QUANTIFYING THE EFFECT OF IN-SITU STRESSES AND PIT DEPTH
ON SLOPE STABILITY BY INCORPORATING BRITTLE
FRACTURING IN NUMERICAL MODEL ANALYSES

by

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A thesis submitted to the Faculty and the Board of Trustees of the Colorado School of Mines in partial fulfillment of the requirements for the degree of Doctor of Philosophy (Mining and Earth Systems Engineering).

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Designing reliable slopes that provide safety and maximize financial return represent one of the main challenges in a mining operation. As open pits get increasingly deeper, the need to better understand the behavior of high rock slopes has become more critical. Currently, it is not clear how mining at increased depths may impact slope behavior. Similarly, the impact of in-situ stress magnitudes in slope stability is still uncertain. High horizontal stresses and increased depths can lead to unfavorable stress conditions, inducing rock mass damage and strength loss. The main goal of this research is to assess the effect of increased mining depth and in-situ stresses on slope stability. Reliable slope behavior predictions require an adequate knowledge of the local pre-mining stress setting, together with suitable numerical tools capable of capturing the brittle characteristics of rock masses. In this research, the response of the rock mass was studied using both a FEM and the Slope Model code, which is based on a simplified DEM approach.

It was found that when pre-existing fractures are present, rock mass damage can develop deep into the rock mass, leading to possible slope failures when combined with non-persistent discontinuities. Rock mass damage levels, represented by the number of induced fractures, tend to increase following a near exponential relationship with depth indicating an important potential to develop in deep open pits. Likewise, damage levels present a strong correlation with in-situ stress magnitudes, with higher horizontal stresses resulting in increased fracturing. In addition, when horizontal stress magnitudes are higher failure surfaces will tend to form at deeper levels, leading to larger failed volumes. These differences in behavior highlight the importance of an accurate determination of the in-situ stress state which together with adequate numerical tools, allow for improved stability assessments. At the same time, improved numerical tools can lead to a better understanding of rock mass behavior in variable stress conditions, reducing the uncertainty in the development of future deep pits.
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LIST OF SYMBOLS

Coefficient of determination .................................................. $R^2$

Cohesion (kPa) ........................................................................ $c$

Disturbance factor ................................................................. $D$

Effective friction angle (degrees) ............................................... $\phi'$

Effective intermediate principal stress (MPa) ......................... $\sigma'_2$

Effective major principal stress (MPa) ....................................... $\sigma'_3$

Effective minor principal stress (MPa) ....................................... $\sigma'_n$

Effective normal stress (MPa) .................................................. $\phi'$

Extension strain critical value .................................................. $\epsilon_{crit}$

Friction angle (degrees) .......................................................... $\phi$

Generalized Hoek-Brown material constant “m” for intact rock .......... $m_i$

Generalized Hoek-Brown reduced value of “m” material constant ........ $m_b$

Generalized Hoek-Brown rock mass constant “a” .................... $a$

Generalized Hoek-Brown rock mass constant “s” .................... $s$

Gravitational stress component (MPa) ..................................... $\sigma_{grav}$

Horizontal in-situ stress (MPa) ................................................. $\sigma_h$

Horizontal to vertical stress ratio .......................................... $k$

Induced stress by gravitational component (MPa) ................... $\sigma'_{grav}$

Induced stress by tectonic component (MPa) ............................. $\sigma'_{tec}$

Initial friction angle (degrees) ............................................... $\phi_i$
Intermediate principal strain \( \epsilon_2 \)

Intermediate principal stress (MPa) \( \sigma_2 \)

Material density (kg/m\(^3\)) \( \rho \)

Major principal strain \( \epsilon_1 \)

Major principal stress (MPa) \( \sigma_1 \)

Minor principal strain \( \epsilon_3 \)

Minor principal stress (MPa) \( \sigma_3 \)

Normal force between spheres in a spherical particle model (kN) \( F^n \)

Normal stress (MPa) \( \sigma_n \)

Plastic strain necessary for full cohesion loss in a CWFS model \( \epsilon^p_c \)

Plastic strain needed for reaching maximum frictional strength in a CWFS model \( \epsilon^p_f \)

Poisson’s ratio \( \nu \)

Residual friction angle after cohesive link rupture in a BPM (degrees) \( \phi_c \)

Residual friction angle (degrees) \( \phi_{res} \)

Rock mass horizontal deformation modulus (GPa) \( E_h \)

Shear strength (MPa) \( \tau \)

Tangential force between spheres in a spherical particle model (kN) \( F^t \)

Tectonic stress component (MPa) \( \sigma_{tec} \)

Tensile strength between bonded particles (Pa) \( T \)

Tensile stress (MPa) \( \sigma_t \)

Total induced stress (MPa) \( \sigma_{ind} \)

Total measured stress (MPa) \( \sigma_{tot} \)

Uniaxial compressive strength of intact rock sample (MPa) \( \sigma_{ci} \)
Uniaxial compressive strength of rock mass (MPa) \( \sigma_c \)

Vertical in-situ stress (MPa) \( \sigma_v \)

Young’s modulus (GPa) \( E \)
<table>
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<tr>
<td>3-Dimensional Universal Distinct Element Code</td>
<td>3DEC</td>
</tr>
<tr>
<td>Acoustic emission method</td>
<td>AE</td>
</tr>
<tr>
<td>Bonded Particle Model</td>
<td>BPM</td>
</tr>
<tr>
<td>Cohesion Weakening Friction Strengthening</td>
<td>CWFS</td>
</tr>
<tr>
<td>Discrete Element Method</td>
<td>DEM</td>
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<tr>
<td>Factor of Safety</td>
<td>FS</td>
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<tr>
<td>Fast Lagrangian Analysis of Continua in 2 Dimensions</td>
<td>FLAC2D</td>
</tr>
<tr>
<td>Fast Lagrangian Analysis of Continua in 3 Dimensions</td>
<td>FLAC3D</td>
</tr>
<tr>
<td>Finite Difference Method</td>
<td>FDM</td>
</tr>
<tr>
<td>Finite Element Analysis</td>
<td>FEA</td>
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<td>Finite Element Method</td>
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<td>Geological Strength Index</td>
<td>GSI</td>
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<td>Hoek-Brown failure criterion</td>
<td>H-B</td>
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<tr>
<td>Microseismic monitoring</td>
<td>MS</td>
</tr>
<tr>
<td>Mohr-Coulomb failure criterion</td>
<td>M-C</td>
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<tr>
<td>Partial differential equation</td>
<td>PDE</td>
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<tr>
<td>Particle Flow Code in 2/3 Dimensions</td>
<td>PFC2D/3D</td>
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<td>Rock Mass Rating</td>
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<td>Rock Quality Designation</td>
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<tr>
<td>Synthetic Rock Mass</td>
<td>SRM</td>
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Universal Distinct Element Code ................................. UDEC
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Sheorey variable $k$ ratio curve for $E_h = 50$ ......................... Sh(50)
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Sheorey variable $k$ ratio curve for $E_h = 125$ ......................... Sh(125)
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CHAPTER 1
INTRODUCTION

Limited understanding about large pit slope behavior has led to current approaches that may not necessarily be satisfactory for the design of open pits that extend to depths over 800 meters. These gaps in knowledge are due in part to the lack of experience at such depths and limitations in current analysis and design tools [46, 93, 102, 106]. Commonly made assumptions in shallower pits regarding failure modes, the effect of the in-situ stress field, and empirical rock mass strength determination methods may not be valid at greater depths due to the higher potential for rock mass damage that can impact slope behavior.

In particular, critical gaps exist in relation to the effect of increased mining depth and pre-mining stress magnitudes in the stability of brittle rock masses. They both can promote unfavorable stress conditions in a slope, potentially leading to rock mass damage and extension of pre-existing fractures which can ultimately compromise slope performance. This fracture formation process can have an important effect on the stability of open pits excavated in hard-brittle rock, potentially resulting in complex and less common failure modes. As several authors have recognized, the current understanding of failure mechanisms is inadequate for the increasing depths of open pit mines [1, 22, 23, 26, 46, 90]. Moreover, it has been stated that it is not unlikely that unknown and more plausible failure modes will be observed as pits become deeper [31, 101, 112]. This thesis addresses some of the potential uncertainties in the stability of large slopes related to intrinsic fracturing and high stress conditions.

1.1 Problem Statement

As open pits get increasingly deeper, the need to better understand the behavior of high rock slopes has become more critical. Currently, it is not clear how mining at increased depths may impact slope behavior, or if there is potential for unknown failure mechanisms.
Similarly, it has been recognized that in-situ stress magnitudes have an effect on ground deformations, but their impact in slope stability is still uncertain [25, 65].

Both the pre-mining stress state and the excavation depth need to be considered in detail because of their important influence on the final stress state. High horizontal stresses and large depths may result in unfavorable stress conditions, which can lead to fracturing within the rock mass and shear displacements along discontinuities. Such damage can reduce overall strength of the rock mass ultimately leading to global slope failure.

In the literature, the failure mechanism in many large scale slope failure processes are said to have been driven by the propagation of brittle tensile fractures interacting with natural pre-existing discontinuities [10, 22, 26, 29, 36, 67]. The majority of the common slope stability analysis approaches, such as Limit Equilibrium, Finite Elements, and Finite Differences based models, have important limitations in representing the role of fractures in deep rock slopes [39, 107, 113]. These methods are not meant to consider the effect of complex fracture development combined with realistic shear slip on failure surfaces [19]. Inadequate tools can potentially over-simplify this behavior, overestimating the reliability of a slope through the analysis of less critical failure modes.

Several authors agree that the use of discrete elements methods (DEM) is a more adequate approach to represent the complicated behavior of fractured brittle rock masses [17, 21, 28, 58, 98]. However, analyses using these kinds of models have been limited to very small scales. Large scale DEM analyses are necessary to study the effect of large depths under varying in-situ stress conditions on the behavior of large pit slopes. Improved understanding of large slope behaviors help reduce uncertainty in the pit slope design process, which in turn contributes to accomplishing successful open pit mining operations.

1.2 Research Objectives

The main objective of this research is to quantify the effect of mining depth and in-situ stresses on slope stability. This will be done by studying the fracturing process and rock mass damage in slopes, which can progressively reduce the strength of brittle rock masses
and contribute to slope failure. The tasks set to accomplish this objective include

- Identifying stress conditions that can promote fracturing in brittle rock masses
- Performing numerical modeling studies for
  - assessing rock mass damage levels in high rock slopes for different in-situ stress states considering homogeneous and jointed rock masses
  - quantifying the effect of increased excavation depth in terms of rock mass damage
  - establishing possible limits for the depths to which open pits can progress
  - studying the influence of pre-mining stress conditions and fracturing in slope failure mechanisms
  - establishing a relation between rock mass damage and slope deformation over time

1.3 Thesis Overview

Chapter 2 includes a review of previous research found in the literature relevant to slope stability in jointed brittle rock masses, where inelastic deformations imply fracturing of intact rock. It starts by covering the anticipated failure modes in deep pit slopes and identifies possible differences in failure mechanisms compared to shallower pits. Special emphasis is put on previous work related to rock mass damage development and its potential impact in rock mass strength. This includes identifying some limitations in rock mass property estimation approaches and difficulties of traditional failure criterions when applied to jointed rock masses. Earlier attempts to establish the influence of in-situ stress on slopes excavated in homogeneous elastic materials are also reviewed. In addition, this chapter covers the main numerical models available for slope stability evaluations, including newer and more appropriate tools for representing fractured brittle rock.

Chapter 3 describes the factors influencing the local stress field magnitudes, and discusses the importance of stress measurements for establishing an adequate description of the local
stress conditions. For this, it reviews the most common measurement techniques, with an emphasis on the ones better suited for mining applications. This chapter also contains a stress measurement compilation obtained from several databases, including an analysis of the measured magnitudes by region, for both vertical and horizontal stress magnitudes. Finally, a description of the most common in-situ stress field modelling approaches is presented, which can account for a wide range of stress settings.

Chapter 4 studies the response of homogeneous rock masses to different in-situ stress fields and increasing pit depth. This response is evaluated in terms of ground deformations, induced stresses, and rock mass damage. By combining Finite Elements analyses and Slope Model results, it was possible to identify stress conditions obtained from elastic analyses that could induce fracturing. This also allowed for the evaluation of different tools for predicting, not only the occurrence, but the location of damaged zones by comparing them with the fractured areas from the Slope Model runs.

Chapter 5 includes an assessment on the effect of pre-mining stress magnitudes and on the impact of increasing excavation depth in the stability of jointed rock masses. High stress magnitudes and stress redistribution resulting from an excavation sequence can promote rock mass damage and fracture extension. This potential is quantified through the number of induced fractures obtained from Slope Model considering different stress states and pit depths. The impact of horizontal stress magnitudes in damage distribution is also analyzed, which is further complicated by pre-existing fractures that promote stress concentrations. Since intact rock fracturing is essential to form a complete sliding surface when non-persistent structures are present, damage location can play a key role in stability and on the location of eventual failures. For this reason, the potential for the stress field to affect failure modes and the shape of failure surfaces is also considered. Finally, this chapter studies the relation between rock mass damage and slope deformations over time.

Chapter 6 completes the thesis by summarizing the conclusions from the previous chapters. Additional studies outside the scope of this thesis are also suggested for future work.
The design of reliable slopes in an open pit mine is one of the biggest challenges in a mining operation [91]. The excavation configuration needs to provide an optimum balance between safety and financial return. Finding this balance can become increasingly complex due to the greater uncertainty related to slope behavior as pits get bigger [93]. These uncertainties can be related to

- expected failure mechanisms in high slopes
- influence of in-situ stress field in stability
- effect of mining-induced damage
- rock mass strength determination
- limitations in current analysis tools
- general lack of experience designing pits over 800 meters deep

2.1 Modes of failure in high slopes

A key part of the slope design process is to understand and predict the potential failure modes that could affect a slope. The selected type of analysis is mainly dictated by the anticipated failure mode, the scale of the slope, available data, as well as the stage of the project [92]. Rock slope failures may involve several mechanisms (Figure 2.1). Some of the ones typically considered include

- sliding along circular or semi-circular failure surfaces (rock mass failure)
- sliding along geological discontinuities (e.g. planar and wedge failures)
• toppling (direct and flexural)

• combined failure modes

![Figure 2.1: Basic failure mechanisms in rock slopes.](image)

Generally, in stronger rock masses structure is likely to be the primary control. In weaker rocks or highly fractured rock masses, the controlling factor is usually the strength of the rock mass (Figure 2.2). However, modes of failure can be far more complicated and usually are a combination of the modes listed above, in particular when the scale of the slope is larger than the fracture persistence.

![Figure 2.2: Perceived failure modes in different rock masses [85].](image)

In fractured rock masses non-persistent structures will form rock bridges of intact rock within the rock mass, which have a significant impact on slope stability [26]. A slope failure
in these conditions will likely show partial structural controls implying local failure of rock bridges driven by stress changes behind and below the slope, combined with sliding along discontinuities (Figure 2.3). The interconnection of non-persistent discontinuities will eventually result in the formation of a full sliding surface, allowing failure to occur where initially it was not kinematically possible. This type of failure, common in moderately jointed rock masses, is generally referred as step-path failure [4, 113].

![Figure 2.3: Examples of modes of failure with partial structural control [101].](image)

An adequate knowledge of rock mass behavior and failure modes can only be achieved by understanding the brittle fracture process taking place in rock slopes [26]. In the same way, the interaction between natural discontinuities and fracture propagation through intact rock is crucial for understanding complex failure mechanisms [31, 94, 105]. Large scale slope failure processes are mainly driven by the initiation and propagation of brittle tensile fractures driven by extensional strain, which interact with natural pre-existing discontinuities to eventually form basal and internal shear planes [29]. In this situation, shear failure only becomes relevant after enough tensile damage have occurred to allow mobilization. Other authors agree that the slope failure process of a rock mass is rarely shearing alone. Tensile failure of intact rock bridges, tensile failure due to bending, block rotation, and flexure all take place together, resulting into what is usually called rock mass shearing [112].

Failure mechanisms in large open pits can be influenced by a number of additional factors in comparison to shallower pits. As an open pit gets deeper, the combination of different materials and properties, potential for intersecting more structural features, higher stress
magnitudes and larger wall deformations can result in increasingly complex failure modes. Since most of these properties and magnitudes can vary with depth, the expected failure mode could be highly dependable on the scale of the slope [102]. For example, in the early stages of an excavation, discontinuities within a rock mass may be continuous in relation to the scale of the slope (Figure 2.4(a)). In this case it is likely that most of the failures are structurally controlled (e.g. wedge or plane failures). As the excavation progresses, these discontinuities will be non-persistent relative to the slope. In this situation, it is more likely that a potential failure surface will run along pre-existing discontinuities but will also imply intact rock failure (Figure 2.4(b) and Figure 2.4(c)).

![Figure 2.4: Example of failure modes at different scales [101].](image)

It has been recognized by several authors that the level of knowledge and understanding of failure mechanisms of increasingly higher slopes is insufficient [24, 26, 46, 90]. It is possible that unseen failure modes are experienced as pits become deeper [101]. For this reason it is critical to consider brittle fracturing and rock mass damage accumulation when studying failure mechanisms in high slopes [107].

### 2.2 Rock mass damage

An important difference in the behavior of shallow and very deep pit slopes is the higher potential for stress-induced rock mass damage in the latter case. As any excavation, an open pit will cause a disruption and redistribution of the in-situ stress field. The new exposed rock
will become unconfined, and will be subject to successive stress changes as mining progresses [112]. In very deep open pits the stress state around the excavation can become increasingly complex, likely showing variable stress conditions and zones of very high to very low stresses within the rock mass [102].

It is generally found that the fracture geometry in the rock governs the stability of near surface structures, and that the natural in-situ stresses govern the stability of deep structures [50]. In other words, failure in hard rocks is a function of the in-situ stress magnitudes and the characteristics of the rock mass, as the intact rock properties and the fracture network. In low stress environments, the failure process is controlled by the continuity and distribution of the natural fractures in the rock mass (i.e. structurally controlled failures).

As stress magnitudes increase, failure can become increasingly dominated by the new stress-induced fractures growing parallel to the excavation boundary, which is commonly referred as brittle failure [77, 78]. A brittle fracture is defined as a fracture that exhibits no permanent (plastic) deformation, as opposed to a ductile fracture which is preceded by considerable plastic deformation [5]. Conversely, a brittle material is defined as a solid that shows an elastic behavior until the failure moment, after which there is little or no irreversible deformation [3].

Most rocks near the Earth’s surface are brittle and contain fractures, cracks, and other inhomogeneities, which cause the rock mass to behave in a non-linear manner. It has long been known that the strength of brittle rocks under compression depends on the growth of flaws and cracks and how these cracks grow, propagate and combine into larger shear faults [36, 41]. Unlike in ductile materials, failure in brittle materials involves a loss of material continuity which is necessary before a kinematically feasible failure mechanism can form.

The damage process in a slope has to begin by fracturing through the intact rock between discontinuous structures, implying that the onset of damage is essentially controlled by the strength properties of the intact rock and its relation to the magnitude and orientation of the induced deviatoric stresses [15]. These fractures form under low confining pressures, propa-
gating in the direction of the maximum applied compressive stress. This fact is supported by the microscopic observation of laboratory rock samples in compression (Figure 2.5), which has shown that most of the formed cracks are tensile and sub-parallel to the maximum principal stress [78, 82]. Previous research has revealed that the occurrence of shear cracks is very limited, which implies that the failure of the sample occur by the interaction of tensile cracks forming a macro shear failure surface [8].

Figure 2.5: Stress-strain curve besides shear and tensile crack number in a BEM model [78].

This behavior has also been verified by numerical modelling studies [22, 41, 73, 78], which have been placing increased emphasis on the role of extension fractures [15]. These studies suggest that the initiation of the damage process in a jointed rock mass is dominated by the formation and propagation of extension fractures, driven by the complex interaction between the macro scale compression and the induced micro scale tensions. This interaction produces non-linear, anisotropic elastic behavior even under low loads before any significant damage
forms [89]. It is expected for the occurrence of extension fractures to be sensitive to changes in the magnitude and direction of the major principal stress [14].

As an excavation gets deeper, higher stress magnitudes are induced parallel to the slope face. This combined with low stress magnitudes near the excavation boundary (i.e. low confinement) in the direction normal to the face can lead to progressive fracturing (Figure 2.6). Moreover, the induced stress magnitudes and orientations will continually change as the excavation develops, affecting the formation of new fractures [73]. For this reason, considering this stress path is critical to understanding the damage accumulation process in a slope, which can have an important effect on the rock mass strength and on potential failure mechanisms.

![Figure 2.6: Variation of principal stresses with pit depth.](image)

In a similar way in which stress magnitudes can disturb the rock mass, changes in the stress field can also affect pre-existing discontinuities [56]. Low stress environment in slopes almost guarantees that existing or incipient structures will be exploited before a full failure surface develops [46]. This suggests that loss of strength along discontinuities can play an important role in the failure process. Strains within the rock mass can manifest as shear displacements or as opening of adversely oriented structural features [85, 94]. Shear
displacements along pre-existing fractures may lead to cohesion loss in addition to a reduction of the friction angle to a residual value (Figure 2.7). Fracture opening can lead to a decrease or even a total loss in shear strength, as the contact forces between rock surfaces will be reduced.

![Figure 2.7: Peak and residual Mohr-Coulomb strength of structures [94].](image)

2.3 Micro seismic activity in open pits

High resolution seismic monitoring has been providing growing evidence on the rock mass damage process affecting slopes in high stress environments [108]. As it was mentioned earlier, stress changes as a result of mining can promote fracture growth through intact rock. These brittle fractures release high frequency seismic waves which can be detected by accelerometers and geophones. The use of microseismic (MS) and acoustic emission (AE) methods provide a reliable way for determining the extent, timing and mechanism of fracturing within a slope [70, 112].

The AE method has been used very successfully in several field applications for monitoring the accumulation of rock mass damage in mining slopes. Lynch [68] reviewed acoustic emission measurements taken at more than 25 open pit slopes, all which showed signatures of brittle rupturing (Figure 2.8). These AE measurements provide practical evidence on the effect of stress changes resulting from mining, and moreover on the role of the stress field in stability. Stress might already be playing an important role in the behavior of slopes,
including shallower pits where most of the experience exists [104, 106].

Figure 2.8: Micro-seismic activity induced by rock removal [68].

It has been concluded from seismic monitoring data that most of the detected damage was caused from extensional fracturing [112]. These fractures are likely to occur below the base of the pit and behind the pit wall, where the confinement has been reduced as a result of mining. Also, they will tend to develop parallel to the major principal stress and perpendicular to the minor principal stress. As a result, fractures at the bottom of the pit will likely form at shallow angles, and the ones behind the slope wall will tend to occur at an angle similar to the slope face. These two orientations can be very unfavorable for stability, in the first case weakening the rock mass to be excavated in following stages, and in the second case causing direct damage to the toe of the slope.

Lynch and Malovichko [68] confirmed the relation between mining rate and seismicity rate (Figure 2.9). They demonstrated that it was the removal of the broken rock, and not blasting what caused fracturing. The occurrence of micro seismic events showed a strong correlation with the volume of rock hauled, which indicates these events are associated with stress changes caused by the missing weight. There also seemed to be a good correlation
between the seismic event data and the slope surface movements, the latter occurring 1 to 2 months after fracturing was detected [68, 108].

Wesseloo [112] recognizes that the current knowledge of the damage accumulation process in a low stress environment is inadequate for the future challenges of open pit mining. He states that the use of MS and AE monitoring is a key component for overcoming this limitation. These methods can provide information on the location and damage accumulation rate at much lower induced strain levels compared to other monitoring techniques [12].

Figure 2.9: Cumulative seismic events (left) and cumulative volume of rock removed (right) comparison [68].

