects between stress and groundwater pressure developing in a slope.

Today, there are a vast number of different numerical methods available, for example, Boundary Element Methods (BEM), Finite Element Methods (FEM) and Finite Difference Methods (FDM). Boundary element methods are commonly referred to as integral methods in which only the boundaries of a problem need to be discretized into elements. Finite element
and finite difference methods are referred to as differential methods, in which the entire problem domain must be discretized into elements. Another class of methods are the Discrete Element Methods (DEM). While BEM, FEM and FDM all are continuum codes, DEM is a discontinuum code in which discontinuities present in the rock mass are modeled explicitly.

Historically, numerical analysis has been used to model the stress state around open pits. This is relatively straight-forward, often assuming linear elastic material behavior. Simple stress analysis of open pit slopes was conducted by Stacey (1968, 1970, 1973) and Blake (1968) using finite element models. Other examples of fairly simple stress analyses are those by Feng (1988) and Du Plessis and Martin (1991). These types of models give a better understanding of the stress conditions in pit slopes but do not directly address the issue of the stability of a slope.

More recently, numerical codes have been used to model failure of rock slopes. Continuum codes are being used in soil mechanics to simulate the formation of a failure surface in a slope, assuming plastic or strain-softening material behavior. One example of this is to use numerical models for determining the shape of the critical failure surface in a slope (Huang and Yamasaki, 1993; Kidger, 1994). Conventional limit equilibrium analysis is then used to calculate the factor of safety against rotational shear failure. A more sophisticated approach of this type involves the use of dynamic programming to determine the critical slip surface (Thompson, 1994; Zou et al., 1995). Furthermore, Chang (1992) used a numerical method (distinct element method) to model individual slices in a slope, but a limit equilibrium approach to calculate the overall factor of safety.

In rock mechanics, discontinuum models have been used to simulate structurally controlled failures such as plane shear and wedge failures. Application of numerical modeling to large scale slope stability is more rare, which probably can be attributed to the fact that the failure mechanisms are less obvious and hence, more difficult to simulate. In addition to this, combinations of sliding along pre-existing discontinuities and failure through intact rock, as well as progressive failure development are more difficult to simulate in a numerical model. However, as pointed out by Starfield and Cundall (1988), models of any kind must be simplifications of the real world for them to be useful. A model which is too complex does not lend any additional understanding to the problem at hand. A successful application of numerical models depend on determining what aspects of, for example, geology are essential for the model. In the methodology outlined by Starfield and Cundall (1988), numerical modeling can also be used to investigate how different factors affect the stability of, for example, a slope. This approach still requires that the several different failure mechanisms affecting large scale slopes can be satisfactorily simulated. For a large scale slope this might
involve the formation of curved failure surfaces, progressive failure behavior, failure along joints, and failure of intact rock. Some of the existing and available numerical methods for analyzing these mechanisms will be presented in this section.

### 4.4.2 Continuum Models

In continuum models, the displacement field will always be continuous. The location of the failure surface can only be judged by the concentration of shear strain in the model. No actual failure surface discontinuity is formed and it can thus be difficult to continue to analyze the behavior of the slope after the first failure surface has formed. Simulation of rotational shear failure in a continuum slope (no discontinuities) using the continuum code **FLAC** (Cundall, 1976; Itasca, 1995a) is shown in Figure 4.4.

![FLAC Simulation](image)

**Figure 4.4** Calculated displacement vectors at failure in a plastic material, using the continuum finite difference program FLAC. The slope height is 300 meters and the slope angle 40°.
The location of the failure surface, judged from the onset of displacement or shear strain (not shown), is in general very similar to that predicted from limit equilibrium methods. Still, the "failure surface" is not very distinct and for large slope deformations, the assumption of a continuum may not be realistic.

To overcome some of these obstacles, several new approaches have been proposed among which two major categories can be identified. The first category is conventional continuum numerical analysis but with new constitutive models which better replicate the actual rock material behavior, while the second category is concerned with simulating localization of shear bands in the material.

In the first category, one finds the Cosserat plasticity models. Classical continuum models, including those where the rock mass (intact material and discontinuities) is modeled as an equivalent continuum, are not accurate in regions of high stress gradients. They cannot account for the bending stiffness of layers, or blocks, of intact rock. The Cosserat continuum model account for both rotational and translational degrees of freedom in a material, unlike classical plasticity models in which only translational degrees of freedom are included. Hence, moments and internal spins are taken into account in the formulation, and a non-symmetric stress tensor is allowed (Dawson and Cundall, 1995). Cosserat models are thus better adapted to modeling discontinuous media. Dai et al. (1994) presented an application of Cosserat models for the simulation of slope stability. A reasonable failure development was reproduced, while still using a very crude model, but an actual failure surface discontinuity is not formed in such a model.

In the second category of numerical models, an attempt is made to simulate the localization of shear bands in the intact material. Most geologic materials exhibit softening behavior as the strength of the material is exceeded, and a residual strength is achieved. Typical for these materials is that the plastic shear strain will localize in thin bands in the material, rather than being uniformly distributed. There is also evidence that such localization can occur in non-softening, plastic materials provided that the material is allowed to dilate (Itasca, 1995a; Zienkiewicz et al., 1995). The simplest approach to model localization involves the use of strain-softening material models in which the rock mass is given different peak and residual strengths (see Figure 3.17). Simulation of a progressive failure behavior is thus made easier and more realistic. There are several commercially available codes which have built-in strain-softening material models, for example, \textit{FLAC} (Itasca, 1995a).

However, the simulation of shear band localization using numerical models presents some particular difficulties. One problem is that the localization tends to be mesh-dependent. This...
means that, as shown by Zienkiewicz et al. (1995) and Schlangen (1995), the shear bands have a tendency to follow the patterns in the discretized mesh of the model rather than develop freely without interference of the mesh. Furthermore, large numbers of elements are required to produce localization in a numerical model, and two models of the same problem geometry but with slightly different discretizations may also produce different results. In many conventional models, the thickness of the shear bands is not simulated accurately. This is the case for the finite difference code FLAC in which the shear band collapses down to the smallest width that can be resolved by the mesh, normally one to three elements wide (Itasca, 1995a). In conventional finite element methods a more diffuse failure surface is formed which does not represent the actual failure mechanism very well (Larsson et al., 1992a). The inherent mesh-dependency is tackled by adjusting the mesh to follow the expected shear bands. This satisfies continuity of displacements over the element boundaries, but requires complete re-meshing (smaller elements, more elements and better orientation of the elements) of the model during the calculations, something which is very time-consuming.

New developments in this area are based on bifurcation theory and new numerical schemes to simulate localization (Chambon et al., 1994). Bifurcation in a certain structural system means that the solution can take different paths, depending upon small changes in the initial conditions. In the inter-element approach, Larsson (1990) and Larsson et al. (1992a, 1992b, 1994) developed a finite element model which could simulate localization in soil materials with the help of bifurcation analysis. In the bifurcation analysis, criteria for the formation of shear bands were formulated in terms of a critical value for the hardening modulus or a critical stress level, and the associated critical bifurcation directions. The bifurcation directions could be evaluated using analytical methods. In Larsson's approach, an incremental plasticity formulation was used along with constitutive relations which were adapted for soil materials (Mohr-Coulomb, double-cap and Cam-Clay models). Furthermore, strains were discontinuous over the element boundaries (as opposed to classical finite element model, see above). The finite element mesh was also adapted to the critical bifurcation directions. During the calculations, some re-meshing is still necessary, and this is carried out by aligning the element sides with the bifurcation directions, thus obtaining the most finely discretized mesh in the region of interest. Larsson et al. (1992a, 1992b) presented an example where the new model was applied to a slope stability problem yielding a good representation of localized deformations at failure (Figure 4.5).
Klisinski et al. (1995) used a different approach termed "inner softening band" in which softening occurs through displacement within an element. Cracks can be introduced anywhere and in any orientation within the element mesh, and at any time during the calculations. The results are less sensitive to the size and geometry of the finite element mesh. This method is, however, still in its early development and has to date only been applied to the cracking of concrete. Other attempts include those in which various new constitutive relations have been combined with mesh refinement techniques to obtain a more precise representation of the shear band. Hicks and Mar (1994) have, in this regard, also included groundwater pressure in their model. In a recent paper, Zienkiewicz et al. (1995) discussed these and other efforts at modeling localization and concluded that more robust formulations are necessary to avoid some of the numerical difficulties commonly encountered. The procedure proposed by Zienkiewicz et al. (1995) is more general and less dependent on the initial mesh. Since re-meshing was used, this procedure is also very time-consuming, and thus none of the developed model techniques can as yet be considered to be practically useful tools for slope stability analysis. It must also be questioned whether a numerical simulation of a very distinct slip surface is warranted. In reality, the failure surface can be relatively thick (see Section 3.3.3) and perhaps a less rigorous but much less time-consuming model is satisfactory for our purposes.

4.4.3 Discontinuum and Crack Propagation Models

Common for the above categories of numerical analysis is that they do not incorporate pre-existing fractures in the material or combinations of failure through intact material and along discontinuities. In a discontinuum computer code, discontinuities are included into the basic model geometry already from the start of calculation. Among the discontinuum codes, one
can distinguish the distinct element programs *UDEC* and *3DEC* (Cundall, 1971; Itasca, 1994a; 1994b), and the discontinuous deformation analysis program, *DDA* (Goodman and Ke, 1995; Pei and Shi, 1995). Both types of programs require that the locations of pre-existing discontinuities are known before an analysis is begun. This often, but not always, requires a rough idea of the governing failure mechanisms. By including a large number of discontinuities it is also possible, to some extent, to simulate the path which failure will take, under the assumption that failure only occurs along discontinuities. Failure through the intact material can be judged in the same manner as for continuum models, but in these programs it is not possible to simulate the formation of a fracture through the intact rock material.

Simulating the failure path and only accounting for pre-existing discontinuities, was addressed by Einstein (1993). Various models for describing a network of fractures, including the persistence and connectivity of discontinuities, were developed. Simple step path failures developing through linking of pre-existing fractures to a continuous failure surface was simulated by checking for kinematic admissibility of the formed rock block. In this approach, the rock material was assumed to be rigid. All displacements therefore are directly associated with the failure surface. Failure occurs when the shear strength of the discontinuities is exceeded. This approach has some similarities with block theory (Goodman and Shi, 1985) since only failure along discontinuities is allowed.

To be able to simulate fracture growth from pre-existing fractures in a rock mass, slightly different methods have been developed. Common for all of these models is that they are based on principles coming from fracture mechanics. Assuming that bridges of intact rock exist between the discontinuities, failure of these bridges can occur as tensile failure (Mode I) at relatively low stress levels, or as shear failure (Mode II) at higher normal stress levels. In the model developed by Einstein et al. (1983), Mode I failure was assumed to generate the primary fracture through the rock bridge, while Mode II failure would generate secondary shear fractures, as shown in Figure 4.6.

Only Mode I fracturing was believed to be important in establishing a failure surface through the rock material. The critical path for a particular joint configuration was defined as that combination of discontinuities and intact rock bridges having the minimum safety margin, where the safety margin was defined as shearing resistance minus the driving force. If the safety margin turns up negative, failure occurs, otherwise not. Failure is assumed to follow the discontinuity until it terminates. For each exit point of a discontinuity, the path of minimum safety margin is found and this constitutes the failure direction in the intact material. The calculation process is then repeated to cover the entire slope geometry. The forces acting on a particular discontinuity were assumed to be due solely to the overburden weight.
It appears as though the developed model and corresponding computer program can only handle configurations with one set of parallel joints. Furthermore, deformation of the intact material is not included in this model. It does not formally satisfy the differential equations of equilibrium, the constitutive equations for the material and the strain compatibility equations, as other numerical methods do (BEM, FEM, FDM and DEM). This approach is very similar to the method used by Call et al. (1977) and West et al. (1985) for calculating the possibility of step path failure and can be regarded as a modification and extension of limit equilibrium methods to account for fracturing through intact rock.

Figure 4.6  Mode I and Mode II fracturing in a rock slope (after Einstein, 1993).

A more rigorous approach to simulating fracture growth was presented by Napier and Hildyard (1992). In this method, a boundary element model was used for modeling the growth of pre-existing fractures in the rock mass. Both extension (Mode I) and shear (Mode II) fracturing were considered. Fracturing was assumed to continue from an existing crack tip. In the algorithm, the growth direction is first determined as the angle at which the maximum "excess shear stress" is obtained. The shear strength is defined through the Mohr-Coulomb criterion. Once the optimum growth angle has been selected, fracture growth occurs if a specified failure criterion is met. Both tension and shear failure criteria are used for this. Although both are based on the Mohr-Coulomb criterion they follow different slopes of the $\sigma_1-\sigma_3$-curve. These criteria must be calibrated against physical observations of cracking. This numerical model satisfies the conditions of equilibrium, compatibility and constitutive relations in continuum mechanics, and is thus a much more comprehensive model than the simplified approach outlined by Einstein et al. (1983). The pre-existing fractures as well as the newly propagated fractures are represented by displacement discontinuity elements in the boundary element method. Practical applications of this model have been limited to fracturing around underground openings in brittle rock and borehole breakouts (Napier and Hildyard, 1992; Kuijpers, 1995), in which fracture "seeds" were generated randomly around the opening or
borehole. Results from the numerical simulations have been found to be in good agreement with field observations of fracturing.

Another approach in which the boundary element method is used and where the pre-existing fractures are represented as displacement discontinuity elements is that presented by Shen (1993). Here, fracture mechanics theory was used for determining whether fracture growth from the tip of a pre-existing fracture can occur. In the calculation scheme, the fracture toughness for Mode I fracturing \( (K_{Ic}) \) is compared to the stress intensity at the fracture tip \( (K_I) \), which is calculated numerically. This automatically determines both whether fracture propagation will take place, and if so in which direction propagation will occur. Once a propagation direction is determined, a new displacement discontinuity element is added to the fracture tip (the fracture is extended), and the calculations repeated for the new fracture tip. Shen (1993) also presented a more comprehensive model in which both Mode I and Mode II fracturing, so called mixed mode fracturing, was incorporated. Fracture toughness for shear fracturing is difficult to determine and hence, a different criterion for fracture growth was adopted. The existing \( G \)-criterion states that fracture propagation will occur when the strain energy release rate \( (G) \) in an elastic medium is larger than the surface energy required to separate the material \( (G_c) \). This criterion was modified to ensure that, as had been observed in laboratory tests, Mode I fracturing would occur prior to Mode II fracturing. Shen (1993) successfully simulated fracture propagation and the coalescence of fractures observed in experimental tests on small scale specimens of gypsum (Figure 4.7).

The last two models are definitely the most comprehensive of the fracture growth models presented here. However, the model by Shen (1993) has not yet been applied to any practical problem of larger scale. Currently, there are limitations to the number of pre-existing fractures, or seeds, that can be modeled. Also, for these models to become practically useful, different sizes of these seeds must be possible to include. The approach by Napier and Hildyard (1992) has only been tested for underground problems, but it is likely that it can handle surface problems as well. For both models, it is essential that calibrations against observed failures are conducted to obtain correct values for the parameters in the growth criterion.
Figure 4.7 Two stages of observed and simulated fracture propagation and coalescence (after Shen, 1993).

The simulation of shear band localization in continuum models has in fact many similarities with the fracture mechanics approach described above. A fracture mechanics criterion for fracture growth could probably also be used as a complement to the standard plasticity formulation. Nonetheless, it is probably easier to use the localization approach for a continuum material. The real benefit of using a fracture mechanics approach comes when applying the model to a discontinuous material in which failure along discontinuities is mixed with fracturing through the intact rock material.

A very recent addition to the family of numerical programs which perhaps can be used for slope analysis is the two-dimensional version of the Particle Flow Code (PFC$^{2D}$) developed by Itasca (1995b), see also Cundall and Strack (1979). PFC is a distinct element program in which the individual blocks are modeled as circular particles. The particles may represent grains in a granular material like sand, or they may be bonded together to represent a solid material. By bonding particles together as blocks, this provides a unique possibility for simulating both fracturing through intact material, as well as failure along pre-existing discontinuities. Whether the program can be used for modeling rock slopes and progressive failure development still remains to be seen.
The above models for simulating progressive failure are all two-dimensional, although FLAC, UDEC and PFC are also available in three-dimensional versions. For a two-dimensional analysis, an assumption must be made regarding the third dimension. For oval open pits, the assumption of plane strain or plane stress is valid at the long sides of the pit. Two-dimensional models could thus be used for the design of large portions of the pit, with a few important exceptions such as the corners. Extension of the above models for localization and fracture propagation to three dimensions does not come easily. Models for simulation of fracture propagation in rock have not yet been developed for application to three-dimensional slope geometries. It is also questionable whether it is feasible to do so, without making substantial simplifications of the fracture geometry in three dimensions.

An important factor to consider is the ability to model groundwater flow. In commercially available programs like FLAC and UDEC (Itasca 1993, 1994), groundwater flow can be modeled explicitly, both as flow in the rock matrix and flow in the rock joints. Consequently, the effect of groundwater pressure on the slope can be quantified directly. It must be remembered, however, that coupled discontinuum analysis can be fairly complex.

4.4.4 Summary

1. Numerical analysis can be used to calculate both stress and deformation in a slope, and different materials and constitutive relations can relatively easily be incorporated. Numerical models can, with the correct input data, be used also for making predictions of slope behavior. Furthermore, sensitivity analyses are readily carried out using numerical modeling.

2. To simulate the actual failure mechanism, an assumption of the failure surface is generally not necessary. Existing continuum codes can, to some extent, simulate the location and shape of the failure surface developing in a slope, although an actual discontinuity is not developed. Discontinuum codes can simulate failure along pre-existing discontinuities. It is, however, much more difficult to simulate failure both along discontinuities and through intact rock in the same model.

3. More development is necessary before a general method is available which can simulate both fracturing through intact material and slip and separation of pre-existing discontinuities. Most of the methods described above have not advanced to the stage where they can be used routinely as a tool for predicting slope failures. Furthermore, simulating crack propagation in a slope may require an explicit description of the
fracture geometry (pre-existing discontinuities and intact rock bridges), and this is almost impossible to accomplish for a large scale slope.

4. Rather than attempting to simulate the exact mechanism in a single numerical model, several different models with different fracture patterns and different material models and properties, can be analyzed. Using this approach, the important factors governing a certain slope behavior can be identified. This approach is perhaps most useful when comparing results with field observations and measurements, see e.g., Board et al. (1996). Simpler models are used in favor of more complex models, which also facilitates collection and choice of input data.

4.5 Empirical Design

The use of past slope behavior experience constitutes an important ingredient in almost all design methods. The earlier described methods all stem from the theory of continuum mechanics, whereas in this section, slope design based solely on precedent is described. An early attempt toward a systematic grouping of empirical data was presented by Lutton (1970). Data from the steepest and highest slope in a specific open pit mine were gathered from several mines, and the slope height was plotted against the slope angle. This was further developed by Hoek and Bray (1981) by adding more cases (Figure 4.8). The currently employed interramp angle for the footwall at Aitik has also been included in this figure.

