MINIMISING DILUTION IN NARROW-VEIN MINES

Penelope Clair Stewart
B.Eng (Mining), Grad.Cert.Eng

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Julius Kruttschnitt Mineral Research Centre
University of Queensland

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STATEMENT OF ORIGINALITY

I declare that the work presented in this thesis is, to the best of my knowledge and belief, original except as acknowledged in the text. No material has been submitted, either whole or in part, for a degree at this or any other university. This dissertation presents original contributions in the following areas:

- A comprehensive review of parameters likely to influence narrow-vein dilution. Dilution parameters not explicitly considered by existing stope stability prediction methods were shown to have a higher potential to influence narrow-vein stope stability than large open stope stability.
- Quantify database requirements for a site-specific stability chart. Evidence was presented to indicate that ‘site-specific’ effects are not truly site-specific, but are instead ‘operating-condition specific’. This conclusion forms the rationale for developing a dilution prediction method addressing narrow-vein operating conditions to improve dilution prediction for narrow-vein stoping.
- Defining and quantifying the effect of three different types of relaxation (partial, full and tangential relaxation) has provided an explanation for the apparent discrepancies in the literature pertaining to the effect of stress relaxation on excavation stability.
- Evidence has been presented to demonstrate that while stability graphs are reliable methods for predicting the probability of geotechnical instability, they are poor predictors of linear dilution because neither stability number (N) nor hydraulic radius (HR) were continuously related to linear overbreak or slough for narrow-vein case studies.
- The effect of stress history on narrow-vein dilution has been quantified and methods for assessing stress damage related dilution potential have been proposed.
- Benchmark stoping and expected stoping widths have been defined for three common narrow-vein longhole blast patterns.
- Development of an empirical narrow-vein design method (NVD method) to improve narrow-vein dilution prediction for greenfield sites and reduce dilution at operating mines.

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ABSTRACT

This thesis project was an initiative of the JKMRC and formed part of the project deliverables for the 2nd AMIRA Blasting and Reinforcement Technology Project. An improved ability to predict dilution in narrow-vein longhole stoping compared to conventional stoping facilitates more accurate economic comparisons between mechanised longhole stoping and other narrow-vein mining methods. Furthermore, understanding the underlying causes of dilution in narrow-vein mining is an essential element in dilution minimisation.

Sponsors of the 2nd AMIRA Blasting and Reinforcement Technology Project expressed concern about the applicability of empirical stability charts to narrow-vein stope design and dilution prediction. To address these concerns, a number of parameters identified as having a high potential to contribute to narrow-vein instability and not generally incorporated into stability charts were reviewed. Of these, several were considered rockmass dependent and therefore, to some extent implicitly taken into account by $Q'$. The possibility of developing site-specific stability charts to capture the potential sensitivity of narrow-vein stopes to $Q'$ was investigated. Monte Carlo simulations analysing trends in the variance of the Extended Mathews logit model indicated that a reliable stable-failure boundary requires at least 150 case histories, of which a minimum of 10 percent should be unstable stope surfaces. The time required to collate sufficient case studies for a site-specific chart is considered a significant limitation of this approach to improved narrow-veins stope design. Furthermore, dilution estimates are required for the feasibility stage of a greenfield project, prior to the availability of site data.

Only marginal site-specific effects were observed for the operating conditions captured within the Extended Mathews database (485 case studies). It has been concluded that the apparent site-specific effects referred to in previous literature are attributable to operating conditions inadequately represented in the database. Such operating conditions could induce erroneous stability predictions at any site, and therefore, are not truly site-specific. Following from these conclusions, a methodology for taking into account narrow-vein operating conditions has been proposed.
Stress relaxation is one of the narrow-vein operating conditions hypothesised to adversely affect narrow-vein stope stability prediction using existing stability chart methods. While many authors refer to the adverse effect of stress relaxation on excavation stability, some authors present compelling empirical evidence indicating that stress relaxation does not have a significant effect. Establishing clear definitions of stress relaxation was critical to understanding and quantifying stress relaxation effects. Three types of stress relaxation have been defined; partial relaxation, full relaxation and tangential relaxation. Once clear definitions were determined, it became clear that the theoretical arguments and empirical evidence presented by various authors to support their respective cases are not contradictory; rather the different conclusions can be attributed to different types of stress relaxation. In particular, when the minor principal stress is negative the intermediate principal stress has been identified as significantly affecting jointed rock mass behaviour. A new set of guidelines to account for the effect of stress relaxation within the stability chart approach has been proposed.