Numerical analyses estimating the stress state during seismic events show that fracturing occur at much lower stress levels than the estimated Hoek-Brown rock mass strength [112]. This is due to the influence of pre-existing discontinuities, which disrupt the stress field and create stress concentrations in rock bridges. The higher stress magnitudes at the end of the fractures can exceed the strength of the rock bridge, causing fracture extension through the intact rock. The relationship between the estimated stress magnitudes at fracturing and the Hoek-Brown strength envelopes for intact rock, undisturbed rock mass (D=0), and disturbed rock mass (D=1) are shown in Figure 2.10. The disturbance factor D used for the Hoek-Brown strength properties is described in more detail in Section 2.4.
2.4 Stress analyses and rock mass damage in slope stability evaluations

The effect of stress on the stability of rock masses is well recognized for underground excavations, but for surface excavations its impact is not as clear. Some work has been done to understand stresses around open pits and their impact on slope stability, yet a significant amount of research is needed. The important role of the regional stress in slope stability due to the effect of stress relief strains on unfavorably oriented structures have been recognized in the past. However, the effect of regional stress was found to be limited to promoting displacements along pre-existing planes [85]. It was considered unlikely that regional stresses could promote failure of intact rock, but also that several exceptions could be possible. However, their remarks were limited to identifying geologic factors and problems that could affect stability in smaller scale slopes. Moreover, no attempts for including these factors in the analyses were made, likely due to the lack of appropriate numerical tools at the time.

Despite the previous observations, still, the common assumption is that stresses play a very limited role in the stability of rock slopes. This statement may be inadequate and needs to be questioned for the design of very large slopes [46]. It has been stated that ignorance of the actual stress regime is curious as it is an essential assumption in all numerical modeling [25].
Stacey [103] discovered in the early stage of finite element methods applied to slopes that a large horizontal in-situ stress has a major influence on the stress distributions in slopes. He also found that when circumferential stresses are compressive they tend to stabilize slopes, while when tensile the slope tends to bulge. Some later work done by Stacey [105] identified a relation between in-situ stresses and pit depth with the occurrence of zones of extension strain in slopes. In these zones the rock has expanded in at least one direction which can be related with the development of extension fractures, potentially contributing to slope failure. Still, there is not enough practical evidence to support his conclusions. As pit slopes become higher, it is necessary to consider the possible impact of in-situ stresses combined with the high induced stresses at the toe of the slopes [106].

Based on the results of continuum numerical analyses, Lorig [65] suggested that in-situ stresses have no significant effect on the safety factor of a slope but that they do have an influence on deformations. However, these analyses were carried out assuming a homogeneous, isotropic and non-softening material. If the rock mass is considered to be brittle, Lorig’s conclusions should not apply. In this case deformations could cause damage and weakening of the rock mass, adversely affecting the stability of a slope. The effects of strain on rock mass strength are not clear but they can be large enough to cause fracturing in more brittle rock masses [105].

2.5 Rock mass strength properties

Reliable estimates of the strength of a rock mass are required for almost any type of analysis for assessing the stability of a slope [71]. Still, the determination of rock mass strength remains as a major deficiency in current rock slope design practice [46]. In general, rock mass analyses look at the potential for slope failures where the mechanism is likely controlled by the rock mass strength. However, the strength of a rock mass and failure potential are influenced by the properties and spatial distribution of different types of pre-existing discontinuities (Figure 2.11). Commonly empirical classification systems, such as GSI or RMR, are needed to account for these features [6, 72]. Additionally, rock mass
strength can be affected by rock mass damage resulting in strength loss, further complicating the assessment of the rock mass mechanical properties.

![Figure 2.11: Factors influencing rock mass strength](image)

Figure 2.11: Factors influencing rock mass strength [46].

In practice, most slope designers use a shear failure criterion such as Hoek-Brown or Mohr-Coulomb to represent the rock mass strength [43, 44, 46, 47]. The main assumptions in this kind of analyses is that the rock mass can be considered equivalent to a homogeneous and isotropic material, and that the rock mass will fail by shear, which might not be the case. This assumption is reasonable in weaker rocks or in highly fractured rock masses with randomly oriented joint sets, but it is not adequate when the expected failure mechanism involves both discontinuity and intact rock strengths as shown in Figure 2.12.

The Hoek–Brown failure criterion, which is the most widely used in mining applications, is an empirical relationship that describes increase in peak strength of isotropic rock with increasing confining stress [30]. This criterion follows a non-linear, parabolic form that distinguishes it from the linear Mohr–Coulomb failure criterion. The original version was developed for the design of underground excavations in hard-brittle rock but it has been modified several times over the years to extend its applicability [45].

In its original form, the Hoek-Brown criterion assumed that failure of the intact rock played no significant role in the overall failure process, but that rock mass failure was con-
controlled by translation and rotation of individual rock pieces [43]. The criterion was later adapted for its use in slope stability problems and on a wider range of geologic conditions [46]. An update of the criterion was presented in 2002 that included improvements in the correlation between the model parameters and GSI (Geological Strength Index) (Figure 2.13) [13, 71, 72]. This update also included the addition of the “disturbance factor” (D) to account for stress relaxation and blast damage in slopes [47] (Figure 2.14).

Figure 2.12: Applicability of the Hoek-Brown failure criterion.

Figure 2.13: Geological Strength Index classification for rock masses [30].
Figure 2.14: Effect of disturbance parameter D in rock mass strength [94].

The Hoek–Brown criterion estimates the strength of a jointed rock mass based on the strength of an intact rock sample. It applies different reduction factors to account for the degree of fracturing of the rock mass and the surface conditions of discontinuities (GSI), as well as for the disturbance level of the excavation (D). The latest version of the criterion in its generalized form is defined by Equation 2.1.

\[
\sigma'_1 = \sigma'_3 + \sigma_{ci} \left( m_b \frac{\sigma'_3}{\sigma_{ci}} + s \right)^a
\]  
(2.1)

In this equation, \( \sigma'_1 \) and \( \sigma'_3 \) are the maximum and minimum effective principal stresses at failure, \( \sigma_{ci} \) is the intact rock uniaxial compressive strength, \( m_b \) is the value of the Hoek-Brown constant \( m_i \) for the rock mass, \( s \) and \( a \) are constants which depend upon the characteristics of the rock mass. Additional expressions for \( m_b \) (Equation 2.2), \( s \) (Equation 2.3) and \( a \) (Equation 2.4) as a function of the GSI and the disturbance factor D are shown below:

\[
m_b = m_i \exp \left( \frac{GSI - 100}{28 - 14D} \right)
\]  
(2.2)

\[
s = \exp \left( \frac{GSI - 100}{9 - 3D} \right)
\]  
(2.3)
\[ a = \frac{1}{2} + \frac{1}{6} \left( \exp \left( -\frac{GSI}{15} \right) - \exp \left( -\frac{20}{3} \right) \right) \]  

(2.4)

One of the main limitations of the Hoek-Brown strength criterion is that it was intended to be applied only where the rock mass can be considered to be homogeneous and isotropic. The Hoek-Brown criterion should not be used in cases where structure is expected to have a significant influence (Figure 2.12). This limitation is particularly important when analyzing failure modes with partial structural controls, as the modes shown in Figure 2.3. Since the rock mass is approximated to a homogeneous material, the criterion ignores the effect of stress concentration around fractures, as well as the effect of local damage before peak strength is reached. In addition, as a shear criterion, Hoek-Brown assumes that a major shear fracture will occur at peak strength. However, many times this assumption does not agree with field observations, particularly in brittle rock where tensile strength seems to play an important role [29, 106]. When the failure mechanism is not shear, it is likely that this criterion will give inadequate results.

Another important shortcoming of the Hoek-Brown criterion is its independency from the intermediate principal stress \( \sigma_2 \). This simplification was justified by Hoek and Brown [43] through triaxial extension and compression tests carried out by Brace [7], which determined that \( \sigma_2 \) had a negligible influence on failure. However, these tests only considered cases where \( \sigma_2 = \sigma_3 \) and \( \sigma_1 = \sigma_3 \) Later studies on rocks under high triaxial compression showed that \( \sigma_2 \) can have an important influence on the behavior of the material. Mogi [81] concluded that the ductility or a rock sample increases with higher confining pressure (\( \sigma_2 = \sigma_3 \)) but decreases with increasing \( \sigma_2 \), promoting brittle failure. Several 3-D versions of the Hoek–Brown failure criterion have been developed to address this problem but their use is still very limited [114, 115].

### 2.5.1 Hoek-Brown criterion with brittle parameters

While the traditional Hoek-Brown parameters may be appropriate for estimating the strength of rock masses around tunnels and slopes at shallow depths in near homogenous
conditions, there is growing evidence that it is not suitable for estimating the strength of rocks in high stress settings [77]. The fundamental difference in behavior is that in a low stress environment slip along discontinuities dominates the failure process, while at depth the process is controlled by stress-induced brittle fracturing [78].

The application of the Hoek-Brown criterion to predict the onset of brittle failure has met with limited success [23]. During brittle failure, peak cohesion and friction are not mobilized together, and the process is dominated by cohesion loss associated with rock mass fracturing [73, 75]. Under this assumption, Martin et al. [77] proposed an adapted version of the Hoek-Brown criterion for representing brittle behavior. In this form of the criterion, the strength envelope is based only in cohesion and the frictional strength component is ignored (Figure 2.15). In this way, the extent of brittle failure around an excavation in massive to moderately fractured rock can be estimated by using an elastic analysis adjusting the Hoek-Brown parameters to \( m = 0 \) and \( s = 0.11 \). Therefore, the Hoek-Brown strength criterion with the brittle parameters proposed by Martin [77] reduces to Equation 2.5

\[
\sigma_1 = \sigma_3 + \sigma_{ci} s^a
\]  

(2.5)

This brittle criterion has shown good agreement with field observations when predicting the depth of brittle failure and rock mass fracturing around tunnels [73], where the traditional Hoek-Brown criterion greatly underpredicts the occurrence of fractures [77]. However, this approach is not adequate for representing post-peak behavior since it ignores the increase in frictional strength mobilization with higher damage levels within the rock mass [35, 36, 112]. Moreover, the limitations of continuum models for representing fracture growth and rock mass damage still remain.

### 2.5.2 Cohesion weakening friction strengthening criterion

Recognizing the limitations of the Hoek-Brown failure criterion, Hajiabdolmajid [36] implemented a constitutive model which considered the plastic strain-dependencies of the strength components in brittle failing rocks. In Hajiabdolmajid’s model, the rock is assumed
to be initially cohesive, and then as the plastic strain increases the cohesion is reduced and the frictional strength builds up (Figure 2.16). This model, known as cohesion weakening friction strengthening criterion (CWFS), has proved to be effective for simulating the behavior of brittle rock. However, this criterion is still based on continuum plasticity for representing the brittle fracturing process. Furthermore, this method still needs field verification as part of its development.

### 2.5.3 Extension strain fracture criterion

After recognizing the difficulty of estimating rock mass strength in brittle conditions, Stacey [104] proposed an extensional-strain criterion based on laboratory tests of intact samples. This criterion states that fracture of the rock will happen when the extension strain exceeds a critical value $\epsilon_{\text{crit}}$ which is dependent on the properties of the rock. As such, this criterion can make a prediction of zones at which local fracturing might occur while carrying out a simple elastic analysis in an isotropic material. The extension strain
Figure 2.16: Cohesion loss and frictional strength increase as a function of failure strain [36].

is defined as the minimum principal strain $\varepsilon_3$ (in a compression positive convention) and is calculated from the principal stresses using the three-dimensional elastic equation (Equation 2.6)

$$\varepsilon_3 = \frac{\sigma_3 - \nu(\sigma_1 + \sigma_2)}{E}$$  \hspace{1cm} (2.6)

In the previous equation, $\sigma_1, \sigma_2$ and $\sigma_3$ are the three principal stresses, $\nu$ is Poisson’s ratio and $E$ is the modulus of elasticity. The extension strain magnitude $\varepsilon_3$ depends on all three principal stresses, including the out-of-plane direction in 2D analyses. As it can be deducted from Equation 2.6, strains can be extension in nature even in a triaxial compressive stress field. This means that it is not necessary to the rock mass to be in tension for it to expand.

The onset of extension strain zones around an open pit implies that the rock mass has expanded in at least one direction, which can lead to brittle fracturing [105]. The extension strain criterion provides an explanation for the development of brittle fractures observed under low confinement levels, like in the vicinity of an excavation. The criterion seems to work well in predicting extension fracture formation in rocks, where it has shown better agreement with the fracturing observed in many tunnels and ore passes compared to the Mohr-Coulomb or Hoek-Brown frictional criterions [104]. It can also predict the orientation of induced fractures, which will occur in the plane normal to the minor principal stress $\sigma_3$ (Figure 2.17). This agrees with evidence described in the literature which indicates that
fracture in rock takes place in the direction of the major principal stress [34].

Figure 2.17: Extension strain magnitudes and principal stress trajectories.

Still, Stacey’s criterion is only intended to predict potential zones of extension fracturing representing rock mass damage. It does not account for material weakening due to these fractures, or attempt to represent post-peak behavior. Furthermore, as an empirical criterion, it is not expected to be applicable under all stress conditions or to all rock types. The criterion assumes a perfectly elastic isotropic material, which is seldom the case in fractured rock masses. Although its application has met with some success in predicting onset and depth of damaged zones, it does not explain the fundamental mechanisms controlling the failure of brittle rock [73].

2.6 Review of current numerical tools

The slope design process of any rock slope involves carrying out stability analyses to the required acceptance level (e.g. factor of safety, probability of failure or displacements). The type of analysis will depend on several factors like the expected failure mode, the scale of the slope, available data and level of the project (Figure 2.18). In order to evaluate a particular slope design it is necessary to represent main characteristics of the rock mass
in a geotechnical model. This model needs to include basic information about material strength properties, as well as structural and groundwater data. However, the mathematical representation of a rock mass can be difficult due to its complex characteristics. Usually, rock masses are discontinuous, inhomogeneous, anisotropic, and non-elastic [49]. Moreover, rock engineering projects are becoming larger and more complex, resulting in more demanding modeling requirements [54].

Figure 2.18: Analysis tools for different slope conditions and expected failure modes [94].

Several methods have been developed for representing the characteristics and behavior of rock masses. Traditionally, most stability analyses have been carried out by Limit Equilibrium (LE) methods. The main assumption of these LE methods is that the rock will behave as a rigid material and that the shear strength is mobilized at the same time along the entire failure surface [101]. For this reason, LE methods are only adequate for analyzing simple failure modes and small scale stability analyses. Over the last decade, most of the pit slope stability analysis and design are done using numerical modeling methods, which can model many of the complex conditions found in rock masses such as nonlinear stress-strain behavior, anisotropy and changes in geometry [66].
Numerical models can be used as a valuable tool to enhance the understanding of the response of a rock mass to an excavation [39]. These models discretize the material into a finite number of elements. Each element has the appropriate constitutive laws for the corresponding material, so stresses and strains for each of these elements can be calculated under a certain loading condition [66]. In this way it is possible to study many of the complex aspects of the stability of a rock slope, like stress and deformation response. This makes numerical models much more useful in managing ongoing slope displacements, where a number of failure mechanisms can exist simultaneously or where the mechanism of failure may change as progressive failure occurs [46]. Another important advantage of numerical models over LE methods is that an assumption of the failure surface is generally not necessary [102]. Failure will develop naturally where stresses overcome the available strength, reducing the risk of not finding the most critical failure mode.

The most commonly applied numerical methods for rock mechanics problems can be divided into the following categories [53]:

- Continuum methods
- Discrete methods
- Hybrid continuum/discrete methods

The use of continuum or discrete methods depends on several problem-specific factors, such as the fracture system geometry and the relative scale of the problem [53]. Continuum methods are more suitable when only a few fractures are present, and if fracture opening and full block detachment are not significant factors. The discrete approach is more adequate in cases where there is a higher number of fractures and where large displacements of discrete blocks are possible. Hybrid continuum-discrete models are applied to avoid the disadvantages of each method by assuming a continuum behavior in areas of the model where a discrete model is not needed.
2.6.1 Continuum methods

Continuum methods assume that the material is continuous, making them more suitable for analyzing soil slopes, intact rock, weak rocks or heavily jointed rock masses [37, 52, 95]. Discontinuities can still be incorporated explicitly through interfaces between continuous zones. This allows the representation of fractures, faults and bedding planes. In contrast, small scale discontinuities are accounted for by reducing the strength and elastic properties of the rock. Additionally, these methods allow the use of several constitutive models (e.g. elastic, elasto-plastic, strain softening, elasto-viscoplastic) to better represent the different rock mass behaviors. The most common continuum approaches applied to slope stability are the finite difference (FDM) and the finite element (FEM) methods. Their main difference is in the way they solve the governing partial differential equations (PDEs) for the stress-strain and the strain-displacement relations.

The Finite Difference method is the older of the two and is based upon the application of a local Taylor expansion to approximate the differential equations [86]. The basic technique in the FDM is the discretization of the governing PDEs by replacing the partial derivatives with differences defined at neighboring grid points. After imposing the adequate initial and boundary conditions, the solution for the system of equations is obtained. This approach provides a very straightforward simulation of complex constitutive material behavior, such as plasticity and damage. However, its weak point for rock mechanics is the inability to incorporate explicit representation of fractures. An additional difficulty of this approach is its less flexible cubic discretization to accommodate to highly irregular shapes. The most popular FDM for mining applications are the explicit codes FLAC (Figure 2.19) and FLAC3D (Itasca Consulting Group), for two-dimensional and three-dimensional analyses respectively.

The Finite Element method was developed motivated by the difficulties the FDM had with handling complex geometries. The FEM requires dividing the problem into elements of standard shapes (e.g. triangle, quadrilateral, tetrahedral) with a fixed number of nodes, which allows for a more flexible discretization than FDM. The PDEs are approximated
by trial functions, generating local algebraic equations which represent the behavior of the elements [86]. These elemental equations are constructed into a global system of equations by following the spatial relation of each element, as well as the initial and boundary conditions. This global equation system is usually represented in matrix form, and the solution of the system can be obtained by inverting the global stiffness matrix. This method has been implemented in several codes, being Phase² (Rocscience Inc.) and Abaqus FEA (Dassault Systèmes Group) the most common for mining and geotechnical engineering applications (Figure 2.20).

Existing continuum codes can simulate the location and shape of the failure surface developed in a slope to some extent, although an actual discontinuity is not developed [102]. The natural development of cracks and rupture surfaces in brittle rock is not well-handled by continuum approaches [19]. Jing [53, 54] agrees with him, stating that the treatment of fractures and fracture growth remains the most important limiting factor in the application of FDM and FEM for rock mechanics problems. These methods are formulated based on continuum assumptions, meaning that large-scale fracture opening, sliding, and complete block detachment and rotation are not allowed [28]. Another important limitation

Figure 2.19: Displacement contours and vectors in the FDM code FLAC [52].
of continuum methods is the need of a constitutive law capable of representing different rock mass behaviors realistically. An adequate constitutive law may not exist or it may be excessively complicated with many obscure parameters [19].

2.6.2 Discontinuous methods

Discontinuous codes are designed specifically to simulate the response of discontinuous media. The fractured medium is represented as an assembly of blocks, which can be rigid or deformable, divided by continuous fractures treated as boundary conditions between blocks. This family of methods is known as Discrete Element Methods (DEM).

Unlike on continuum codes, the contact patterns between components of the system are continuously changing with the deformation process in DEM [53, 54]. For this reason, the contacts among blocks need to be identified and continuously updated during the entire deformation process, and represented by proper constitutive models. As a result, slip, separation, rotation and complete detachment along explicit structures can occur, while the individual blocks can deform and yield [96]. This allows the simulation of complex failure mechanisms in rock masses containing multiple, intersecting joint structures.

The most common DEM codes used in geotechnical engineering and rock mechanics are based on the Distinct Element Method created by Cundall [18]. These include the Universal
Distinct Element Code (UDEC) code (Figure 2.21) and its three-dimensional version 3DEC (Itasca Consulting Group). This approach uses an explicit solution scheme which can model complex, non-linear behaviors. Blocks can be deformable and are defined by a continuum mesh of finite-difference zones. The method has been developed specifically to study complex failure mechanisms involving large numbers of explicit structures that divide a rock mass into blocks [51].

![Figure 2.21: UDEC code showing velocity contours and vectors [51].](image)

### 2.6.3 DEM formulations for particle systems

The principle of the DEM technique for granular materials is basically the same as for the blocks, with the additional simplification that particles are rigid and their shape can be regular or irregular [53]. In this case, the material is represented as a collection of particles interacting among themselves in the normal and tangential directions, where the material deformation is assumed to be concentrated at the contact points [48, 84]. Appropriate contact laws at these contact points at the particle level will result in the adequate material properties at the macroscopic level (Figure 2.22).
In order to represent intact rock with a DEM, assemblies of rigid particles can be fused together by cohesive bonds as in the bonded-particle model (BPM) [89]. These bonds can fail by shear or by tension, allowing the simulation fracture formation and propagation within the material. A local failure criterion similar to the Mohr-Coulomb criterion is generally used for this purpose, as the one shown in Figure 2.23 [28]. Typically, for a bonded contact (cohesive) the normal direction force is represented by a spring that acts in tension and compression, while for a frictional contact (broken bond) the normal direction spring acts only in compression and the shear force is limited by a friction law [109] (Figure 2.24).

As a result, the bonded particle approach is capable of simulating the development of new fractures within the intact rock, which is the main advantage over the Distinct Element
method. Using numerical models to further explain the observed cracking and failure in brittle rock is becoming more and more feasible as computer power increases ([41]). The future trend for numerical modelling in soil and rock may consist of the replacement of continuum methods by particle methods [19]. These methods are able to capture the complex overall behavior of actual material through simple assumptions and few parameters at the particle level. However, the computational effort involved in this approach has limited its application to small scale problems. The most well-known code in rock mechanics is perhaps the PFC codes for both 2-D and 3-D problems (Itasca Consulting Group).

A recent method for simulating the mechanical behavior of jointed rock masses, called the Synthetic Rock Mass approach (SRM), has been developed by Pierce et al. [87]. This method combines a Bonded Particle Model with an embedded Discrete Fracture Network (DFN) for representing the rock mass (Figure 2.25). Joints are modelled as unbonded particles applying the Smooth-Joint Model (SJM), which ensures slip and opening in the direction of the joint, independently of the local directions of the contacts particles [110]. Overall failure of a synthetic rock mass depends on both fracture of intact material (i.e. bond breaks) as well as yield of joint segments [17, 88]. This method allows studying the interaction of brittle failure of the intact rock with failure along pre-existing discontinuities in the same model, which is essential to understand the behavior and failure mechanisms of jointed rock masses [21, 79, 87].
2.6.4 Lattice models

The application of particle methods to large-scale problems is currently difficult or impossible because of high computational demands [19]. For this reason, an alternative method closely related to the DEM model for particle systems called the Dynamic Lattice Network model was developed [17, 53, 63]. This method is also capable of simulating fracture initiation and propagation in rock but with some computational advantages over bonded particle models.

Lattice models represent the intact rock through an assembly of particles and springs, rather than as a direct discrete medium as in DEM models. In this case contact forces between particles (normal and shear) are represented by mass-less springs, and the mass of each particle is concentrated at the spring vertices (Figure 2.26). The dynamic motion of the medium is simulated by the equations of motions of the mass particles and the deformation of the springs, whose stiffness and strength are derived from those of the medium [53]. Similarly, the particle masses are derived from the density of the material. Lattice models have been successfully applied to investigate the interaction of crack formation and heterogeneities in elastic-plastic and elastic-brittle materials [97, 116].
The lattice model achieves a high computational efficiency by assuming small displacements, which eliminates the need of contact detection as in the BPM [17, 63]. The full generality of PFC3D is often unnecessary when relatively small movements occur during fracturing and movement on discontinuities. In this case, geometrical relations can be pre-calculated and the constitutive and motion equations simplified, leading to big savings in computer resources [17].