The dotted line in Figure 4.8 represents an estimated (Lutton, 1970; Hoek and Bray, 1981) upper limit for stable slopes. The higher the slope, the lower must the slope angle be to maintain stability. However, for the higher slopes the angle becomes almost constant which might lead to the conclusion that there is a lower limit to the required slope angle. In reality, this is probably an effect of having too little data for higher slopes. Furthermore, the shape and location of the design curve appears to be chosen somewhat arbitrarily, judging from the cases in Figure 4.8, since there are several unstable slopes located below the design curve (on the safe side).

Empirical data on stable and unstable slope angles and slope heights has also been collected from natural slopes by Coates (1977, 1981). Coates also included data from excavated slopes in his study and concluded that the difference between slopes in different rocks decreased as the slope height increased, which could partly explain the shape of the curve in Figure 4.8. These are only general guidelines, but the limited data do not permit the establishment of more detailed design rules. As pointed out by Lutton (1970), the design curve in Figure 4.8
represents a combined effect of many factors which makes it difficult to judge its applicability in a specific geomechanical environment.

Figure 4.8  Slope height (in feet) versus slope angle relation for hard rock slopes (after Lutton, 1970; Hoek and Bray, 1981). The currently used interramp slope angle for the north portion of the footwall at Aitik is also shown (FN - footwall north).
Other ongoing work on empirical slope design includes the very extensive field work on both natural and engineered slopes in China (Chen, 1995). More than 100 slope cases have been collected, ranging from very small (less than 10 meters in height) to very large slopes (more than 1000 meters in height), in various rock types, and exhibiting various types of failures. Rock mass classification (see below) has been conducted but no guidelines for slope design have as yet been developed from this work.

Rock mass classification is a more comprehensive and better structured form of empirical design. Different factors believed to affect stability are given different ratings which then are combined into a total rating representing the rock mass quality. An early attempt at a rock mass classification was that presented by Deere (1975). This classification scheme was used in the Aitik slope stability study by Call et al. (1976) and West et al. (1985). Several classification systems have later been developed, primarily for use in tunneling design (Barton, Lien and Lunde, 1974; Bieniawski, 1976, 1989). The $RMR$-system (Rock Mass Rating), developed by Bieniawski (1976, 1989), has been modified into a classification system specifically aimed at rock slopes — the $SMR$ (Slope Mass Rating) system (Romana, 1993). The standard $RMR$-system is applied and four adjustment factors are then added or subtracted. The adjustment factors account for joint and slope geometry, and the excavation method for the slope. The resulting $SMR$-rating is grouped into one of five stability classes which then determine the overall stability for the slope and the suggested support. The $SMR$ classification scheme has been used, for example, in the assessment of slope stability along a highway in Spain (Miño, 1991).

A similar, although less comprehensive approach to slope stability classification was proposed by Haines and Terbrugge (1991) who based their classification on the $MRMR$-system (Mining Rock Mass Rating) by Laubscher (1977). The $MRMR$-rating and the slope height are used to obtain the stable slope angle, based on a number of case studies from Africa and South America. A simplified classification system was also developed by Hawley, Gilmore and Newcomen (1994) for use in the preliminary slope design of large scale pits in South America and Canada. Relatively recently, a more universal classification system for slope design has been developed (Arnold, 1991; Mazzocola, 1992). These studies have also served as input data to the "matrix"-based system known as $REMIT$ — Rock Engineering Mechanisms Information Technology (Hudson, 1992). The $REMIT$-system is a nice way of graphically presenting the factors which affect the stability of a slope and how they interact. It is not, however, a true design system and thus of limited use for our purposes. The same can be said for the recent application of expert systems in this field (Sinha and Sengupta, 1989; Hao and Zhang, 1994), since the development of expert systems rely heavily on complete knowledge of the problem at hand. There have also been some attempts made at developing a single design
formula for slopes, similar to the formulas often used in pillar design. Sah et al. (1994) formulated an equation for the safety factor by applying regression analysis to a number of case studies. This was purely empirical and different failure modes were not differentiated amongst. The reliability of such a formula for general application must therefore be questioned.

To summarize, all slope classification systems exhibit the same major weakness in that they simply are not precise enough for the design of final slopes in open pit mining. They may be applicable for the design of small scale slopes, at least as first estimates when detailed data regarding potential failure mechanisms in the slope are not available. The concept of learning from past experience is, however, very important and should always constitute a major portion in slope design work.

Finally, a few words on trial slopes. Conducting a full scale test of slope stability is perhaps the most extreme form of empirical design. Although very cumbersome and costly, it can yield extremely valuable results on large scale strength parameters for the rock mass (Coates, 1977, 1981). Among the few tests actually carried out, the one at the Kimbley Pit in Nevada is probably the best documented. Despite the fact that the overall slope angle was increased from 45° to 61°, no massive failures occurred in this relatively weak rock mass, see also Chapter 6 of this report. For a full scale test to be successful, it is important that the objective of conducting such a test be stated in detail along with what can be expected and achieved before the test is conducted. Careful planning is imperative to all such activities. It is also important to recognize that even a successful trial slope test, i.e., a test in which the failure mechanisms and the governing parameters can be determined, does not imply that the obtained data are representative and can be used for the entire open pit, or even for a higher slope in the same area.

**4.6 Physical Model Tests**

Physical model tests have developed within the field of geomechanics basically because of the difficulties and costs associated with full scale testing in the field. Model tests provide the means of simulating the conditions of an actual slope in a controlled environment, where parameters more easily can be varied and their effect on the stability of the slope studied. They also provide the opportunity of testing up to, and beyond, the point of failure, something which can be cumbersome in the field. Model tests are perhaps not a true design method since it is not possible to calculate a slope angle directly from the results. For this, several tests with varying slope angles would need to be carried out and the results compared. On the other
hand, physical models have been very successful in that they have dramatically increased the knowledge and understanding of the possible failure modes in rock slopes (Pentz, 1971; Stacey, 1973), as was described in Section 3.3.4 in this report. Today, numerical methods have to some extent replaced physical model tests as a means of conducting sensitivity studies. Nevertheless, model tests should not be neglected as tools to investigate the fundamental failure mechanisms in rock masses.

Three different types of model tests can be distinguished. In the first type of tests, a model material is used in a down-scaled slope model. Loading is only by the gravity forces developed from the self-weight of the model material. However, horizontal loads can also be applied to the boundaries of the model, thus simulating a horizontal virgin stress field. Since only gravitational loading is used, this type of testing requires relatively large model dimensions and a model material which is substantially weaker than rock. Good examples of such model tests were those conducted by Barton (1971, 1972, 1974). Model materials are often different types of plaster-sand-water mixes. To ensure correct scaling of the material properties, a number of similitude laws must be fulfilled. This is by far the greatest difficulty associated with this testing method, and the results are often very sensitive to the choice of model material. Boundary effects can also come into play, as well as the manner in which joints are simulated in the model material.

In the second group of model tests, larger loads are applied to a model using conventional testing machines in a laboratory. Uniaxial, biaxial or triaxial loading can be applied. In these tests, stronger materials can be used which are more reminiscent of a hard rock, for example, high strength concrete. For most testing machines, there is a restriction on specimen size which can make it difficult to test a realistic slope geometry when including discontinuities. Moreover, artificial loading by testing machines does not replicate the actual stress state in a pit slope. This severely restricts the applicability of these tests. Nevertheless, very interesting results have been obtained regarding failure mechanisms using these types of model tests, as was discussed in Section 3.3.4 (Ladanyi and Archambault, 1969, 1972, 1980; Einstein et al., 1970).

The third group of model tests is centrifuge testing. Here, increased body forces are applied by rotating the model horizontally in a high speed centrifuge, thus generating centrifugal forces in the sample (Figure 4.9). This is equivalent to increasing the gravity forces acting on the model slope. The centrifugal acceleration constitutes the scale coefficient for the physical dimensions of the model. Provided that high accelerations can be generated, this approach greatly reduces the demand for a weak model material. In soil mechanics, centrifuge testing is frequently carried out using the actual soil and hence, no material property scaling is necessary.
(Veder, 1981; Ko, 1988; Schofield, 1988; Ohshima et al., 1991). However, only gravitational forces can be generated in centrifuges, making centrifuge testing less applicable to cases where high horizontal virgin stresses are believed to be of importance. A big advantage of using centrifuge models is that it is relatively easy to verify the correctness of the models by running several tests with different model sizes and different accelerations ("modeling of models"). Scaling relations can thus be validated, as well as size effects (Ko, 1988).

![Centrifuge Testing Diagram](image)

**Figure 4.9 Principal of centrifuge testing (after Schofield, 1988).**

The possibility of avoiding the use of a model material makes centrifuge testing very appealing also for rock slopes, although only gravitational forces can be simulated. There is, however, a problem associated with centrifuge testing of rock slopes. Since rocks have much higher strengths than soils, high capacity centrifuges are required. Soil testing is normally conducted with accelerations of less than 300 \(g\) (\(g\) being the gravitational acceleration at ground surface). For testing rocks up to the point of failure, much higher accelerations are required. Clark (1988) estimated that for medium to hard rocks, an acceleration of at least 1000 \(g\) would be required.

There are only a few centrifuges available which can generate these accelerations. Because of the need to include discontinuities in the model, centrifuge testing of rocks also requires somewhat larger model dimensions, compared to soil testing. Larger model dimensions require a centrifuge which, besides a high acceleration, also can handle a large mass. Unfortunately, these two objectives are not easily met simultaneously. Among the centrifuges which can generate 1000 \(g\) or more, the load capacity for the models is generally in the range of 1 to 50 kg (Clark, 1988). To put this into perspective, assume that a 500 meter high rock slope is to be simulated. With an acceleration of 1000 \(g\), the slope height would be 0.5
meters. Testing a section through the pit including some surrounding rock increases the size to something of the order of 1 m$^2$. Even with a very thin model of say 5 cm thickness, the mass of the model would be 135 kg. This is more than most of these high speed centrifuges can handle, and the lateral model size is still quite small if natural discontinuities are to be used. A new extremely powerful centrifuge is being developed and built at the WES (Waterways Experiment Station) in Vicksburg, Mississippi. This centrifuge will have a radius of 6.5 meters and have the ability to accelerate 2000 kg at $350 \, g$ or 8000 kg at $143 \, g$. In spite of the fact that this is stated to be the world's most powerful engineering centrifuge, it still cannot fulfill the requirements for rock testing discussed above.

The material scale effect introduced due to the substantial down-scaling of the model dimensions can be very significant. For coarse-grained soils, the grain size can be relatively large in relation to the model slope size. This effect is even more pronounced for a discontinuous rock mass. First, it can be difficult to obtain representative samples of jointed rock masses and secondly, there is a problem with the joint properties and their scale-dependency. As was discussed in Section 3.5, joint shear strengths depend on the roughness of the joint surface. In a small scale sample the size of the joint surface asperities would be greatly overestimated in relation to the size of the model pit slope. Furthermore, displacements are smaller on a model scale than in full scale situations and since the mobilized joint shear strength depends on the absolute magnitude of displacement along the joint surface, this violates the scaling rules. This phenomenon was verified experimentally by Iglesia et al. (1991), who concluded that centrifuge testing is less reliable for testing models involving discontinuities.

One way of avoiding the problem with high speed, high capacity centrifuges is to use a model material. This was the approach taken by Stacey (1973), which is an example of one of the few centrifuge tests specifically aimed at investigating rock slope stability (see Section 3.3.4). On the downside, this approach greatly diminishes the appeal of centrifuge testing since material property scaling using similitude laws becomes necessary. Other problems associated with centrifuge testing are that it is difficult to completely avoid vertical velocities due to vertical shaking of the centrifuge arm, and boundary effects from the model frame cannot be neglected. Miniature instruments may also have to be developed to be able to measure the performance of the model. It is also relatively complicated, although not impossible, to test saturated model samples, and care must be taken to simulate the correct stress path up to failure in the model (Ko, 1988; Schofield, 1988).

Despite these obstacles, it is believed that centrifuge testing of physical scale models can be of some value for further studies of large scale slope stability. One approach is to use centrifuge
testing for the validation of numerical models (Ko, 1988; Iglesia, 1991). It is much easier to compare the results from a numerical analysis of a certain type of slope behavior with model test results than against field observations. Parameter studies can be more easily conducted in centrifuge testing. Moreover, the model slope does not necessarily have to replicate all aspects of the behavior of an actual slope, which permits simplifications of model procedures and reduces the demands on the centrifuges. For this type of testing, the emphasis should be placed on the investigation of new phenomena and fundamental mechanisms of failure.

At this stage it must also be considered whether an even simpler test, such as the loading of a sample in a conventional testing machine, also can provide sufficient data for verification and validation of numerical or analytical design methods. The advantage of using such a test arrangement is that the test facilities are much more common compared to high capacity centrifuges. The test procedure would also be much simpler and less expensive thus permitting more tests to be conducted. The loading conditions in such a test are obviously not exactly similar to those in an actual open pit slope. The issue as to whether the correct failure mechanisms can be reproduced must first be clarified before conducting such tests.

4.7 Probabilistic Methods

4.7.1 Probability Theory

The basis for probabilistic design methods is the recognition that the factors which govern slope stability all exhibit some natural variation. Ideally, this variation should be accounted for in the design method. Using a deterministic approach, this is only possible by means of a sensitivity analysis. Although a sensitivity analysis can yield a good qualitative understanding of which factors are most important for a specific rock slope, such an analysis cannot quantify the actual chance of a slope failure. In a probabilistic design method, the stochastic nature of the input parameters are included and the resulting chance, or probability, of failure is calculated. Dealing with probabilities of failure rather than safety factors (see Section 4.3.2) means that one acknowledges that there is always a finite chance of failure, although it can be very small. This is more realistic than stating that a slope with a certain factor of safety is perfectly stable. Also, a quantitative description of the failure probability can be used in a risk analysis and linked to economical decision criteria. In the following, a short description is given of the basic theory for probabilistic design methods. Applications to slope stability and the special problems associated with this are discussed, followed by a section on risk analysis and decision theory.
Probabilistic methods have long been used in other engineering disciplines and with some success. Examples of this can be found in civil engineering where probabilistic design methods are used almost routinely to assess the failure probability of building structures. It is therefore not surprising that the approach has been imported into the field of geomechanics for design of construction elements. To illustrate the general methodology, assume that the load and the strength of a structure or construction element, for example, a slope, can be described by two probability density functions, respectively, as shown in Figure 4.10. The strength, or resistance, of the construction element is termed $R$ and the load is denoted $S$. The respective mean and standard deviations of each distribution is denoted $m_r$ and $s_r$ for the resistance, and $m_s$ and $s_s$ for the load. From Figure 4.10 one can see that the two curves overlap meaning that there exist values of the resistance which are lower than the load, thus implying that failure is possible. In a purely deterministic approach using only the mean strength and load, the resulting factor of safety would have been significantly larger than unity which implies stable conditions.

![Figure 4.10 Hypothetical distributions of the strength, or resistance, $R$, and the load, $S$, for a construction element.](image)

To be able to calculate the probability that the load exceeds the strength of the construction element it is common to define a safety margin, $SM$, as:

$$SM = R - S$$  \hspace{1cm} (4.3)

The safety margin is one type of performance function which is used to determine the probability of failure. The performance function is often denoted $G(X)$, hence:
\[ G(X) = R(X) - S(X) \quad (4.4) \]

where \( X \) is the collection of random input parameters which make up the resistance and the load distribution, respectively. An alternative formulation of the performance function which often is used in geomechanics involves the factor of safety, \( F_s \). Failure occurs when \( F_s \) is less than unity, hence the performance function is defined as:

\[ G(X) = F_s - 1 \quad (4.5) \]

The probability density function for the safety margin is illustrated in Figure 4.11. In this case, failure occurs when the safety margin is less than zero. The probability of failure, \( P(\text{failure}) \), is the area under the density function curve for values less than zero, as shown in Figure 4.11.

The reliability of a structure, on the other hand, is defined as the probability that the construction will not fail. The same concept applies to any performance function.

Assuming for now that the performance function can be expressed according to either Equation 4.4 or 4.5 and that the resistance and load distributions can be defined, how can the failure probability be calculated? For this, one can distinguish between Level 1, Level 2 and Level 3 analyses. A Level 1 analysis is basically a deterministic analysis, i.e., only one parameter value is used for every variable. In a Level 2 analysis, each stochastic variable is characterized by two parameters, the mean and the standard deviation, as described above. A Level 3 analysis is the most complete method of assessing the probability since the exact statistical characteristics of all variables are taken into account and the joint probability density
functions are calculated. Level 3 analysis is fairly uncommon, in particular in rock mechanics applications, since it often is very difficult to describe and quantify the joint probability density function (Mostyn and Li, 1993).

In practical design, Level 2 analysis is definitely most common. In this approach, the probability of failure is evaluated using a reliability index, $\beta$, defined as:

$$\beta = \frac{m_G}{s_G}$$  \hspace{1cm} (4.6)

where $m_G$ and $s_G$ are the mean and standard deviation of the performance function, respectively. The reliability index is thus a measure of the distance from the origin to the mean value of the performance function, expressed in number of standard deviations, (Figure 4.11). In building construction design, the reliability index has been linked to safety classes for buildings.

Exact solutions for calculating the failure probability is only possible for simple cases. The performance function contains several variables describing the load and resistance and is therefore often non-linear, which prohibits exact analytical solutions. A commonly used approximate method is the first-order-second-moment method (FOSM) in which the performance function is approximated by a polynomial (Taylor) expansion into a linear expression. Using a linear expression, the mean and standard deviation of the performance function can be easily calculated using standard statistical formulas and from this the reliability index determined (Mostyn and Li 1993; Thoft-Christensen and Baker, 1982). The resulting distribution of the performance function can with good reason be assumed to be a normal distribution, according to the central limit theorem (Kreyszig, 1988). Consequently, the resulting failure probability can be calculated as $\Phi(-\beta)$, where $\Phi$ is the standardized normal distribution which can be found tabulated, see e.g., Kreyszig (1988).

An alternative technique is the point estimate method (PEM) in which the performance function is evaluated $2^N$ times ($N$ being the number of input variables) to obtain the mean and standard deviation of the performance function (Rosenblueth, 1975; Mostyn and Li, 1993). This method does, however, become impractical for large numbers of input parameters. Another slightly different definition of the reliability index is that given by Hasofer and Lind (1974), in which the reliability index is defined as the distance between the origin and the boundary of the limit state. The limit state is determined from the performance function by transforming to statistically uncorrelated variables. The reliability index, $\beta$, can then be determined iteratively. Hasofer-Lind's method is common in building construction design but
has limitations regarding how complex the performance function can be to be able to do the transformation to uncorrelated parameter space.

All of the above methods are analytical means of determining the reliability index from a number of stochastic variables which make up the performance function. In cases where the performance function is complex and contains a large number of variables, a simulation technique can instead be used. The most common simulation technique is the Monte Carlo method. In this method, the distribution functions of each stochastic variable must be known. From each distribution, a parameter value is sampled randomly and the value of the performance function calculated for each set of random samples. If this is repeated a large number of times, a distribution of the performance function is obtained. The probability of failure can be calculated as the ratio between the number of cases which failed and the total number of simulations. Alternatively, the mean and standard deviation of the performance function distribution can be calculated to yield the reliability index from which the failure probability can be determined using tabulated values for the standardized normal distribution (Kim, Major and Ross-Brown, 1978; Mostyn and Li, 1993).