115 narrow-vein case studies from the Barkers mine in Western Australia showed a poor correlation to both the stability number (N) and hydraulic radius (HR). Given that both N and HR correlate well with stability in the vast majority of stability chart case studies, this suggests there is an overriding influence on stability at Barkers not accounted by stability charts. Blast pattern was found to have a statistically significant effect on overbreak. There was no evidence that tight backfill abutments (not continuous moving) behave differently from solid rock abutments in terms of determination of stable stope dimensions. These findings provided justification for further explicit consideration of narrow-vein operating conditions.

The effect of stress damage associated with the incremental extraction of long-hole rings resulting in a retreating high stressed zone at the stope brow was analysed. The study involved analysis of overbreak from 412 case studies from Barker mine. Numerical modelling of a 32 month extraction sequence for each case study demonstrated that stress damaged stope walls at this mine had an average 0.27 metres more overbreak than stope walls where stresses had not exceeded the site calibrated damage criterion.

Narrow-vein case studies from Barkers, Callinan and Trout Lake mine suggest that overbreak is not continuously related to N and HR well inside the stable zone. Based on this result it was concluded if a stope plots well inside the stable zone the cause of dilution is unlikely to be related to geotechnical instability. This interpretation implies that the causes of narrow-vein dilution can be separated into two independent causes:
• Geotechnical instability (stope size dependent).
• Blast overbreak (unrelated to stope size).

A probabilistic benchmarking method has been used to estimate a benchmark stoping widths for three commonly used narrow-vein longhole blast patterns. The benchmark stability stoping width for each pattern defines realistic planned dilution limits. These limits provide the basis from which true unplanned dilution can be assessed. The probabilistic overbreak model has also been used to predict expected (average) stope widths for each of the patterns. Benchmark average stoping width, in conjunction with vein or ore width, can be used to estimate total dilution.

Benchmark stoping widths are primarily a function of the longhole stoping method (operating-condition specific). On this basis, the benchmark stoping widths determined for Barkers mine are applicable to narrow-vein longhole stoping with similar operating conditions. However, it was recognised that complex geology (e.g. cross-cutting structures) would require adjustments be made to both the benchmark stability stoping width and the benchmark average stoping width.

The narrow-vein dilution methodology (NVD method) proposed in this thesis is a tool for predicting narrow-vein dilution based on the benchmark average stoping widths. In addition to dilution prediction, the NVD method also includes recommendation and strategies for narrow-vein dilution minimisation generally, including; filling, cablebolting, stress relaxation, stress damage and blast overbreak. It is envisaged that improved dilution prediction will lead to more accurate comparisons of the expected cost of dilution in longhole stopes compared to other mining methods. In this sense, the title of this thesis could more accurately be described as ‘optimising dilution in narrow-vein mines’. This recognises that optimal mining method selection will, in some instances, accept a higher level of dilution as part of a higher overall NPV for an operation. Conversely, in other instances the higher mining unit costs may be acceptable to increase overall NPV.


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CHAPTER 1

INTRODUCTION

1.1 Background

Narrow-vein mining methods are used to extract thin tabular orebodies. Dilution, in the context of mining underground orebodies, refers to the entrainment of waste material within the ore delivered to the mill. In this thesis dilution refers to the unplanned dilution associated with stope instability or blasting overbreak. An improved ability to predict dilution enables the economic risks associated with unplanned dilution to be reduced.

Dilution is associated with indirect and direct costs (Pakalnis, 1986; Elbrond, 1994; Pakalnis et al., 1995; Villaescusa, 1995; Bock, 1996; Bock et al., 1998; Revey, 1998; Scoble and Moss, 1994). Dilution can be defined as the contamination of ore by non-ore material during the mining process (Wright, 1983). Dilution is commonly expressed as the percentage of overbreak in metres or tonnes or cubic metres divided by the planned stope extraction width or tonnes or volume. For example, one metre of overbreak constitutes 25 percent dilution in a four metre wide stope, whilst in a 20 metre wide stope one metre of overbreak constitutes only 5 percent dilution. Therefore, narrow-vein stope dilution is more sensitive to overbreak than large open stopes.