2.6.5 Slope Model code

Slope Model, which uses a modified version of the Synthetic Rock Mass (SRM) approach, is a new code written by Itasca Consulting Group as part of the Large Open Pit (LOP) international research project. The code can capture the behavior of jointed rock masses where failure is a combination of opening and sliding along pre-existing discontinuities, and intact-rock tensile failure [20, 21]. Although a SRM model based on bonded particles can achieve the same, its high computational demands make them unsuitable for large scale analyses. In contrast to the original SRM method, in Slope Model the BPM is replaced by a lattice for representing the intact rock. The lattice, which is a 3D quasi-random array of springs, can also break in shear or tension resulting in microcracks. Additionally, as in the SRM formulation, a smooth joint model (SJM) is also used (Figure 2.27). This allows respecting the joint plane independently of the local contact normals [21].
Slope Model can represent a 3D section of a rock slope consisting of planar benches (Figure 2.28). The rock mass can contain joint segments derived from a user-specified Discrete Fracture Network (DFN). In addition, the code can model non-steady fluid flow and pressure in a fracture network, considering several aspects of fluid-rock interaction like effective stress and transient pressure changes. The program implements a fully dynamic solution, which allows the representation of highly nonlinear behavior without numerical problems [20]. This approach seems very promising for understanding the key mechanisms leading to slope failure in certain conditions. However, the code is still relatively new and more validation work is necessary before it is applied to critical cases. Furthermore, obtaining realistic input data for both the intact rock properties and for the fracture networks are some of the main limitations of this and other similar methods based on discrete elements.

Figure 2.27: Spring lattice and Smooth Joint Model (Itasca).

Figure 2.28: Slope Model displaying displacement contours and induced fractures.
2.7 Summary and conclusions

Designing deep open pit slopes involves an additional degree of uncertainty compared to the design of shallower pits. This uncertainty can be related to a variety of factors influencing the rock mass response to an excavation, which can become more relevant as a pit gets deeper. In addition to the several gaps in knowledge and the limitations in current analysis tools presented in this chapter, the very limited experience in the design of deep open pits makes this process very difficult.

Understanding and predicting the most likely failure mode for a slope is a significantly important part of the pit slope design process, since this will mainly dictate the type of analysis to be carried out. The current knowledge of failure mechanisms in deep pit conditions is lacking, and it is not unlikely that unknown failure modes could be observed in the future. Failure modes in progressively deeper excavations can potentially get more complex. When fractures are non-persistent relative to the scale of the slope, potential failures will be partially controlled by these structures but will also involve fracture extension and failure of intact rock bridges. Similarly, large open pits have a greater potential for interacting with large scale geological structures. This interaction of failing discontinuities and rock fracturing is difficult to work with the current analysis tools. Conventional numerical modeling tools that use ductile and shear failure based constitutive models lack brittle failure capability, which can become a significant limitation for the design of large open pits.

Another important gap in the understanding of deep open pit behavior is the effect of in-situ stresses. The norm that pre-mining stresses have a negligible effect on stability, and that their impact is limited to slope deformations, might only apply in shallow pits where induced stresses are low or in plastic materials that are unaffected by extension strains. There is evidence that in high stress hard rock environments pre-mining stresses can be important for slope stability. In these conditions stress changes caused by the excavation sequence can promote tensile failure of the intact rock, resulting in rock mass damage and fracture extension. This damage can have an important effect on the strength properties
of the rock, and can potentially impact the mode of failure in a slope by interacting with preexisting discontinuities.

Numerical models, in particular continuum models, have limitations in their representation of the behavior of jointed rock masses. Rock masses cannot be modelled realistically without considering their structural defects. Unlike continuum methods, discrete methods are capable of representing a discontinuous rock mass formed by individual blocks, permitting large displacements along fractures and complete block detachment. These blocks can deform and yield, allowing more complex failure modes to be represented. However, yielding of intact rock is represented through a constitutive law, as in continuum methods, and fracturing of rock is not developed naturally. In a jointed rock mass, fracture initiation and interconnection with pre-existing fractures can be highly non-linear, and very hard to describe through a pre-determined stress-strain relation.

In order to represent the behavior of fractured rock masses more accurately, models need to be able to represent intact rock brittle failure and yielding along discontinuities simultaneously. Recent work in modelling damage processes in brittle rocks has put increasing emphasis on the role of extension fractures, which can form under high compressive stresses and low confining levels. Models based on bonded particles and lattice methods are able to represent this fracture formation process by considering the rock brittleness and strength characteristics at the particle level. They can also include a discrete fracture network, as in the SRM approach, allowing representing the interaction of pre-existing fractures with intact rock failure. These approaches overcome the limitations of other numerical methods, making a pre-defined constitutive model unnecessary.

The less demanding formulation of lattice models makes them more suitable for large scale problems. This makes codes like Slope Model an adequate tool for studying the stability of deep open pit rock slopes. In particular these codes can be capable of assessing the effect of stress, as well as the impact of in-situ stresses in rock mass strength and stability. Despite the advantages of bonded particle and lattice methods, the lack of knowledge of the geometry
and distribution of preexisting fractures is perhaps their main limitation for more general applications.
Knowledge of the in-situ stress field is necessary to predict the response of a rock mass to the disturbance caused by an excavation [2]. Reliable data of these stresses is a prerequisite for the design of underground excavations and its importance is well recognized, particularly if these stress magnitudes are high. In contrast, the effect of stresses in large surface excavations has not been studied as much.

In general, in-situ stress measurements for pit slope design are rarely performed and in most cases the stress field is roughly estimated from regional data. An explanation to this limited emphasis on pre-mining stresses in slope design is the belief that these have a very limited influence in stability. It is often assumed that the stresses around an excavation are low and only gravitational stresses are considered. This assumption may be adequate for small slopes but it needs to be questioned for the design of very large slopes [46].

3.1 Local in-situ stress field

Frequently, the local stress field can be significantly different compared to the regional one. The state of stress in an element of rock depends on both the current loading conditions in the rock mass and the stress path defined by its geologic history. Mechanical processes such as brittle fracture growth and sliding along discontinuities can modify the stress field. Changes in the state of stress can also be associated to temperature changes, as well as chemical processes such as leaching, precipitation and recrystallization [9]. As a result, the state of stress at a particular site can be highly complex and heterogeneous.

In a broad scale, the state of stress in the upper part of the earth’s crust is determined mainly by tectonic forces induced by several sources (Figure 3.1) and by gravitational forces [119]. However, cooling and shrinking of intrusive rocks, faulting and geologically-active thrusting seem to control the local magnitude and orientation of in-situ stresses near the
surface [61]. Furthermore, local in-situ stresses are also affected by several site-specific factors such as

- local geology
- faults and other discontinuities
- rock mass properties
- topography

Rock masses are not homogeneous and different geologic features will tend to alter the stress field. This is particularly true near geological structures where stress magnitudes can increase or decrease significantly, and stress orientations can rotate as much as 90° when these discontinuities are crossed [74]. Even in a large relatively homogeneous rock mass, in-situ stress magnitudes and orientations are highly variable [77]. For this reason, measurements taken close to geologic structures might not necessarily represent the far-field stress situation.

![Sources of Tectonic Stress](image)

**Figure 3.1:** Sources of tectonic stress [119].

Similarly, rock mass strength and stiffness can also impact the stress distribution within a rock mass. In tectonically stressed areas, stronger rocks are likely to show major principal stress magnitudes significantly greater than both the vertical stress and the other horizontal component [9]. Stronger rocks are able to withstand higher horizontal stresses compared to
weaker rock. In the same way, stiffer rock masses will tend to take a bigger portion of the horizontal stresses if the adjacent rock is softer.

The topography of a site can not only influence stress magnitudes but the orientation of the principal stresses as well [80]. The measured stress at a point usually is a combination of the pre-mining state of stress plus components induced by other sources such as topography or nearby large excavations, which needs to be considered when interpreting measurement results (Appendix A). In mountainous areas with high topographic reliefs, the common assumption that one of the principal stresses is vertical is not always valid. In this case, additional measurements should be carried out to establish the principal stress directions as well as their magnitudes.

### 3.2 Local in-situ stress determination

Pre-mining stresses may be approximated from regional stress compilations, but it is usually necessary to measure the local stresses in-situ since local stress conditions can vary considerably from regional conditions. The process of estimating the complete in-situ state of stress can be technically difficult and costly. Stress cannot be determined directly and can only be inferred by disturbing the rock. This disturbance needs to be measured (e.g. strain, displacement, hydraulic pressure) and analyzed under some assumptions on the rock mass constitutive behavior, resulting in an estimate of the stress components.

Since stress is a second order tensor, at least six independent measurements need to be carried out in different directions at a certain location. However, if the directions of the principal stresses are known, only three measurements are necessary at each location. Several measuring techniques are available which can provide reasonable estimates for the different stress magnitudes when applied in favorable rock conditions.

Between the many techniques available for performing stress measurements, hydraulic fracturing the overcoring method are the most widely used approaches in mining and civil engineering projects [27]. Hydraulic fracturing has been used in deep vertical holes in various rock conditions since it was proposed by Fairhurst [33]. At high depths, hydraulic pressure
creates a vertical fracture parallel to the least horizontal stress, which allows determining the stress magnitude normal to the induced fracture (Figure 3.2). However, when horizontal stresses are higher than the vertical stress, the fracture plane will be horizontal so the method can only provide the vertical stress magnitude[57].

![Figure 3.2: Typical pressure vs. time plot during a hydraulic fracturing test.](image)

In this situation, the magnitude obtained will correspond to the weight of the overlying rock, so the method cannot provide information on horizontal stress magnitude. In general, this will be the case at shallower depths, where the horizontal to vertical stress ratio tends to be greater than one. This reduces the applicability of hydraulic fracturing to mining projects.

Overcoring methods allow determining in-situ stresses by analyzing strain measurements taken before and during overcoring [27] (Figure 3.3). Despite its wide use in mining applications, this method can be very hard to apply in hard stress conditions [64]. If stresses are too high, micro fractures will develop around the borehole and the necessary assumption of linear elastic behavior is violated, limiting the confidence on this method [76]. Moreover, the maximum depth at which overcoring has been used successfully is less than 500 meters in good rock conditions, which is insufficient for the increasing depths of open pits [92].

Recently, the acoustic emission (AE) method has become more common for carrying out stress measurements [59, 99, 111] but still has not gained wide acceptance in mining.
Figure 3.3: Steps during overcoring strain measurements.

applications. This method can be used as an indirect method for estimating the stress field from core samples[59]. When these cores are loaded in uniaxial compression, acoustic emission increases when the stress reaches a level greater than that which the rock has previously experienced [62] (Figure 3.4).

Figure 3.4: Typical AE count for rock sample [111].

The AE method has the advantage that measurements can be done in the lab, rather than in the field, unlike the hydraulic fracturing and overcoring methods which can greatly
reduce its cost and complexity [111]. However, there are several uncertainties in this method and has been providing variable results. It is not always clear if it supplies the current in-situ stress or the maximum stress during the rock’s deformation history [25]. Also, rock samples are subject to core damage, as well as core disking in hard stress situations which makes testing not possible.

### 3.3 In-situ stress measurements compilation

Several stress measurements were compiled from different databases in order to study the spatial distribution of in-situ stresses. These measurements were obtained from numerous mines and tunnels, as well as from published data [11, 38, 60, 61]. Measurements were grouped in five different regions, Australia, Canada, United States, Scandinavia and Southern Africa. In most of these databases it was assumed that one of the principal stresses was vertical. Also, some of them only included the average magnitude between the two horizontal principal stresses.

#### 3.3.1 Vertical stress

It can be noted from Figure 3.5 that measured vertical stresses show a relatively low dispersion, following a near linear relation with depth. Usually the vertical stress is estimated by calculating the weight of the overlying rock, with \( \sigma_v = 0.027z \). In this case \( \sigma_v \) is the vertical stress component in MPa and \( z \) is the depth in meters. Departures from this theoretical value can be explained by the variation in unit weight of rocks, the use of different measuring techniques, and due to specific geologic circumstances.

The dispersion between measured vertical stresses and the values predicted by considering the overburden stress was quantified through the coefficient of determination \( (R^2) \). The value of \( R \) ranges from zero to one, and the better the correlation between the data and the linear regression the closer this value is to one. According to these results, the assumption that the vertical stress component can be estimated by the weight of the overburden seems reasonable in most cases (Figure 3.6). There are some exceptions like in Scandinavia and Australia,
where the vertical stress component tends to be higher than the predicted ones. In most of the other regions, the measured vertical stress tends to correlate with the weight of the overlying rock. Despite this correlation, if measurements for the vertical stresses are available these should be applied and not corrected with the theoretical values.

3.3.2 Horizontal stress

Measured horizontal stresses tend to increase with depth. This is expected as the higher confinement provided by the overburden allow the rock mass to sustain higher horizontal stress magnitudes. However, stress measurements present a high dispersion, and some important differences are observed between different regions (Figure 3.7). For instance, Australian data show the highest horizontal stress magnitudes, particularly at shallow depths. As depth increases, the magnitudes exhibit a moderate increase while maintaining a high dispersion. The United States data also present a high dispersion in the stress magnitudes with the measurements both above and below the lithostatic stress magnitudes. In contrast, Scandinavia and Canada show high horizontal stresses, which remain relatively high for all of the depth
range. Measurements from Southern Africa tend to have a lower dispersion and moderate stress magnitudes.

Horizontal stresses are usually represented as a proportion of the vertical stress. The horizontal to vertical stress ratio, referred as the $k$ ratio, is defined as the horizontal stress magnitude divided by the vertical stress magnitude at a certain point. Figure 3.8 shows the average $k$ ratio between the sub-horizontal principal stresses for different regions. It can be noted that there are important differences between these regions on the magnitudes of $k$. It can also be observed that $k$ can present very different trends with depth. For example, some regions as South Africa tend to show small variations of $k$ as depth increases, converging to a value between 0.5 and 1. In contrast, the United States seems to present highly variable $k$ ratios, with high magnitudes near the surface and low values at higher depths. Some other regions like Australia or Scandinavia show relatively high magnitudes of $k$ at all depths.

The $k$ ratio is known to vary spatially. Figure 3.8 also shows an overall trend for the $k$ ratio as being higher near the surface, and decreasing with depth. The dispersion of $k$ is the highest in the first 500 meters and it declines as depth increases. In general, $k$ ratios over 1 are found in compression zones as in most intra-plate regions, while $k$ ratios of 0.5 or less are usually found in extensional zones as in the East African Rift zone [11, 40, 117]. Continental areas known to have anomalous high $k$ ratios include the Yilgarn region in
Figure 3.7: Average horizontal stress measurements by depth and region.

Figure 3.8: Average $k$ ratio vs. depth for different regions.
Western Australia [60], Scandinavia [83], the Eastern United States [119] and the Canadian Shield [69]. Conversely, areas with recognized low horizontal stresses include Arizona, as well as South and Eastern Africa [38, 119].

Despite the regional stress patterns described above, in-situ stress magnitudes are highly influenced by local tectonics and some of the factors presented in Section 3.1. As a result, these regional trends are not a replacement for stress measurements, but they might be useful for preliminary analyses or if the number of measurements are limited. The scatter shown in stress measurements near the surface could be due to local geologic settings, in particular to variable rock and discontinuity conditions.

The stress magnitudes that a rock mass can withstand may be limited by the orientation and shear strength of local structures. As a consequence, high horizontal stresses would not be sustained in highly fractured rock masses near the surface, where the confinement provided by the weight of the overburden is lower. In other words, geologic structures might be controlling or moderating the stresses that can locally exist in the rock mass. As a result of these local conditions, an important dispersion in the $k$ ratio magnitudes should be expected within a particular region, which emphasizes the need of performing stress measurements at a particular site for design purposes. An adequate description of the stress field can be achieved through a stress measurement program applying the methods described in Section 3.2. The far-field stresses can be then determined by using the procedure in Appendix A.

### 3.4 In-situ stress modeling approaches

Pre-mining stress models currently used in rock mechanics have been recognized to be inadequate [38]. There have been multiple attempts on establishing empirical relationships for predicting the in-situ stress field at a certain location, motivated by the technical difficulties and cost on carrying out stress measurements. Despite these various attempts, there is still no mathematical theory that accounts for the wide range of in-situ stress patterns recorded around the world [11].
A common approach for determining the in-situ stress field is to assume that the pre-mining principal stresses are vertical and horizontal. Then, the vertical stress component can be estimated by the weight of the overburden. However, this assumption is not always true, as it was shown in Section 3.3.1.

In the case of horizontal stresses, there are several approaches that can be applied for determining their magnitude. The more commonly used methodologies in slope design are based in the assumption of a fixed \( k \) ratio, a variable \( k \) ratio or a fixed \( k \) ratio plus locked-in horizontal stresses. Which approach is more adequate will depend on each particular case.

### 3.4.1 Constant \( k \) ratio with depth

Applying a constant \( k \) ratio results in a linear increase in horizontal stress with increasing depth, following the relation \( \sigma_h = k \sigma_v \). In this case stresses at the surface will always be zero. However, as it was shown in the previous section, in some regions high \( k \) ratio values have been obtained near the surface. In some settings the \( k \) ratio tends to change with depth, being higher at shallower depths and getting smaller with increasing depth.

### 3.4.2 Decreasing \( k \) ratio with depth

Several mathematical expressions proposed by different authors are available for representing the variation of the in-situ stresses with depth [2]. Some of these relations are intended to be used at specific sites while others are aimed for different regions in the world. One of these expressions, which was presented by Sheorey [100], attempts to describe the variation of the \( k \) ratio as a function of depth and the horizontal stiffness of the rock mass. Sheorey’s expression for \( k \) is shown in Equation 3.1, where where \( E_h \) is the Elastic Modulus of the rock mass in the horizontal direction, and \( z \) is the depth below the surface of the location in which \( k \) is being estimated.

\[
k(z) = 0.25 + 7 \, E_h \left( 0.001 + \frac{1}{z} \right)
\]  

(3.1)
Sheorey’s relation was obtained from an elasto-static thermal stress model of the earth, considering the variation of elastic constants, density and thermal expansion coefficient through the earth’s crust and mantle. This relation implies that higher stresses can be sustained in hard, stiff rocks. In contrast, softer rocks can only withstand a limited horizontal stress without yielding or fracturing. This fact is supported by observations on stress measurements, which show that higher stress magnitudes are only obtained in very competent rock masses [32, 118].

Figure 3.9 shows several of the Sheorey’s curves for different assumed Elastic Modulus. These curves are superposed with $k$ ratios obtained from stress measurements data. It can be observed that most of the measurements fall between these curves, and that in some regions they present a better correlation than a fixed $k$ ratio.

Figure 3.9: Sheorey’s curves superposed to $k$ ratios obtained from stress measurements.
3.4.3 **Horizontal stress magnitudes**

Despite the general observed trend of the $k$ ratio decreasing with depth, many exceptions are possible and the most adequate in-situ stress model needs to be carefully assessed for each particular case. Figure 3.10 shows the horizontal stresses resulting from a variety of fixed $k$ ratios, together with several variable $k$ ratio curves. Assuming a constant $k$ ratio results in lower in-situ horizontal stresses at shallower depths when compared to Sheorey’s curves. At higher depths, constant $k$ ratios result in higher horizontal stresses. In contrast, the different Sheorey curves tend to give higher in-situ stress magnitudes near the surface and lower magnitudes at higher depths relative to fixed $k$ ratios.

![Figure 3.10: In-situ horizontal stresses from Sheorey’s curves and fixed k ratios vs depth.](image)

From Figure 3.10 it can be noted that the horizontal stress magnitudes resulting from the variable $k$ ratio curves can be represented as a constant, locked-in tectonic stress component plus a variable stress component which increases with depth. Table 3.1 shows the equivalent locked-in stress component and the constant $k$ ratio for the Sheorey curves. The equivalent $k$ ratio values are significantly smaller, indicating a very moderate increase of horizontal stress magnitudes with depth.
Table 3.1: Horizontal stress equivalency for Sheorey’s curves and fixed $k$ ratio.

<table>
<thead>
<tr>
<th>Sheorey curve $E_h$ [GPa]</th>
<th>Fixed $k$ ratio equivalent $k$ ratio</th>
<th>locked-in stress [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>0.32</td>
<td>1.9</td>
</tr>
<tr>
<td>25</td>
<td>0.43</td>
<td>4.7</td>
</tr>
<tr>
<td>50</td>
<td>0.60</td>
<td>9.5</td>
</tr>
<tr>
<td>75</td>
<td>0.78</td>
<td>14.2</td>
</tr>
<tr>
<td>100</td>
<td>0.95</td>
<td>18.9</td>
</tr>
<tr>
<td>125</td>
<td>1.13</td>
<td>23.6</td>
</tr>
</tbody>
</table>

Measurements at different depths are necessary to assess the change of $k$ below the ground surface. This assessment is more critical for the design of deep pits, as the ones in massive porphyry deposits, due to the larger depth range involved. An increased depth range will lead to larger differences horizontal stresses resulting from the different in-situ modelling approaches. For instance, if only a single measurement or multiple measurements at similar depths are available, more than one stress-depth curve can be fitted to the stress data. In this case it will not be possible to establish a realistic variation of the stress magnitudes with depth, which can affect the modelling results.

Similarly, if high stress magnitudes are measured near the surface and no additional information is available, considering a fixed $k$ ratio can lead to an artificially high value of $k$ at higher depths. Establishing the variation of the horizontal stress magnitudes with depth is particularly important when studying stress-induced damage in rock since, together with the excavation geometry and rock mass properties, is one of the main factors controlling the final stress state around around an open pit.

3.5 In-situ stress in massive porphyry deposits

Porphyry type deposits give origin to the world’s largest open pit mines, as the Chuquicamata and Escondida mines (Chile), Grasberg (Indonesia), and the Bingham Canyon mine (Utah) [16]. For this reason, the in-situ stress settings in which they exist is relevant to this study. The term porphyry copper refers to large, relatively low grade deposits occurring
close to or in granitic intrusive rocks with porphyritic texture. These massive deposits are significant sources of copper, gold, and molybdenum.

Porphyry rocks and the surrounding host rock are typically veined and closely fractured and show strong alteration zones. Fractures are predominantly near vertical, dipping into the ore body. Mineralization occurs mostly on fractures or on alteration zones.

Based on the fact that porphyries occur in similar and very specific geologic environments, Lee et al. [61] suggested that the ore body and their immediate host rocks should have similar large-scale rock mass strengths and structural characteristics. As a result, they should be able to sustain similar stress regimes, regardless of the local principal stress orientations. Figure 3.11 shows measured stress magnitudes form different porphyries plotted against the sum of the principal stresses, where the intermediate principal stress is vertical.

![Figure 3.11: Sum of principal stresses vs. principal stress magnitudes in porphyry rocks [61].](image)

The average principal stress relations proposed by Lee et al. [61] for porphyry copper deposits and their surrounding rocks are the following:

- $\sigma_1/\sigma_2 \approx 1.4$
- $\sigma_2/\sigma_3 \approx 1.4$
- $\sigma_1/\sigma_3 \approx 2.0$
3.6 Summary and conclusions

This chapter addressed different aspects related to the in-situ stress field. The local stress state can be highly complex and variable, presenting significant differences to the regional stress field. These differences can include principal stress deviation in terms of magnitudes and orientation. As a consequence, stress measurements are usually necessary to obtain an adequate representation of the local stress conditions for slope stability analyses.

Some interesting findings were obtained from the stress measurement compilation. First, it seems like the common assumption of considering the vertical stress component as the weight of the overlying rock is reasonable in most regions. However, some areas tend to present greater vertical magnitudes that the ones predicted by this relation, as in Scandinavia and Western Australia.

In the case of the horizontal stress magnitudes, the dispersion of the measured values is greater than for the vertical component. Also, different trends of the $k$ ratio against depth can be observed depending on the region. Some regions are better described by constant $k$ ratios, while others seem to follow a decreasing $k$ ratio as depth increases.

General in-situ stress relations are not a replacement for an adequate stress measurement program. Still, regional stress data can be valuable for preliminary studies, or when site specific measurements are limited. In these cases, in order to increase the confidence in numerical analyses, it can be useful to carry out a parametric study considering different possible stress states assessing the impact of the in-situ stress state. This procedure can help determine if additional stress measurements are needed at a certain location. In addition, it is important to establish not only stress magnitudes at a point, but also their orientation and how these stresses change with depth at a specific site.
CHAPTER 4
STRESS ANALYSES AND DAMAGE IN HOMOGENEOUS ROCK MASSES

The final state of stress surrounding an excavation is a function of the in-situ stress, the excavation geometry and rock mass properties. It is anticipated that new geotechnical problems caused by unfavorable stress conditions will develop in future deep open pit mines [108]. Therefore, this chapter will assess the response of a rock mass to different in-situ stress fields and increasing pit depth in terms of ground deformations, final stress state and rock mass damage in homogeneous materials.