Monte Carlo simulation is thus a procedure in which a deterministic problem is solved a large number of times to build up a statistical distribution. It is simple and can be applied to almost any problem and there is practically no restriction to the type of distribution for the input variables. The drawback is that it can require substantial computer time. This becomes especially important when relatively small probabilities are expected and hence many iterations are required to obtain a reliable measure of the tails of the distribution. To overcome this, more efficient sampling techniques have been developed among which can be mentioned the Latin Hypercube sampling technique. In this method, stratified sampling is used to ensure that samples are obtained from the entire distribution of each input variable. This results in much fewer samples to produce the distribution of the performance function, in particular for the tails of the distribution (Nathanail and Rosenbaum, 1991; Pine, 1992). With today's powerful computers, computational time has become less of a problem and Monte Carlo methods prevail as the most common simulation techniques.

It is clear that a probabilistic approach is very attractive since it remedies some of the limitations of a purely deterministic approach. The limitations are mainly the necessary assumption of a distribution function for each stochastic variable. As will be shown in the next section there are, however, more severe limitations to applying probability theory to slope stability which must be considered.
## 4.7.2 Slope Stability Applications

In civil engineering, probabilistic design has advanced to the stage that virtually all building regulations are based on a probabilistic approach. An example of this is the use of partial safety factors which have been calibrated using more sophisticated methods of analysis. The development has not yet reached this point in the field of geomechanics. One of the reasons for this is the difficulty associated with describing a rock mass quantitatively and defining a model which describes both the load and the strength acting on a specific construction element. In the previous section the assumption was made that both the load and the strength could be described explicitly. This requires, similarly to deterministic design methods, knowledge of the failure mechanisms and a model which describes how failure occurs. In slope design, probabilistic methods have therefore primarily been based on the same failure models as those employed in limit equilibrium methods but with statistical distributions assigned to all input parameters (Franklin and Dusseault, 1991). Using this approach one must rely completely on the assumed failure mechanism, but several other uncertainties, such as the distribution of discontinuity parameters can be taken into account.

Pioneering attempts in this area are those by McMahon (1971, 1974, 1975). In the same category one find the work by Call (1972, 1985), Call, Savely and Nicholas (1977), Call et al. (1977), Call (1985), and Call and Savely (1990). These attempts emphasize the importance of the geological structure on slope behavior. The commonly assumed failure models are plane shear, step path, and different types of wedge failures. The failure mechanism is, to some extent, determined by the distributions of the discontinuity parameters such as dip, dip direction, length and spacing. This is accomplished by using a simulation technique (normally Monte Carlo simulation) and sampling values from each distribution of discontinuity parameters. This set of samples is then compared with the slope geometry and the failure mode determined depending upon what is kinematically possible for that specific set of samples. Once the kinematically possible failure mode is determined, random samples are taken from the distributions of joint strengths and the factor of safety calculated. The probability of failure is then calculated as the ratio between the number of iterations which yielded a safety factor less than unity and the total number of Monte Carlo iterations. The overall failure probability is the product of the probability that failure is kinematically possible (i.e., that a structure exists and that it is long enough) and the probability that the strength is exceeded. This may be written as:

\[
P\text{(failure)} = P\text{(kinematically possible failure)} \cdot P\text{(strength exceeded)} \quad (4.7)
\]
The slope geometry is analyzed in increments, typically of one bench height. By analyzing different bench face angles, the probability of failure as a function of bench face angle can be determined (Figure 4.12). Alternatively, the net failure volume (for each failure mode) can be calculated for various bench face angles. This approach has been used in various open pit mines including Aitik (Call et al., 1976; Call et al., 1977; West et al., 1985), the Cassiar Mine (Call & Nicholas, 1981) and Palabora (Piteau et al., 1985; Martin, Steenkamp and Lill, 1986). It is clear that a graph as that shown in Figure 4.12 can be very helpful in deciding what bench face angle to choose depending upon what failure probability the mine is willing to accept.

![Figure 4.12 Hypothetical failure probability versus bench face angle.](image)

Similar approaches are those by Kim, Major and Ross-Brown (1978), and Major, Ross-Brown and Kim (1978), who analyzed plane shear and various types of wedge failures using Monte Carlo simulation. Einstein et al. (1979) also based their approach on a limit equilibrium failure model, as did Carter and Lajtai (1992). Another case example in which this type of analysis has been applied to open pit slopes can be found in Morriss and Stoter (1983). A more comprehensive analysis of wedge failures including a more complex slope geometry and utilizing the first-order-second-moment method was presented by Low and Einstein (1992), while Düzgün et al. (1995) did the same for plane shear failures. Bolle et al. (1987) applied the point estimate method to plane shear failure. Progressive failure in the form of multiple plane shear failures occurring one after another was addressed by Chowdhury (1986). Savely (1987) analyzed block flow failures, in particular rolling blocks in intensely fractured rock.
masses. In this study, a comparison with other assumed failure modes showed the importance of assessing the correct failure mechanism before analyzing slope stability.

All of the above approaches are examples of Level 2 analysis. Common for these is that they can give good results for cases when the failure mechanism is relatively simple and well-established through observations of previous failures. There is still the problem of using the correct failure criterion, in particular for step path failure. In this case, failure occurs as shear (and probably also as tensile) failure along pre-existing discontinuities, in combination with failure through the intact rock bridges between the discontinuities (Mode I and Mode II fracturing, see Figure 4.6). Since the actual failure mechanism for this type of failure is not completely known, the probabilistic approach to analyzing step path failure cannot be regarded as fully satisfactory. This leaves us with the other failure modes controlled by the geological structure, such as plane shear and wedge failures. In this respect, the above approach provides a nice extension of the commonly used stereographic projection and block theory, and a link between discontinuity orientation data and the probability of slope failure. Consequently, probabilistic design of this type is more suitable to bench scale slopes, since this is where structural control is of most concern. On a bench scale, a large number of discontinuities can be mapped and statistical analysis applied. Bench design also involves determining catch bench widths, but this is often done on a purely deterministic basis, see e.g., Call (1985).

For a large scale slope, there are fewer structures of the same scale as the slope height and because of this, they may not have to be treated statistically in order to assess the stability of the slope. On the other hand, rotational shear failure must be considered in the design of large scale slopes. There have been several attempts at analyzing this failure mode using a probabilistic approach. These are also based on a limit equilibrium failure model, typically one of the slice methods described in Section 4.3.5. Examples of this type of analysis can be found in Markland (1972), Kim, Major and Ross-Brown (1978), Fairs (1982), Priest and Brown (1983), and Leventhal et al. (1993). In these cases, Monte Carlo simulation was used to determine the probability of failure for a certain rock slope. In neither of these attempts was a full search routine used for determining the factor of safety for a specific set of random samples of the input parameters. Instead, assumptions were made regarding the location of the most critical failure surface. This reduces the calculation time but is not very rigorous. However, both circular and non-circular failure surfaces can be analyzed in this way. A full search routine for circular failure surfaces (see Section 4.3.5) was used by, amongst others, Dai, Fredlund and Stolte (1993), but this is not feasible for non-circular failure surfaces.

An interesting aspect to consider is the system failure probability, i.e., the probability of failure of the slope as a whole. It was shown by Oka and Wu (1990) that the failure surface with the
minimum factor of safety does not necessarily have to be the failure surface with the maximum probability of failure. It is therefore necessary to evaluate the probability of the entire system, including perhaps several possible failure surfaces at the same instant (Chowdhury, 1987; Mostyn and Li, 1993). The combined probability of failure can be significantly higher than the probability of failure along a single failure surface.

To be able to use a probabilistic design method one must have sufficient samples for each input parameter to form a statistical distribution with some amount of certainty. Let us first briefly discuss the variability of parameters describing the discontinuity pattern in the rock mass, including orientation, trace length and spacing. In the literature, there is some agreement that the dip and dip direction of discontinuities are normally distributed (Call, Savely and Nicholas, 1977; Coates, 1981; Herget and Garg, 1982) or distributed according to the Fisher distribution (Einstein et al., 1979; Priest, 1993a, 1993b). Joint spacing and joint length on the other hand, both appear to be negatively exponentially distributed although a log-normal distribution also is possible for the joint trace length (Call, Savely and Nicholas, 1977; Einstein et al., 1979; Coates, 1981, Herget and Garg, 1982; Priest, 1993a, 1993b; Grossman, 1995).

These findings can only be used as a rough indicator of the statistical distribution for a certain variable at a specific site. To obtain the actual distribution, extensive sampling is necessary but this introduces the problem of bias, i.e., discrepancies between a measured value and the real property value (Mostyn and Li, 1993; Chowdhury, 1994). Surface mapping of discontinuities will, no matter what mapping technique is being used, always cause some bias and the true statistical distribution may not be possible to determine. This can be serious since the choice of distribution for the stochastic variables can have a significant impact on the final result. Furthermore, determining the kinematically possible failure modes involves delineating mapped structures into sets of joints with similar orientation. Call (1985) suggested that design joint sets should be identified based on pit wall orientation in relation to joint orientations. These sets need not correspond to geologically different sets of discontinuities but are more appropriate for design analysis.

The problem of sampling bias becomes even more important when assessing the strength distributions for the rock. For structurally controlled failures, joint cohesion and friction angle are the two most important strength parameters to consider. Research has shown both to be normally or log-normally distributed (Coates, 1981; Herget and Garg, 1982), but there are, in general, little data available to substantiate this. In reality, obtaining enough data on joint shear strengths to construct a reliable distribution of the two strength parameters is seldom feasible. Field tests are too expensive to be considered for more than a few tests, whereas
Laboratory tests are less reliable because of the difference in scale. Even if the strength properties were not scale-dependent, the required number of laboratory tests would still be unrealistically large. This was amply demonstrated in the Aitik study (Call et al., 1976; Call et al., 1977; West et al., 1985) in which only very few (less than ten) shear tests were conducted for each rock type. Consequently, the resulting distributions cannot be regarded with very high confidence.

For assessing the overall slope stability, obtaining representative strength values is even more cumbersome, partly because of the scale and partly because of the uncertainty regarding failure mechanism. Without the assumption of a failure mechanism, the variability of the structure must first be described explicitly together with the variability of the strength of all parts of a rock mass, and these two then coupled together (Pentz, 1982). This is an almost unattainable task and simplifications in the form of an assumed shape of the failure surface and the use of composite rock mass properties have therefore prevailed. Priest and Brown (1983) used the Hoek-Brown failure criterion in conjunction with the $RMR$-classification system to arrive at a strength envelope for the rock mass. From this, they determined the equivalent cohesion and friction angle. The approach used by Call et al. (1976), Call et al. (1977) and West et al. (1985) involved weighting the respective strength distributions of the intact rock and the discontinuities into a strength distribution for the rock mass.

Geostatistical approaches have been tried in order to better quantify the spatial variation of variables, i.e., the fact that the property values vary from one point to another and cannot be accurately represented by point tests. An example of this is given by Kimmance and Howe (1991). The use of this approach to some extent reduces the amount of input data. The problem of correlation between different strength parameters also needs to be considered. It is beyond the scope of this review to discuss this topic in detail but it appears likely that some correlation exists (Mostyn and Small, 1987) and that this can significantly influence the resulting failure probability if not properly included in the analysis. Li (1991) also pointed out the danger of letting the random variation of a material property be represented by only one single random variable. Instead, the property values at different locations should be treated as separate random variables. This has not always been done in most of the cases listed above. The practical significance of this insufficiency has not yet been quantified.

Alternatives to probabilistic methods based on a limit equilibrium failure model are very scarce. Numerical modeling could potentially be used, at least in the sense that random variations (within a specified distribution) of some input parameters can be simulated in a model. This does not, however, give any indication of the failure probability and only serves to investigate how sensitive the results are to the choice of input data. Using numerical
modeling in simulation techniques such as the Monte Carlo method is impractical due to the resulting calculation times.

Finally, one must discuss what failure probabilities are acceptable for a large scale slope. This was brought up by Priest and Brown (1983) and Pine (1992), who defined acceptance criteria according to Table 4.1.

Table 4.1 Acceptance criteria for rock slopes (after Priest and Brown, 1983; Pine, 1992).

<table>
<thead>
<tr>
<th>Category and consequence of failure</th>
<th>Example</th>
<th>Reliability index, $\beta$</th>
<th>Failure probability $P(F_s &lt; 1)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Not serious</td>
<td>Non-critical benches</td>
<td>1.4</td>
<td>0.1</td>
</tr>
<tr>
<td>2. Moderately serious</td>
<td>Semi-permanent slopes</td>
<td>2.3</td>
<td>0.01-0.02</td>
</tr>
<tr>
<td>3. Very serious</td>
<td>High/permanent slopes</td>
<td>3.2</td>
<td>0.003</td>
</tr>
</tbody>
</table>

For benches, failure probabilities of around 10% would seem acceptable, whereas for an overall slope a failure probability of less than 1% would be more suitable. This is still much higher than the commonly employed failure probabilities in building construction design. However, these values are not unreasonable considering that an open pit slope in general has a shorter life. The actual acceptance criteria to be used in a specific mine cannot be determined from general guidelines like these but should be subject to a more thorough analysis of the consequences of failure. Ideally a cost-benefit analysis should be used to quantify this. In the next section, these and other decision tools will be presented.

### 4.7.3 Risk Analysis and Decision Theory

The risk associated with a certain event can be defined as the probability of failure combined with the consequence or potential loss associated with that event as:

$$\text{Risk} = P(\text{failure}) \cdot (\text{Consequence of failure}) \quad (4.8)$$

The consequence can involve loss of life, capital loss or non-monetary environmental effects (Einstein, 1995). For an open pit mine it is most practical to define the consequence in monetary terms since this then can be used in a cost optimization process. The total cost, $C_T$, is thus defined as the sum of the initial cost, $C_0$, and the product of the failure probability and
the costs associated with the failure, $C_D$, (Shuk, 1970; Sexsmith and Mau, 1972, Chowdhury, 1994) as follows:

$$C_T = C_0 + P(\text{failure}) \cdot C_D$$  \hspace{1cm} (4.9)

Cost optimization according to Equation 4.9 is conceptually simple but can be cumbersome to apply due to the difficulties with obtaining accurate cost estimates for failure events. A few examples will be given later in this section. Assuming for now that the different costs actually can be quantified, determining how to act in a specific circumstance can be facilitated by using decision theory. A relatively simple method is to construct a decision tree, as shown in Figure 4.13. For this, different alternative actions are compiled and their probabilities calculated as well as the costs associated with each outcome (Befo, 1985; Berggren et al., 1991).

Decision trees are very instructive and simple to use once established. Different decision criteria have been developed to, for example, maximize profit or minimize loss. There are unfortunately few cases in which decision trees have been used for the design of open pit slopes, which probably can be attributed to the difficulties in obtaining both the probabilities and the costs associated with failure. Berggren et al. (1991) give examples on how decision theory can be used for dealing with soil slope stability and concludes that perhaps the largest benefit lies in the ability to quantify the effect of additional exploration. For example, if the rock mass characteristics to a large extent are unknown, additional exploration would reduce the uncertainty. If on the other hand knowledge of the rock mass properties is good, the money could be better spent on other activities. Further field or laboratory work is therefore also a question of comparing potential benefits with expected costs (Einstein et al., 1978; Pentz, 1982; BeFo, 1985). In general, decision trees can be used to quantify the value of any risk-reducing measure (Berggren et al., 1991).

An example of cost optimization for slope design is the methodology described by Kim et al. (1977a, 1977b). In this benefit-cost model the probability for failure is calculated for various mining geometries. The costs associated with different failures are included and a full economic analysis including all mining costs is conducted using Monte Carlo simulation. Different mining alternatives (different slope angles) are then compared with respect to net present value ($NPV$). A similar approach was used by Dinis da Gama (1994). Clearly, these two cases are a step in the right direction and should, if properly conducted, provide excellent feed-back to mine planning. Besides being able to calculate the failure probabilities, the methodology requires that the cost impact of failure can be specified in advance.
Figure 4.13 Example of a decision tree with hypothetical probabilities and costs (partly after Berggren et al., 1991).

A very wide spectrum of instability-related costs exists. This includes cleanup costs, loss or re-establishment of haulage system, cost of lost or delayed production, unrecoverable ore, loss or damage to plant, mill and equipment, costs for engineering (design) and monitoring, and, in the worst case, the cost of a complete shutdown (Coates, 1977; Call, 1985; Savely, 1985). These costs can be very difficult to estimate accurately, something which was evident in the slope stability study at Aitik conducted by West et al. (1985). There are also different methods of dealing with failure in an open pit which may reduce the costs associated with failure (see Chapter 5). The difficulty in accurately quantifying failure costs currently presents the largest difficulty with using a cost-benefit approach in slope design. Whether this can be overcome or not is uncertain. The same problem applies for decision trees, although valuable information can be obtained even if the costs are only approximated.
4.7.4 Summary

There are, as one can see, several potential merits for adopting a probabilistic approach to slope design but also several important drawbacks and limitations. These may be summarized as follows:

1. Probabilistic methods have the ability to include the inherent variation exhibited by almost all parameters which influence slope stability. Also, a probabilistic approach emphasizes the fact that a slope collapse cannot be completely neglected for any choice of slope angle, although the likelihood for failure can be very small.

2. These methods can be applied to almost any type of problem but a major drawback is the necessary assumption of failure model, commonly based on existing deterministic limit equilibrium methods. Probabilistic methods share this problem with almost all other design methods.

3. Probabilistic methods require extensive input data. Statistical uncertainty, sampling bias and spatial variation must be accounted for, which is not always easily done. Specifically, probabilistic methods are less suited to deal with variations in the load acting on a slope.

4. A probabilistic approach to design is more easily integrated with mine design than a deterministic approach, as it can serve as input to risk analysis for an open pit. Cost-benefit analyses of failures are very attractive but suffer from the difficulty associated with estimating all costs involved.

5. Judging from this it can be concluded that application of probabilistic design methods to large scale slope stability is relatively cumbersome, mainly because of the model uncertainty. A probabilistic design approach appears to be much better suited for the design of a relatively large number of small scale slopes, such as benches. This is also confirmed by the relatively large amount of such cases and the scarcity of applications concerning design of overall slope angles in open pit mining.
4.8 Discussion and Conclusions

From the reviewed literature, it is apparent that no design method is completely satisfactory for the design of large scale slopes in open pit mining. The relative merits and shortcomings of the currently existing design methods are summarized as follows:

- Limit equilibrium methods are simple to use and thus very popular. The drawbacks are the assumption of (1) the rock behaving as a rigid material, and (2) the shear strength being mobilized at the same time along the entire failure surface.

- Numerical modeling is very versatile and can simulate progressive failure behavior and deforming materials. Standard commercial software for performing rock mechanics analyses do not, however, allow fracture propagation through intact material, and new developments in this field have not yet reached full maturity for practical applications in slope design.

- Purely empirical design methods such as rock mass classification are, although attractively simple, not precise enough to be of value for the design of overall and final pit slopes in open pit mining.