The Mathews Stability Graph (Mathews et al. 1981), the Modified Stability Chart (Potvin, 1988) and the Extended Mathews Stability Chart (Trueman et al., 2000; Mawdesley, 2001) databases contain fewer than nine narrow-vein stopes. For this reason, there is some concern about the applicability of these charts to narrow-vein underground stopes. Based upon a comprehensive literature survey and the anecdotal evidence therein, Figure 1.1 illustrates the hypothesised relative importance of stope dilution variables for narrow vein stopes and large open stopes. The stope dilution variables contained in Figure 1.1 are not explicitly taken into account by commonly used empirical stope models (Mathews et al. 1981; Potvin, 1988; Clark and Pakalnis, 1997; Trueman et al. 2000). The stability graph approach includes some averaging effect of blasting, undercutting, time corresponding to the database. When these
factors differ from the average assessments, the observed stability of a stope may be more or less than the predicted stability.

However, over the past ten years authors have proposed methodologies to account for the following parameters; in-stope pillars and moving backfill abutments (Milne, 1996), undercutting of stope walls (Wang, 2002b), stress relaxation (Diederichs, 1999; Wang, 2002a), blast damage (Villaescusa, 1998) and stress damage (Sprott et al., 1999). One of the aims of this thesis was to review and evaluate these methodologies with respect to narrow-vein dilution prediction.

Despite the exclusion of the additional variables contained in Figure 1.1, stability charts are approximately 80 percent reliable for large open stopes (Stewart and Trueman, 2001). Reliability has been measured in terms of specificity and sensitivity (Parker and Davis, 1999), where 80 percent of stable points are correctly predicted as stable and 80 percent of unstable points are correctly predicted as unstable. As an empirical model, there is a trade off between model complexity and the practicality and cost of data collection. In the case of large open stopes, the broad usage of stability charts in both Canada and Australia, suggests the level of
complexity is well balanced by practicality of data collection. However, narrow-vein mine operators report less confidence in existing stability charts (Ascott, 2000; Li, 2000). In addition, the subjectivity of the stability category is an important limitation of the method when applied to narrow vein mines. For this reason an objective measurement of overbreak is considered an important aspect of risk management in narrow vein mines. In the past five years wider application of laser cavity monitoring technology (Miller et al. 1992) has seen the incorporation of equivalent linear overbreak/slough (ELOS) as an objective measure of instability (Clark and Pakalnis, 1997; Suorineni, 1998; Suorineni et al., 2001b; Wang, 2002a and 2002b).

1.2 Hypothesis
The hypothesis of this thesis is that existing stope stability prediction methodologies are inadequate for narrow-vein stope stability prediction. The premise for this hypothesis is that many of the factors believed to significantly affect narrow-vein dilution are inadequately accounted for by existing methods.

1.3 Aim of Study
The aim of the study was to empirically quantify the effect of each parameter hypothesised to have a significant effect on narrow-vein dilution, as well as review and evaluate methodologies proposed in the literature to take into account a number of these parameters. It was envisaged that an improved methodology for narrow-vein dilution prediction would be an outcome of this thesis.

1.4 Methodology
Due to the complexity of the problem of stope stability and the uncertainty associated with the engineering properties and behaviour of rock masses generally, empirical and semi-empirical approaches have been adopted for this study. Numerical, analytical and theoretical techniques have been employed as part of the iterative process of determining the most suitable parameters for empirical analysis.

The methods of analysis varied according to the type of data available. In the case of the categorical variables such as stable and failure, logistical and misclassification methods were employed. When continuous variables such as linear overbreak (metres of overbreak) were available comparative statistical analyses were performed. Whenever possible alternative
analytical methods have been employed to the same hypothesis as part of a critical review of conclusions reached.

Neural networking techniques were considered. However, because of the mismatches between fields it was decided that the inaccuracy associated with converting between systems of categorising stability, for example, would reduce accuracy and thus affect the power of statistical tests to detect small differences between groups. In addition, controlling variability of the other parameters in a database reduces the amount of data required to detect a small difference in dilution.

As a largely empirical study of narrow-vein dilution, the collection of site case studies forms an important component of this thesis. During the course of research study for this thesis the following Western Australian sites were visited: Kundana Gold operations (Barker mine), Kanowna Belle gold mine, Mt Charlotte mine and Kambalda Nickel operations (Junction mine). Case studies were collected from all four operations. In addition, stope records from the Mt Isa lead mine stopes were evaluated for suitability to the study of narrow-vein dilution parameters. However, due to missing fields in the database and or dependencies between parameters under consideration only the Barker mine case studies are presented in this thesis. While the case studies collected from Kanowna Belle, Mt Charlotte and Junction mines are not included in this thesis, the observations and anecdotal experiences related by site personnel while on site were invaluable in formulating the direction and priorities of research study for this thesis. Literature sources of case studies have been utilised in two out of the five studies presented in this thesis.