Numerical models allow computing the stress field and deformation response around complex-shaped excavations while incorporating different far-field stress conditions. In this chapter, Finite Element elastic analyses are used for determining slope wall displacements, stress path and zones of extension strain induced by an excavation. The results of these elastic analyses are complemented with the ones obtained from Slope Model, where a brittle material with equivalent elastic properties is applied. However, since Slope Model represents the rock mass as an array of discrete masses interconnected by springs, it cannot provide stress information as continuum codes. As a result, combining these two approaches can allow studying the effect of depth and pre-mining stress in homogeneous materials in terms of both rock mass damage and induced stress conditions.

4.1 Model geometry

The model geometry considered for the different analyses represent a 1200 meter pit slope excavated in steps of 200 meters with interramp angles of 45 degrees and 50 meter ramps, resulting in an overall angle of 40 degrees. The complete slope geometry is shown in Figure 4.1. This slope is assumed to be of infinite length in the out of plane direction so plain strain conditions are applied. The model is fixed at the base and has rollers on both vertical boundaries. These boundary conditions consider symmetry along the right side of
the model, resulting in a symmetrical open pit with a floor width of 400 meters. The left vertical boundary is set far enough so that its influence in the excavation can be ignored.

![Figure 4.1: Basic model geometry and boundary conditions.](image)

4.2 Models description

Two basic situations are analyzed in this chapter. The first one considers the rock mass as one homogeneous material, which is studied using both a Finite Element code and Slope Model. For the FEM analyses, the material properties used are shown in Table 4.1 for the elastic case. An additional Finite Element analysis was performed applying the Hoek-Brown brittle parameters suggested by Martin [77] for predicting extension fracturing, shown in Table 4.2. The properties applied to Slope Model are listed in Table 4.4.

<table>
<thead>
<tr>
<th>Material name</th>
<th>Density [kg/m³]</th>
<th>Young’s Modulus [GPa]</th>
<th>Poisson’s ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartz-Feldspar Gneiss</td>
<td>2650</td>
<td>78</td>
<td>0.2</td>
</tr>
</tbody>
</table>
Table 4.2: Hoek-Brown brittle parameters and elastic properties for homogeneous slope used in F.E. model (Case 1)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartz-Feldspar Gneiss</td>
<td>2650</td>
<td>78</td>
<td>0.2</td>
<td>175</td>
<td>0</td>
<td>0.11</td>
</tr>
</tbody>
</table>

The second case contemplates a layered slope composed of three homogeneous materials. In this case the upper portion of the slope is excavated in weaker rock, and as depth below the surface increases the rock mass includes stronger units (Figure 4.2). The three materials are assumed to be brittle for which Slope Model is used for the analyses. The elastic and strength properties for the different rock types are shown in Table 4.5. These material properties were selected from a list of rock types included in Slope Model and calibrated by Itasca.

Table 4.3: Hoek-Brown frictional parameters and elastic properties (Case 1)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartz-Feldspar Gneiss</td>
<td>2650</td>
<td>78</td>
<td>0.2</td>
<td>175</td>
<td>10</td>
<td>0.19</td>
</tr>
</tbody>
</table>

Table 4.4: Slope Model strength and elastic properties for homogeneous slope (Case 1)

<table>
<thead>
<tr>
<th>Material name</th>
<th>Density [kg/m³]</th>
<th>Young’s Modulus [GPa]</th>
<th>Poisson’s ratio</th>
<th>UCS [MPa]</th>
<th>Tensile strength [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartz-Feldspar Gneiss</td>
<td>2650</td>
<td>78</td>
<td>0.2</td>
<td>175</td>
<td>17.5</td>
</tr>
</tbody>
</table>

4.3 In-situ stress field cases

Several pre-mining stress regimes were applied to the two cases described in the previous section. The vertical stress component was determined from the weight of the overburden rock. The horizontal stress magnitudes were considered by applying both constant and
variable $k$ ratios with depth following Sheorey’s curves. As it is shown in Chapter 3, the
decreasing $k$ ratio curves can be expressed as a constant $k$ ratio plus a locked-in stress
magnitude (Figure 3.10). The nine resulting stress states used for the analyses are listed in
Table 4.6.

![Figure 4.2: Rock mass materials in layered slope case (Slope Model).](image)

Table 4.5: Slope Model strength and elastic properties for layered slope (Case 2)

<table>
<thead>
<tr>
<th>Material name</th>
<th>Density [kg/m³]</th>
<th>Young’s Modulus [GPa]</th>
<th>Poisson’s ratio</th>
<th>UCS [MPa]</th>
<th>Tensile strength [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandstone</td>
<td>1926</td>
<td>12</td>
<td>0.2</td>
<td>75</td>
<td>7.5</td>
</tr>
<tr>
<td>Quartz-Feldspar Gneiss</td>
<td>2650</td>
<td>78</td>
<td>0.2</td>
<td>175</td>
<td>17.5</td>
</tr>
<tr>
<td>Amphibolite Gneiss</td>
<td>2828</td>
<td>99</td>
<td>0.2</td>
<td>200</td>
<td>20</td>
</tr>
</tbody>
</table>

4.4 Slope displacements vs. rock mass damage

The first step taken for evaluating the response of a homogeneous rock mass to an exca-
vation was to quantify slope wall deformations with increasing depth for different pre-mining
stress conditions. The differences in expected ground deformations were determined for the
in-situ stress situations listed in Table 4.6. For this analysis, Slope Model was used to repre-
Table 4.6: Stress states considered for the numerical analyses

<table>
<thead>
<tr>
<th>In-situ stress case</th>
<th>$k$ ratio</th>
<th>Sheorey($E_h$) [GPa]</th>
<th>Locked-in stress [MPa]</th>
<th>Equivalent $k$ ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.0</td>
<td>-</td>
<td>0.0</td>
<td>1.0</td>
</tr>
<tr>
<td>2</td>
<td>1.5</td>
<td>-</td>
<td>0.0</td>
<td>1.5</td>
</tr>
<tr>
<td>3</td>
<td>2.0</td>
<td>-</td>
<td>0.0</td>
<td>2.0</td>
</tr>
<tr>
<td>4</td>
<td>2.5</td>
<td>-</td>
<td>0.0</td>
<td>2.5</td>
</tr>
<tr>
<td>5</td>
<td>$k = 0.425 + 175/z$</td>
<td>25</td>
<td>4.7</td>
<td>0.43</td>
</tr>
<tr>
<td>6</td>
<td>$k = 0.6 + 350/z$</td>
<td>50</td>
<td>9.5</td>
<td>0.6</td>
</tr>
<tr>
<td>7</td>
<td>$k = 0.775 + 525/z$</td>
<td>75</td>
<td>14.2</td>
<td>0.78</td>
</tr>
<tr>
<td>8</td>
<td>$k = 0.95 + 700/z$</td>
<td>100</td>
<td>18.9</td>
<td>0.95</td>
</tr>
<tr>
<td>9</td>
<td>$k = 1.125 + 875/z$</td>
<td>125</td>
<td>23.6</td>
<td>1.13</td>
</tr>
</tbody>
</table>

sent a brittle material (Table 4.4). Additionally, elastic analyses with equivalent properties were performed through Finite Element models. Comparing the results obtained from Slope Model with the FEM helped determining the relation between rock mass damage and deformation (Table 4.7). An example of these analyses, corresponding to the Sh(125) stress curve, can be found in Figure 4.3(a) which shows the Slope Model displacement contours while Figure 4.3(b) displays the ones resulting from the FE elastic model.

Table 4.7: Maximum slope displacements, relative error and fracture count for final pit

<table>
<thead>
<tr>
<th>Stress state</th>
<th>FE elastic displacement [m]</th>
<th>Slope Model displacement [m]</th>
<th>Slope Model fracture count</th>
<th>Displacements relative error</th>
</tr>
</thead>
<tbody>
<tr>
<td>$k = 1.0$</td>
<td>0.30</td>
<td>0.35</td>
<td>11</td>
<td>18%</td>
</tr>
<tr>
<td>$k = 1.5$</td>
<td>0.37</td>
<td>0.41</td>
<td>190</td>
<td>12%</td>
</tr>
<tr>
<td>$k = 2.0$</td>
<td>0.47</td>
<td>1.02</td>
<td>9741</td>
<td>119%</td>
</tr>
<tr>
<td>$k = 2.5$</td>
<td>0.57</td>
<td>1.45</td>
<td>2268</td>
<td>153%</td>
</tr>
<tr>
<td>Sh(25)</td>
<td>0.27</td>
<td>0.32</td>
<td>0</td>
<td>15%</td>
</tr>
<tr>
<td>Sh(50)</td>
<td>0.30</td>
<td>0.35</td>
<td>11</td>
<td>16%</td>
</tr>
<tr>
<td>Sh(75)</td>
<td>0.37</td>
<td>0.41</td>
<td>99</td>
<td>14%</td>
</tr>
<tr>
<td>Sh(100)</td>
<td>0.45</td>
<td>0.52</td>
<td>368</td>
<td>16%</td>
</tr>
<tr>
<td>Sh(125)</td>
<td>0.54</td>
<td>0.83</td>
<td>1136</td>
<td>53%</td>
</tr>
</tbody>
</table>

Figure 4.4 presents the maximum wall displacements for the final pit from constant $k$ ratios, while Figure 4.5 displays the ones obtained from decreasing $k$ ratio curves. It can be
Figure 4.3: Final pit displacement contours and damage for Sh(125) in a homogeneous material.
noted that the occurrence of stress-induced damage seen in Slope Model leads to considerable higher displacements compared to the FE elastic model. At lower stress levels, when the material is intact and in its elastic range, Slope Model tend to give maximum displacements for the final pit wall that are around 15% lower than the FE models. At higher stress levels, and when rock mass damage increases, the divergences in displacements become bigger. This means that despite the bias in the calculated displacements from both analysis tools, an important part of this deformation difference can be achieved to intact rock fracturing and dilation [55].

Figure 4.4: Final pit maximum displacements, rel. error and fracture count (constant k).

Figure 4.6 shows the maximum displacements obtained from Slope Model for the final pit wall together the number of induced fractures resulting from the different in-situ stress conditions (Table 4.6). In this plot fractures start growing when the maximum displacement exceed 0.34 meters in the final 1200 meter-deep pit. These results imply a near linear relation between maximum pit displacement and rock mass damage caused by intact rock fracturing, under the assumption that the rock mass is homogeneous. In this case it can be assumed that displacements are mainly elastic when maximum slope displacements are under 0.34
4.5 Critical extension strain determination

As it is discussed in Chapter 2, zones of extension fractures can be determined through an elastic analysis by establishing the locations where the minor principal strain exceed a critical value for a certain material. However, this approach is based on continuum elasticity, which does not account for formation of fractures within the rock mass. In contrast, Slope Model is a mechanics based approach which can represent strength loss due to intact rock fracturing. In this model, fracture growth is simulated through springs breaking in tension and shear when their maximum capacity is exceeded.

Extension strain magnitudes are a function of the elastic properties of the rock mass, as well as the principal stress magnitudes at a certain point, which can be calculated through a Finite Elements analysis. The challenge for applying the extension strain fracture criterion is to determine the critical magnitude that results in fracturing for a particular material. In this study the critical strain magnitude was found by comparing the results of the Finite
Elements analyses and Slope Model.

In order to find the critical extension strain value, the maximum extension strain computed from every elastic model is compared to the number of fractures resulting from the equivalent Slope Model run. Figure 4.7 displays the extension strain magnitude and the corresponding fracture number. The 54 data points resulted from the nine different stress states and the multiple excavation steps. In this figure it can be observed that the first cracks appeared at $\epsilon_3 = 0.00015$, which is assumed to be the critical strain value for the material ($\epsilon_{crit}$). This magnitude is within the range of critical values which have been published for different rock types obtained from uniaxial compressive tests [104].

4.6 Prediction of damage zones locations

Based on the critical strain magnitude, the damaged zones defined as areas where the minor principal strain is greater than $\epsilon_{crit}$ can be determined using numerical models. Since these zones change for every excavation stage, in order to predict fracture locations it is necessary to consider the envelope of zones where the critical extension strain was exceeded at any time during the excavation sequence. Figure 4.8 shows the zones where strains were larger than the critical value, superposed with an image of the fracture locations resulting
from tensile failure of springs in an equivalent Slope Model run. It can be seen that there is a reasonable agreement between the two models when using the critical strain magnitude determined above ($\epsilon_{\text{crit}} = 0.00015$). In other words, the extension strain criterion applied to elastic FE simulations compare well with the zones of fracturing determined by Slope Model. In contrast, Figure 4.9 shows the yielded elements in an equivalent FE resulting from applying the Hoek-Brown brittle parameters for fracture onset prediction. In this figure it can be noted that the brittle form of H-B was not as close as the extension strain approach to the Slope Model results for determining potential damaged locations.

### 4.7 Extension strain magnitudes in slopes

The maximum extension strain magnitudes obtained from each model were plotted against the different pit depths resulting from the excavation steps. Figure 4.10 shows the maximum extension strain for fixed $k$ ratios, while Figure 4.11 presents the extension strain magnitudes for the variable $k$ ratio curves. These figures show a clear increase in strain magnitudes after each excavation step. This increase is proportional to the horizon-
Figure 4.8: Induced fractures superposed by zones of $\epsilon_3 \geq 0.00015$ and deformation contours for Sh(125).

Figure 4.9: Yielded elements resulting from Hoek-Brown brittle parameters for Sh(125).
tal stresses resulting from the different in-situ stresses. Comparing the constant $k$ to the variable $k$ ratio results, it can also be noted that the maximum extension strain values are more sensitive to depth when constant $k$ ratios are applied. For instance, when a fixed ratio was used, extension strain magnitudes increased around 2.6 times when going from a 600 meter to a 1200 meter pit (Figure 4.10). In contrast, for the decreasing $k$ ratio curves, these magnitudes only increased 1.8 times for the same variation in pit depth (Figure 4.11).

![Figure 4.10: Maximum extension strain magnitude vs. pit depth (constant $k$ ratio).](image)

These figures also illustrate at which depth the critical extension strain is exceeded for the different in-situ stress states. For example, for $k = 1$ and $k = 1.5$, the maximum extension strain remain below the critical value for pit depths of less than 600 meters. The same can be observed for the Sh(25) and Sh(50) variable $k$ ratio curves. This means that below this depth, rock mass damage caused by stress-induced fractures will be very limited and should not affect rock mass strength. However, in this same in-situ stress situation, if depths increase there is a growing potential for the development of extension fractures. In high in-situ stress settings, as in some Australian mines, extension fractures could develop.
at much shallower depths. In some areas measured k ratios of 2.5 are not rare, condition that could promote fracturing as depths as low as 400 meters.

![Figure 4.11: Maximum extension strain magnitudes vs. pit depth (variable k ratio).](image)

According to the previous results, it can be anticipated that as pits increase their depths, stress-induced fracturing leading to rock mass damage might occur in strong and brittle rock masses. However, these results are only valid under the assumption that the rock mass is homogeneous and behaves elastically. The extension strain magnitudes as well as the critical strain value will likely change for different materials, so these results cannot be generalized.

### 4.8 Stress path and rock mass damage

Different areas within the rock mass are subject to varying stress conditions as an excavation progresses. In this section the loading history followed by two different points is studied through two-dimensional elastic analyses. Additionally, the results are compared with the number of stress-induced fractures resulting from equivalent Slope Model runs.

The fracture count combined with the stress history provided by the Finite Elements models allow identifying stress situations which could potentially result in rock mass damage,
and hence in a loss of strength. Damage is predicted through the extension strain criterion established in this study as $\epsilon_{\text{crit}} = 0.00015$ for the material used. As it is shown in the previous sections, the extension strain criterion seems to give better results for assessing fracture occurrence compared to the Hoek-Brown criterion in both frictional and brittle form (Table 4.3 and Table 4.2), based on the comparison with the Slope Model explicit fractures locations.

### 4.8.1 Control point 1

First, the stress path was determined in terms of principal stress for a fixed point located 20 meters above the final pit floor level, where the extension strain magnitude reached its maximum. This location is expected to show high damage levels, so tracking its loading history helps understanding how the different mining steps contribute to the damage of the final slope toe. A loss of strength at this location can be particularly unfavorable for slope stability.

Figure 4.12 and Figure 4.13 show the stress paths for constant and variable $k$ ratios, the extension strain criterion (with $\epsilon_{\text{crit}} = 0.00015$), as well as the frictional and brittle form of the Hoek-Brown criterion (Table 4.3 and Table 4.2). The furthest point to the right of the stress path represents the in-situ stress at a depth of 1200 meters. As the excavation advances there is a progressive loss of confinement which can be seen as a reduction in the magnitude of $\sigma_3$. In general $\sigma_1$ remains near constant in the first steps of the excavation but as the excavation floor moves deeper, $\sigma_1$ magnitudes start to increase due to stress concentrations. It can be noted that the existence of sharp corners on the model boundary can result in artificially high stress magnitudes as in the toe of a slope due to a stress intensity.

When the excavation reaches the control point, which at this step corresponds to the slope toe, there is a sudden increase in the $\sigma_1$ values. In this last excavation step the $\sigma_3$ magnitudes decrease abruptly, which could be expected since the control point is now located on a free surface. In this way areas near the slope toe will tend to have high $\sigma_1$ magnitudes but with very low confinement levels resulting in more likely or earlier damage.
Figure 4.12: Stress path for final pit slope toe (fixed k ratio).

Figure 4.13: Stress path for final pit slope toe (variable k ratio).
Looking at the stress paths resulting from the different in-situ stress states it is possible to anticipate at what stage of the excavation damage is initiated at the final pit level. This damage was assessed through extension strain criterion with $\epsilon_{\text{crit}} = 0.00015$. For almost every stress state evaluated, with the exception of the Sh(25) variable $k$ ratio curve, rock mass damage was predicted when the excavation reaches its final depth (Figure 4.12 and Figure 4.13). Also, damage in the final pit toe area originated mostly in the last excavation step. In this way, it is likely that the damage caused to the final slope toe by the preceding steps will be very limited. However, this situation will change if the excavation is modeled with smaller steps, possibly showing damage in the last few stages but at a similar excavation depth.

### 4.8.2 Control point 2

The second location considered for the stress path analyses was a point at the excavation floor. In this way the control point is always at the current slope toe. This can allow studying the stress conditions that promotes fracturing at the slope toe for a range of open pit depths, and establishing the excavation depth at which damage is likely to occur. As for the previous control point, the extension strain criterion with $\epsilon_{\text{crit}} = 0.00015$ was used for assessing the onset of damage.

Figure 4.14 and Figure 4.15 show the stress path followed by this moving point through the excavation sequence. The frictional and brittle Hoek-Brown criterions are also shown for the properties listed in Table 4.3 and Table 4.2. As it can be expected, there are important differences in the principal stress magnitudes for the several in-situ stress states considered. It can be observed that both $\sigma_1$ and $\sigma_3$ magnitudes grow linearly with pit depth. For fixed $k$ ratios, the critical strain value is exceeded in all cases, except when $k = 1$ (Figure 4.14). For $k = 2.5$, extension fractures are predicted after the third excavation step, corresponding to a pit depth of 600 meters. For $k = 2$, this occurs at a pit depth of 800 meters. For variable $k$ ratios, the critical value was only exceeded for the Sh(100) and Sh(125) curves, occurring at pit depths of 800 and 600 meters respectively (Figure 4.15).
Figure 4.14: Pit bottom stress path for different in-situ stress states (constant k ratio).

Figure 4.15: Pit bottom stress path for different in-situ stress states (variable k ratio curves).
4.8.3 Effect of stress path on fracture growth

In addition, stress path trajectories were compared with the fracture growth rate obtained from the Slope Model runs for the same stress conditions. This also allowed evaluating the different stress-based failure criterions for predicting the occurrence of extension fractures. The stress path and fracture growth rate plots for all of the in-situ stress conditions can be found in Appendix B.

Figure 4.16 presents an example of this analysis for $k = 2$. This figure shows the stress path against fracture growth rate as the excavation sequence progressed together with the different failure criterions. It can be noticed that when the stress path gets closer to the extension strain criterion, which occurs during the fourth excavation step, the fracture growth rate starts to increase. For the last two steps of the excavation, the stress path progresses further away from the fracture criterion which coincide with an increase in the fracture growth rate. In this case, the Hoek-Brown criterion with brittle parameters slightly over predicted fracturing, which matches what was observed on previous sections. In contrast, the traditional Hoek-Brown criterion could not anticipate extension fractures in the slope.

![Figure 4.16: Pit bottom stress path and rock mass damage rate for k=2.](image-url)
The results shown above confirm that the extension strain criterion can be capable of predicting the occurrence and also the extent of rock mass damage. As it was said earlier, the main difficulty for applying this criterion is finding the critical strain value of the material that result in fracturing, which can be done with the procedure described in Section 4.5. However, this criterion may only be applicable for the near-field rock mass around an excavation and is not appropriate for estimating strength properties far from the excavation boundary.

4.9 Influence of pit depth and in-situ stress in rock mass damage

In the previous sections, rock mass damage was estimated through the extension strain criterion. In this section, rock mass damage is determined through the Slope Model code by applying different in-situ stress conditions. The damage level is quantified by tracking the number of broken springs in the lattice for each excavation step. This allows establishing relationships between fracturing levels, pit depth and in-situ stresses, as well as how these factors can impact stability.

Two different rock masses, corresponding to the ones described in Section 4.2, are considered. In the first case, the rock is composed entirely by a homogeneous material with the properties shown in Table 4.2. The second case contemplates a layered rock mass composed by three homogeneous materials (Table 4.4). As in the earlier sections, the analyses include pre-mining stresses for both fixed and variable $k$ ratios.

4.9.1 Case 1: Homogeneous rock mass (1 material)

Figure 4.17 shows an example of the Slope Model runs for the Sh(125) variable $k$ ratio curve in a homogeneous material. This figure displays the induced fractures in black after the excavation is completed. Additionally, a plot with the crack number history versus computing time is superposed to the model to show the effect of the excavation steps in terms of fracture count.
After each step is excavated a rapid increase in the number of stress-induced fractures can be observed in the cracks history plot. After a few timesteps, the fracture growth rate start to decrease until new fractures eventually stop forming. When a new step is excavated the same process is observed and the fracture count converges to a constant value, indicating that the model has reached equilibrium. The fracture count history plot shows clearly that the rock mass damage level progressively increases for each additional mining step, more than doubling in the last step of the excavation Figure 4.17.

A similar trend in the fracture count can be observed for the different stress states, which are shown in Table 4.8 as a function of the pit depth. If the fracture count does not converge to a constant value, after the model has been computed for sufficient time, it can be an indicator of slope instability. In this case the stability of the slope needs to be assessed by analyzing the slope displacements history plots at different locations. If the slope is stable, these displacements will converge to a fixed value.
Table 4.8: Number of fractures by excavation step for different stress states (homogeneous rock mass)

<table>
<thead>
<tr>
<th>Depth</th>
<th>k=1</th>
<th>k=1.5</th>
<th>k=2</th>
<th>k=2.5</th>
<th>Sh(25)</th>
<th>Sh(50)</th>
<th>Sh(75)</th>
<th>Sh(100)</th>
<th>Sh(125)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>200</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>400</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>4</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>3</td>
<td>14</td>
</tr>
<tr>
<td>600</td>
<td>0</td>
<td>1</td>
<td>2</td>
<td>27</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>11</td>
<td>48</td>
</tr>
<tr>
<td>800</td>
<td>0</td>
<td>2</td>
<td>32</td>
<td>455</td>
<td>0</td>
<td>0</td>
<td>3</td>
<td>40</td>
<td>213</td>
</tr>
<tr>
<td>1000</td>
<td>1</td>
<td>36</td>
<td>358</td>
<td>2031</td>
<td>0</td>
<td>1</td>
<td>21</td>
<td>144</td>
<td>706</td>
</tr>
<tr>
<td>1200</td>
<td>12</td>
<td>210</td>
<td>1162</td>
<td>9738</td>
<td>0</td>
<td>11</td>
<td>103</td>
<td>433</td>
<td>1505</td>
</tr>
</tbody>
</table>

The results from Table 4.7 are displayed in Figure 4.18 using a log scale for the fracture count. Here the near exponential relation between pit depth and rock mass damage is very clear. This figure also shows a strong correlation between in-situ stress and rock mass damage.