- Physical model tests can be useful for determining fundamental failure mechanisms and for the verification of analytical and numerical methods. They are not a true design method, and the problems associated with simulating the correct loading conditions and accurately modeling rock mass properties can be overwhelming.

- Probabilistic methods can be used to (1) assess the risk for certain failures in a much more quantitative way than the use of the factor of safety, and (2) account for the inherent uncertainties of rock mechanics parameters. They require large amount of input data and assumptions regarding the distribution functions. Also, probabilistic design methods are mostly based on a deterministic model of failure (typically limit equilibrium methods) and are thus subject to the same limitations as these methods.

- Cost integration into design methods can to some extent be accomplished using probabilistic methods. However, the vast amount of input data required has rendered these cost-benefit-methods difficult to use in practical applications.

In addition to these findings, there are some general features of the existing design methods which need to be discussed. Most design methods are two-dimensional. This is a valid
assumption for large and long open pits, but may be a severe source of error for more circular pits. Nevertheless, the price (in the form of increased complexity and calculation times) for moving to a three-dimensional method is probably too high. In comparison to other deficiencies, the assumption of two-dimensional geometry may not be that severe. In fact, it appears that the choice of design method is far less important than the choice of input parameters into the design. Also, the assumption of a certain failure mechanism probably has more influence on the end results than the choice of design method. The mechanism of failure and the strength of the rock mass are obviously linked together and a good design method must be based on relevant knowledge of both these parameters.

Today, design methods are being extrapolated from the design of bench scale slopes, or from soil mechanics, to very large scale slopes in rock. It may very well be that the soil mechanics approach with circular failure surfaces and limit equilibrium analysis is satisfactory but this must first be verified. As was evident from the reviewed literature, verifications of actual failure mechanisms are lacking. As an example, the transfer from local, bench scale instabilities to more deep seated, large scale failures is not well understood, and the interaction between failure along pre-existing discontinuities and failure through portions of intact rock has not been quantified in detail. At the same time, it is necessary to realize that huge efforts have actually been made earlier in this area and despite this, the results are sparse. This is evidence of the difficulty involved associated with this topic.
5 REMEDIAL MEASURES AND MINING STRATEGY

5.1 Support and Drainage

When designing large scale rock slopes one deal both with controllable and uncontrollable variables. Uncontrollable variables are (1) the geologic structure at a site including orientation and frequency of discontinuities, (2) the regional groundwater levels, (3) the virgin stress state, and (4) the mechanical properties of the rock mass. These factors cannot be altered, in contrast to the pit slope geometry which to a large extent is controllable. Slope geometry, and the slope angles, in particular, are thus the main design parameters for an overall pit slope. Furthermore, the groundwater drawdown in a pit slope can be controlled through drainage systems and some types of potential failures can be controlled by artificial support. Consequently, drainage and support can be important alternatives or valuable complements to merely changing the slope angle (Coates, 1977).

5.1.1 Support and Reinforcement of Slopes

Mechanical support of rock is frequently used in underground mining. Relatively recently, the application of rock support and reinforcement has also become popular in surface mining. Most of these cases concern support of small scale slopes and benches, see e.g., Seegmiller (1974, 1984), Dight (1982), Nilsen and Hagen (1990) and Thompson et al. (1995). Commonly used support techniques are rock bolts, rock anchors or cable bolts, and to some extent wire mesh and chain link fencing (Sage, 1977; Seegmiller, 1982). The latter two are probably more common in civil projects. Retaining walls have also been used in civil projects and in a few instances in open pit mining to protect very critical areas. Standard rock bolts and longer cable bolts are good for controlling the stability of single benches, ramps and other critical locations in a pit.

Not all failure modes are equally amenable to artificial support. In general, plane shear and wedge failures are amenable to successful support, at least on a small to moderate physical scale. Rotational shear and block flow failures are much more difficult to support, in particular for large scale slopes (Sage, 1977; Seegmiller, 1982). Using rock bolts or cable bolts to support a potential rotational shear failure may result in a new critical and more deep seated failure surface, as shown in Figure 5.1. Block flow failures initiated by overpressuring at the slope toe are also very difficult to support due to their progressive nature (Sage, 1977).
For very high and large rock slopes, artificial support is not at all a very feasible strategy. The mere scale of an overall pit slope and the weight of a potential failure volume makes it almost impossible to use, for example, cable bolts or any type of mesh for support. Several authors suggest that rotational shear failure in soil slopes can be stabilized by removing material from the top of the sliding area thus reducing the driving force (see e.g., Brawner, 1982). This is only valid for a material with both friction and cohesion. If the material is only frictional, unloading changes both the driving and resisting force equally, thus not resulting in any change of the safety factor for the slope. If groundwater pressures exist in the slope, the situation becomes more complicated and it cannot generally be said that unloading increases the stability of a slope. Furthermore, for an open pit mine, unloading is hardly a practical solution.

Figure 5.1  Unsuccessful support of rotational shear failure (top left) and rock buttress support (bottom right). After Sage (1977) and Seegmiller (1982).

Other support methods sometimes employed in civil engineering are buttresses and retaining walls (Richards and Stimpson, 1977). Retaining walls are commonly made out of concrete and rely on their flexural stiffness to provide support for the slope. Although these types of walls have worked well in soil applications, they are obviously less applicable for support of a large scale open pit slope. A rock buttresses on the other hand, is simply a massive weight placed at the toe of a pit slope to provide horizontal restraint and increase confinement at the toe (Figure 5.1). Waste rock could be used to construct the buttress. Experience from highway rock cuts indicate, however, that both retaining walls and buttresses work best for
moderate slope heights up to around 60 meters (Richards and Stimpson, 1977). Furthermore, the required size of a buttress for stabilizing large scale slope failures makes it difficult to use buttresses in open pit mining where space is already limited at the pit bottom (Seegmiller, 1982).

To conclude, artificial support of large scale slopes is seldom a practical solution to increase slope stability, other than for isolated areas of smaller size. Even then, the choice of support strategy must be based on a cost comparison among different support methods and changes in the slope geometry. Whether rock support should be used instead of other measures is ultimately an economic decision.

5.1.2 Drainage of Slopes

The important influence of groundwater pressure upon slope stability was discussed in Section 3.1 and 3.5. Although the regional groundwater levels and the annual precipitation cannot be controlled, the local groundwater pressure distribution in an open pit is very much a controllable factor. Compared to mechanical rock support, a properly conducted drainage program affects a much larger volume of rock thus making it a very attractive stabilizing measure even for large scale slopes.

Reduction of the groundwater pressure can be accomplished by preventing surface water from entering the slope through cracks and discontinuities. This can be done by continuously grading the slope crest and filling and sealing all tension cracks, as well as diverting away surface water from the pit crest using surface drains (Hoek and Bray, 1981). To further reduce the water pressure in a slope, different drainage methods can be used. When selecting a drainage system care must be taken to ensure that maximal efficiency is obtained. Local drainage around a suspected failure surface may require a different strategy than attempting to lower the water table around the entire pit. An array of drainage techniques is shown in Figure 5.2 (Sharp et al., 1977; Hoek and Bray, 1981; Brawner, 1982).

The most common drainage method is definitely the use of horizontal drains drilled into the slope face. Water is allowed to drain through these and then lead away in collector drains and pumped out of the pit. This is a fairly simple and inexpensive method but the length of the drain holes can be difficult to determine beforehand and new drain holes are required as the pit is deepened. Horizontal drains can also be sheared off if the slope is moving, thus requiring new holes to be drilled constantly. The effectiveness of horizontal drain holes strongly
depends on the rock mass permeability and if the drain holes intersect major discontinuities in which water is transported.

Figure 5.2  Examples of drainage methods in open pit mining. After Sharp et al. (1977) and Hoek and Bray (1981).

An alternative to horizontal drains are vertical drainage wells drilled from the slope surface and equipped with down-hole pumps to pump out water from the slope. Wells can be installed before open pit mining is commenced thus providing depressurized conditions already from the start of mining. Vertical wells also function relatively well in rock masses with low permeability, although longer pumping times obviously are required in these rocks. Vertical wells equipped with pumps are more expensive than horizontal drains. Determining the number and the position of wells around an open pit can also be difficult and may require some trial and error.

Finally, drainage adits and galleries are extremely efficient but are also the most expensive means of slope drainage. The drainage potential for an adit is much larger than for individual drain holes and can be improved further by drilling fans of radial holes from the adit. However, the cost associated with excavating one or perhaps several adits into the pit slope often prohibits this drainage technique. Vacuum depressurization is another drainage method
which can be used on both horizontal drains and in drainage tunnels (Brawner, 1982). This technique is theoretically more efficient than drainage under purely gravitational conditions but also more expensive which has limited its application in practice.

It is clear that drainage is a very effective means of slope stabilization also for large scale slopes (see Figure 3.25), provided that it is well planned. Practical examples of successful drainage can be found, for example, at (1) the Barrick Goldstrike Post Pit where the water table constantly is kept below the current pit bottom; (2) the Jeffrey Mine where large scale failures were stabilized through extensive drainage; and (3) several pits at Highland Valley Copper (see also Chapter 6 of this report). There is, however, no unique solution as to how to install a drainage program. Each mine and each specific mining situation may require a certain type of dewatering program. Ultimately, an evaluation of cost versus benefit will determine what to adapt. An example of applying a cost-benefit analysis to the design of a drainage system was given by Sperling and Freeze (1987). Knowledge of the groundwater flow pattern and the resulting groundwater pressure distribution as well as the hydraulic properties of the rock mass is also necessary to optimize drainage. The obstacles listed above should, however, not discourage one from using drainage since the benefits far outweigh the disadvantages. In absence of good knowledge of the groundwater situation, a trial and error approach can prove valuable. Installation of horizontal drains and the continuous monitoring of groundwater pressures and inflow from these can be a first step. Experience from such a system can then be used to improve the drainage system.

5.2 Monitoring

Monitoring of pit slope behavior constitutes an important ingredient in the slope design process. The objectives of a slope monitoring program are to (i) maintain safe operational practices, (ii) provide advance notice of instability, and (iii) provide additional geotechnical information regarding slope behavior (Call, 1982; Call and Savely, 1990). Displacement measurements are the most common type of monitoring, complemented by monitoring of groundwater pressure. In addition, microseismic monitoring is becoming increasingly popular in open pit mining.

The techniques available for slope monitoring can be divided into surface and subsurface measurements. In the first category one find survey networks, surface wire extensometers and tension crack mapping. Survey networks, consisting of target prisms placed on the pit slope and one or several base stations from which the distances to the targets are being measured, are very common in open pit mining. A total station EDM instrument is often used to measure
the distances. These measurements can be made on a continuous basis. A survey network should have a good areal coverage and, if measurements are taken regularly and over a long period of time, provide a movement history for the slope. For a survey network to function properly, the base station must be located on completely stable ground, but at the same time close enough to the pit crest so that all prisms can be seen. This can be difficult to achieve in practice and a backsight, i.e., a stable point further away from the pit, can be used as a complement (Figure 5.3). Readings are taken also to the backsight and the actual movement of the base station recorded and subtracted from the other measurements. In large open pits, dust and haze can affect the measurement accuracy. Damage to installed prisms can also be a problem and new prisms may need to be installed as the pit is being deepened. Nevertheless, survey networks are probably the most cost-effective monitoring method and should be used on a routine basis in all types of open pit mining.

![Survey network for pit slope monitoring](image)

**Figure 5.3** Survey network for pit slope monitoring. After Call (1982).

Simple leveling of points on the slope crest can be a valuable addition to survey networks (Larocque, 1977). Furthermore, survey networks can be complemented with surface wire extensometers. These are, in the simplest case, a tensioned wire positioned over, for example, a tension crack at the slope crest. Wire extensometers are easy to use and can be moved to other unstable areas but do not provide the long-term monitoring of a survey network. Visual mapping of tension cracks is another very simple method which provides some information on the extent of an unstable area. On the other hand, tension crack mapping does not give a quantitative measure of the amount of instability.
Subsurface measurement methods include borehole inclinometers and extensometers, failure indicators, piezometers and microseismic monitoring (Call 1982; Call and Savely, 1990). Borehole inclinometers measure the angular deflection of the borehole and can be used to locate the failure surface in a slope which is moving. Inclinometers are relatively expensive and are not well suited to routine monitoring but can be extremely useful for help with identifying failure mechanisms. Conventional borehole extensometers only measure the deformation parallel to the borehole and are of less benefit in slope applications. Furthermore, borehole extensometers can only withstand very small shear displacements normal to the hole before they become useless. Failure indicators such as shear strips and coaxial cables can be used to locate failure surfaces but both have limited life in terms of sustainable shear displacement. They are thus best suited for indicating onset of failures before large displacements have developed. Monitoring of groundwater pressures can be done using piezometers (see also Section 3.1). These are invaluable tools for monitoring the effectiveness of a drainage program and planning program extensions.

Microseismic monitoring has also found its way into open pit mining, see e.g., Wisecarver, Merrill and Stateham (1969), and Stateham and Merrill (1979). Currently, a sophisticated microseismic monitoring system is being placed into use at the Chuquicamata mine in northern Chile. Experience has shown that there is some correlation between measurable rock noise and slope movement, but quantitative criteria are still lacking. Filtering out the noise from mining equipment is one problem; another is correcting arrival times for the actual length of travel for the wave (around the pit bottom rather than across the open void). In spite of the fact that the technical equipment already exist, microseismic monitoring cannot yet be regarded as very reliable. Microseismic monitoring still has its niche in open pit mines located in seismically active areas. Here, microseismic monitoring is used to detect seismically active zones which can cause rockbursts and earthquakes which in turn might trigger slope failure.

It is also interesting to discuss the use of alarms connected to various monitoring systems. Several of the methods presented above, for example, wire extensometers, borehole extensometers and even survey networks, can be hooked to a warning device. The warning device is activated when a prescribed amount of displacement occurs. For survey networks which use computerized data processing, displacement limits could be specified as total displacement, displacement rate or acceleration. The difficulty is not one of measurement technology but concerns determining what the limits should be. As was shown in Section 3.4, the amount of displacement for an unstable failure varies widely from one mine to another. Calibration against previously observed failures is thus a necessity. This means that an alarm criterion used at one mine cannot be freely transmitted to another mine with different geology and production (Košták and Rybár, 1993). Using alarms which have not been properly
calibrated can in fact be very dangerous since a slope failure could occur before the alarm has been activated. Alternatively, it can result in false alarms thus creating uncertainty and ultimately decreased belief in the benefits of the entire monitoring system.

This underlines the importance of interpretation for measurement data. Merely plotting measured displacements does not provide enough information for the mine planning department. Careful inspection and interpretation of the underlying mechanisms at work are necessary. To accomplish this it is necessary to have not only one but several measurement systems in function at the same time. In this respect, simple and reliable techniques are preferred over more complex and expensive systems. It is also important that these systems are largely independent and can function even if another system fails completely. Measurement accuracy is a question of scale and should match the anticipated magnitude of displacement. For a large scale slope, extremely sensitive measurements are not required and the money could instead be spent on additional monitoring points to track areas of instability.

Implementation of a monitoring program could be done in several steps. Basic monitoring, such as a survey network with a moderate amount of reflector prisms should always be conducted, even if there are no signs of instability. As unstable areas are being detected, more prisms should be installed and one or several other monitoring systems implemented to provide redundancy and better knowledge of the mechanisms at work. In critical stages of mining, very intense monitoring should be considered to ensure mining with safety (Larocque, 1977).

In conclusion, monitoring of slope displacements are an invaluable tool for optimizing open pit mining. Monitoring can be seen as an insurance premium for continued mining (Coates, 1977). There exist a number of cases in which slope monitoring has enabled a more optimized mining, see e.g., Kennedy and Niermayer (1970), Sharp (1988), Call et al. (1993) and Martin and Mehr (1993). These cases are evidence of how production interruptions due to failures have been minimized through careful monitoring. In these cases relatively large displacements were recorded before unstable failure and the warning time was relatively long. There might also be cases in which the time before measurable movements and slope collapse is very short thus putting more strain on the accuracy and reliability of the monitoring system.
5.3 Accepting and Dealing with Failure

As was discussed earlier, benches are often designed at a reliability of around 80 to 90%, thus accepting that up to 20% of the benches are exposed to failures. Naturally, these figures are lower for the overall slope but an economically optimal slope design almost always implies some degree of slope instability. Accepting that slope failures will occur requires methods for dealing with failures. Of primary importance is to make contingency plans to minimize the impact of slope failure. Savely (1993) provided the following list of contingencies that have proven successful in open pit mining:

- Provide multiple access to the ore faces and pushbacks.
- Stockpile ore.
- Establish additional ore faces before instability occurs.
- Plan wide pushbacks.
- Plan a stepout.
- Design to prevent noses in the plan geometry.
- Provide for failure costs in scheduling and budgeting.
- Add lag times in production scheduling.

Flexibility is the keyword here. Ensuring that several ore faces are accessible and that there is more than one access road to the active mining area reduces the risk that a failure will result in complete termination of production. The same applies to planning wide pushbacks since they provide the flexibility to mine around a local failure area. Narrow pushbacks are in general much more difficult to maintain (Call et al., 1993) and excessive time could be spent in cleanup which would reduce production efficiency substantially. Furthermore, stockpiling ore provides some backup in case mining must be stopped for a shorter period of time. In cases when the expectation for failure is high, planning a preliminary stepout (Figure 5.4) can be beneficial. This is an additional cost and should thus be included both in budgeting and in the economic analysis. If the stepout becomes unnecessary, additional ore can be gained at a later stage. By adding potential lag times proportional to the risk of failure in the production schedule, a more realistic time plan is obtained and a time buffer created.

Mining geometry also deserves some attention. It is important to avoid noses in the pit geometry since they are much more unstable than a concave pit wall (see Section 3.1). Also, the time of exposure should be minimized as well as the length of pit walls with very steep angles (Munn, 1985). Noses can result as different phases of mining join together but these can be alleviated by better mine planning. Good planning also makes it possible to mine critical areas selectively to restrict the extent of potential failure areas. In general, it is
important to include the eventualities of a slope failure at an early stage in the production planning. Providing multiple access and planning wide pushbacks requires additional stripping and this must be weighed against the consequences of failure if these precautions are not taken.

Part of a good contingency planning is also to try to anticipate when and where failure can occur. This can be done by constantly comparing the pit geology with the advance of mining. Furthermore, keeping track of old unstable areas is important. The geotechnical slope design should also be continuously updated as mining progresses with depth.

In instances when failure actually occurs, what actions can be taken to deal with the failure? One can identify a few different alternatives as follows (Stewart and Kennedy, 1971; Munn, 1985; Call and Savely, 1990; Savely, 1993; Call et al., 1993):

- Leave the unstable area alone.
- Continue mining without changing mine plans.
- Unload the failure area through additional stripping (only efficient under certain conditions).
- Leave a stepout.
- Do a partial cleanup.
- Mine out the failure.
- Install support and/or drainage.