![Figure 4.18: Rock mass damage envelope vs. pit depth for different pre-mining stress fields in a homogeneous rock mass.](image)

In addition, it is observed that when in-situ stresses resulting from fixed $k$ ratios are applied, damage tends to start later in the excavation sequence and showing a relative slow
increase in the first steps. As the excavation advances the number of induced fractures grow rapidly with most of the damage being developed in the very last excavation step. In some cases the number of fractures increased in more than five times during this last stage.

In contrast, for the variable $k$ ratio curves, damage tends to start at earlier stages of the excavation, growing slower compared to the fixed $k$ ratio cases. This can be explained by the fact that the decreasing $k$ ratio curves result in higher horizontal stresses at shallow depths, which will tend to induce more damage at this level, and lower magnitudes at the final pit depth.

### 4.9.2 Case 2: Layered rock mass (3 homogeneous materials)

The results from the Slope Model runs for the layered rock mass, composed of three homogeneous materials, show similar trends compared to the single homogeneous material case. An example of these models corresponding to Sh(125) is displayed in Figure 4.19. The fractures count obtained for each case are listed in Table 4.9.

![Figure 4.19: Layered slope run in Slope Model for Sh(125).](image-url)
Table 4.9: Number of fractures by excavation step for different stress states (layered slope)

<table>
<thead>
<tr>
<th>Depth</th>
<th>k=1</th>
<th>k=1.5</th>
<th>k=2</th>
<th>k=2.5</th>
<th>Sh(25)</th>
<th>Sh(50)</th>
<th>Sh(75)</th>
<th>Sh(100)</th>
<th>Sh(125)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
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<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>200</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>9</td>
<td>225</td>
<td>1537</td>
</tr>
<tr>
<td>400</td>
<td>0</td>
<td>0</td>
<td>7</td>
<td>25</td>
<td>0</td>
<td>0</td>
<td>42</td>
<td>364</td>
<td>2068</td>
</tr>
<tr>
<td>600</td>
<td>0</td>
<td>1</td>
<td>9</td>
<td>36</td>
<td>0</td>
<td>0</td>
<td>44</td>
<td>376</td>
<td>2124</td>
</tr>
<tr>
<td>800</td>
<td>0</td>
<td>2</td>
<td>21</td>
<td>247</td>
<td>0</td>
<td>0</td>
<td>46</td>
<td>397</td>
<td>2240</td>
</tr>
<tr>
<td>1000</td>
<td>1</td>
<td>14</td>
<td>164</td>
<td>1286</td>
<td>0</td>
<td>1</td>
<td>54</td>
<td>455</td>
<td>2524</td>
</tr>
<tr>
<td>1200</td>
<td>12</td>
<td>77</td>
<td>581</td>
<td>1794</td>
<td>0</td>
<td>4</td>
<td>80</td>
<td>594</td>
<td>2984</td>
</tr>
</tbody>
</table>

Figure 4.20: Rock mass damage envelope vs. pit depth for different pre-mining stress fields in a layered rock mass.

As in the homogeneous slope situation, the values for the different in-situ stress states are plotted in a logarithmic scale as a function of the pit depth (Figure 4.20). This figure shows that damage tends to increase with each excavation step. However, in this case damage levels are much higher in the first stages due to the weaker material being excavated in the first 400 meters.
When the excavation reaches the stronger material (with the properties shown in Table 4.5), damage increases at a much lower rate. This was particularly true for the variable k ratio curves where most of the fractures developed in the first mining steps while the weaker material was being excavated. In contrast, for the constant k ratio situations the fracture growth rate showed a steady increase until reaching its maximum rate when the excavation was at its final depth. These results suggest that damage distribution can get increasingly complex as more materials are included in the analyses, especially when there is a high contrast in strength properties. In this case rock mass damage will not necessarily occur near the slope toe, and its potential for impacting slope behavior can be more difficult to assess.

4.10 Summary of results and conclusions

In this chapter the effect of depth and in-situ stress on the behavior of a rock slope excavated in homogeneous materials was evaluated. This assessment was done through elastic Finite Elements analyses together with the Slope Model code, which was used for representing the brittle behavior of rock. The Slope Model runs also allowed studying the relation between rock mass damage and deformation, which was observed to be near-linear.

Zones of potential rock mass damage can be identified through the extension strain criterion, based on a known critical strain magnitude. This approach is able to predict fracture occurrence through an elastic analysis that is in good agreement with the Slope Model results. A procedure for finding the critical extension strain value through numerical modeling is also presented in this chapter. This critical magnitude can be found by relating the maximum extension strain values from FEM analyses with the number of fractures resulting from the Slope Model runs for each combination of in-situ stress and pit depth.

Stress path analyses also show that the extension strain criterion is more adequate for predicting brittle fracture onset than the Hoek-Brown failure criterion. However, this criterion is based on continuum elasticity, which does not account for formation of fractures within the rock mass. As a result, the extension strain criterion can only be used for identifying damage potential and for establishing the need for more advanced numerical modeling.
techniques.

Rock mass damage analyses revealed that there is a near exponential relationship between pit depth and the number of stress-induced fractures, indicating a considerable potential for rock mass damage to develop in deep open pits. In a similar way, in-situ stress magnitudes present a strong influence on rock mass damage levels, with higher stress magnitudes resulting in a noticeable increase in the number of induced fractures. Areas near the slope toe and pit floor can be particularly susceptible to damage, and as a result to a possible loss of strength. However, based on the results from this chapter, it is unlikely that rock mass damage can lead to a global slope failure, at least in homogeneous materials.
CHAPTER 5
STRESS-INDUCED DAMAGE IN JOINTED ROCK MASSES

It is possible that new geotechnical problems resulting from increasing pit depths and pre-mining stress magnitudes will develop in the future. In the previous chapter it was shown that some stress conditions could promote fracturing in homogeneous rock. This chapter will study the effect of stress in jointed rock masses, where failure mechanisms involve a combination of shear on pre-existing structures and stress-induced fracturing.

Failure modes with partial structural control are particularly complex to analyze with traditional design tools, which cannot represent fracture growth and sliding along discontinuities simultaneously. Since classical design approaches do not account for such failure mechanisms, future deep pits may be under designed, leading to potential pit-scale failures and unexpected business risk. In order to address their limitations, more advanced numerical tools capable of modeling more complex processes have been developed. One of these codes, Slope Model, was used in this chapter to evaluate the effect of in-situ stresses and depth in strong, jointed rock masses.

5.1 Model geometry and boundary conditions

The basic slope geometry considered for the analyses is the same as in Chapter 4 for homogeneous rock masses. The 1200 meter pit slope is excavated in 6 steps of 200 meters each, resulting in an overall slope angle of 40 degrees (Figure 5.1). In this case, the slope is represented through a three-dimensional slice of material with a thickness of 170 meters using the Slope Model code.

The base of the model is fixed, while the four vertical boundaries have rollers allowing displacements in the corresponding in-plane direction. As it was said in Chapter 2, Slope Model represents the rock mass as an array of masses connected by springs which can break
in shear or tension, simulating intact rock fracturing. The code also allows defining explicit
fractures, making it capable of simulating failure modes with partial structural controls.

5.2 Description of analyzed cases

Four different rock masses are considered for the analyses. The different materials and
the discontinuities friction coefficients used for every case are listed in Table 5.1. The intact
rock properties for each of these materials, as well as the fracture strength properties are
shown in Table 5.2 and Table 5.3 respectively.

The selected cases represent common situations where traditional continuum and dis-
continuum codes have difficulties simulating the rock mass behavior. In all of these cases,
a slope failure will necessarily involve intact rock failure combined with sliding along pre-
existing fractures. The formation of a complete failure surface can only occur after intact
rock bridges have been damaged. Parallel faults that are dipping steeply around an ore body
are common in massive deposits like copper porphyries. Depending on the slope orientation

Figure 5.1: Slope geometry used in the Slope Model runs.
and pit location, these structures will be dipping into the slope or running parallel to the pit face, as in the situations described below.

Table 5.1: Rock mass materials and discontinuities friction coefficients

<table>
<thead>
<tr>
<th>Case #</th>
<th>Material name</th>
<th>Joint strength (tan φ)</th>
<th>Fault strength (tan φ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>Kimberlite</td>
<td>weak (0.1)</td>
<td>-</td>
</tr>
<tr>
<td>Case 2</td>
<td>Kimberlite</td>
<td>weak (0.1)</td>
<td>med strong (0.75)</td>
</tr>
<tr>
<td>Case 3</td>
<td>Sandstone (upper 400m)</td>
<td>weak (0.1)</td>
<td>med strong (0.75)</td>
</tr>
<tr>
<td></td>
<td>Kimberlite (lower 800m)</td>
<td>weak (0.1)</td>
<td></td>
</tr>
<tr>
<td>Case 4</td>
<td>Sandstone (upper 400m)</td>
<td>weak (0.1)</td>
<td>med strong (0.75)</td>
</tr>
<tr>
<td></td>
<td>Kimberlite (lower 800m)</td>
<td>weak (0.1)</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.2: Intact rock properties

<table>
<thead>
<tr>
<th>Material name</th>
<th>Density [kg/m³]</th>
<th>Young’s Modulus [GPa]</th>
<th>Poisson’s ratio</th>
<th>UCS [MPa]</th>
<th>Tensile strength [MPa]</th>
<th>Porosity [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kimberlite</td>
<td>2757</td>
<td>53</td>
<td>0.3</td>
<td>110</td>
<td>11.5</td>
<td>20</td>
</tr>
<tr>
<td>Sandstone</td>
<td>1926</td>
<td>12</td>
<td>0.2</td>
<td>75</td>
<td>7.5</td>
<td>20</td>
</tr>
</tbody>
</table>

Table 5.3: Discontinuities strength properties

<table>
<thead>
<tr>
<th>Discontinuity</th>
<th>Surface condition</th>
<th>Friction coefficient</th>
<th>Friction angle [deg]</th>
<th>Cohesion [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint set 1</td>
<td>weak</td>
<td>0.1</td>
<td>6</td>
<td>0</td>
</tr>
<tr>
<td>Fault 1</td>
<td>med strong</td>
<td>0.75</td>
<td>37</td>
<td>0</td>
</tr>
<tr>
<td>Fault 2</td>
<td>med strong</td>
<td>0.75</td>
<td>37</td>
<td>0</td>
</tr>
</tbody>
</table>

5.2.1 Case 1

The first case considers a slope excavated in a rock mass containing one set of discontinuous joints, dipping 15 degrees into the slope face (Figure 5.2(a)). In this situation, the expected failure mechanism is a combination of sliding along discontinuous structures and brittle failure of rock bridges (step-path failure).
5.2.2 Case 2

The second analyzed case is similar to the previous one, but a fault dipping 60 degrees is added to the discontinuous joint set (Figure 5.2(b)). In this situation, the anticipated failure mode involves a combination of a step-path failure mechanism and sliding along a large scale structure. As in the previous case, fracture extension and intact rock failure are necessary for developing a continuous failure surface.

5.2.3 Case 3

In the third case, which is similar to Case 2, the rock mass in the upper portion of the slope is considered to be weaker as it is shown in Table 5.1. This situation represents a rock mass where the rock near the surface has lost part of its strength due to weathering. The distribution of the non-persistent joint set, as well as the location and dip angle of the fault is identical to Case 2 (Figure 5.2(c)).

5.2.4 Case 4

The last case analyzed, includes a series of parallel faults dipping 80 degrees into the slope combined with a non-persistent joint set dipping into the slope face at an angle of 15 degrees (Figure 5.2(d)). The expected failure mode in this situation is a toppling kind of failure, including flexing of the rock columns between faults, and sliding along the joint set and failed rock bridges.

5.3 Considered in-situ stress states

Nine different pre-mining stress regimes are applied to the four cases described in Section 5.2, which include four fixed $k$ ratio and five variable $k$ ratio stress conditions. These stress states are shown in Table 5.4, and correspond to the same cases considered in Chapter 4. In this chapter the vertical stress component is also estimated from the weight of the overburden rock.
Figure 5.2: Slope Model diagrams for analyzed rock masses.
Table 5.4: Stress states considered for the Slope Model analyses

<table>
<thead>
<tr>
<th>Stress state</th>
<th>$k$ ratio</th>
<th>Sheorey($E_h$)</th>
<th>$\sigma_h$ at z=0 [MPa]</th>
<th>$\sigma_h$ at z=2000 [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.0</td>
<td>-</td>
<td>0</td>
<td>54</td>
</tr>
<tr>
<td>2</td>
<td>1.5</td>
<td>-</td>
<td>0</td>
<td>81</td>
</tr>
<tr>
<td>3</td>
<td>2.0</td>
<td>-</td>
<td>0</td>
<td>108</td>
</tr>
<tr>
<td>4</td>
<td>2.5</td>
<td>-</td>
<td>0</td>
<td>135</td>
</tr>
<tr>
<td>5</td>
<td>$k = 0.425 + 175/z$</td>
<td>25</td>
<td>4.7</td>
<td>27.7</td>
</tr>
<tr>
<td>6</td>
<td>$k = 0.6 + 350/z$</td>
<td>50</td>
<td>9.5</td>
<td>41.9</td>
</tr>
<tr>
<td>7</td>
<td>$k = 0.775 + 525/z$</td>
<td>75</td>
<td>14.2</td>
<td>56.0</td>
</tr>
<tr>
<td>8</td>
<td>$k = 0.95 + 700/z$</td>
<td>100</td>
<td>18.9</td>
<td>70.2</td>
</tr>
<tr>
<td>9</td>
<td>$k = 1.125 + 875/z$</td>
<td>125</td>
<td>23.6</td>
<td>84.4</td>
</tr>
</tbody>
</table>

5.4 Effect of pit depth in rock mass damage

In order to study the impact of pit depth on damage formation, the excavation of the slope shown in Figure 5.1 was simulated with 200 meter mining steps while tracking the number of fractures resulting from each step. This process was repeated for the four rock masses described in Section 5.2, and applying the in-situ stress states listed in Table 5.4 for each situation.

Figure 5.3 presents an example of the effect of pit depth in fracture formation. This model corresponds to Case 1, where a non-persistent joint set daylights into the slope. In this situation, the applied in-situ stress field correspond to the Sh(75) decreasing $k$ ratio curve. The induced fractures are displayed in black. A plot with the fracture count history as a function of the model computing time is also included. This figure shows a noticeable increase in the fracture count due to stress changes during each of the excavation steps, which are indicated by the vertical red lines in the horizontal axis, and in most cases it is possible to identify the timestep where each step was mined.

The number of fractures resulting from each excavation step are presented in Table 5.5, Table 5.6, Table 5.7 and Table 5.8, corresponding to Case 1 through Case 4 respectively. In these tables it can be observed that in some cases the number of fractures was not registered. This occurred in two different situations.
Figure 5.3: Effect of excavation depth in rock mass damage (in black) for Sh(75) (Case 1).

Table 5.5: Number of fractures by excavation step (Case 1):

<table>
<thead>
<tr>
<th>Depth</th>
<th>k=1</th>
<th>k=1.5</th>
<th>k=2</th>
<th>k=2.5</th>
<th>Sh(25)</th>
<th>Sh(50)</th>
<th>Sh(75)</th>
<th>Sh(100)</th>
<th>Sh(125)</th>
</tr>
</thead>
<tbody>
<tr>
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<td></td>
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<td>0</td>
</tr>
<tr>
<td>200</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
<td>0</td>
<td>0</td>
<td>55</td>
<td>248</td>
<td>869</td>
</tr>
<tr>
<td>400</td>
<td>4</td>
<td>9</td>
<td>105</td>
<td></td>
<td>0</td>
<td>11</td>
<td>152</td>
<td>890</td>
<td>3397</td>
</tr>
<tr>
<td>600</td>
<td>22</td>
<td>85</td>
<td>1509</td>
<td></td>
<td>18</td>
<td>64</td>
<td>949</td>
<td>3310</td>
<td>8927</td>
</tr>
<tr>
<td>800</td>
<td>85</td>
<td>635</td>
<td>5341</td>
<td></td>
<td>81</td>
<td>248</td>
<td>1458</td>
<td>5423</td>
<td>22807</td>
</tr>
<tr>
<td>1000</td>
<td>227</td>
<td>4564</td>
<td>13893</td>
<td></td>
<td>151</td>
<td>643</td>
<td>4400</td>
<td>13050</td>
<td>24916</td>
</tr>
<tr>
<td>1200</td>
<td>734</td>
<td>7172</td>
<td>10883</td>
<td></td>
<td>328</td>
<td>1352</td>
<td>6523</td>
<td>23985</td>
<td>41465</td>
</tr>
</tbody>
</table>

Table 5.6: Number of fractures by excavation step (Case 2):

<table>
<thead>
<tr>
<th>Depth</th>
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<th>k=1.5</th>
<th>k=2</th>
<th>k=2.5</th>
<th>Sh(25)</th>
<th>Sh(50)</th>
<th>Sh(75)</th>
<th>Sh(100)</th>
<th>Sh(125)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>200</td>
<td>0</td>
<td>13</td>
<td>107</td>
<td></td>
<td>0</td>
<td>5</td>
<td>53</td>
<td>247</td>
<td>-</td>
</tr>
<tr>
<td>400</td>
<td>9</td>
<td>22</td>
<td>1222</td>
<td></td>
<td>13</td>
<td>16</td>
<td>146</td>
<td>1452</td>
<td>-</td>
</tr>
<tr>
<td>600</td>
<td>86</td>
<td>101</td>
<td>2659</td>
<td></td>
<td>39</td>
<td>71</td>
<td>917</td>
<td>3396</td>
<td>-</td>
</tr>
<tr>
<td>800</td>
<td>640</td>
<td>852</td>
<td>6272</td>
<td></td>
<td>98</td>
<td>255</td>
<td>1446</td>
<td>6112</td>
<td>-</td>
</tr>
<tr>
<td>1000</td>
<td>4315</td>
<td>5267</td>
<td>-</td>
<td>-</td>
<td>174</td>
<td>647</td>
<td>3934</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1200</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>348</td>
<td>2194</td>
<td>6283</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

86
Table 5.7: Number of fractures by excavation step (Case 3):

<table>
<thead>
<tr>
<th>Depth</th>
<th>k=1</th>
<th>k=1.5</th>
<th>k=2</th>
<th>k=2.5</th>
<th>Sh(25)</th>
<th>Sh(50)</th>
<th>Sh(75)</th>
<th>Sh(100)</th>
<th>Sh(125)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
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<td>0</td>
<td>-</td>
<td>-</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>200</td>
<td>4</td>
<td>13</td>
<td>-</td>
<td>-</td>
<td>0</td>
<td>0</td>
<td>90</td>
<td>403</td>
<td>-</td>
</tr>
<tr>
<td>400</td>
<td>5</td>
<td>24</td>
<td>-</td>
<td>-</td>
<td>0</td>
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<td>235</td>
<td>1109</td>
<td>-</td>
</tr>
<tr>
<td>600</td>
<td>19</td>
<td>93</td>
<td>-</td>
<td>-</td>
<td>15</td>
<td>69</td>
<td>1826</td>
<td>5253</td>
<td>-</td>
</tr>
<tr>
<td>800</td>
<td>88</td>
<td>664</td>
<td>-</td>
<td>-</td>
<td>72</td>
<td>274</td>
<td>2333</td>
<td>-</td>
<td>-</td>
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<tr>
<td>1000</td>
<td>230</td>
<td>4554</td>
<td>-</td>
<td>-</td>
<td>132</td>
<td>699</td>
<td>3677</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1200</td>
<td>744</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>311</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 5.8: Number of fractures by excavation step (Case 4):

<table>
<thead>
<tr>
<th>Depth</th>
<th>k=1</th>
<th>k=1.5</th>
<th>k=2</th>
<th>k=2.5</th>
<th>Sh(25)</th>
<th>Sh(50)</th>
<th>Sh(75)</th>
<th>Sh(100)</th>
<th>Sh(125)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
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First, when the rock mass showed noticeable and unexpected damage before the excavation sequence started, as a result of applying the initial stress conditions to the model. The subsequent high level of fracturing was interpreted as an unrealistic combination or in-situ stresses and rock mass strength. In this case, high stresses led to fracture extension due to stress concentrations at the discontinuity terminations. For avoiding bias in the results, these cases were not considered due to their strengths not being comparable to the undamaged cases.

The second situation occurred when the slope was already failing during the excavation sequence. A slope failure can be assumed when the model does not reach equilibrium (Figure 5.4). In the case of an overall failure the rate of cracking accelerates as the excavation progresses and will not converge to a constant value at the end of the simulation, or after preceding excavation stages (plotted in black in Figure 5.4). This, assuming that the model was run for a sufficient length of time to achieve equilibrium if it were possible. In this con-
dition the fracture growth is already unstable, and the final number of fractures will depend on how long the model is processed.

An additional confirmation of an overall failure in the model can be provided by the wall displacements history plot not stabilizing at a constant value (plotted in red in Figure 5.4). In this case relative displacements within the rock mass will be observed, indicating the presence of a sliding surface. In addition, an overall failure can be established by examining the particles velocities indicating an unstable portion of the slope.

The fracture count obtained from the different Slope Model runs is plotted in a log scale as a function of the excavation depth. Figure 5.5, Figure 5.6, Figure 5.7 and Figure 5.8 show these plots for Case 1 through Case 4 correspondingly. It can be observed that in all four analyzed cases, damage increased for every stress state as the excavation got deeper.

However, as opposed to what was observed in homogeneous rock masses (Figure 4.17), in fractured rock the increase in the micro crack number during each excavation step was not as smooth. In this case, the number of new fractures also increased with each step but not

Figure 5.4: Example of overall slope failure showing velocity field, fracture count and displacement history.
in proportion to the pit depth. This can be explained by the inhomogeneous distribution of pre-existing fractures, which can result in weaker areas within the rock mass resulting in higher damage levels in those locations. As it can be noted in Figure 5.3, there was a near continuous weak plane at the fifth excavation step level that resulted in additional damage near this area. This example shows that the location of pre-existing fractures can have a significant influence in the damage distribution behind the slope face.

The trends discussed in this section, and illustrated in Figure 5.5 through Figure 5.8, confirm the relation between pit depth and rock mass damage. This means that there is a bigger potential for damage occurring in larger open pits. Unlike in homogeneous materials where rock mass damage concentrates in the slope toe and pit floor, in fractured rock masses, damage can form fairly deep into the slope depending on the pre-existing fractures spatial configuration. It should also be noted, when comparing the homogeneous and jointed rock masses cases, that induced fractures can contribute to slope failure only when they can combine with pre-existing structures.
Figure 5.6: Fracture count vs pit depth (Case 2).

Figure 5.7: Fracture count vs pit depth (Case 3).
5.5 Effect of in-situ stress magnitudes in rock mass damage

The results presented in Section 5.4 can also be used for studying the impact of pre-mining stress magnitudes on rock mass damage potential. Similarly to the homogeneous case analyzed in Chapter 4, in fractured rock masses there is a strong correlation between in-situ stress and fracturing levels. These results show that the occurrence of rock mass damage is mainly dictated by the stress magnitudes, given that the slope geometry and rock mass characteristics are unchanged. It can be observed that higher horizontal stress magnitudes result in increased damage levels.