Simply leaving the failure area is only an option in inactive mining areas or at the very end of the life of the pit when restoring would be too costly. Continued mining may be an option provided that the failure mechanism is understood, that the failure is slow and stable (see Section 3.4), and that a good monitoring system is in place. Nevertheless, some changes to the original mining plans may be necessary including revision of slope angles for deepening of the pit and a change of sequencing. This could also include leaving a stepout, as shown in Figure 5.4. A stepout reduces the overall slope angle and also provides some additional confinement at the toe of the slope. Also, planning a stepout means that cleanup cost can be reduced. There is also the potential that a stepout can be recovered at the end of the mine life by doing a mass blast and loading out the ore using remotely operated equipment. On the other hand, in cases when the failure is relatively limited and blocks an important area of the pit, a partial cleanup can be a good alternative.
Figure 5.4 Leaving a stepout in response to a slope failure.

Mining out the failure is an alternative only if the failure occurs along a specific geological structure and there is more competent rock behind the failure surface. In the case of a large scale rotational shear failure along discontinuities and through intact rock, mining out the failure is less feasible since a new failure surface could form behind the old one, unless the slope geometry or loading conditions are changed at the same time. Unloading the sliding area cannot, as was discussed in Section 5.1.1, generally be said to increase the stability of a failing slope, and it is seldom practically possible for large scale failures due to the large amount of stripping necessary and the costs associated with this. Support and drainage was discussed in Section 5.1 and one can conclude that support is an option for small scale failures whereas drainage can work also for large scale slope failures. This requires, however, that the failure is relatively slow so that there is ample time to drain the slope before a potential slope collapse.

In dealing with failures, controlling the excavation rate can be crucial. The idea is to maintain the displacement velocity of an impending failure below certain limits by mining intermittently in the unstable area (Call et al., 1993, Savely, 1993). This is an example of how to "live with a failure" and continue mining although significant slope movements are occurring. In Chapter 6, several examples are given of mines which have succeeded in continuing mining despite large slope displacements. It is extremely important to realize that this is only possible if the failure is of stable nature (see Section 3.4) and that unstable failure is not immediate. Current knowledge regarding the kinetic behavior of large scale failures in strong, competent rock is very limited. It might even be suspected that failures can be rapid and uncontrollable, in particular for steep pit slopes, thus rendering most of the above strategies useless. In this
case, the time between the first detection of instability and slope collapse may even be too short for removing equipment from the failure area. Providing enough operational flexibility to continue mining in other portions of the pit becomes even more important in this case, but aside from this, there is very little that can be done in terms of preventive measures. Mining under such circumstances constitutes a risk but it is not currently possible to quantify this risk. There are also very few if any examples of mines where such a risk analysis has been made. Mining under the uncertainty of an uncontrollable failure may be necessary to economically extract low grade ore in hard rock and thus deserves more attention in the future.
6 CASE STUDIES

6.1 Introduction

Case studies can be an invaluable source of information. Studies of existing slope failures and slope collapses can lead to a better understanding of the failure modes and the governing mechanisms of failure. Furthermore, if the failure mechanisms can be identified and the geomechanical environment quantified, the back-analysis of strength parameters is possible.

On the following pages, a review of some case studies from large scale open pits is presented. The selection of cases was based primarily on the criteria that (1) the slopes should be of large height, (2) the rock mass relatively hard and of high strength (similar to Aitik), and (3) large scale slope failures should have occurred, or were expected to occur. Rather than excluding cases because they did not fulfill all three criteria, several cases with somewhat weaker rock and a long history of stability problems were included in the list. Some of these date back to the early 1970's; hence, it is difficult to obtain further information from these mines. These cases do, however, provide a wider spectrum of the possible problems that can occur in large scale slopes. One of these cases is also an actual full scale test. In addition, open pits in Norway and Finland were included due to their close proximity to Aitik which makes it easy to obtain more field data from these mines.

The list of cases is far from complete and should be viewed as a first attempt to single out case studies of large scale slopes. All cases have been grouped by geographic location, starting with North and South America, and followed by Africa, Asia and Europe. A short description of the operation itself and the geomechanical environment is given for each pit along with a description of the encountered stability problems. The observations, descriptions and interpretations of stability problems included are those given by the authors referenced under each case. A concluding discussion is given at the end of this chapter.

6.2 Canada

6.2.1 Afton Mine, British Columbia

The Afton copper deposit is located 13 km west of Kamloops. The orebody occurs in a dioritic rock mass which has been extensively altered. The south wall of the pit is dominated by diorites, whereas in the north portion, sandstones, mudstones, shales, tuffs and arkoses occur. Faulting within the pit has been intense. Intact rock strengths for the rocks in the
northern portion vary from 3.5 MPa for the mudstones to 210 MPa for dacites. In the south wall, the diorites exhibit a uniaxial compressive strength in the range of 20 to 110 MPa.

The north wall of the pit was designed with a 40° overall slope angle, whereas the remainder of the pit including the south wall was designed at 45°. Several failures have been experienced in the pit. In the north wall, a large slide occurred in 1984 involving some 200,000 tons of overburden and rock, probably sliding along a pre-existing fault. Movement rates were very high, up to 1200 mm/hour, but production was only partly affected by the failure due to stockpiling of ore beforehand and fast remedial work including cleanup and crest flattening. Several small to medium scale wedge failures were also encountered in the north wall, as well as a larger failure in mudstones surrounding a dacite laccolith structure. This failure involved around 200,000 tons of rock and the slope was flattened from 40° to 30°. A circular failure involving over 1 Mton of overburden and highly weathered sediments also occurred in the northeast corner of the pit (Stewart and Reid, 1986).

In the south wall, a large scale failure occurred in 1985-1986. The failure mechanism was believed to be large scale toppling along pre-existing east-west striking faults. These dip at 65° to 90° to the south and have an average spacing of around 30 meters. The faults are continuous and have mineral infillings. The overall slope height at the time of failure was nearly 300 meters but the portion of the slope involved in the failure was approximately 170 meters high. (Figure 6.1).

Movement rates were at first relatively modest, around 2 mm/day. During 1985 and 1986 around 3000 meters of horizontal drainholes were drilled, but as the spring thaw of 1986 advanced, movement rates increased to 30 to 60 mm/day. The measured displacements prior to failure were largest at the slope crest, see also Figure 6.1. In early June, unstable failure occurred over a period of 45 minutes and involved approximately 300,000 tons of material. After failure, movement rates decreased to around 10 mm/day. The volume of rock which showed measurable movements was, however, significantly larger. It was estimated that the entire area of deep seated toppling involved a total of 7.4 Mton (Stewart and Reid, 1986; Reid and Stewart, 1986; Martin, 1990). Production at the Afton mine was suspended in 1991. The mine and the mill are still maintained for a potential restart of production in the future.
Figure 6.1  Section through the south wall of the Afton pit, showing slope outline, mapped structures and movement vectors (from Martin, 1990).

6.2.2  Brenda Mine, British Columbia

The Brenda mine is located at Peachland, 40 km west of Kelowna in southern British Columbia. Copper and molybdenum were mined from the orebody which is part of a relatively homogenous quartz diorite. The rock also exhibits some schistosity. Numerous clay gouges exist throughout the mine. The most dominate set of discontinuities strike approximately east-west, dip at around 70° to 80° to the south, and have an average spacing of about 15 to 27 meter. The long axis of the pit is oriented east-west. Two other joint sets were also identified; one striking north-south and dipping steeply to the northeast and one striking east-west and dipping around 30° to the north. Both of these sets are less continuous compared to the first one, but exhibit smaller joint spacing. The uniaxial compressive strength of the intact rock (for the south wall) was estimated to be greater than 150 MPa. The meteorological conditions are adverse with highly varying temperatures and high annual snowfall resulting in a heavy spring runoff (Blackwell and Calder, 1982; Martin, 1990).
Production from the mine started in 1970 with overall wall slopes designed at 45°. The south wall had experienced instability since the mid-1970's. In the spring of 1978, a failure initiated in the lower part of the pit, extending from the ramp to the pit bottom—a height of some 90 meters. The failure was believed to be structurally controlled by the clay gouges, and the spring runoff was probably the triggering factor (Blackwell and Calder, 1982). This failure was of relatively limited size.

During final mining of the south wall, more extensive instabilities were observed. The final cut was started in mid 1988. The south wall was designed at a slope angle of 45° but experienced rockfalls after the first 30 meter high bench was mined. Continued mining resulted in movement rates of around 50 mm/day but increasing up to 750 mm/day immediately after blasting of the benches. Flattening of the slope angle in the lower portion of the south wall to an overall angle of 40°, and extensive drainage (6000 meters of drainage holes) kept the failure under control. The south wall was then approximately 200 meters high (end of 1988). Measured vertical deformations were significantly larger at the slope crest compared to the lower portions of the slope, see Figure 6.2. The failure mechanism was identified as large scale toppling along the steeply dipping gouge-filled faults in the south wall.

![Figure 6.2](image)

*Figure 6.2 Vertical section through the Brenda mine south wall (from Pritchard and Savigny, 1990)*
Observations and numerical modeling indicated that the base of the toppling movements was relatively deep seated in the slope and that "graben" toppling (downward movement of blocks at the crest resulting in an outward thrust on the other blocks) occurred at the slope crest. (Martin, 1990; Pritchard and Savigny, 1990).

An uncontrollable failure event occurred in 1990. At that time, the slope height was 335 meters and the slope angle 45°. The failure involved around 5.10^6 m^3 of rock (approximately 15 Mton). An interesting aspect of this failure is that it occurred without prior warning in terms of cracks or displacements. The failure had an estimated velocity of 175 km/h (Sharp, 1995). The Brenda Mine is now closed.

6.2.3 Cassiar Mine, British Columbia

Asbestos was mined at the Cassiar mine, owned and operated by the Cassiar Mining Corporation. The mine is located in the mountains in northern British Columbia, some 1200 km north of Vancouver. Major slope instabilities developed during the final phase of mining in the hangingwall (east wall) of the open pit. The final height of this wall was 370 meters. The upper 250 meters of the hangingwall consisted of relatively competent volcanic rocks and argillites with compressive strengths of about 80 MPa. The lower portion of the hangingwall consisted of serpentinite of lower strength (σ_c < 50 MPa). Several shear zones and major fault structures were also present, as well as clearly defined sets of discontinuities. The overall slope angles were 40.5° for the ore and the shear zones, 43° for the serpentinite and 54.5° for the volcanic rocks and the argillite.

Several instabilities of smaller scale have been observed in benches and the layout adjusted accordingly (Call & Nicholas, 1981). A larger slope failure was initiated in 1989 during final mining of the hangingwall. The final slope was steepened an additional 3° (from 39° to 42°) in the northeast corner of the pit. Instabilities started during the spring thaw in June 1988. It was not economically feasible to flatten the slope and hence, a program of accelerated mining was adopted to complete the pit before the spring thaw in 1989. During this period 2.5 Mton of ore were mined until accelerated movements forced the pit to close in June 1989. At this time, movement rates were greater than 130 mm/day but then slowed to about 40 mm/day during the winter. Some additional mining from the bottom of the pit was conducted at this time. The failure process was relatively slow (stable failure process) and occurred over two years time.
Continuous monitoring and mapping enabled the failure surface to be determined. The mechanism proved to be quite complex and involved sliding of a large block on a steep-dipping serpentinite dyke which in turn caused toppling of neighboring blocks (Figure 6.3).

Figure 6.3  Diagrammatic representation of the failure mechanism at Cassiar mine (after Martin and Mehr, 1993).

The whole assemblage of blocks was bounded by a flat-dipping structure, dipping away from the slope. At the toe of the slope, intact failure in the weak serpentinite occurred and blocks were pushed out from the slope along the flat-dipping structure. Steeply dipping cracks between larger blocks were formed by the intersection of numerous short discontinuities in the rock mass and shearing and tensile failure occurred throughout the rock mass. High groundwater pressures were also acting on the slope. The failure occurred over a height of about 180 meters and involved approximately 17.6 Mton of material. Measurements showed that the upper section of the slope moved much more than the lower section (Martin, 1990; Martin and Mehr, 1993). Open pit mining at the Cassiar Mine ended in 1990. Underground mining started the same year and continued until 1993 when the mine closed.
6.2.4 **Highland Valley Copper, British Columbia**

The Highland Valley mine is located outside Kamloops and some 20 km west of Logan Lake in British Columbia. Highland Valley Copper is a partnership between Cominco Ltd., Rio Algoma Ltd. and Teck Corporation. The partnership was formed in 1986. Currently, two pits are being actively mined in the Highland Valley; the Lornex pit and the Valley pit. The combined annual production is 45 Mton of ore, and with a stripping ratio of 1:1, a total of 90 Mton of rock is mined annually. The average ore grade is 0.40% Cu and 0.007% Mo. The cutoff grade is 0.20% Cu (Scales, 1989; Sporleder, 1989; Holmgren et al., 1994).

**Jersey Pit**

Before the partnership was formed, the Jersey pit was being mined. A major slide involving approximately 300 000 m³ (less than 1 Mton) occurred in this pit in 1969. At that time, the pit had reached a depth of 137 meters with a final depth planned at 244 meters. The rock was a relatively hard quartz diorite and granite with intrusions of quartz, monzonite, porphyry and breccia. Investigations into the slide mechanisms showed that the sliding volume was bounded by discontinuities in the rock mass (Brawner, 1970). The Jersey pit is now closed.

**Valley Pit**

Of the two pits which currently are being mined, the Valley Pit stands for 80% of the total production. Mining at the Valley Pit, which is located in the north portion of the Highland Valley, started in 1983. The west wall of the pit currently measures 380 meters in height, whereas the northeast wall is slightly lower but with up to 200 meters of weaker overburden. The final pit depth will be around 630 meters and is planned for the year 2008. The host rocks are mainly diorite, granodiorite or quartz monzonite, and porphyry, all of which are relatively jointed. Three to four well-defined joint sets have been mapped. The uniaxial compressive strengths for the intact, unaltered rock are in the range of 120-140 MPa. Argillaceous alteration occurs sporadically, which reduces the intact rock strength to less than 3 MPa.

The slopes are designed with interramp slope angles of between 38° and 45°. Large scale failure has been observed in the west wall. The failure is believed to be deep seated and associated with toppling movements on steeply dipping faults and joints. An interramp angle of 38° is being used in regions of observed toppling failure. Slope monitoring includes distance measurements on prisms, surface extensometers to measure crack openings and
pneumatic piezometers for groundwater monitoring. The failed zone moves at a rate of between 25 and 200 mm/day, with the largest movements occurring in the spring due to snow melting and increased groundwater pressure. A relatively extensive drainage program is also in use at the pit (Brawner, 1970; Sperling, Munro and Freeze, 1989; Stacey, 1993; Krauland and Sjöberg, 1995).

**Lornex Pit**

The Lornex Pit accounts for 20% of the total production at the Highland Valley. It is located in the south wall of the Valley. The copper-molybdenum orebody is some 2100 meters long and 700 meters wide and dips at about 20°. Mining of the pit started in 1972 and the pit currently measures 2500 meters in length, 1700 meters in width and 350 meters in depth. The host rocks are diorite, granodiorite and quartz porphyry, which all show some alteration. The strength and quality of the rock mass is fairly good, but several intersecting joint sets can be found. Rock strengths are similar to the Valley Pit. The orebody is bounded on the west by the Lornex Fault which strikes north-south and dips between 45° and 85° to the west. This fault zone is 50 to 90 meters wide. There are also several smaller faults with similar orientation, typically 5-6 meters wide and containing clay gouge and breccia.

Instabilities and large scale failures have been observed in the east, west and southwest wall of the pit. In the east wall, a relatively complex and deep seated toppling mechanism is believed to have developed, aided by adverse groundwater conditions. Movement rates of up to 200 mm/day have been measured. Overall slope angles had to be limited to around 30° (interramp angles of 35°). In the west wall, a similar failure mechanism developed when the slope was around 110 meters high with an interramp angle of 36° - 42°, (Figure 6.4). Slope monitoring revealed that the slope had moved a horizontal distance of up to 70 meters during a period of 8 years. An extensive dewatering program slowed the movement rates down to around 35 mm/day, but drainage holes tend to get sheared off and new holes must be drilled constantly. Surface cracking is currently visible up to 100 meters behind the slope crest. These cracks partly follow the weak fault zones. In the southwest wall, very large but relatively local movements were recorded.

The failure mechanism for the above failures is believed to involve toppling along the steeply dipping faults in the pit wall. The base of rotation is believed to be deep seated, almost down to the toe elevation of the slope (see Figure 6.4 and also Figure 3.14). Flattening of the slope angles, planning of stepouts and extensive drainage have been conducted to control failures, accompanied by continuous monitoring. The ability of both these pits to "live with failure" is
aided by the relatively slow and stable failure process (Daly, Munro and Stacey, 1988; Krauland and Sjöberg, 1995).

6.2.5 Highmont Mine, British Columbia

The Highmont mine is located near Logan Lake, approximately 50 km southwest of Kamloops. The porphyry orebodies were mined for copper and molybdenum, starting in 1980. The two principal rock types found at Highmont are granodiorite and quartz diorite. Various degrees of alteration are found throughout the mine. Intact rock strengths vary significantly, from 1 to 140 MPa. A large, 15 meter wide and gouge-filled fault intersects the pit, striking 20-30° toward the east and dipping at 60° to the west. In addition to this, six sets of structural discontinuities have been identified in the pit. Slope angles are around 40°.

In May 1983, failure occurred in the southeast portion of the pit. At this time the pit depth was around 60 meters. Mining continued down to 110 meters depth, accompanied by frequent slope movements involving 1-2 Mton of rock. Increases in movement rates could be correlated to mining at the toe of the slope and increased precipitation and runoff. Failure was believed to be a combination of sliding along structural discontinuities and failure through the poor quality low strength rock mass in the lower section of the slope. The failure was also relatively deep seated. Back-calculation of the strength parameters for the rock mass indicated that for a friction angle of 33°, the rock mass cohesion would be of the order of 0.1 to 0.4 MPa (Newcomen and Martin, 1988).
6.2.6 Island Copper, Vancouver Island

The Island Copper Mine is located near Port Hardy on Vancouver Island. The mine is owned and operated by BHP (Broken Hill Proprietary) Minerals Canada Ltd. Mining started in 1971 and was completed in July of 1995. Processing of stockpile ore continued until January 1996, when the mine was closed down. The ore averages 0.4-0.5% Cu and the cutoff (1995) was 0.3% Cu, although the ore has been mined at a cutoff grade of only 0.2% Cu. The annual production in 1994-1995 was around 12 Mton of ore and a total of 20 Mton of rock which is a significantly less than during the 1980's, when the total production was around 60 Mton of rock. The porphyry orebody occurs as two lenses in an andesite together with some breccia and limestone. The host rocks are basalt and rhyolite, all of fairly high strength and stiffness.

The final pit depth is 400 meters. An interesting aspect is that the slope crest is at sea level, meaning that the bottom of the pit is 400 meters below sea level. The pit is located immediately north of the Rupert Inlet on Vancouver Island and special precautions have been made to control the potential water problems. A barrier wall was constructed to prevent seepage from the sea during the final pushback of the south wall, as shown in Figure 6.5.

The barrier wall has functioned very well but the south slope experienced a relatively large failure during mining of the pushback. Failure appeared to follow a large fault zone in the upper portion of the slope and pass through a zone of weaker rock at the toe. Inclinometer measurements and surface monitoring indicated a failure surface which would outcrop behind the barrier wall. It was decided to do a stepout of the slope to reduce the slope angle from a planned 40° to 35°. The total movement of the slope amounted to 35 meters but flattening of the slope stabilized the area. There have also been some failures in the east wall in a weaker sericite zone. Apart from this, the slopes have behaved very well and there are very few failures even on a bench scale. The interramp angles employed at the mine range from 35° for the south slope close to the sea to 50° for the even higher (500 meter) north slope. Estimated rock mass strengths (from classification) range from a friction angle of 24° and a cohesion of 0.05 MPa, to a friction angle of 40° and a cohesion of 0.09 MPa (Mathis et al., 1993; Holmgren et al., 1994; Krauland and Sjöberg, 1995).
Figure 6.5  Horizontal and vertical section of the Island Copper Pit (partly after Mathis et al., 1993)
6.2.7 Jeffrey Mine, Asbestos, Quebec

The Jeffrey mine is located at Asbestos, Quebec, and is operated by JM Asbestos Inc. (formerly Canadian Johns-Manville Co. Ltd.) Asbestos fiber is mined from ultrabasic host rocks dominated by peridotites, dunites and serpentinites. The rock mass is intersected by several thick shear zones and smaller scale discontinuities. In general, the rock does not exhibit any structural consistency. Strength and deformability for the rock materials vary widely from very soft and weak ($E=0.14$ GPa, $\phi=23^\circ$) to moderately stiff and strong rock ($E=8.6$ GPa, $\phi=60^\circ$). The mine, in particular the south-east corner, has suffered several large scale failures during the period from 1970 to 1986. Failures are very critical in this region due to its close proximity to the town of Asbestos.

In 1970, the slope height was 180 meters plus 60 meters of overburden (clay, silt and sand). Failure occurred in relatively fractured, serpentinized peridotite with a major shear zone present. Sliding in the overburden was first observed, followed by local wedge failures and finally a major slide in 1971, involving some 33 Mton of rock. The dimensions of the slide area was 210 meters in height, 610 meters in width and 75 meters in depth (Figure 6.6).

![Figure 6.6 Cross section through the Jeffrey mine showing the 1971 failure surface (from Pariseau and Voight, 1979).](image)

The overall slope angle in this wall was of the order of $30^\circ$. Failure was believed to have started in the weak shear zone which then lead to failure of the upper portions of the slope. Overstressing of the slope toe followed. Mining at the toe also helped to initiate the failure
and adverse water conditions reduced the effective shear strength of the rock. Movement rates of the order of 1500 mm/month were recorded. The slide was stabilized with comprehensive drainage of the slope and unloading by removal of the overburden.

In 1974-75, a reactivation of the 1971 year failure occurred. Displacement rates of 1000-1500 mm/month were again recorded. The slide was stabilized by water control measures. Up to this point, a total of 30 meters of displacement had occurred in the slope. In 1982, a new major zone of movement developed in the south east corner area, this time in more undisturbed rock. A major pushback had resulted in slope heights of around 390 meters.

Movement rates were much lower for this failure, of the order of 10-30 mm/month, but in 1985 a more rapid failure occurred, with movement rates of up to 300 mm/month. In fact, multiple failures probably occurred, with two failure surfaces sliding on top of each other. The total mass of the moving material was estimated to be between 40 and 50 Mton. The displacement rates for this failure showed a strong correlation with the amount of precipitation in the area, and in the spring of 1986 and 1987, movement rates increased up to almost 2000 mm/month. Common for these failures is that the failure surface was relatively shallow in relation to the slope dimensions (Figure 6.7).

Figure 6.7 Cross section through the 1971, 1974 and 1983 failure zones at Jeffrey mine with relative slope locations (from Sharp, Lemay and Neville, 1987).

Furthermore, the failure surface was sub parallel to the slope face, except for the upper and lower zones. It was also concluded that the failure surface did not appear to be controlled by
any particular geological weaknesses. Inclinometer measurements revealed that the failure zone (shear zone) was around 25 meters thick, with more concentrated shear displacements in a 4 to 5 meter thick zone (Brawner, 1977; Sharp et al., 1977; Sharp, Lemay and Neville, 1987; Sharp, 1988). Recently, numerical modeling using both distinct element and finite element codes has been conducted, but the results have not yet been published (Sharp, 1995).

The north wall of the Jeffrey mine mainly consists of slate with a well-developed bedding, or foliation. The uniaxial compressive strength of the slate ranges from 15 to 50 MPa, depending upon the sample orientation relative to the foliation. A major expansion of the north wall was conducted between 1976 and 1986, resulting in a 300 meter high slope at an interramp slope angle of 50°. The foliation dips at around 55°, a situation not unlike the Aitik mine. Controlled blasting and bench face reinforcement enabled the 50 degree-slope to be stable and only exhibit elastic displacements (Sharp, Bergeron and Ethier, 1987).

6.2.8 Nickel Plate Mine, British Columbia

This gold mine in southern British Columbia is owned and operated by Homestake Canada Ltd. The orebody occurs in a skarn-altered limestone and the host rock is also skarn, along with smaller lenses of diorite. The mineralized zone contains several ore lenses overlain on each other, which all dip at around 25-30° to the west. The bedding is poorly developed and the majority of faults and joint sets are steeply dipping. The rock is very strong and competent with uniaxial compressive strengths ranging between 250 and 450 MPa. The groundwater levels are believed to be low since the mine is located at the crest of a ridge (Figure 6.8).

The ore was previously mined underground using room and pillar mining. Underground mining ceased in 1955 leaving several open stopes on four different mining levels. The stopes are up to 90 meters long with 12 meter high pillars. Open pit mining began in 1986. Several small pits have been mined but currently only the North pit remains. Mining follows the plunge of the ore zones; hence, the pit is horse-shoe shaped with pit walls in the north, west and south. The north and the south walls are the major pit walls with a current height of 225 meters. Mining with new pushbacks is continued toward the west; hence the west wall becomes lower with each new pushback (see Figure 6.8). After the final (fourth) pushback, the north and south pit walls will be 280 meters high (from the ridge crest) and the pit will measure 600 by 450 meters in horizontal projection. Current production is around 5.5 Mton per year. The mine will close down at the end of 1996.
The slope is designed with bench face angles of 83° and a modified double bench design which results in interramp angles of 63°. The slope is mined through the underground stopes by blasting and collapsing the stope roofs. Larger stopes are first backfilled with waste rock. So far, there have not been any signs of large scale instabilities in the pit walls. Some small scale failures have been observed at the bench crests, but catch benches, which are cleaned periodically, have proven very effective. A monitoring system is being installed and miners have been trained to observe potential instability problems with the very steep pit walls. A failure of intermediate size has occurred in the east wall where a fault and an altered, weaker zone intersected, causing a wedge-type failure. This has been stabilized using tie-backs and steel rails (Golder Associates Vancouver, 1992; Stacey, 1993; Krauland and Sjöberg, 1995).
6.3 United States

6.3.1 Bingham Canyon, Utah

Kennecott Copper Corporation operates one of the largest open pit copper mines in the world in Bingham Canyon, Utah. The rock mass consist mainly of quartz monzonite (an intermediate igneous rock) along with some quartzite and limestone. Four different rock units were identified ranging from very competent to very weak rock. Intact rock strengths vary widely from 1 to 140 MPa. The rock mass is in general very fractured with both minor joints and larger fault structures. The mine has experienced several large scale instabilities, most notably in the Main Hill area in the NW corner of the pit. A large failure involving some 2 Mton of material occurred in 1967. This was followed by another slide in 1968 and one in 1974. The 1974 slide was partly stabilized by successful dewatering. Slope angles in this portion of the pit were around 29°. The large failures in the Main Hill area were believed to be a combination of rotational shear failure initiated at the weak toe by excessive water pressure, and plane shear failure along pre-existing discontinuities in the upper portions of the slope (Zavodni and McCarter, 1977).

Currently, the overall slope height is 2800 ft (850 meter) with an overall slope angle of 37°, but the upper 600 ft (180 m) has an angle of 50°. Rotational shear failure has occurred in the lower portion of the slope, whereas the upper, steeper portion show no signs of instability. A classification scheme developed by Call & Nicholas Inc. has been used to estimate friction and cohesion of the rock mass. Friction angles range from 28° to 46°, and average cohesion is around 0.1 MPa. The stress state is characterized by horizontal stresses being slightly higher than the vertical stresses ($K=1.1$). Furthermore, the water table is believed to be almost parallel to the slope and about 200 ft (60 m) behind the slope face, going from the toe to near the crest of the slope. There has also been some recent attempts to predict future failures using numerical analysis and the computer code FLAC but the results have not yet been published (Hustrulid, 1995).

6.3.2 Carlin Trend, Nevada

The Carlin Trend consists of several gold orebodies in a larger "belt" close to the towns of Carlin and Elko in Nevada. Newmont Gold operates a number of open pits in this area, the largest being the Gold Quarry and the Genesis/Blue Star Pit. Newmont Gold also owns half of the Post Pit, the other half being owned by Barrick Goldstrike which also operates the Post Pit. Average ore grade is around 1 ppm Au for all pits. The annual production from all of
Newmont Gold's mines in the Carlin Trend amounts to 165 Mton. Barrick Goldstrike produces 130 Mton per year from the Post Pit, resulting in a total production of over 300 Mton of rock per year from this area. The geology is very complex with siltstones, limestones, mudstones and breccias as the dominate rock types. The rock exhibits altering and weathering and has in general very low strength. There are, however, also intrusive rocks with uniaxial compressive strengths greater than 200 MPa, for example, in the west portion of the Post Pit. Faulting is very intense with an average spacing between faults of the order of 30 meters. There are also wider fault zones, up to 30 meters in width, containing clay minerals. The rocks are typically overlain by 60-200 meters of overburden.

Current pit depths are 240 meters in the Gold Quarry, 175 metres in the Genesis/Blue Star Pit and 210 meters in the Post Pit. Interramp slope angles vary from 28° to 50° at Gold Quarry, depending on rock type. In the Post Pit, interramp slope angles are all close to 35°. Several large scale failures have been observed in the Post Pit, involving up to 8 Mton of rock. These have been stabilized with buttresses and stepouts. Extensive drainage is conducted to keep the water table approximately 30 meters below the current pit bottom at all times. There are also zones with perched water in the pit walls. The Post Pit is currently planned to a final depth of between 430 (south portion) and 550 (north portion) meters (Krauland and Sjöberg, 1995).

6.3.3 Cleveland Cliffs Republic Pit, Michigan

This iron ore pit is located in hard, massive, igneous rocks. Careful blasting has enabled mining with nearly vertical pit walls of up to 120 meters height. Wire mesh and bolting has been used to protect against minor rock falls (Brawner, 1977).

6.3.4 Cyprus Bagdad and Sierrita, Arizona

Cyprus-Amax operates a copper and molybdenum mine (Cyprus Bagdad) some 160 km northwest of Phoenix. The average ore grade is 0.43% Cu and the annual ore production is 25 Mton. The orebody occurs within a stockwork of small quartz veins developed within a large quartz monzonite stock. The quartz monzonite has intruded into a complex of metamorphosed volcanic and sedimentary rocks. Some weathering and erosion has taken place and the rocks have been overlain by alluvial fans and gravel, and capped by rhyolite tuff and olivine basalt. In the south wall, which has experienced stability problems, the dominate rock type is a relatively competent quartz monzonite with an estimated uniaxial compressive strength of more than 100 MPa. Rocks in the upper section of the slope are weathered and
have compressive strengths less than 30 MPa. Several sets of discontinuities, mostly steeply
dipping, are also present.

The south wall was designed with an overall slope angle of 45°. Instability problems started
already in 1974 and have continued ever since. The total displacements amount to several tens
of meters. Typical movement rates are only about 2 to 3 mm/day, increasing to 10 to 20
mm/day when the slope is being actively mined. The current overall slope angle is 39° and the
slope height is approximately 250 meters. No uncontrollable failures have been experienced.
There is some evidence indicating that the failure mechanism is associated with large scale
toppling along a steeply dipping joint set. However, movements in the weaker upper portion
of the slope could also be due to a rotational shear failure. Drainage has been conducted but
this has only slightly reduced the movement rates. Increased movement rates follow when
mining activity occurs at the toe of the slope (Martin, 1990).

Cyprus-Amax also operates the Sierrita open pit in southern Arizona. The average ore grade
is 0.29% Cu and 0.03% Mo and annual production amounts to 80 Mton of rock of which 36
Mton is ore. Cyprus-Amax also owns the nearby Twin Buttes mine, which currently is in its
final mining stage (see Section 6.3.6). Ore from the Twin Buttes mine is used to supplement
production from the Sierrita mine. The current pit depth at the Sierrita mine is around 500
meters and the pit is planned to be mined down to 700 meters depth. However, ore grades are
decreasing with depth and the future of the mine is somewhat uncertain. The rock is relatively
hard and strong and the mine has experienced large scale failures, but no details regarding
these failures are currently available (Holmgren et al., 1994).

6.3.5 Robinson/Ely/Ruth Mining District, Nevada

The Robinson mining district is a group of open pit mines located just outside the town of Ely
in eastern Nevada. The area is interesting since it has a history of slope failures but also
because a full scale test on slope stability was conducted in one of the pits in the 1960's. At
that time, Kennecott Copper Corporation owned and operated the five larger open pits in the
area; Kimbley, Ruth, Liberty, Tripp and Veteran. The last two were subsequently merged into
one pit, the Veteran-Tripp pit (Figure 6.9). Kennecott closed the last of these mines in 1978.

Nevada Mining Company (a subsidiary of Magma Copper) bought the mining rights for the
Robinson district. Magma is currently cleaning up the old pits and constructing a new plant.
Mining of the old pits started in late 1995 with a production target of 50 Mton of rock per
year and an expected life of the project of 16 years. Mining commenced in the Liberty pit, and will be followed by the Veteran-Tripp pit, the Ruth pit and finally the Kimbley pit. All the copper orebodies are porphyry copper deposits in quartz monzonite host rocks. These rocks are relatively weak with various degrees of alteration and oxidation. In the west region of the district (Veteran-Tripp and Liberty pits) there is also a rhyolite intrusion which is stronger but the other rock types are weaker here compared to the east region of the district (Kimbley and Ruth pits). The average ore grades are 0.55% Cu and 0.3 g/ton of gold (Au).

![Diagram of mining district](image)

**Figure 6.9** Map showing the relative location of the open pits in the Robinson/Ely/Ruth mining district, Nevada (from Magma, 1995).

**Liberty Pit**

The Liberty Pit was mined until the late 1960's by Kennecott. Initially, the ore was mined underground using block caving. The pit has experienced several large scale failures, some due to caving of underground stopes and some due to the open pit mining itself. A large slide occurred in May, 1966 (Brawner, 1977). The slide involved somewhere between 5 and 12 Mton of material, and occurred in the form of a large wedge bounded by two faults. At the time of failure, the slope was 76 meter high and the overall slope angle was 33°. The slide
took place during the spring melt suggesting that increased groundwater pressure was a triggering factor. Movements were observed several years before the slide but the major failure displacements occurred in comparison very rapidly (Zavodni and Broadbent, 1978). Current pit dimensions are 1800 by 1500 meters and the pit depth is approximately 150 meter. Monitoring has shown that the pit walls are currently stable with movements of only a few millimeters over 3 years time. The new slope design to be used by Magma involves interramp slope angles of 26° near fault zones and up to 50° in the stronger rhyolite rocks.

**Veteran-Tripp Pit**

The Veteran-Tripp Pit was originally mined as two pits but subsequently merged into one pit. Mining was carried out by Kennecott from 1950 to 1971. The rock is more fine-grained here, but rock strengths are similar to the other pits. A large failure named the Northeast Tripp Slide occurred in 1970, and involved some 11.7 million m³ (approximately 30 Mton) of material. At the time of failure, the pit was about 220 meters deep. The failing volume was bounded by faults forming a large wedge. The intersection of these faults dipped at 13° toward the open pit, but the existence of a thick (6-12 meter) clay gouge in the fault zones enabled sliding. Measurements of the shear strength of the clay gouge and back-calculation of strength parameters revealed a friction angle of only around 10° and a cohesion of 0.06 MPa. The slide was at first accelerating, probably initiated by heavy precipitation, and then reached a recessive stage with decelerating displacements. This was most likely due to the cessation of mining at the slope toe and to the fact that rubblized rock came down into the pit which added some constraint to the toe of the failure (Miller, 1982). There has been significant raveling of the individual benches since the pit closed down but no more large scale failures.

**Ruth Pit**

The Ruth Pit is located in the east region of the Robinson district. It measures 1000 by 800 meters, with a depth of about 120 meters. Several faults transverse the pit. Massive raveling has been observed since Kennecott closed the pit in the 1970's, but no large scale failures. The pit is currently partly water-filled, and will be pumped dry before Magma continues mining.
Kimbley Pit

The Kimbley Pit is the smallest of the pits in the Robinson district. The pit is slightly elliptical, being approximately 500 meters wide in the east-west direction and 400 meters wide from north to south. Kennecott Copper Corporation conducted a joint research project with USBM (United States Bureau of Mines) in this mine, starting in 1961. The objective was to safely mine additional ore contained in the pit walls by steepening them from the original 45° overall slope angle. The Kimbley pit was chosen because it had not been mined for six years and had remained stable during this time. Also, it was located outside the stressed zone from other pits in the area (Figure 6.9). The dominate rock is monzonite, varying from altered to fresh. Young's modulus for the rock was determined from laboratory tests and was found to be in the range of 3.5 to 10.5 GPa, i.e., relatively soft rock. Flat-dipping faults, dipping away from the pit wall, were found in the western wall of the pit, as shown in Figure 6.10. In addition, a dominate, steeply dipping (60° to 70°) joint set was found to exist. At this time, the pit was generally quite dry with the water table at the pit bottom and only isolated areas with water inflow, mostly in the south wall of the pit. Based on this, USBM proposed interramp slope angles of 67° for the north and 60° for the west wall. Including the ramp and some minor modifications, the overall slope angles came out as 61° for the north wall and 56° for the west wall (Broadbent, 1967; Blake, 1968; Merrill, 1968, 1995).

![Figure 6.10](image-url) Vertical section through the Kimbley Pit, showing the original and the final (steepened) slope outline (from Blake, 1968).
An elliptical pit shape was chosen to accommodate slightly steeper angles in the north wall. Hence, the slope angles continuously increased from 45° in the south wall to 56° in the west wall and 61° in the north wall of the pit. In addition to steepening the walls, the pit was also deepened some 15 meters, from a pit depth of 150 meter to a final depth of 165 meters. Mining of this slope configuration started in February 1966 and was completed in October of the same year. Scrapers were used to remove the upper 30 meters of more weathered material and conventional drilling and blasting was used for the bottom section (Bellum, 1967). A total of 2.55 Mton were mined from the pit during this period (Merrill, 1968).

The slope was monitored carefully during the mining. Two instrumentation adits were excavated in the west wall and instrumented with borehole extensometers, stress monitoring cells and convergence measurement stations. Surface monitoring was also conducted as well as microseismic monitoring (Wisecarver, 1968; Wisecarver, Merrill and Stateham, 1969). This was followed by a comprehensive stability analysis (Blake, 1968; Broadbent, 1969; Merrill, 1995). Stress changes and induced displacements due to mining were, in general, quite small. Significant displacements were only recorded at points where the slope was both steepest and highest. Microseismic rates were also low and decreased to almost zero after mining was completed.