In addition, some differences can be found between stress states assuming constant $k$ ratios and the ones considering a decreasing $k$ ratio with depth. In general, when fixed $k$ ratios are applied, damage starts later in the excavation sequence. After fractures start forming, these show a faster growth compared to the variable $k$ ratio curves. In contrast, when a decreasing $k$ ratio is used damage tends to start in earlier stages of the excavation, presenting a more moderate growth as the excavation develops. Again, these differences in
damage levels can be expected given the dissimilar horizontal stress profile resulting from these two stress situations (Figure 3.10).

While rock mass damage does not necessarily lead to an overall slope failure, it has the potential to contribute to it when combined with pre-existing discontinuities. Figure 5.9 presents a good example on how stress magnitudes can impact the stability of a slope. Figure 5.9(a) shows a model in which the Sh(25) decreasing $k$ ratio curve is used, while the model in Figure 5.9(b) considers a Sh(50) curve. This situation corresponds to Case 3, where the only difference between these two models are the pre-mining stress magnitudes. In these two figures, stress-induced fractures are shown as black disks. As it can be observed, the first model remained stable after the excavation was completed, while the second one failed in the final excavation step. In the Sh(50) case, the additional horizontal stresses induced slightly more damage, which was enough for connecting some of the pre-existing fractures behind the slope face. This partial failure surface eventually reached the major structure and the whole volume in front of this fault turned unstable.

5.6 Effect of in-situ stress in failure modes

As it was shown in the previous sections, rock mass damage combined with pre-existing discontinuous fractures can form a complete failure surface, and eventually cause a slope to collapse. Since the damage levels and the location of damaged zones are controlled by the in-situ stress field, the shape of the possible failure surfaces could also be impacted. Therefore, the pre-mining stress field might also impact the failed volume of an eventual slope collapse. This potential was evaluated by comparing the failure surface for different pre-mining stress settings.

Figure 5.10 shows how the failure mechanism can be affected by the in-situ stress magnitudes. Both models represent Case 3 and considered decreasing $k$ ratio curves. Figure 5.10(a) corresponds to the Sh(75) curve while Figure 5.10(b) represents the Sh(100) curve. On the lower stress case, the slope remained stable but a partial step-path surface started forming, combining pre-existing fractures through tensile failure of intact rock. In the second case,
Figure 5.9: Effect of in-situ stress field in stability and fracture distribution (shown in black) for Case 3.
where the applied stresses were higher, a complete failure surface formed which was also deeper. Here, rock mass damage led to the formation of a near circular sliding surface that combined with the fault behind the slope.

Figure 5.11 shows another situation in which in-situ stress magnitudes impacted the shape of the failure surface, which corresponds to Case 1. The stress states considered were obtained from the Sh(100) and Sh(125) curves for the models shown in Figure 5.11(a) and Figure 5.11(b) respectively. In both of these cases, the resulting horizontal stresses were relatively high and led to similar failure mechanisms. Failure implied sliding of an unstable block along a near linear surface, but also a secondary semi-circular failure surface developed deeper into the slope. Comparing these two models it can be noted that in the higher stress case a tension crack is starting to develop far behind the slope. This tension crack could eventually progress into the development of an additional step-path failure surface, resulting in an overall slope failure involving more than twice the volume of the initial failure.

The potential effect of stress on the failed volume can be seen more clearly in Figure 5.12, which compares the results obtained from Case 4 for the $k = 1.5$ (Figure 5.12(a)) and Sh(75) (Figure 5.12(b)) in-situ stress states. In the first model, significant damage only develops in the last excavation step. In this situation, the slope failure involves a relatively low volume near the bottom of the pit. It has to be noted that in this case the fracture count does converge to a constant value. However, the lower portion of the slope is already unstable and sliding along a failure plane. This can be observed in the displacement history of the detached block, which is plotted in red. The velocity field of the model also confirmed the instability of this portion of the slope. In the higher stress case, damage starts occurring earlier in the excavation sequence, leading to the formation of a partial sliding surface in the top half of the slope. In the last excavation step, a second failure surface forms, which in this occasion leads to a considerably higher failed volume compared to the first model.

These results show that the estimation of the in-situ stress field can be of great importance for the quality of the slope behavior predictions. An accurate description of the stress field
Figure 5.10: Effect of in-situ stress on shape of failure surface (Case 3).
Figure 5.11: Effect of stress on failure surface (Case 1).
Figure 5.12: Effect of stress on failure surface and fracture distribution (shown in black) for Case 4.
should include establishing how in-situ stress magnitudes change with depth. This can be achieved through stress measurements which need to cover the whole excavation depth range.

Relying on one stress measurement, or on multiple measurements obtained at very similar depths, can lead to inadequate assumptions about the variation of \( k \) with depth. For instance, a single measurement carried out at a depth of 750 meters returns 30 MPa which corresponds to \( k = 1.5 \) at this point. However, this could be interpreted as a constant \( k \) ratio of 1.5 or as a variable \( k \) ratio following the Sh(75) curve, which also results in \( k = 1.5 \) at this particular depth (Figure 3.9 and Figure 3.10). This situation corresponds to Figure 5.12, which shows that these two assumptions resulted in very different slope behaviors, while both satisfied the measured stress at that point.

### 5.7 Slope movements and rock mass damage vs. time

This last section includes an analysis of the relation between slope movements and rock mass damage over time. As it is shown in Section 5.4, the removal of rock on each excavation step results in stress-induced fractures and increased slope displacements. However, it can be observed that fractures and displacements do not occur simultaneously. After each excavation step there is a noticeable delay between the onset of fracture formation and slope wall displacements. This delay was also found to be proportional to the depth of the excavation.

Figure 5.13 shows the fracture count and total crest displacement over time for Case 3 and Sh(50), which corresponds to the model displayed in Figure 5.9(b). In this plot, the vertical dashed lines represent the onset of fracturing, as well as the instant when slope displacements manifest after each excavation step. In this figure the first 10 seconds of the simulation time corresponds to the stress initialization for the model, after which displacements were resetted.

It can be noted that fracturing starts immediately after each step is excavated while slope displacements tend to initiate a few time steps later, only after an important number of fractures are formed. For instance, after the fourth excavation step (\( t=38 \) s) slope movements did not initiate until after fractures finished propagating. Since induced fractures will precede any slope movement, detecting the occurrence of these fractures might help recognize the
first signs of a possible instability. Extension fractures can be observed in high rock slopes, specially in rock cores obtained from deep open pits. In these cores it is possible to distinguish fresh induced fractures from pre-existing ones. Also, the orientation of extension fractures can be assumed to be perpendicular to $\sigma_3$. In addition, some authors have stated that fracture growth produces measurable AE events but its applicability for open pit monitoring needs to be studied in more depth.

![Figure 5.13: Delay between fracturing and slope displacements for Sh(50) (Case 3).](image)

While comparing slope displacements with the resulting fracturing levels at the different excavation steps, it was also observed that the measured total displacements can include an important component of elastic deformation. These movements are mainly caused by elastic rebound when the rock mass is unloaded as a result of mining. This statement is supported by the relatively high displacements observed in the first excavation steps where damage is very limited.

### 5.8 Summary of results and conclusions

This chapter studied the effect of stress in rock slopes resulting from increasing pit depths and pre-mining stress magnitudes, on the relative damage levels induced within a jointed
rock mass. In these conditions, failure mechanisms involve a combination of shear on pre-existing structures and stress-induced fracturing, which cannot be represented correctly with traditional design tools. The Slope Model code, based on a simplified DEM approach, could successfully represent this behavior.

Similarly to the homogeneous material case considered in Chapter 4, it was found that in jointed rock masses there is also a strong correlation between in-situ stress magnitudes and damage levels. As in the previous chapter, the application of variable $k$ ratios lead to earlier damage in the excavation, while with constant $k$ ratios most of the damage concentrated in the later stages. However, in the case of jointed rock masses, rock mass damage led to slope failure in some situations. It was also observed that the differences in damage levels resulting from different in-situ stress states can have an important effect in stability. In some situations, additional damage due to higher horizontal stresses lead to slope failure by connecting discontinuous structures.

The results obtained in this chapter confirm the relation between pit depth and rock mass damage, meaning that there is a bigger potential for damage occurring in larger open pits. Fractures increased for every stress state as the excavation got deeper in all of the considered cases. However, this increase was not always proportional to the excavation depth. This can be explained by the irregular distribution of pre-existing fractures, which result in weaker areas more susceptible to damage. The location of pre-existing fractures can have an important influence in the damage spatial distribution. In jointed rock masses damage will not necessarily occur in the slope toe and pit floor. In this case damage will tend to develop at pre-existing fractures terminations, promoting fracture extension due to stress concentrations.

Because the in-situ stress field can impact damage levels and the location of damaged zones, it can also affect the location and shape of the failure surface. When in-situ stresses are higher, a potential failure surface tends to form deeper, resulting in larger failed volumes. Moreover, depending on the in-situ stress field considered, in some situations instability
occurred in different areas of the slope. This highlights the importance of the in-situ stress field estimation for the quality of the slope behavior predictions. This estimation should address the variation of stresses with depth, for which stress measurements at different depths are essential.

The relation between slope movements and rock mass damage over time was also assessed. It was observed that fractures started forming immediately after a step was excavated, confirming that rock mass damage is stress induced and not a result of slope deformations. In contrast, displacements tended to start a few time steps later, only after a significant number of fractures were formed. This delay between fracturing onset and deformation was found to be proportional to the excavation depth.
Designing reliable slopes that provide safety and also maximize financial return represent one of the main challenges in a mining operation, with an unexpected large scale slope failure being one of the biggest threats to the business. The design of increasingly deeper open pits includes an additional degree of uncertainty that can be related to the limited experience with the design of very high slopes, and to numerous deficiencies in the understanding of deep pit behavior.

Another significant source of uncertainty, which is often ignored, is the local stress state. The in-situ stress field can be highly complex and variable due to several site specific factors, presenting significant differences to the regional stress field. Reliable slope behavior predictions require an adequate knowledge of the local pre-mining stress setting, together with the application of suitable numerical tools capable of capturing the brittle characteristics of rock masses.

Several numerical methods for slope stability evaluations are currently available, and most of them still have essential limitations for modeling the behavior of fractured rock masses realistically. Failure in jointed rock masses involves an interaction between pre-existing discontinuities and stress-induced fractures which can only be captured by more advanced numerical techniques. The Slope Model code was found to be capable of simulating this behavior, making it the ideal tool for evaluating the effect of in-situ stress and increasing depth in slope stability. As traditional analysis tools do not account for such failure mechanisms, future deep pits could be potentially under-designed and lead to unpredicted slope behavior.

6.1 Summary of conclusions

The response of a homogeneous rock mass to increasing pit depths for different in-situ stress fields was studied using the Finite Element and Slope Model approaches. Combining
these two methods allowed for assessing the influence of depth and stress magnitudes in terms of both stress conditions and rock mass damage as a result of stress-induced fractures. These analyses showed that the number of stress-induced fractures increased following a near exponential relation with depth, indicating an important potential for rock mass damage to develop in deep open pits. Considerable damage was observed as depths exceeded 800 meters, even for low in-situ stress situations. Similarly, fracturing levels also showed a strong correlation with in-situ stress magnitudes, with higher horizontal stresses resulting in greater damage levels. However, damage did not lead to a global slope failure in any of the cases when a homogeneous rock mass was considered.

When jointed rock masses were considered, it was observed that rock mass damage could lead to slope failure when it combined with non-persistent discontinuities. The potential for damage to develop in a given rock mass was found to be mainly dictated by in-situ stress magnitudes and pit depth. However, pre-existing fractures can have a noticeable impact on damage distribution, causing fractures to develop deep into the rock mass and potentially impacting failure mechanisms. It was observed that when horizontal stress magnitudes are higher, failure surfaces tend to form deeper. In other words, the stress field cannot only impact the likelihood of a slope failure but also its potential consequences by influencing the failed material volume.

The previous results highlight the importance of an accurate determination of the in-situ stress state. An appropriate assessment of the local stress field, together with adequate numerical approaches capable of a realistic representation of the slope failure processes, will allow improved stability assessments leading to more reliable slope designs. At the same time, improved numerical tools can lead to a better understanding of rock mass behavior in variable stress conditions and increasing mining depths, reducing the uncertainty in the development of future deep pits.

A review of the more relevant conclusions by chapter is provided below, which summarize the main findings discussed on each of the previous chapters.
Chapter 2 – Literature review

A review of the current literature showed that several deficiencies exist in the understanding of slope behavior, which become even more critical as open pits develop to depths in which previous experience is very limited. Advances in numerical tools and improved design methods are necessary to overcome several limitations and in this way reduce the risk related to mining at increasing depths. A list of the identified deficiencies is presented below:

- Traditional design tools have fundamental limitations for representing the interaction of yielding discontinuities and intact rock failure within the rock mass, applying inappropriate ductile, shear models instead. Oversimplified and unrealistic approaches can possibly lead to inaccurate stability assessments and added uncertainty.

- Predicting the most likely failure mode for a slope is essential since it will determine the type of analysis to be carried out. Still, the current knowledge of failure mechanisms in deep pit conditions is lacking, and it is not unlikely that unknown failure modes could be observed in the future.

- The effect of in-situ stresses in stability is still unknown and often ignored. However, there is enough evidence that some stress conditions can promote intact rock fracturing resulting in damage.

- Similarly, rock mass damage can have a critical effect on the strength properties of the rock, and can potentially impact the mode of failure in a slope by interacting with preexisting discontinuities. As a result, in-situ stresses can possibly induce slope failures, which should be considered when assessing the stability of a slope.

- Numerical models based on bonded particles and lattice methods are able to simulate the interaction of preexisting fractures with intact rock fracturing; still, the lack of knowledge of the geometry and distribution of preexisting fractures within a rock mass is perhaps their main limitation.
Chapter 3 – Pre-mining stress models

The local in-situ stress state represents an additional source of uncertainty to the slope design process. This uncertainty could be easily reduced, but the importance of stresses in slope performance is still not fully recognized. The stress field can very likely impact slope behavior and it is necessary to put more effort on its assessment. Some additional findings relevant to the effect of in-situ stresses in stability are listed below:

- Available measurement techniques can provide reasonable estimates for representing the local stress field despite some limitations.

- General in-situ stress relations are not a replacement for an adequate stress measurement program; still, they might be useful in some situations but should always be used with caution.

- The vertical in-situ stress component follows a near linear relation with depth in most regions but some exceptions exist. In these cases, the vertical stress magnitudes should be determined from stress measurements, which will not necessarily correspond to the weight of the overlying rock.

- Horizontal in-situ stress magnitudes present a high dispersion with some regions being better represented by near constant $k$ ratios, while in others $k$ show a clear decrease with increasing depth.

- It is important to establish not only stress magnitudes at a point, but also their orientation and how these stresses change with depth at a specific site. For this, it is necessary to carry out stress multiple measurements at different depths, allowing establishing a more realistic approximation of the stress field.

- A parametric study through numerical modeling, considering several likely stress states, can be used for assessing the effect of stress. This allows determining the need of addi-
tional site-specific measurements, increasing the confidence in the rock mass behavior predictions.

Chapter 4 – Stress analyses and damage in homogeneous rock masses

In-situ stress and increased depth can play an important role in the development of rock mass damage in homogeneous materials. Higher stress magnitudes result in a noticeable increase of rock mass damage levels, particularly in the slope face near the toe and the pit floor. Similarly, the likelihood of stress-induced damage is considerably higher for deeper pits, with fracturing levels following a near exponential relation with depth. The potential for damage development can be identified even in low stress environments when an excavation reaches significant depths. Conversely, when higher in-situ stress magnitudes are present, it can be expected that geotechnical problems associated with induced fracturing can occur at very moderate depths, and could already be playing a role in existing pits.

Fractured zones could be anticipated through a simple elastic Finite Element analysis by applying the extension strain criterion. These predictions seem to be reasonably close to the ones obtained with more advanced tools. Despite this fact, elastic models cannot represent the effect of induced fractures in rock mass behavior limiting its usefulness for stability evaluations. Slope behavior assessments can only be achieved with models that can capture the brittle characteristics of a rock mass. However, the extension strain criterion can be useful identifying situations where these more advanced models are needed.

Also, the extension strain criterion can be applied for assessing the impact of different slope designs in rock mass damage potential, before more complicated and time consuming models are built. Similarly, the criterion can be used for sensitivity analyses studying the effect of possible in-situ stress states and their potential impact in slope performance, and helping determine the need for a more detailed stress field evaluation.

Additional conclusions from this chapter include:

- Increased excavation depth results in a higher potential for rock mass damage development. This damage potential is also proportional to the horizontal stress magnitudes
resulting from the different pre-mining stresses.

- The extension strain criterion can allow the prediction of fracture occurrence through simple elastic analyses that are comparable to fractured zones obtained through advanced numerical techniques.

- It is possible to determine the critical value for the extension strain magnitudes through the approach proposed in this chapter. Combining elastic and brittle models allow establishing the relation between elastic strains and fracturing levels.

- In homogeneous rock masses, it is likely that rock mass damage will not lead to failure by itself; however, its interaction with pre-existing fractures needs to be considered. In homogeneous materials, intact rock damage might only manifest as additional slope deformations. This relationship between maximum slope displacements and damage levels seems to be near-linear.

**Chapter 5 – Stress-induced damage in jointed rock masses**

The analyses carried out in Chapter 5 confirm the increased potential of rock mass damage development in deep pits. It was also found that in jointed rock masses there is a strong correlation between in-situ stress magnitudes and damage levels, with larger magnitudes leading to more likely instabilities related to induced fracturing.

When pre-existing discontinuities are present damage distribution can be largely affected, and can be highly prevalent within the rock mass. High horizontal stress magnitudes will tend to promote fracture extension, eventually resulting in the interconnection of fractures. In this way, rock mass damage interacting with pre-existing fractures can lead to the development of a complete failure surface resulting in a slope collapse.

In addition, horizontal in-situ stress magnitudes might influence the depth and shape of an eventual failure surface, with higher magnitudes promoting larger failed volumes. Depending on the rock mass characteristics, it is possible that in-situ stress magnitudes can also impact the anticipated failure mechanism of a particular slope.
Some additional findings from this chapter are summarized below:

- The location of pre-existing fractures can have a significant influence in the damage spatial distribution. In jointed rock masses, rock mass damage will not necessarily occur near the slope toe and will tend to develop at fractures terminations. In this way, damage can promote fracture extension and interconnection of discontinuities.

- Small variations in damage levels resulting from different in-situ stress states can have a critical effect on the stability of fractured rock masses. Additional damage due to higher horizontal stresses can lead to slope failure in some situations by connecting discontinuous structures.

- When in-situ horizontal stresses are higher, a potential failure surface tends to form deeper, resulting in larger failed volumes and bigger possible consequences.

- In-situ stress settings characterized by decreasing $k$ ratios with depth will likely show early damage in the excavation sequence, while in constant $k$ ratio environments damage will tend to occur at later stages.

- There is a clear relationship between pit depth and stress-induced fracture formation, resulting in an increased damage potential for deep open pits.

- When a new step is excavated, fractures start forming immediately due to instant stress changes, while slope wall displacements tend to manifest later. This delay between fracturing onset and wall deformation is likely proportional to the excavation depth.

### 6.2 Future work

Several areas for future research were identified during this dissertation which are listed below:

1. Study effect of stress in back-analyzed slope failures considering deformations and seismic monitoring.
2. Compare Slope Model results with a UDEC model applying a more realistic cohesion weakening friction strengthening (CWFS) constitutive model.

3. Determine critical strain magnitudes for a wider range of materials.

4. Establish the relation between induced fracture density and strength loss for materials at different scales.

5. Compare rock mass damage predicted by models with field data from rock cores obtained around the slope toe of existing deep open pits.

6. Study the combined effect of rock mass damage and pore water pressures through coupled hydro-mechanical models.

7. Quantify the effect of rock mass damage in permeability.

8. Evaluate the combined effect of rock mass damage and surface runoff in slope failures.

9. Consider multiple joint distributions which can eventually be used for probabilistic and risk analyses.

10. Apply different joint set properties and assess their effect in rock mass damage distribution
REFERENCES CITED


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APPENDIX A - STRESS CALIBRATION

It is often not possible to perform in-situ stress measurements far enough from excavations or topographic features, so that their influence in the stress field can be ignored. McKinnon [80] developed a method to interpret stress measurements and to obtain the pre-mining stress field where these measurements are affected by the excavation or by local topography. His technique can also be used to calibrate stress boundary conditions for numerical models.

The stress measured at any point \( \sigma_{tot} \) is the sum of the pre-mining stress and the stresses induced by the excavation\( \sigma_{ind} \). The pre-mining stress is at the same time composed by a gravitational component \( \sigma_{grav} \), and a horizontal component, usually referred as the tectonic or far field stress \( \sigma_{tec} \). Therefore, the total measured stress tensor can be written as the sum of these three components, as it is shown in Equation A.1.

\[
\sigma_{tot} = \sigma_{grav} + \sigma_{tec} + \sigma_{ind}
\]  

(A.1)

The induced stress tensor can also be decomposed in a gravitational and tectonic component (Equation A.2):

\[
\sigma_{ind} = \sigma_{grav}^{I} + \sigma_{tec}^{I}
\]  

(A.2)

Substituting Equation (3.2) into (3.1) results in the following expression for the total stress at a point (Equation A.3):

\[
\sigma_{tot} = \sigma_{grav} + \sigma_{grav}^{I} + \sigma_{tec} + \sigma_{tec}^{I}
\]  

(A.3)

In order to determine the tectonic stress magnitudes, it is necessary to create numerical models including the geometry before and after the excavation. First, these two models are computed considering gravitational loading only, which gives the first two terms on Equation A.3 at the measurement point. Next, the pre-mining and excavated models need to be solved considering different boundary stress conditions which represent the unknown tectonic stress components.
Since there are more equations than unknowns, more than one solution might be possible. For this reason it is necessary to find a solution that minimize the error between the tectonic component of the measured stress tensor and the one obtained from the numerical model. For finding the adequate far-field stress components, unit normal or shear stress conditions can be applied to the model, and then scaled through a least-squares procedure to match the magnitudes of components using Equation A.4.

\[
(\sigma_{tec} + \sigma_{tec}^I) = \sigma_{tot} - (\sigma_{grav} + \sigma_{grav}^I)
\] (A.4)

Once the scaling factors for the tectonic stress are obtained, the far-field stresses are determined by multiplying the unit stresses by the appropriate coefficient. Then, it is possible to compute several useful quantities at any point in the model, such as total stress, pre-mining stress, mining-induced stress, and the stress field for any sequence of excavations made in the model (McKinnon, 2001). Additionally, the method can be applied to single or multiple measurements. When analyzing groups of stress measurements, stress domains can be established for group locations with a similar combination of gravitational and tectonic stresses.
APPENDIX B - STRESS PATH FOR SLOPE TOE WITH INCREASING PIT DEPTH

The stress path for a moving point located at the toe of a slope was determined in terms of principal stresses through FE elastic analyses as the excavation deepened. These results were combined with the fracture rate determined from equivalent Slope Model runs (Figure B.1 through B.9) for different stress situations. The geometry and properties used correspond to the ones presented in Chapter 3 for a homogeneous material.

Figure B.1: Stress path and fracture rate for k=1.
Figure B.2: Stress path and fracture rate for $k=1.5$.

Figure B.3: Stress path and fracture rate for $k=2$. 
Figure B.4: Stress path and fracture rate for $k=2.5$.

Figure B.5: Stress path and fracture rate for $Sh(25)$. 
Figure B.6: Stress path and fracture rate for Sh(50).

Figure B.7: Stress path and fracture rate for Sh(75).
Figure B.8: Stress path and fracture rate for Sh(100).

Figure B.9: Stress path and fracture rate for Sh(125).
APPENDIX C - PRINCIPAL STRESS MEASUREMENTS BY REGION

The following figures show stress measurement data obtained from the World Stress Map Project [42]. This information was divided by region and the principal stress magnitudes were plotted against the measurement depth. The first plot included data from all regions (Figure C.1), while the following ones display measurements from Australia (Figure C.2), Canada (Figure C.3), United States (Figure C.4) and Scandinavia (Figure C.5). Other regions were not included due to limited stress data information.