Measurement results confirmed the fact that the pit walls remained stable when steepened. During the mining, only two small failures occurred. One failure took place in the north wall where tensile cracking around a "nose" and a subsequent wedge failure was observed (Bellum, 1967; Merrill, 1995). This failure involved less than 500 m³ of material (approximately 1500 tons). The other failure occurred in the lower (90 meter high) south wall, which was in weaker material and designed with a 50° slope angle. This failure occurred in September, 1966 and involved some 3000 m³ (8000 tons) of rock in a soft and weak rhyolite zone. The failure surface was relatively shallow and sub parallel to the slope face, and was bounded by the weak Kimbley fault at the toe and laterally by stronger limestone, sandstone and porphyry. Failure developed over a period of 2 days but the main failure was relatively rapid. Initially, the failing volume moved as a rigid body, rotating about the toe of the slope, but then it disintegrated as more cracking developed. A contributing factor to this failure could have been the higher water pressure known to exist in the south wall. Back-analysis of the failure in the south wall yielded a rock mass friction angle of between 35° to 42° and a cohesion of 0.05 to 0.07 MPa (Hamel, 1971; Pariseau and Voight, 1979). Both of the above failures occurred when mining was almost completed (Merrill, 1995).

After the pit wall steepening, the two adits where charged with explosives and blasted in 1970 (a so called coyote blast). This effectively undercut the west wall of the pit and the slope
angle was thus increased to 69°. The slope failed and caved back to a residual angle of around 58°. Copper was then recovered from the caved material using leaching (Michaelson, 1979; Broadbent and Zavodni, 1982). Today, the pit is partly backfilled since it has been used as a waste dump by Kennecott. Slight raveling of individual benches can be observed but there are no signs of large scale instabilities (Krauland and Sjöberg, 1995).

6.3.6 Twin Buttes, Arizona

This mine is located just south of Tucson, Arizona and was formerly known as the Anamax mine. The typical rock types are quartzite, conglomerate, tuff, andesite and dacite. These are overlain by up to 135 meters of alluvial gravel. The upper part of this mine is thus very soft and is removed with scrapers. The underlying rock is stronger with uniaxial compressive strengths of up to, and exceeding, 140 MPa (unweathered). Numerous faults and several joint sets are found throughout the rock mass. Seegmiller (1972, 1979) reported a large slope failure which occurred in the south slope of the pit in 1970-1971. The annual production at that time amounted to 12 Mton of ore. The pit depth was 330 meters, where the upper 110 meters consisted of alluvial gravel. Slope angles of 37° in the overburden and 45° in the rock were employed. The failure developed over several years. Failure initiated in the form of small wedge failures which grew to a massive block flow failure. The failure was very shallow, approximately 18 meters deep, and ran parallel to the slope face. Involving almost the entire slope height, the total failure volume was approximately 3 Mton.

Another major slide occurred in 1974. This failure involved over 10 Mton of material and occurred in a relatively weathered rock. Evidence of toppling failure and sliding on discontinuities was found. Water was believed to be a governing factor as well as excessive blast damage from early workings (Brawner, 1977). There have also been other failures of large scale in this rock which probably is weaker than the typical rock at Aitik, mostly due to weathering (Call, 1994). The mine is currently in its final mining stage and is now operated by Cyprus-Amex (see also Section 6.3.4).

6.4 Mexico

6.4.1 Las Encinas, Colima

Las Encinas is a nearly circular pit with a diameter of 400 meters which was mined to about 200 meters in depth. In an attempt to recover more ore, the overall slope angle was increased
from $55^\circ$ to $80^\circ$. Heavy reinforcement in the form of cable bolts, wire mesh and Split Set bolts were installed to keep wedges and smaller blocks in place. The rock is considered slightly weaker than the rocks at Aitik. A large scale slope collapse occurred in May, 1994, with only 2 months of mining remaining. Deformation was observed before failure but the final collapse was very rapid (Call, 1994).

6.5 Chile

6.5.1 Chuquicamata

Codelco operates the worlds largest metal mine and open pit at Chuquicamata in northern Chile. This copper mine currently produces around 140 Mton of rock annually, of which around 56 Mton is ore. The current pit depth is 658 meters (May, 1994). Future mining plans call for a pit depth of 788 meters in the year of 2012 and a final depth in excess of 1000 meters. The Chuquicamata mine is divided by a fault zone into two distinctly different geomechanical environments. The East Sector is fairly competent (granodioritic rock) with compressive strengths for the ore of the order of 60 to 80 MPa. This rock mass tends to behave elastically and stability problems are governed by failure along pre-existing joint planes. Joints are also more persistent and continuous in the East Sector compared to the rest of the pit. The West Sector also consists of granodiorite, but of lower strength. The fault zone is much weaker and exhibits plastic behavior. In the West wall, there is also a very continuous joint set dipping at around $70^\circ$ into the wall. Compressive strengths vary from around 30 MPa for the ore and the fault zone to 100 MPa for the granodiorite. The West side of the pit displaces continuously (stable failure), sometimes as much as 100 mm per day. Consequently, the slope angles differ in the East and the West Sector of the mine. In the West wall an overall slope angle of $37^\circ$ is used, whereas slope angles ranging from $40^\circ$ to $46^\circ$ are used in the East wall (Rapiman, 1993).

The mine has experienced several large scale slope failures during the years. One of the most famous was the large failure of the East wall in 1969 (Kennedy and Niermeyer, 1970). The total amount of failed material was around 1 Mton, but the volume of rock which experienced substantial movements corresponded to somewhere between 12 and 15 Mton of material (Broadbent and Zavodni, 1982). The complex failure surface was composed of pre-existing fractures and major structures. Careful monitoring and a relatively slow failure process (up to the point of collapse) resulted in a minimum of production delays due to this failure. The East wall is still experiencing wedge failures both on the bench scale and multiple bench scale.
More recent large scale instabilities have been concentrated to the West wall of the pit. These failures are a combination of structurally controlled failures and failure through the intact but weak rock in the fault zone. The failure mechanism is believed to be toppling along the steeply dipping continuous joints in the West wall and shearing along a failure surface almost parallel to the slope face and at a depth of around 100 meters. This is accompanied by tension cracks at the crest and heaving and failure of the fault zone at the toe of the slope, as shown in Figure 6.11. The weaker fault zone is believed to act as a "passive" wedge restricting sliding of the upper failure zone (Kvapil and Clews, 1979; Call et al., 1993; Board et al., 1996).

Figure 6.11  Conceptual sketch of the failure behavior of the west wall at Chuquicamata (after Board et al., 1996).

6.6  South Africa

6.6.1  Palabora

The Palabora Mining Company Ltd. operates a copper open pit in the north-east corner of the Republic of South Africa. Mining started in 1966 and current annual production amounts to around 29 Mton of ore and a total of 92 Mton of material (ore and waste rock). The orebody is about 1440 meters long and 800 meters wide and the pit is thus slightly elliptical in shape. Current mining is planned for a final pit depth of 836 meters which would be reached in year 2000. The interramp slope angles used in the pit vary from 37° at the crest to 58° near the pit bottom (Leroy and Lill, 1990).
The igneous rocks at Palabora are dominated by carbonatites (ultra-basic igneous intrusive dykes) of various types (Evans, 1980), such as banded carbonatite and transgressive carbonatite. There are also zones of foskerite (coarse-grained basic rock) as well as dolerite dykes. The copper content varies from 0.3 to 1.0% in these rock types. The rocks are generally unweathered with uniaxial compressive strengths of the order of 70 MPa for the foskerite, 130 MPa for the carbonatites and up to 300 MPa for the dolerite dykes. Weaker and more weathered rocks are found near the ground surface. Three major faults have been identified in the pit and several sets of joints have been delineated. Slopes with heights of up to 370 meters have performed very well and without major instabilities for interramp slope angles in excess of $50^\circ$. Monitoring of movements have shown displacements of up to 200 mm in these slopes with movement rates of about 25-35 mm/year (Martin, Steenkamp and Lill, 1986; Du Plessis and Martin, 1991; Stacey, 1993). The Palabora Mining Company is presently considering plans for continued underground mining of this orebody using block caving (McKinnon, 1994).

6.7 Papua New Guinea

6.7.1 Bougainville

The Bougainville open pit is currently planned to a depth of 700 meters. The rock is intensively fractured with very low strength. An annual rainfall of 5000 mm adds to the complexities at this mine. The initially designed slope angles of $45^\circ$ for competent rock and $30-35^\circ$ in weathered rock had to be revised as a slide involving more than 10 million m$^3$ (approximately 25 Mton) occurred (Franklin and Dusseault, 1991; Stacey, 1993). The mine has been closed for some time but is currently being reopened.

6.8 Finland

6.8.1 Kemi Mine

Outokumpu Oy operates a chromium open pit in Elijärvi, 10 km NE of Kemi, Finland. The dominate rocks are basic to ultrabasic intrusions in granite and schist host rocks. The rock types range from hard, high-strength albite ($\sigma_c=147$ MPa) to very weak talc-carbonate rocks ($\sigma_c=12$ MPa). The annual production is around 1 Mton of chromium ore and the depth of the pit as of 1988 was 120 meters. The length and width are 1400 and 370 meters, respectively.
The pit slopes are currently stable and the recommended slope angles range from 35° to 55° (Feng, 1988).

6.9 Norway

6.9.1 Bjørnevatn dagbrudd, Kirkenes

A/S Sydvaranger operates this iron ore open pit, which is located in north-east corner of Norway, close to the Russian border. The ore is a taconite (quartz-magnetite) with 32% Fe, and the host rock is gneiss. The strength of both the ore and the waste rock is very high. The orebody consist of two lenses, one to the east and one to the west. Both lenses have been mined and the current pit depth is 190 meters in the western pit and 210 meters in the eastern pit. Interramp slope angles vary from 47° to 54°. The current annual production is 2.0 Mton of ore with an additional 0.5 Mton of waste rock. Stability problems has included slip along schistosity planes in the footwall, mainly affecting bench stability. A larger failure has occurred in the footwall of the eastern pit, involving some 70,000 m³ of rock. Failure was governed by slip along a large discontinuity, oriented parallel to the footwall. This failure resulted in the production being terminated for 1 week. A switch to underground mining of the western ore lens has started. Poor financial results over a long period of time have, however, lead to the decision to close the mine in 1996 (Hansen, 1994; Krauland et al., 1996).

6.9.2 Ørtfjell dagbrudd, Rana

Rana Gruber A/S operates the Ørtfjell iron ore open pit, which is located in northern Norway. The rock at Ørtfjell is dominated by various schists, often with high mica content along with marble. Consequently, there is a very pre-dominate schistosity which runs parallel to the footwall. The intact rock strength is relatively high and rocks are moderately stiff. Young's modulus is in the range of 12-52 GPa and the uniaxial compressive strength is approximately 49-75 MPa. The initial mine plan called for a final pit depth of 500 meters for this iron ore mine. A thorough stability analysis was conducted at the startup of mining to aid in the design of the overall slope angle. In this study, particular interest was focused on the influence of rock stresses on the wall stability. This was due to the relative absence of large structural features and the fairly high tectonic stresses measured in this mountainous region. The planned overall slope angle was 51° and a stability analysis confirmed that the pit would remain stable even with a slight increase in overall slope angle (Broch and Nilsen, 1977; Nilsen, 1979; Broch and Nilsen 1982).
At present, the pit is 230 meters deep and the current mining plans call for a final pit depth of (only) 260 meter. The annual production is 1.4 Mton of ore and an additional 0.6 Mton of waste rock. No stability problems have been observed and the interramp angles have been increased to 54°. This corresponds to the overall angle in the hangingwall which has no ramps. After the pit is mined out (in 1998), a switch to underground mining will be made. (Hansen, 1994; Krauland et al, 1996).

6.9.3 Tellnes dagbrudd, Hauge i Dalane

Titania A/S operates the Tellnes open pit, located south of Stavanger in southern Norway. The ore is a norite-ilmenite intrusion from which ilmenite (FeTiO₃) is being extracted. Ilmenite is used for manufacturing pigment. The surrounding rock is anorthosite, frequently intersected by diabase dykes. Intact rock strengths are generally high, but the rock mass is also relatively fractured, in particular the anorthosite. The ore extends to approximately 300 meters depth and mining has reached around 155 meters depth as of 1996. The interramp pit slope angles vary between 50° and 55°. The annual production amounts to 3 Mton of ore and an additional 3 Mton of waste rock.

Local instability problems in benches have been experienced. A slightly larger slope failure started to develop in 1989 following a heavy rainfall. A large rock wedge formed by intersecting discontinuities started to move. The dominate discontinuities dipped at 30° to 35° and striked sub parallel to the footwall slope. The discontinuities contained clay minerals, some which also exhibited swelling behavior. It was estimated that the moving mass was of the order of 300 000-400 000 tons of rock. Measurable subsidence could be recorded close to a haulage ramp and in close vicinity to the crusher. A stability analysis was undertaken and support recommendations were proposed. Due to difficulties in accessing the slope face, it was recommended to drill large diameter vertical holes from the haulage ramp and install several rebar rock bolts in each hole. This was to be accompanied by extensive scaling of the loose rock masses above the haulage ramp (Nilsen and Hagen, 1990).

The proposed reinforcement was never undertaken due to the high costs associated with this. Instead, approximately 110 000 tons of the unstable volume was mined out. Also, a safety berm was constructed below the unstable area. A new crusher is currently being built in a safe area. The unstable area is monitored using a survey network, and wire extensometers are used to monitor tension cracks. Smaller failures have also been noted more recently in this area (Krauland et al., 1996).
6.10 Spain

6.10.1 Aznalcollar Mine, Seville

The Aznalcollar mine is located in southern Spain, northwest of Seville, and is operated by Andaluza de Piritas SA (APIRSA), a subsidiary of Boliden Mineral AB. The lead-zinc orebody strikes east-west and dips to the north at an angle of 45° to 60° (becoming steeper with depth). The dominant lithology of the footwall is slate with a well-developed cleavage, or foliation. The cleavage discontinuities are generally planar, dip between 45° and 70°, and are continuous over at least 80 meters length. In addition to the slates, phyllites and quartz veins occur in the footwall, along with a body of intrusive felsite. In the hangingwall, the geology is dominated by slates, tuffs, felsites and rhyolites. The rock material is slightly weathered, and strengths are generally low (estimated uniaxial compressive strength of 25 MPa for the slate). In addition to the cleavage/foliation, a total of six joint sets have been identified. Groundwater conditions at Aznalcollar are very complex, with two phreatic surfaces present, which in turn have been modified by mining of the pit. Heavy rainfalls during periods of the year add to the complex hydrogeological conditions.

The Aznalcollar mine started as an open pit operation in the early 1970's, but portions of the orebody was mined underground much earlier than this. The pit currently measures approximately 1300 by 700 meters, and the pit depth is around 220 meters. The overall slope angle is approximately 30-35° for the footwall, and 45° for the hangingwall. The pit is planned to be mined to a final depth of around 270 meters. Current production amounts to 2.5 Mton of ore annually. The stripping ratio is around 4:1 giving a total annual production of approximately 12 Mton of rock.

Despite the relatively moderate slope height, the Aznalcollar mine has suffered several large scale failures in the footwall slope. The first large scale movements were observed in October 1979, and these were followed by new failures in January 1983, December 1987, and November 1988 (Krauland 1987, 1988; Golder Associates, 1989a; 1989b, 1990). The corresponding slope heights at these instances were approximately 40, 80, and 180 meters. A comprehensive monitoring program has been in use at the mine, involving aerial photogrammetry, surface stations and inclinometers for measuring deformations, and piezometers for recording changes in groundwater pressure. It has thus been possible to deduce the shape of the failure surface. The failure surface proved to be relatively shallow and almost parallel with the slope dip, and with a tension crack at the slope crest. In fact, several failure surfaces sliding on top of each other could be observed in the footwall (Figure 6.12).
The failure mechanism is relatively complex and probably involves failure both through intact rock and along pre-existing discontinuities. The tension cracks are believed to develop along the cleavage planes, but the major portion of the failure surfaces cuts across the dominate cleavage planes in the footwall. Some instabilities of moderate scale have also been noted in the hangingwall. The large scale failures of the footwall have probably been initiated by changes in the mining geometry. Large increases in movements have, however, in most cases been preceded by periods of heavy rainfall, suggesting that changes in groundwater pressures triggered larger displacements along the developed failure surface. During dry weather, the slope moves at a rate of about 5 mm/day, but after periods of heavy rainfall, the movement rate can be as high at 170 mm/day. The total displacement of the slope at the end of this measurement campaign (1990) was of the order of 7 meters.

In July 1992, slope movement rates increased dramatically from 3 to 5 mm per day to up to 40 mm/day (July 22). This was followed by a period of extremely high movement rates between July 22 and July 24 when the maximum recorded movement rates reached 1680 mm/day.
corresponding to 66 mm/hour. Unlike most of the other failures, this failure could not be correlated to any rainfall events (Golder Associates, 1992, 1995).

Despite the large displacements associated with these failures, the pit is still in operation. The relatively slow and non-violent, stable failure behavior has enabled mining to continue with only slight interruptions during the failures. Stability analysis has so far included (1) fairly simple limit equilibrium models with various groundwater conditions assumed (Golder Associates, 1989a, 1995), and (2) attempts to simulate the slope behavior using numerical models (Proughten, 1991, McCullough, 1993). A revised detailed monitoring program has been installed to secure final mining (August-September, 1996) of the eastern portion of the pit. The slope angles will actually be increased slightly during final mining, as the ore dip is steeper at depth. Monitoring has also been installed to watch the hangingwall during final mining. The hangingwall is significantly stronger and has not yet experienced any failures.

6.11 Sweden

6.11.1 Leväniemi, Svappavaara

This iron ore pit is located some 60 km north of the Aitik mine. It was shut down in 1983 due to the low prices of iron ore, and there are no current plans to reopen the pit. Slope design has been carried out by Call & Nicholas Inc., using a probabilistic approach similar to what was used in the Aitik studies (Hustrulid, 1995).

6.11.2 Kiirunavaara, Kiruna

LKAB operates one of the world's largest underground mines in Kiruna, northern Sweden. The Kiirunavaara mine was initially mined as an open pit, but during the 1950's, a switch to underground mining using sublevel caving was conducted. The Kiirunavaara iron orebody is about 4000 meters long and extends probably to more than 2000 meters depth below the ground surface. The orebody is relatively strong magnetite surrounded by competent quartz porphyry on the hangingwall and syenite porphyry on the footwall. Weaker zones also occur both in the ore and in the surrounding rocks.

The mine has experienced large scale stability problems which to some extent are similar to large scale slope failures. Sublevel caving requires that the hangingwall continuously cave as the ore is mined. This has the unwanted side-effect of large subsidence on the ground surface,
which has been observed in Kiruna. More serious is the fact that instability problems of large scale have also begun to occur in the footwall of the Kiirunavaara mine (Figure 6.13).