Figure C.1: Principal stresses vs. depth (All regions).
Figure C.2: Principal stresses vs. depth (Australia).
Figure C.3: Principal stresses vs. depth (Canada).
Figure C.4: Principal stresses vs. depth (United States).
Figure C.5: Principal stresses vs. depth (Scandinavia).
APPENDIX D - SLOPE MODEL RUNS EXAMPLE FOR INCREASING PIT DEPTH

Figure D.1 presents an example of a slope excavated in several steps simulated in Slope Model. This corresponds to Case 1 (Section 5.2.1) for Sh(50).

Figure D.1: Excavation sequence simulation for Sh(50).
Figure E.1 and Figure E.2 show the maximum slope displacement versus pit depth, for constant k and variable k ratios respectively. These results were obtained from elastic analyses using the Finite Element code Phase\textsuperscript{2}. In the case of constant k ratios, at depths below 400 meters there are no significant differences in displacements between the different k ratios.

As depth increases, relative differences in deformations become bigger. For instance, at a depth of 1200 meters the displacements obtained from k=2.5 are almost double the ones resulting from k=1. When variable k ratios were applied, there was also a strong correlation between the stress magnitudes and deformations. However, in this case the relative differences in displacements between the applied k ratio curves remained similar for the full range of the excavation depth.
Figure E.2: Maximum slope displacement versus pit depth for variable k ratios.
APPENDIX F - FINAL PIT DEFORMATION PROFILE FOR AN ELASTIC MATERIAL

This appendix includes the final pit crest deformation profile obtained from both FE and Slope Model with the properties listed in Table F.1. The slope final geometry is shown in Figure F.1. The displacements obtained from FE and Slope Model are plotted in Figure F.2 and Figure F.3 for constant and variable $k$ ratios respectively.

![Final pit geometry](image)

Figure F.1: Final pit geometry.

<table>
<thead>
<tr>
<th>Material name</th>
<th>Density [kg/m$^3$]</th>
<th>Deformation Modulus [GPa]</th>
<th>Poisson’s ratio</th>
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<tr>
<td>Siltstone</td>
<td>2838</td>
<td>100</td>
<td>0.2</td>
</tr>
</tbody>
</table>

Table F.1: Material properties
Figure F.2: Displacement profile for final pit from constant k ratios.

Figure F.3: Displacement profile for final pit from Sheorey curves.
APPENDIX G - SLOPE MODEL BASE CASE INPUT FILE

This Appendix includes an example of a Slope Model input file, which in this corresponds to Case 4 described in Section 5.2 and shown in Figure G.1. Table G.1 includes a list of the units used in this input file for several parameters.

Figure G.1: Slope Model discontinuities (blue) and excavation boundaries (black).

Table G.1: Units for different model parameters.

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</tr>
<tr>
<td>Confining stress</td>
<td>MPa</td>
</tr>
<tr>
<td>Density</td>
<td>kg/m³</td>
</tr>
<tr>
<td>Bench height</td>
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</tr>
<tr>
<td>Bench length</td>
<td>meters</td>
</tr>
<tr>
<td>Bench width</td>
<td>meters</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>Pa</td>
</tr>
<tr>
<td>UCS</td>
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<td>Young’s modulus</td>
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  <item hint="normal stiffness" type="double" name="normal_stiffness"/>
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  <item type="bool" name="uses global coordinate">no</item>
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  <item hint="normal stiffness" type="double" name="normal stiffness"/>
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  <item hint="rotation angle applied to the joint set dip direction" type="double" name="dip direction rotation">0</item>
  <item hint="translation applied to the joint set X coordinate" type="double" name="translation x">0</item>
</item>
<item hint="local or global global coordinate Z" type="double" name="reference point up"/>
<item type="bool" name="uses global coordinate">yes</item>
<item hint="joint aperture" type="double" name="aperture"/>
<item hint="joint tensile strength" type="double" name="tensile strength"/>
<item hint="joint friction angle" type="double" name="friction"/>
<item hint="joint cohesion" type="double" name="cohesion"/>
<item hint="normal stiffness" type="double" name="normal_stiffness"/>
<item hint="shear stiffness" type="double" name="shear_stiffness"/>
<item hint="dilation angle" type="double" name="dilation_angle"/>
<item hint="zero dilation slip" type="double" name="zero_dilation_slip"/>
<item hint="the name of a file used to create the joint set" type="string" name="filename"/>
</group>
</properties>
</page>

<page name="DXF Joint Sets">
  <properties>
    <group prototype="yes" name="DXF Joint Set">
      <item hint="Jointset description" table="joint condition" type="list" name="description"/>
      <item hint="joint aperture" type="double" name="aperture"/>
      <item hint="joint tensile strength" type="double" name="tensile strength"/>
      <item hint="joint friction angle" type="double" name="friction"/>
      <item hint="joint cohesion" type="double" name="cohesion"/>
      <item hint="normal stiffness" type="double" name="normal_stiffness"/>
      <item hint="shear stiffness" type="double" name="shear_stiffness"/>
      <item hint="dilation angle" type="double" name="dilation_angle"/>
      <item hint="zero dilation slip" type="double" name="zero_dilation_slip"/>
      <item hint="the name of a file used to create the joint set" type="string" name="filename"/>
      <item hint="rotation angle applied to the joint set on the Z axis" type="double" name="rotation"/>
      <item hint="translation applied to the joint set X coordinate" type="double" name="translation x"/>
      <item hint="translation applied to the joint set Y coordinate" type="double" name="translation y"/>
      <item hint="translation applied to the joint set Z coordinate" type="double" name="translation z"/>
    </group>
  </properties>
</page>
<page name="seams">
  <properties>
    <group prototype="yes" name="Seam">
      <item hint="local or global east-coordinate" type="double" name="origin east"/>
      <item hint="local or global north-coordinate" type="double" name="origin north"/>
      <item hint="local or global up-coordinate" type="double" name="origin up"/>
      <item type="double" name="point1 X"></item>
      <item type="double" name="point1 Y"></item>
      <item type="double" name="point1 Z"></item>
      <item type="double" name="point2 X"></item>
      <item type="double" name="point2 Y"></item>
      <item type="double" name="point2 Z"></item>
      <item hint="dip of seam [Deg]" name="dip">0</item>
      <item hint="dip direction of seam [Deg]" name="dip direction">0</item>
      <item hint="seam thickness/height [m] or arc (degrees)" name="thickness">0</item>
      <item hint="seam width or diameter [m]" name="width">0</item>
      <item hint="seam length [m]" name="length">0</item>
      <item hint="initial condition for injection well [m3/s]" name="injection rate">0.0</item>
      <item hint="seam shape [rect|cylinder]" name="shape">Rectangular</item>
      <item hint="Possible types: (0) Infinite Plane, (1) ClosedVolume, (2) excavation/hole, etc." type="int" name="seam type">0</item>
      <item hint="Indicates if excavation is inactive, active or removed from model" name="excavation state">0</item>
      <item type="int" name="item id"/>
      <item type="bool" name="uses default resolution">yes</item>
      <item type="double" name="seam resolution (cm)">20.0</item>
      <item type="bool" name="uses flat joint model">no</item>
      <item type="int" name="flat joint model number of contact points">3</item>
      <item type="double" name="flat joint model disk radius multiplier">0.5</item>
    </group>
  </properties>
</page>
<item type="double" name="point1 Z">0</item>
<item type="double" name="point2 X">0</item>
<item type="double" name="point2 Y">0</item>
<item type="double" name="point2 Z">0</item>
<item hint="dip of seam [Deg]" name="dip">0</item>
<item hint="dip direction of seam [Deg]" name="dip direction">0</item>
<item hint="seam thickness/height [m] or arc (degrees)" name="thickness">200</item>
<item hint="seam width or diameter [m]" name="width">200</item>
<item hint="seam length [m]" name="length">170</item>
<item hint="initial condition for injection weel [m3/s]" name="injection rate">0</item>
<item hint="seam shape [rect|cylinder]" name="shape">Rectangular</item>
<item hint="Possible types: (0)Infinite Plane, (1) ClosedVolume, (2) excavation/hole,etc." type="int" name="seam type">2</item>
<item hint="Indicates if excavation is inactive,active or removed from model" name="excavation state">2</item>
<item type="int" name="item id">2</item>
<item type="bool" name="uses default resolution">yes</item>
<item type="double" name="seam resolution (cm)">700</item>
<item type="bool" name="uses flat joint model">no</item>
<item type="int" name="flat joint model number of contact points">3</item>
<item type="double" name="flat joint model disk radius multiplier">0.5</item>
</group>
<group name="Seam 2">
<item hint="local or global east-coordinate" type="double" name="origin east">-225</item>
<item hint="local or global north-coordinate" type="double" name="origin north">0</item>
<item hint="local or global up-coordinate" type="double" name="origin up">1100</item>
<item type="bool" name="uses global coordinate">no</item>
<item table="rock" type="list" name="material">None</item>
<item type="double" name="point1 X">0</item>
<item type="double" name="point1 Y">0</item>
<item type="double" name="point1 Z">0</item>
<item type="double" name="point2 X">0</item>
<item type="double" name="point2 Y">0</item>
<item type="double" name="point2 Z">0</item>
<item hint="dip of seam [Deg]" name="dip">0</item>
<item hint="dip direction of seam [Deg]" name="dip direction">0</item>
<item hint="seam thickness/height [m] or arc (degrees)" name="thickness">200</item>
<item hint="seam width or diameter [m]" name="width">450</item>
<item hint="seam length [m]" name="length">170</item>
<item hint="initial condition for injection weel [m3/s]" name="injection rate">0</item>
<item hint="seam shape [rect|cylinder]" name="shape">Rectangular</item>
<item hint="Possible types: (0) Infinite Plane, (1) ClosedVolume, (2) excavation/hole, etc." type="int" name="seam type">2</item>
<item hint="Indicates if excavation is inactive, active or removed from model" name="excavation state">2</item>
<item type="int" name="item id">3</item>
<item type="bool" name="uses default resolution">yes</item>
<item type="double" name="seam resolution (cm)">700</item>
<item type="bool" name="uses flat joint model">no</item>
<item type="int" name="flat joint model number of contact points">3</item>
<item type="double" name="flat joint model disk radius multiplier">0.5</item>
</group>
<group name="Seam 3">
<item hint="local or global east-coordinate" type="double" name="origin east">-350</item>
<item hint="local or global north-coordinate" type="double" name="origin north">0</item>
<item hint="local or global up-coordinate" type="double" name="origin up">1300</item>
<item type="bool" name="uses global coordinate">no</item>
<item table="rock" type="list" name="material">None</item>
<item type="double" name="point1 X">0</item>
<item type="double" name="point1 Y">0</item>
<item type="double" name="point1 Z">0</item>
<item type="double" name="point2 X">0</item>
<item type="double" name="point2 Y">0</item>
<item type="double" name="point2 Z">0</item>
<item hint="dip of seam [Deg]" name="dip">0</item>
<item hint="dip direction of seam [Deg]" name="dip direction">0</item>
<item hint="seam thickness/height [m] or arc (degrees)" name="thickness">200</item>
<item hint="seam width or diameter [m]" name="width">700</item>
<item hint="seam length [m]" name="length">170</item>
<item hint="initial condition for injection weel [m3/s]" name="injection rate">0</item>
<item hint="seam shape [rect|cylinder]" name="shape">Rectangular</item>
<item hint="Possible types: (0) Infinite Plane, (1) ClosedVolume, (2) excavation/hole, etc." type="int" name="seam type">2</item>
<item hint="Indicates if excavation is inactive, active or removed from model" name="excavation state">0</item>
<item type="int" name="item id">4</item>
<item type="bool" name="uses default resolution">yes</item>
<item type="double" name="seam resolution (cm)">700</item>
<item type="bool" name="uses flat joint model">no</item>
<item type="int" name="flat joint model number of contact points">3</item>
<item type="double" name="flat joint model disk radius multiplier">0.5</item>
</group>
<group name="Seam 4">
    <item hint="local or global east-coordinate" type="double" name="origin east">-475</item>
    <item hint="local or global north-coordinate" type="double" name="origin north">0</item>
    <item hint="local or global up-coordinate" type="double" name="origin up">1500</item>
    <item type="bool" name="uses global coordinate">no</item>
    <item table="rock" type="list" name="material">None</item>
    <item type="double" name="point1 X">0</item>
    <item type="double" name="point1 Y">0</item>
    <item type="double" name="point1 Z">0</item>
    <item type="double" name="point2 X">0</item>
    <item type="double" name="point2 Y">0</item>
    <item type="double" name="point2 Z">0</item>
    <item hint="dip of seam [Deg]" name="dip">0</item>
    <item hint="dip direction of seam [Deg]" name="dip direction">0</item>
    <item hint="seam thickness/height [m] or arc (degrees)" name="thickness">200</item>
    <item hint="seam width or diameter [m]" name="width">950</item>
    <item hint="seam length [m]" name="length">170</item>
    <item hint="initial condition for injection weel [m3/s]" name="injection rate">0</item>
    <item hint="seam shape [rect|cylinder]" name="shape">Rectangular</item>
    <item hint="Possible types: (0) Infinite Plane, (1) ClosedVolume, (2) excavation/hole, etc." type="int" name="seam type">2</item>
    <item hint="Indicates if excavation is inactive, active or removed from model" name="excavation state">0</item>
    <item type="int" name="item id">5</item>
    <item type="bool" name="uses default resolution">yes</item>
    <item type="double" name="seam resolution (cm)">700</item>
    <item type="bool" name="uses flat joint model">no</item>
    <item type="int" name="flat joint model number of contact points">3</item>
    <item type="double" name="flat joint model disk radius multiplier">0.5</item>
</group>
<group name="Seam 5">
    <item hint="local or global east-coordinate" type="double" name="origin east">-600</item>
    <item hint="local or global north-coordinate" type="double" name="origin north">0</item>
    <item hint="local or global up-coordinate" type="double" name="origin up">1700</item>
    <item hint="uses global coordinate" name="uses global coordinate">no</item>
    <item table="rock" type="list" name="material">None</item>
    <item type="double" name="point1 X">0</item>
    <item type="double" name="point1 Y">0</item>
    <item type="double" name="point1 Z">0</item>
</group>
<item type="double" name="point1 Y">0</item>
<item type="double" name="point1 Z">0</item>
<item type="double" name="point2 X">0</item>
<item type="double" name="point2 Y">0</item>
<item type="double" name="point2 Z">0</item>
<item hint="dip of seam [Deg]" name="dip">0</item>
<item hint="dip direction of seam [Deg]" name="dip direction">0</item>
<item hint="seam thickness/height [m] or arc (degrees)"
name="thickness">200</item>
<item hint="seam width or diameter [m]" name="width">1200</item>
<item hint="seam length [m]" name="length">170</item>
<item hint="initial conditionn for injection weel [m3/s]"
name="injection rate">0</item>
<item hint="seam shape [rect|cylinder]"
name="shape">Rectangular</item>
<item hint="Possible types: (0) Infinite Plane, (1) ClosedVolume,
(2) excavation/hole,etc." type="int" name="seam type">2</item>
<item hint="Indicates if excavation is inactive,active or removed
from model" name="excavation state">0</item>
<item type="int" name="item id">6</item>
<item type="bool" name="uses default resolution">yes</item>
<item type="double" name="seam resolution (cm)">700</item>
<item type="bool" name="uses flat joint model">no</item>
<item type="int" name="flat joint model number of contact
points">3</item>
<item type="double" name="flat joint model disk radius
multiplier">0.5</item>
</group>

<group name="Seam 6">
<item hint="local or global east-coordinate" type="double"
name="origin east">-725</item>
<item hint="local or global north-coordinate" type="double"
name="origin north">0</item>
<item hint="local or global up-coordinate" type="double"
name="origin up">1900</item>
<item type="bool" name="uses global coordinate">no</item>
<item table="rock" type="list" name="material">None</item>
<item type="double" name="point1 X">0</item>
<item type="double" name="point1 Y">0</item>
<item type="double" name="point1 Z">0</item>
<item type="double" name="point2 X">0</item>
<item type="double" name="point2 Y">0</item>
<item type="double" name="point2 Z">0</item>
<item hint="dip of seam [Deg]" name="dip">0</item>
<item hint="dip direction of seam [Deg]" name="dip direction">0</item>
<item hint="seam thickness/height [m] or arc (degrees)"
name="thickness">200</item>
<item hint="seam width or diameter [m]" name="width">1450</item>
<item hint="seam length [m]" name="length">170</item>
<item hint="initial conditionn for injection weel [m3/s]"
name="injection rate">0</item>

<item hint="seam shape [rect|cylinder]"
name="shape">Rectangular</item>

<item hint="Possible types: (0) Infinite Plane, (1) Closed Volume, (2) excavation/hole, etc." type="int" name="seam type">2</item>

<item hint="Indicates if excavation is inactive, active or removed from model" name="excavation state">0</item>

<item type="int" name="item id">7</item>

<item type="bool" name="uses default resolution">yes</item>

<item type="double" name="seam resolution (cm)">700</item>

<item type="bool" name="uses flat joint model">no</item>

<item type="int" name="flat joint model number of contact points">3</item>

<item type="double" name="flat joint model disk radius multiplier">0.5</item>

</group>

<group name="Seam 7">

<item hint="local or global east-coordinate" type="double"
name="origin east">-1450</item>

<item hint="local or global north-coordinate" type="double"
name="origin north">0</item>

<item hint="local or global up-coordinate" type="double"
name="origin up">1800</item>

<item type="bool" name="uses global coordinate">no</item>

<item table="rock" type="list" name="material">None</item>

<item type="double" name="point1 X">-1450</item>

<item type="double" name="point1 Y">0</item>

<item type="double" name="point1 Z">2000</item>

<item type="double" name="point2 X">-1650</item>

<item type="double" name="point2 Y">0</item>

<item type="double" name="point2 Z">2000</item>

<item hint="dip of seam [Deg]" name="dip">0</item>

<item hint="dip direction of seam [Deg]" name="dip direction">0</item>

<item hint="seam thickness/height [m] or arc (degrees)"
name="thickness">200</item>

<item hint="seam width or diameter [m]" name="width">1450</item>

<item hint="seam length [m]" name="length">170</item>

<item hint="initial conditionn for injection weel [m3/s]"
name="injection rate">0</item>

<item hint="seam shape [rect|cylinder]"
name="shape">Triangular</item>

<item hint="Possible types: (0) Infinite Plane, (1) Closed Volume, (2) excavation/hole, etc." type="int" name="seam type">2</item>

<item hint="Indicates if excavation is inactive, active or removed from model" name="excavation state">0</item>

<item type="int" name="item id">17</item>

<item type="bool" name="uses default resolution">yes</item>

<item type="double" name="seam resolution (cm)">700</item>

<item type="bool" name="uses flat joint model">no</item>
<item type="int" name="flat joint model number of contact points">3</item>
<item type="double" name="flat joint model disk radius multiplier">0.5</item>
</group>
<group name="Seam 8">
    <item hint="local or global east-coordinate" type="double" name="origin east">-1200</item>
    <item hint="local or global north-coordinate" type="double" name="origin north">0</item>
    <item hint="local or global up-coordinate" type="double" name="origin up">1600</item>
    <item type="bool" name="uses global coordinate">no</item>
    <item table="rock" type="list" name="material">None</item>
    <item type="double" name="point1 X">-1200</item>
    <item type="double" name="point1 Y">0</item>
    <item type="double" name="point1 Z">1800</item>
    <item type="double" name="point2 X">-1400</item>
    <item type="double" name="point2 Y">0</item>
    <item type="double" name="point2 Z">1800</item>
    <item hint="dip of seam [Deg]" name="dip">0</item>
    <item hint="dip direction of seam [Deg]" name="dip direction">0</item>
    <item hint="seam thickness/height [m] or arc (degrees)" name="thickness">200</item>
    <item hint="seam width or diameter [m]" name="width">1200</item>
    <item hint="seam length [m]" name="length">170</item>
    <item hint="indicates if excavation is inactive, active or removed from model" name="excavation state">0</item>
    <item type="int" name="id">18</item>
    <item type="bool" name="uses default resolution">yes</item>
    <item type="double" name="seam resolution (cm)">700</item>
    <item type="int" name="uses flat joint model">no</item>
    <item type="int" name="flat joint model number of contact points">3</item>
    <item type="double" name="flat joint model disk radius multiplier">0.5</item>
</group>
<group name="Seam 9">
    <item hint="local or global east-coordinate" type="double" name="origin east">-950</item>
    <item hint="local or global north-coordinate" type="double" name="origin north">0</item>
    <item hint="local or global up-coordinate" type="double" name="origin up">1400</item>
    <item type="bool" name="uses global coordinate">no</item>
<item table="rock" type="list" name="material">None</item>

<item type="double" name="point1 X">-950</item>
<item type="double" name="point1 Y">0</item>
<item type="double" name="point1 Z">1600</item>
<item type="double" name="point2 X">-1150</item>
<item type="double" name="point2 Y">0</item>
<item type="double" name="point2 Z">1600</item>
<item hint="dip of seam [Deg]" name="dip">0</item>
<item hint="dip direction of seam [Deg]" name="dip direction">0</item>
<item hint="seam thickness/height [m] or arc (degrees)" name="thickness">200</item>
<item hint="seam width or diameter [m]" name="width">950</item>
<item hint="seam length [m]" name="length">170</item>
<item hint="initial condition for injection well [m3/s]" name="injection rate">0</item>
<item hint="seam shape [rect|cylinder]" name="shape">Triangular</item>
<item hint="Possible types: (0) Infinite Plane, (1) ClosedVolume, (2) excavation/hole, etc." type="int" name="seam type">2</item>
<item hint="Indicates if excavation is inactive, active or removed from model" name="excavation state">0</item>
<item type="int" name="item id">19</item>
<item type="bool" name="uses default resolution">yes</item>
<item type="double" name="seam resolution (cm)">700</item>
<item type="bool" name="uses flat joint model">no</item>
<item type="int" name="flat joint model number of contact points">3</item>
<item type="double" name="flat joint model disk radius multiplier">0.5</item>
</group>