![Diagram of large scale footwall and hangingwall failures at the Kiirunavaara mine](image)

*Figure 6.13 Large scale footwall and hangingwall failures at the Kiirunavaara mine (after Herdocia, 1991; Dahnér-Lindqvist, 1992).*

Ventilation shafts, ore passes, and portions of the main access ramps are all located in the footwall relatively close to the orebody and are thus subject to these instabilities. The mechanisms for failure are not known in detail, but it is suspected that the failure mode is a form of large scale rotational shear. Limit equilibrium analysis assuming rotational shear failure with a circular failure surface have also been conducted and it has been possible to predict new failure surfaces with some accuracy (Dahnér-Lindqvist, 1992). A more recent approach involves a limit equilibrium method which analyzes the footwall and the hangingwall at the same time and explicitly takes into account the effect of the caved rock (Lupo, 1996). There is still some uncertainty regarding the strength parameters for the rock mass, which gives this problem many similarities with the situation at Aitik. In this case, however, the strength parameters can be back-calculated from previous failures. The rock types and rock strengths are believed to be relatively similar to what can be expected at Aitik, which makes this a very good case. There is one particular issue which must be addressed separately and that is the potential support provided by the caved rock. This must be quantified before determined strengths can be applied to an open pit mine.
6.11.3 Malmberget

LKAB also operates the Malmberget iron ore mine. The mine consist of several relatively large orebodies which are currently being mined using sublevel caving. Similar to the Kiirunavaara mine, caving of the hangingwall has occurred accompanied by extensive ground subsidence. In particular the Kapten orebody has caused extensive surface caving and portions of the Malmberget city have been moved due to this. The Kapten orebody was also first mined as an open pit. Unlike the Kiirunavaara mine, no footwall failures have been noted at Malmberget (Rutqvist and Sjöberg, 1987).

6.11.4 Other Mines in Sweden

Among the open pits which are actively being mined today in Sweden can be mentioned the Långdal mine owned and operated by Boliden Mineral, and the Björkdal mine, owned and operated by Terra Mining. Both of these pits are very small in comparison to Aitik, and have not suffered any large scale failures. The same is true for other active open pits in Sweden today.

Other underground sublevel caving mines, such as the Grängesberg mine, has experienced large scale failures, and can perhaps be used as cases from which to obtain large scale rock mass strength parameters (see Section 3.5 and Herdocia, 1991). In this group of cases one also finds the Långsele mine, which experienced an uncontrollable large scale hangingwall failure, see Section 3.4 (Krauland, 1975; Kolsrud and Krauland, 1979). Both the Grängesberg and Långsele mines are now closed, and back-calculation must rely on previously reported data.

6.12 Summary

A condensed summary of the slope geometry and failure characteristics from some of the cases listed above is given in Tables 6.1 and 6.2. The intact rock strength obtained from laboratory tests is given for comparison. Although the intact strength is not a perfect measure of the rock mass strength it does give some indication of whether the rock in question is strong or weak. There is quite a large scatter in slope heights, slope angles and corresponding rock strengths. At this stage, it is not possible to draw any definite conclusions from Tables 6.1 and 6.2, but it is noteworthy that there are several mines with similar slope angles as Aitik but in weaker rock.
Table 6.1  Summary of case studies outside Europe. The slope height is the height of
the slope at failure or the currently highest stable slope in the pit.

<table>
<thead>
<tr>
<th>Mine</th>
<th>Slope Height [m]</th>
<th>Slope Angle [°]</th>
<th>(\sigma_c) [MPa]</th>
<th>Failure type and comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Afton Mine</td>
<td>170/300</td>
<td>45 (O)</td>
<td>20-110</td>
<td>Unstable failure</td>
</tr>
<tr>
<td>Brenda Mine</td>
<td>335</td>
<td>45 (O)</td>
<td>150</td>
<td>Unstable failure</td>
</tr>
<tr>
<td>Cassiar Mine</td>
<td>180/370</td>
<td>42 (O)</td>
<td>80</td>
<td>Slow and stable failure</td>
</tr>
<tr>
<td>Lornex Pit, Highland Valley</td>
<td>380</td>
<td>35(I); 30(O)</td>
<td>(3)-140</td>
<td>Slow and stable failure</td>
</tr>
<tr>
<td>Valley Pit, Highland Valley</td>
<td>350</td>
<td>38(I); 35(O)</td>
<td>(3)-140</td>
<td>Slow and stable failure</td>
</tr>
<tr>
<td>Highmont Mine</td>
<td>60-110</td>
<td>40 (O)</td>
<td>1-140</td>
<td>Slow and stable failure</td>
</tr>
<tr>
<td>Island Copper, South Wall</td>
<td>365</td>
<td>40 (O)</td>
<td>Medium</td>
<td>Slow failure; Stepout</td>
</tr>
<tr>
<td>Island Copper, North Wall</td>
<td>500</td>
<td>50 (O)</td>
<td>Medium</td>
<td>No failures</td>
</tr>
<tr>
<td>Jeffrey Mine, Southeast Wall</td>
<td>390</td>
<td>30 (O)</td>
<td>Low</td>
<td>Multiple, stable failure</td>
</tr>
<tr>
<td>Jeffrey Mine, North Wall</td>
<td>300</td>
<td>50 (O)</td>
<td>15-50</td>
<td>No failures</td>
</tr>
<tr>
<td>Nickel Plate Mine</td>
<td>225</td>
<td>63 (I)</td>
<td>250-450</td>
<td>No failures</td>
</tr>
<tr>
<td>Bingham Canyon</td>
<td>670/850</td>
<td>37 (O)</td>
<td>Low</td>
<td>Slow and stable</td>
</tr>
<tr>
<td>Gold Quarry, Carlin Trend</td>
<td>240</td>
<td>28-50 (I)</td>
<td>Low</td>
<td>Slow and stable failure</td>
</tr>
<tr>
<td>Post Pit, Carlin Trend</td>
<td>210</td>
<td>35 (I)</td>
<td>Low</td>
<td>Slow and stable failure</td>
</tr>
<tr>
<td>Cyprus Bagdad</td>
<td>500</td>
<td>45 (O)</td>
<td>30-100</td>
<td>Slow and stable failure</td>
</tr>
<tr>
<td>Kimbley Pit, Ely</td>
<td>150</td>
<td>56 (O)</td>
<td>Low</td>
<td>No failure</td>
</tr>
<tr>
<td>Ruth Pit, Ely</td>
<td>120</td>
<td>-</td>
<td>Low</td>
<td>Only bench raveling</td>
</tr>
<tr>
<td>Liberty Pit, Ely</td>
<td>76</td>
<td>33 (O)</td>
<td>Low</td>
<td>Large failure</td>
</tr>
<tr>
<td>Veteran/Tripp Pit, Ely</td>
<td>220</td>
<td>-</td>
<td>Low</td>
<td>Large failure</td>
</tr>
<tr>
<td>Twin Buttes</td>
<td>330</td>
<td>37-45 (O)</td>
<td>140</td>
<td>Slow, shallow</td>
</tr>
<tr>
<td>Las Encinas</td>
<td>200</td>
<td>80 (O)</td>
<td>-</td>
<td>Unstable</td>
</tr>
<tr>
<td>Chuquicamata - East Sector</td>
<td>645</td>
<td>40-46 (O)</td>
<td>60-100</td>
<td>Multiple bench</td>
</tr>
<tr>
<td>Chuquicamata - West Sector</td>
<td>645</td>
<td>37 (O)</td>
<td>30-100</td>
<td>Slow and stable</td>
</tr>
<tr>
<td>Palabora</td>
<td>370</td>
<td>37-58 (O)</td>
<td>70-300</td>
<td>No failures</td>
</tr>
</tbody>
</table>

O = Overall Slope Angle
I = Interramp Slope Angle
Table 6.2  Summary of case studies in Europe. The slope height is the height of the slope at failure or the currently highest stable slope in the pit.

<table>
<thead>
<tr>
<th>Mine</th>
<th>Slope Height [m]</th>
<th>Slope Angle [°]</th>
<th>$\sigma_c$ [MPa]</th>
<th>Failure type and comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kemi Mine</td>
<td>120</td>
<td>35-55 (O)</td>
<td>12-147</td>
<td>No failures</td>
</tr>
<tr>
<td>Bjørnevatn</td>
<td>190-210</td>
<td>47-54 (I)</td>
<td>High</td>
<td>Multiple bench scale</td>
</tr>
<tr>
<td>Ørtfjell</td>
<td>120-230</td>
<td>54 (I)</td>
<td>49-75</td>
<td>No failures</td>
</tr>
<tr>
<td>Tellnes</td>
<td>155</td>
<td>50-55 (I)</td>
<td>High</td>
<td>Multiple bench scale</td>
</tr>
<tr>
<td>Aznalcollar</td>
<td>210</td>
<td>30-35 (O)</td>
<td>25</td>
<td>Multiple, stable failure</td>
</tr>
<tr>
<td>Aitik - Hangingwall</td>
<td>225-270</td>
<td>51-56 (I)</td>
<td>70-140</td>
<td>No failures</td>
</tr>
<tr>
<td>Aitik - Footwall</td>
<td>225-270</td>
<td>42-49 (I)</td>
<td>70-140</td>
<td>No failures</td>
</tr>
</tbody>
</table>

O = Overall Slope Angle  
I = Interramp Slope Angle

Some of the mines can be grouped by similar failure behavior. The Chuquicamata, Jeffrey and Aznalcollar mines all have had failures with fairly shallow failure surfaces, running almost parallel to the slope outline. In these cases, movements have been relatively slow. The rocks are quite weak in these cases and the slope angles generally less than 40°. Among the slightly stronger rock types one finds the Afton, Brenda and Highland Valley mines, all of which have suffered from what is believed to be large scale toppling failures. The failure behavior is different though, with rapid, unstable failure at Afton and Brenda, and a slow, stable failure process at Highland Valley.

A few of the mines have already been visited within the current research project (Bjørnevatn, Carlin Trend, Highland Valley, Island Copper, Nickel Plate Mine, Robinson District, Tellnes and Ørtfjell). Candidates for future visits are, amongst others, the Jeffrey Mine in Canada and Palabora in South Africa.

The Aznalcollar mine is a candidate for a more in-depth study and perhaps back-analysis of failures and strength parameters, since measurement data and failure observations are very good, although the rock is much weaker than at Aitik. It is also worthwhile to try to obtain more information about the stability analysis that has been carried out for the Jeffrey Mine, as the failures at this mine show several similarities with the large scale failures at Aznalcollar. Other interesting cases are the Afton and Brenda mines in Canada, and Las Encinas in Mexico, since all three have experienced uncontrollable failures in relatively high strength rock. A major drawback with these mines is that they all are closed and it might be difficult to obtain
more information about the failures. A potential case study close to home is the hard and strong rock at the Kiirunavaara sublevel caving mine. Provided that the effect of the caved rock can be quantified, this case can prove to be enormously valuable in quantifying the large scale rock mass strength parameters.

It is also clear that the above list of cases is far from complete. It is likely that there are more cases, for example, in Australia and Canada, which can be of use for this project. A continued search for better case studies is thus necessary.
7 RECOMMENDATIONS FOR FUTURE RESEARCH

7.1 Current Status and Future Needs

Design of slopes is arguably one of the areas in the field of rock mechanics which have received the most attention during the last few decades. The massive amount of work conducted has primarily focused on structurally controlled failures in small to moderate rock slopes. Considerably less efforts appear to have been devoted to large scale slopes. From the reviewed literature, the current status of the design of large scale open pit slopes can be summarized as follows:

• The knowledge of the mechanisms for large scale rock slope failures is limited, in particular for slopes in hard, brittle and jointed rock masses.

• The strength of large scale rock masses is poorly known, and there are few, if any, methods available for determining the large scale strength with the required accuracy for use in the design of ultimate pit slope angles.

• There are a large amount of design tools available, such as limit equilibrium methods and numerical model analysis. Depending on what the mechanisms for failure are, one has the choice of several design methods which appear to be reasonably accurate.

The problem is thus not one of choosing design tool, but one of identifying the failure mechanisms and determining the rock mass strength. Nevertheless, the development toward better analysis tools will without doubt continue. Combinations of limit equilibrium and numerical analysis is one likely path of development. Furthermore, a numerical tool which can handle failure propagation both along discontinuities and through intact rock will certainly be available in the future. The latter will enable more investigations into the fundamental mechanisms of failure in rock slopes.

The question of how precise design methods one actually needs must, however, be raised. Is it perhaps adequate to use limit equilibrium methods for predicting the overall shape and approximate location of the failure surface? A simpler method has many practical advantages compared to more complex, although theoretically more satisfying, analysis methods. It is also clear that complex numerical modeling is not suitable for use in practical mine design. Therefore, development of a simple and versatile design methodology which includes guidelines on the choice of strength parameters as well as tools for determining the slope angle, is very much warranted.
More important than all of the above is the determination of the rock mass strengths for large scale slopes. In Section 3.5, it was shown that the choice of strength parameters have an extremely large impact on the calculated stability for the slope. The development toward more sophisticated analysis tools should therefore be complemented by equal efforts devoted toward the determination of rock mass strength. Closely linked to the rock mass strength are the failure mechanisms in large scale slopes. The discovery of "new" failure modes such as deep seated toppling in high rock slopes is somewhat alarming. It is not certain whether also other (and potentially more dangerous failure modes) can develop when steepening up high pit slopes in hard, jointed rocks. This is definitely an issue which needs to be studied in more detail that what has been done to date.

To summarize, the following issues need to be addressed to advance the status of large scale slope design:

- Quantification of the importance of geologic structure of different scale and type, for different slope heights. An interesting outcome of this work would be to see if there is a continuous transfer from structurally controlled failures to rock mass failures as the physical scale increase, or if there is a more distinct "transition point"

- Collection of field data, in particular back-calculated rock mass strengths, from pit slopes in hard, jointed rock. These data could be compiled in a database for use in the design of slopes which have not yet experienced failures.

- Development of quantitative criteria for the prediction of whether failures will be slow and stable, or rapid and uncontrollable.

- Quantification of the factors contributing to the strength of large scale rock masses. From the above list of suggested activities, this would be the largest and most important contribution. However, the difficulties associated with this are large. Hence, a solution to this problem is not anticipated in the foreseeable future.

7.2 Thesis Project Proposal — Objective and Scope of Work

One of the objectives of this report was to provide the basis for a thesis project proposal. The thesis project must satisfy both the requirements of practical applicability for the Aitik mine, as well as the scientific and academic requirements for a doctoral thesis. For the Aitik mine, the criteria for bench design, involving double benching with catch benches, appear to be fairly
acceptable (Section 1.2). As was stated by Call (1994), important activities for continued bench and interramp slope design is a continuous update of structural data, strength properties, and hydrologic conditions throughout the pit. These are activities that to a large extent can be carried out by the staff at Aitik. Following this, it may be necessary to update or revise the design criteria used by Call & Nicholas, Inc., but this does not constitute a research task itself. The question of controlling blast damage has also been addressed previously by the Boliden staff. Results are promising although some implementation work still remains.

The main issue remaining is the question of how to assess the large scale stability of the pit walls. This issue has been treated very superficially in previous studies and deserves more attention, in particular now that the pit is being planned for the final mining. Assuming that small scale instabilities can be designed for accurately and that failures along the foliation can be kept under control, the thesis project can be focused entirely on the large scale aspects of slope instabilities at Aitik. In view of what the mine is currently facing in terms of mine planning for the next ten years of mining, a methodology for the design of ultimate pit walls is very much warranted. The time constraints for all planning activities are very hard. The current, final pushback of the footwall will carried out during 1996 to 1997. Consequently, all changes to the ultimate slope angles must be done soon. For the hangingwall, however, several pushbacks will be mined before the final wall is being formed. This means that a developed design methodology also has a long term value for the design of the hangingwall (and if additional pushbacks are decided upon for the footwall).

From a mining perspective it is important to establish criteria both for failure initiation (location and shape of failure surface) and for failure kinetics (stable or unstable, controllable or uncontrollable failure). Judging from the reviewed literature, establishing criteria for failure kinetics would be a much more difficult task. To limit the scope of work to a reasonable amount, and still provide results which are beneficial for the mine, consider the following statements:

i. The resulting failure surface for a large scale slope failure will probably be curved but the conditions and criteria for this are not known in detail, nor are the processes leading to the formation of this failure surface.

ii. The shape and the location of the failure surface is not known.

iii. The stress level at which failure initiates and criteria for failure propagation are not known.
iv. Large scale slope failure in hard rock (such as Aitik) will probably be uncontrollable (steep slopes in hard, brittle rocks).

The last statement is a very reasonable assumption judging from experiences in other open pit mines in similar rock types. It is also a conservative assumption since it implies that once failure has initiated there will be no time for remedial measures or even evacuation of the pit. From this follows that it is necessary to design the slopes at Aitik for failure initiation, to prevent failure from developing fully.

The thesis project can thus focus on the first three issues. This involves studying the failure mechanisms for large scale slope failures in more detail and quantifying the strength parameters based on identified failure mechanisms. Even this is a very ambitious task and the approach to take must be outlined in more detail. Since the most important factor to determine is the rock mass strength, the largest efforts should be focused on this. Development of a new theory which describes the strength of the rock mass explicitly is not, however, a realistic target. Rather, a semi-empirical approach will be used in which strength data from Aitik, back-analyzed strengths from other open pits, and theoretical and numerical analyses, are combined to arrive at a methodology for choosing slope angles. Some of the activities that are anticipated for this approach are outlined below. As the project continues this list will be revised and more detailed work plans made.

• Develop a geological/geomechanical model for the Aitik mine. This includes quantifying the rock mass in terms of geology, jointing, intact rock strength, discontinuity shear strength, groundwater conditions etc.

• Back-analyze previous failures at Aitik. These failures are of small scale, but back-analysis of these can yield additional data on the shear strength for dominant discontinuities at Aitik.

• Gather back-calculated values on rock mass strength from large scale slope failures in similar geomechanical environment as Aitik.

• Analyze failure mechanisms in large scale slopes. Collected case studies from other open pit mines should be analyzed in more detail with regard to the failure mechanisms. Numerical modeling can be used better understand the mechanics, through parameter and sensitivity studies.
• Analyze the factors contributing to the rock mass strength. Numerical modeling can be used to investigate what factors contribute to the strength of a large scale rock mass, such as joint dilation and block rotation. It is envisioned that an "equivalent" cohesion can be back-calculated from such modeling work. Simple conceptual models will be used for this. Furthermore, it can be possible to investigate the effects of confinement and fracturing of intact rock bridges on the resulting "equivalent" rock mass cohesion.

• Verify models and mechanisms. Comparisons against gathered case studies should be conducted. It is also envisioned that simple laboratory tests can aid in better verification of identified failure mechanisms. Full scale testing is not, however, within the scope of this project.

• Develop and apply design methodology. A simple, easy-to-use, design methodology should be developed based on the work outlined above. Also, guidelines for more detailed analysis of large scale slope stability should be postulated including how to choose strength parameters. This is followed by application of the design methodology to the Aitik case.


Magma Nevada Mining Company. 1995. Information material.


Merrill, R. H. 1995. Personal communication.