<group name="Seam 10">
<item hint="local or global east-coordinate" type="double" name="origin east">-700</item>
<item hint="local or global north-coordinate" type="double" name="origin north">0</item>
<item hint="local or global up-coordinate" type="double" name="origin up">1200</item>
<item type="bool" name="uses global coordinate">no</item>
<item table="rock" type="list" name="material">None</item>
<item type="double" name="point1 X">-700</item>
<item type="double" name="point1 Y">0</item>
<item type="double" name="point1 Z">1400</item>
<item type="double" name="point2 X">-900</item>
<item type="double" name="point2 Y">0</item>
<item type="double" name="point2 Z">1400</item>
<item hint="dip of seam [Deg]" name="dip">0</item>
<item hint="dip direction of seam [Deg]" name="dip direction">0</item>
<item hint="seam thickness/height [m] or arc (degrees)" name="thickness">200</item>
<item hint="seam width or diameter [m]" name="width">700</item>
<item hint="seam length [m]" name="length">170</item>
<item hint="initial conditionn for injection weel [m3/s]" name="injection rate">0</item>
<item hint="seam shape [rect|cylinder]" name="shape">Triangular</item>
<item hint="Possible types: (0) Infinite Plane, (1) ClosedVolume, (2) excavation/hole, etc." type="int" name="seam type">2</item>
<item hint="Indicates if excavation is inactive, active or removed from model" name="excavation state">0</item>
<item type="int" name="item id">20</item>
<item type="bool" name="uses default resolution">yes</item>
<item type="double" name="seam resolution (cm)">700</item>
<item type="bool" name="uses flat joint model">no</item>
<item type="int" name="flat joint model number of contact points">3</item>
<item type="double" name="flat joint model disk radius multiplier">0.5</item>
</group>
<group name="Seam 11">
<item hint="local or global east-coordinate" type="double" name="origin east">-450</item>
<item hint="local or global north-coordinate" type="double" name="origin north">0</item>
<item hint="local or global up-coordinate" type="double" name="origin up">1000</item>
<item type="bool" name="uses global coordinate">no</item>
<item table="rock" type="list" name="material">None</item>
<item type="double" name="point1 X">-450</item>
<item type="double" name="point1 Y">0</item>
<item type="double" name="point1 Z">1200</item>
<item type="double" name="point2 X">-650</item>
<item type="double" name="point2 Y">0</item>
<item type="double" name="point2 Z">1200</item>
<item hint="dip of seam [Deg]" name="dip">0</item>
<item hint="dip direction of seam [Deg]" name="dip direction">0</item>
<item hint="seam thickness/height [m] or arc (degrees)" name="thickness">200</item>
<item hint="seam width or diameter [m]" name="width">450</item>
<item hint="seam length [m]" name="length">170</item>
<item hint="initial conditionn for injection weel [m3/s]" name="injection rate">0</item>
<item hint="seam shape [rect|cylinder]" name="shape">Triangular</item>
<item hint="Possible types: (0) Infinite Plane, (1) ClosedVolume, (2) excavation/hole, etc." type="int" name="seam type">2</item>
<item hint="Indicates if excavation is inactive, active or removed from model" name="excavation state">2</item>
<item type="int" name="item id">21</item>
<item type="bool" name="uses default resolution">yes</item>
<item type="double" name="seam resolution (cm)">700</item>
<item type="bool" name="uses flat joint model">no</item>
<item type="int" name="flat joint model number of contact points">3</item>
<item type="double" name="flat joint model disk radius multiplier">0.5</item>
</group>
<group name="Seam 12">
  <item hint="local or global east-coordinate" type="double" name="origin east">-200</item>
  <item hint="local or global north-coordinate" type="double" name="origin north">0</item>
  <item hint="local or global up-coordinate" type="double" name="origin up">800</item>
  <item type="bool" name="uses global coordinate">no</item>
  <item table="rock" type="list" name="material">None</item>
  <item type="double" name="point1 X">-200</item>
  <item type="double" name="point1 Y">0</item>
  <item type="double" name="point1 Z">1000</item>
  <item type="double" name="point2 X">-400</item>
  <item type="double" name="point2 Y">0</item>
  <item type="double" name="point2 Z">1000</item>
  <item hint="dip of seam [Deg]" name="dip">0</item>
  <item hint="dip direction of seam [Deg]" name="dip direction">0</item>
  <item hint="seam thickness/height [m] or arc (degrees)" name="thickness">200</item>
  <item hint="seam width or diameter [m]" name="width">200</item>
  <item hint="seam length [m]" name="length">170</item>
  <item name="initial conditionn for injection weel [m3/s]" name="injection rate">0</item>
  <item hint="seam shape [rect|cylinder]" name="shape">Triangular</item>
  <item hint="Possible types: (0) Infinite Plane, (1) ClosedVolume, (2) excavation/hole, etc." type="int" name="seam type">2</item>
  <item hint="Indicates if excavation is inactive, active or removed from model" name="excavation state">2</item>
  <item type="int" name="item id">22</item>
  <item type="bool" name="uses default resolution">yes</item>
  <item type="double" name="seam resolution (cm)">700</item>
  <item type="bool" name="uses flat joint model">no</item>
  <item type="int" name="flat joint model number of contact points">3</item>
  <item type="double" name="flat joint model disk radius multiplier">0.5</item>
</group>
<group name="Seam 13">
  <item hint="local or global east-coordinate" type="double" name="origin east">-1750</item>
  <item hint="local or global north-coordinate" type="double" name="origin north">0</item>
  <item hint="local or global up-coordinate" type="double" name="origin up">300</item>
  <item type="bool" name="uses global coordinate">no</item>
</group>
<item table="rock" type="list" name="material">Quartz/Feld Gneiss</item>
<item type="double" name="point1 X">0</item>
<item type="double" name="point1 Y">0</item>
<item type="double" name="point1 Z">0</item>
<item type="double" name="point2 X">0</item>
<item type="double" name="point2 Y">0</item>
<item type="double" name="point2 Z">0</item>
<item hint="dip of seam [Deg]" name="dip">0</item>
<item hint="dip direction of seam [Deg]" name="dip direction">0</item>
<item hint="seam thickness/height [m] or arc (degrees)" name="thickness">600</item>
<item hint="seam length [m]" name="length">170</item>
<item hint="initial condition for injection well [m3/s]" name="injection rate">0</item>
<item hint="seam shape [rect|cylinder]" name="shape">Rectangular</item>
<item hint="Possible types: (0) Infinite Plane, (1) ClosedVolume, (2) excavation/hole,etc." type="int" name="seam type">1</item>
<item hint="Indicates if excavation is inactive, active or removed from model" name="excavation state">0</item>
<item type="int" name="item id">211</item>
<item type="bool" name="uses default resolution">yes</item>
<item type="double" name="seam resolution (cm)">700</item>
<item type="bool" name="uses flat joint model">no</item>
<item type="int" name="flat joint model number of contact points">3</item>
<item type="double" name="flat joint model disk radius multiplier">0.5</item>

</group>
<group name="Seam 14">
<item hint="local or global east-coordinate" type="double" name="origin east">-1750</item>
<item hint="local or global north-coordinate" type="double" name="origin north">0</item>
<item hint="local or global up-coordinate" type="double" name="origin up">1800</item>
<item type="bool" name="uses global coordinate">no</item>
<item table="rock" type="list" name="material">Sandstone</item>
<item type="double" name="point1 X">0</item>
<item type="double" name="point1 Y">0</item>
<item type="double" name="point1 Z">0</item>
<item type="double" name="point2 X">0</item>
<item type="double" name="point2 Y">0</item>
<item type="double" name="point2 Z">0</item>
<item hint="dip of seam [Deg]" name="dip">0</item>
<item hint="dip direction of seam [Deg]" name="dip direction">0</item>
<item hint="seam thickness/height [m] or arc (degrees)" name="thickness">400</item>
<item hint="seam width or diameter [m]" name="width">3500</item>
<item hint="seam length [m]" name="length">170</item>
<item hint="initial conditionn for injection weel [m3/s]" name="injection rate">0</item>
<item hint="seam shape [rect|cylinder]" name="shape">Rectangular</item>
<item hint="Possible types: (0) Infinite Plane, (1) ClosedVolume, (2) excavation/hole, etc." type="int" name="seam type">1</item>
<item hint="Indicates if excavation is inactive, active or removed from model" name="excavation state">0</item>
<item type="int" name="item id">296</item>
<item type="bool" name="uses default resolution">yes</item>
<item type="double" name="seam resolution (cm)">700</item>
<item type="bool" name="uses flat joint model">no</item>
<item type="int" name="flat joint model number of contact points">3</item>
<item type="double" name="flat joint model disk radius multiplier">0.5</item>
</group>
</properties>
</page>

<page name="histories">

<properties>

<group prototype="yes" name="history">
  <item type="string" name="type"/>
  <item type="string" name="name"/>
  <item type="double" name="reference point east"/>
  <item type="double" name="reference point north"/>
  <item type="double" name="reference point up"/>
  <item type="int" name="flux bloundary"/>
</group>

<group name="history 1">
  <item type="string" name="type">cracks</item>
  <item type="string" name="name"></item>
  <item type="double" name="reference point east">0</item>
  <item type="double" name="reference point north">0</item>
  <item type="double" name="reference point up">0</item>
  <item type="int" name="flux bloundary">0</item>
</group>

<group name="history 2">
  <item type="string" name="type">x-displacement</item>
  <item type="string" name="name">x0 crest</item>
  <item type="double" name="reference point east">-1650</item>
  <item type="double" name="reference point north">75</item>
  <item type="double" name="reference point up">2000</item>
  <item type="int" name="flux bloundary">0</item>
</group>

<group name="history 3">
  <item type="string" name="type">z-displacement</item>
  <item type="string" name="name">z0 crest</item>
  <item type="double" name="reference point east">-1650</item>
  <item type="double" name="reference point north">75</item>
</group>

</properties>
</page>
<item type="double" name="reference point up">2000</item>
<item type="int" name="flux bloundary">0</item>
</group>

<group name="history 4">
  <item type="string" name="type">z-displacement</item>
  <item type="string" name="name">z1 crest</item>
  <item type="double" name="reference point east">-1400</item>
  <item type="double" name="reference point north">75</item>
  <item type="double" name="reference point up">1800</item>
  <item type="int" name="flux bloundary">0</item>
</group>

<group name="history 5">
  <item type="string" name="type">x-displacement</item>
  <item type="string" name="name">x1 crest</item>
  <item type="double" name="reference point east">-1400</item>
  <item type="double" name="reference point north">75</item>
  <item type="double" name="reference point up">1800</item>
  <item type="int" name="flux bloundary">0</item>
</group>

<group name="history 6">
  <item type="string" name="type">x-displacement</item>
  <item type="string" name="name">x2 crest</item>
  <item type="double" name="reference point east">-1150</item>
  <item type="double" name="reference point north">75</item>
  <item type="double" name="reference point up">1600</item>
  <item type="int" name="flux bloundary">0</item>
</group>

<group name="history 7">
  <item type="string" name="type">z-displacement</item>
  <item type="string" name="name">z2 crest</item>
  <item type="double" name="reference point east">-1150</item>
  <item type="double" name="reference point north">75</item>
  <item type="double" name="reference point up">1600</item>
  <item type="int" name="flux bloundary">0</item>
</group>

<group name="history 8">
  <item type="string" name="type">z-displacement</item>
  <item type="string" name="name">z3 crest</item>
  <item type="double" name="reference point east">-900</item>
  <item type="double" name="reference point north">75</item>
  <item type="double" name="reference point up">1400</item>
  <item type="int" name="flux bloundary">0</item>
</group>

<group name="history 9">
  <item type="string" name="type">x-displacement</item>
  <item type="string" name="name">x3 crest</item>
  <item type="double" name="reference point east">-900</item>
  <item type="double" name="reference point north">75</item>
  <item type="double" name="reference point up">1400</item>
  <item type="int" name="flux bloundary">0</item>
</group>

<group name="history 10">
  <item type="double" name="reference point up">2000</item>
  <item type="int" name="flux bloundary">0</item>
</group>
<item type="string" name="type">x-displacement</item>
<item type="string" name="name">x4 crest</item>
<item type="double" name="reference point east">-650</item>
<item type="double" name="reference point north">75</item>
<item type="double" name="reference point up">1200</item>
<item type="int" name="flux boundary">0</item>
</group>
<group name="history 11">
<item type="string" name="type">z-displacement</item>
<item type="string" name="name">z4 crest</item>
<item type="double" name="reference point east">-650</item>
<item type="double" name="reference point north">75</item>
<item type="double" name="reference point up">1200</item>
<item type="int" name="flux boundary">0</item>
</group>
<group name="history 12">
<item type="string" name="type">z-displacement</item>
<item type="string" name="name">z5 crest</item>
<item type="double" name="reference point east">-400</item>
<item type="double" name="reference point north">75</item>
<item type="double" name="reference point up">1000</item>
<item type="int" name="flux boundary">0</item>
</group>
<group name="history 13">
<item type="string" name="type">x-displacement</item>
<item type="string" name="name">x5 crest</item>
<item type="double" name="reference point east">-400</item>
<item type="double" name="reference point north">75</item>
<item type="double" name="reference point up">1000</item>
<item type="int" name="flux boundary">0</item>
</group>
<group name="history 14">
<item type="string" name="type">x-displacement</item>
<item type="string" name="name">x6 crest</item>
<item type="double" name="reference point east">-200</item>
<item type="double" name="reference point north">75</item>
<item type="double" name="reference point up">800</item>
<item type="int" name="flux boundary">0</item>
</group>
<group name="history 15">
<item type="string" name="type">z-displacement</item>
<item type="string" name="name">z6 crest</item>
<item type="double" name="reference point east">-200</item>
<item type="double" name="reference point north">75</item>
<item type="double" name="reference point up">800</item>
<item type="int" name="flux boundary">0</item>
</group>
</properties>
</page>
$page name="Solution">
<properties>
<item type="double" name="model resolution (cm)">1700</item>
</properties>
<item type="double" name="fluid initi. convergence factor">0.05</item>
<item type="bool" name="fluid flow option">no</item>
<item type="bool" name="matrix flow option">no</item>
<item type="bool" name="aperture change">no</item>
<item type="double" name="maximum aperture">0</item>
<item type="double" name="tolerance angle for joint intersection">10</item>
<item hint="calibration velocity" type="double" name="calibration velocity">0</item>
<item hint="calibration confinement pressure" type="double" name="calibration confinement pressure">0</item>
<item hint="calibration mode" type="bool" name="calibration mode">no</item>
<item hint="calibration symmetry" type="bool" name="calibration symmetry">yes</item>
<item hint="calibration test type" type="int" name="calibration test type">1</item>
<item hint="Rotation scheme, spin on or off" type="bool" name="spin">yes</item>
<item hint="use voronoi on model initialization" type="bool" name="use voronoi">no</item>
<item type="bool" name="water table change">no</item>
<item type="double" name="water table change factor">10</item>
<item type="int" name="seam current id">328</item>
<item hint="flat joint model active" type="bool" name="flat joint model active">no</item>
<item type="int" name="flat joint model number of contact points">3</item>
<item type="double" name="flat joint model disk radius multiplier">0.5</item>
<item type="double" name="gravity">9.81</item>
</properties>
</page>

<page name="batch simulation">
<properties>
<item type="string" name="saved files directory">C:\Users\cherrero\Desktop\slope runs\EXTENDED MODEL\9B-toppling faults joints\eh=75\eh=75</item>
<item type="string" name="saved files base name">1200 Eh=75</item>
<group prototype="yes" name="batch">
<item hint="simulation time" type="double" name="simulation time"/>
<item hint="if yes, engine will perform mechanical calculation" type="bool" name="mechanical flag"/>
<item hint="if yes, engine will perform mechanical calculation" type="bool" name="fluid flag"/>
<item hint="if yes, state will be saved after step is completed" type="bool" name="save state"/>
<item hint="if yes, engine will reset displacements" type="bool" name="reset node displacement"/>
</group>
</properties>
</page>
<item hint="if yes, engine will increment the micro crack counter" type="bool" name="increment crack counter"/>
<item hint="if yes, water table flag in comp. engine will be set to true" type="bool" name="water table change flag"/>
<item type="double" name="water table change factor">10.0</item>
<item hint="list of excavation ID's, comma separated" type="string" name="excavation list"></item>
<item hint="list of excavation layers defined in the DXF file, comma separated" type="string" name="excavation layers"></item>
<item hint="status of the run" type="string" name="status">Completed:</item>
<item hint="the time that the simulation started" type="string" name="start time">14:48:4 pm</item>
<item hint="the time that the simulation has completed" type="string" name="end time">15:4:21 pm</item>
<group prototype="yes" name="fluid boundary condition group">
  <item hint="the name of the DXF layer" type="string" name="layer name"/>
  <item hint="fluid boundary condition, constant head = 0, constant pressure = 1" type="int" name="fluid boundary condition">0</item>
  <item hint="value for constant head (m) or pressure (Pa) depending on fluid boundary condition" type="double" name="constant head or pressure"/>
</group>
<group prototype="yes" name="time dependency group">
  <item hint="time in secs" type="double" name="time"/>
  <item hint="pressure in pa" type="double" name="pressure"/>
</group>
</group>
</group>
<group name="batch 2">
  <item hint="simulation time" type="double" name="simulation time">10</item>
  <item hint="if yes, engine will perform mechnical calculation" type="bool" name="mechnical flag">yes</item>
  <item hint="if yes, engine will perform mechnical calculation" type="bool" name="fluid flag">no</item>
  <item hint="if yes, state will be saved after step is completed" type="bool" name="save state">yes</item>
  <item hint="if yes, engine will reset displacements" type="bool" name="reset node displacement">yes</item>
  <item hint="if yes, engine will increment the micro crack counter" type="bool" name="increment crack counter">yes</item>
  <item hint="if yes, water table flag in comp. engine will be set to true" type="bool" name="water table change flag">no</item>
  <item type="double" name="water table change factor">10</item>
  <item hint="list of excavation ID's, comma separated" type="string" name="excavation list">6,7</item>
  <item hint="list of excavation layers defined inf the DXF file, comma separated" type="string" name="excavation layers"></item>
  <item hint="status of the run" type="string" name="status">Completed:</item>
  <item hint="the time that the simulation started" type="string" name="start time">15:4:21 pm</item>
  <item hint="the time that the simulation has completed" type="string" name="end time">15:18:38 pm</item>
</group>
</group>
<item hint="value for constant head (m) or pressure (Pa)
depending on fluid boundary condition" type="double" name="constant
head or pressure"/>
<group prototype="yes" name="time dependency group">
  <item hint="time in secs" type="double" name="time"/>
  <item hint="pressure in pa" type="double" name="pressure"/>
</group>
</group>
<group name="batch 3">
  <item hint="simulation time" type="double" name="simulation
time">10</item>
  <item hint="if yes, engine will perform mechanical calculation"
type="bool" name="mechanical flag">yes</item>
  <item hint="if yes, engine will perform mechanical calculation"
type="bool" name="fluid flag">no</item>
  <item hint="if yes, state will be saved after step is completed"
type="bool" name="save state">yes</item>
  <item hint="if yes, engine will reset displacements" type="bool"
name="reset node displacement">no</item>
  <item hint="if yes, engine will increment the micro crack
counter" type="bool" name="increment crack counter">no</item>
  <item hint="if yes, water table flag in comp. engine will be set
to true" type="bool" name="water table change flag">no</item>
  <item hint="list of excavation ID's, comma separated" type="int"
name="excavation list">5,8</item>
  <item hint="list of excavation layers defined in the DXF file,
comma separated" type="string" name="excavation layers"></item>
  <item hint="status of the run" type="string" name="status">Completed:</item>
  <item hint="the time that the simulation started" type="string"
name="start time">15:18:38 pm</item>
  <item hint="the time that the simulation has completed"
type="string" name="end time">15:32:22 pm</item>
</group>
<group prototype="yes" name="fluid boundary condition group">
  <item hint="the name of the DXF layer" type="string" name="layer
name"/>
  <item hint="fluid boundary condition, constant head = 0,
constant pressure = 1" type="int" name="fluid boundary
condition">0</item>
</group>
<item hint="simulation time" type="double" name="simulation time">10</item>
<item hint="if yes, engine will perform mechanical calculation" type="bool" name="mechanical flag">yes</item>
<item hint="if yes, engine will perform mechanical calculation" type="bool" name="fluid flag">no</item>
<item hint="if yes, state will be saved after step is completed" type="bool" name="save state">yes</item>
<item hint="if yes, engine will reset displacements" type="bool" name="reset node displacement">no</item>
<item hint="if yes, engine will increment the micro crack counter" type="bool" name="increment crack counter">no</item>
<item hint="if yes, water table flag in comp. engine will be set to true" type="bool" name="water table change flag">no</item>
<item hint="value for constant head (m) or pressure (Pa) depending on fluid boundary condition" type="double" name="constant head or pressure"/>
<group prototype="yes" name="time dependency group">
  <item hint="time in secs" type="double" name="time"/>
  <item hint="pressure in pa" type="double" name="pressure"/>
</group>
</group>
</item>
<group name="batch 5">
  <item hint="simulation time" type="double" name="simulation time">10</item>
  <item hint="if yes, engine will perform mechanical calculation" type="bool" name="mechanical flag">yes</item>
  <item hint="if yes, engine will perform mechanical calculation" type="bool" name="fluid flag">no</item>
  <item hint="if yes, state will be saved after step is completed" type="bool" name="save state">yes</item>
  <item hint="if yes, engine will reset displacements" type="bool" name="reset node displacement">no</item>
</group>
<item hint="if yes, engine will increment the micro crack counter" type="bool" name="increment crack counter">no</item>
<item hint="if yes, water table flag in comp. engine will be set to true" type="bool" name="water table change flag">no</item>
<item type="double" name="water table change factor">10</item>
<item hint="list of excavation ID's, comma separated" type="string" name="excavation list">3,10</item>
<item hint="list of excavation layers defined inf the DXF file, comma separated" type="string" name="excavation layers"></item>
<item hint="status of the run" type="string" name="status">Running</item>
<item hint="the time that the simulation started" type="string" name="start time">15:46:25 pm</item>
<item hint="the time that the simulation has completed" type="string" name="end time"></item>
<group prototype="yes" name="fluid boundary condition group">
  <item hint="the name of the DXF layer" type="string" name="layer name"/>
  <item hint="fluid boundary condition, constant head = 0, constant pressure = 1" type="int" name="fluid boundary condition">0</item>
  <item hint="value for constant head (m) or pressure (Pa) depending on fluid boundary condition" type="double" name="constant head or pressure"></item>
  <group prototype="yes" name="time dependency group">
    <item hint="time in secs" type="double" name="time"/>
    <item hint="pressure in pa" type="double" name="pressure"/>
  </group>
</group>
</group>
<group name="batch 6">
  <item hint="simulation time" type="double" name="simulation time">10</item>
  <item hint="if yes, engine will perform mechanical calculation" type="bool" name="mechanical flag">yes</item>
  <item hint="if yes, engine will perform mechanical calculation" type="bool" name="fluid flag">no</item>
  <item hint="if yes, state will be saved after step is completed" type="bool" name="save state">yes</item>
  <item hint="if yes, engine will reset displacements" type="bool" name="reset node displacement">no</item>
  <item hint="if yes, engine will increment the micro crack counter" type="bool" name="increment crack counter">no</item>
  <item hint="if yes, water table flag in comp. engine will be set to true" type="bool" name="water table change flag">no</item>
  <item type="double" name="water table change factor">10</item>
  <item hint="list of excavation ID's, comma separated" type="string" name="excavation list">2,11</item>
  <item hint="list of excavation layers defined inf the DXF file, comma separated" type="string" name="excavation layers"></item>
  <item hint="status of the run" type="string" name="status"></item>
</group>
<item hint="the time that the simulation started" type="string"
name="start time"></item>
<item hint="the time that the simulation has completed"
type="string" name="end time"></item>
<group prototype="yes" name="fluid boundary condition group">
<item hint="the name of the DXF layer" type="string" name="layer
name"/>
<item hint="fluid boundary condition, constant head = 0,
constant pressure = 1" type="int" name="fluid boundary
condition">0</item>
<item hint="value for constant head (m) or pressure (Pa)
depending on fluid boundary condition" type="double" name="constant
head or pressure"/>
<group prototype="yes" name="time dependency group">
<item hint="time in secs" type="double" name="time"/>
<item hint="pressure in pa" type="double" name="pressure"/>
</group>
</group>
</group>
<group name="batch 7">
<item hint="simulation time" type="double" name="simulation
time">10</item>
<item hint="if yes, engine will perform mechnical calculation"
type="bool" name="mechanical flag">yes</item>
<item hint="if yes, engine will perform mechnical calculation"
type="bool" name="fluid flag">no</item>
<item hint="if yes, state will be saved after step is completed"
type="bool" name="save state">yes</item>
<item hint="if yes, engine will reset displacements" type="bool"
name="reset node displacement">no</item>
<item hint="if yes, engine will increment the micro crack
counter" type="bool" name="increment crack counter">no</item>
<item hint="if yes, water table flag in comp. engine will be set
to true" type="bool" name="water table change flag">no</item>
<item type="double" name="water table change factor">10</item>
<item hint="list of excavation ID's, comma separated"
type="string" name="excavation list">1,12</item>
<item hint="list of excavation layers defined inf the DXF file,
comma separated" type="string" name="excavation layers"/>
<item hint="status of the run" type="string"
name="status"></item>
<item hint="the time that the simulation started" type="string"
name="start time"></item>
<item hint="the time that the simulation has completed"
type="string" name="end time"></item>
<group prototype="yes" name="fluid boundary condition group">
<item hint="the name of the DXF layer" type="string" name="layer
name"/>
<item hint="fluid boundary condition, constant head = 0,
constant pressure = 1" type="int" name="fluid boundary
condition">0</item>
</group>
<item hint="value for constant head (m) or pressure (Pa) depending on fluid boundary condition" type="double" name="constant head or pressure"/>
<group prototype="yes" name="time dependency group">
  <item hint="time in secs" type="double" name="time"/>
  <item hint="pressure in pa" type="double" name="pressure"/>
</group>
</group>
</properties>
</page>
</model>
</LOP>