1.5 Thesis Outline

This thesis is comprised of four separate empirical studies. The studies were conducted separately according to the parameters available for study in each of the four databases collated. Due to excessive mismatches in fields between databases the various databases were not merged.

Chapter 2 examines the effect of dilution on economic performance, as well as defining and discussing the various methods to quantify dilution. This chapter also considers how the increasing prevalence of longhole stoping methods in narrow vein mines is affecting dilution. Three approaches to dilution minimisation were identified. This chapter provides direction and
focus for this thesis by defining evaluation of stope design with respect to dilution potential as the primary dilution minimisation strategy to be examined.

In Chapter 3 empirical, analytical and numerical stope design methods are reviewed with respect to their potential to predict dilution in narrow-vein mines. Chapter 4 reviews literature regarding each of the parameters hypothesised to have a significant effect on narrow-vein dilution with the aim of establishing thesis priorities. The objectives of this thesis were updated during the course of study to reflect both new literature and progress study.

Chapter 5 contains the results of the first study evaluating the potential for site-specific charts to improve dilution prediction in narrow-vein mines. The aims of this study were two-fold. The first aim was to quantify, using a database of 485 case studies sourced from the literature, the amount and type of data that would be required to predict a reliable stability chart. Once the data requirements for a site-specific chart had been determined, the second aim was to investigate whether site-specific effects are significant. This was achieved by comparing a site-specific stability chart to the generic stability chart.

In Chapter 6 methods for quantifying the effect of stress relaxation on stability have been reviewed and evaluated. 55 relaxed case studies were collated from the literature. Two and three dimensional linear elastic modelling was undertaken for each case study. Three types of relaxation have been defined. Recommendations regarding the treatment of each type of stress relaxation within the stability chart framework have been proposed.

In Chapter 7 back-analysis of 115 case studies collated from Kundana Gold operations Barkers mine in Western Australia demonstrated the limitations of applying existing stability charts to predict narrow-vein stope stability. Drill and blast issues were isolated as the most likely cause of the poor correlation of overbreak to stability chart parameters. Statistical analyses of linear overbreak indicated that blast pattern had a statistically significant affect on overbreak. The effect of undercutting footwalls, stationary backfill abutments and drillhole accuracy on overbreak were analysed using indirect semi-quantifiable techniques.

Chapter 8 evaluates the potential for stress damage to affect narrow-vein stability. In particular, the effect of brow stresses to affect large areas of hangingwall and footwall has been investigated. A linear elastic model of a 32 step extraction sequence was developed from stope record sheets and enabled the stress history at a point corresponding to the mid stope span to
be logged and analysed. Each step represents one month in the life of the Barkers mine. Peak stresses were analysed with respect to the site calibrated damage criterion and observed linear overbreak.

Chapter 9 evaluates the applicability of stability charts to narrow-vein stability prediction. This was achieved by applying the adjustments determined in earlier chapter to evaluate the accuracy of stability graphs for narrow-vein. In addition the logistical nature of the stability graph is demonstrated by illustrating the lack of continuity in the relationship between stability graph parameters and dilution. Arguments are presented to justify separating the causes of narrow-vein dilution into two independent groups: blast overbreak related and geotechnical stability.

Chapter 10 uses probabilistic analysis of overbreak distributions from the Barkers mine to develop benchmark practical stoping widths and expected stoping widths for three common longhole narrow-vein blast patterns. Because the parameters affecting benchmark stoping width and expected stoping width are specific to longhole narrow-vein stoping they have not been considered site-specific.

In Chapter 11 benchmark stoping width and expected stoping width are applied as part of the narrow-vein dilution method (NVD Method). The NVD Method summarises the results and conclusions of this thesis with a view to improved narrow-vein dilution prediction.

Chapter 12 summarises conclusions and provided recommendation for future work.
This chapter considers the economic effects of dilution and reviews mining methods with respect to dilution potential. The application of longhole stoping in narrow-vein mines has increased dramatically over the past 20 years. This increase in longhole stoping has been associated with increased dilution at narrow-vein mines. In terms of analysing the economic consequences of dilution, a percentage of tonnes or volume approach is well suited. However, when the primary aim is empirical analysis aimed at determining the causes of dilution, equivalent linear overbreak/slough (ELOS) is the preferred measure. This measure enables mines with different mining widths to be readily compared. This is a particularly important consideration when comparing narrow-vein stoping to large open stoping. The development and wide use of cavity monitoring surveys (CMS) has facilitated accurate measurement of ELOS.

2.1 INTRODUCTION

Brewis (1995) defines narrow mining to be the working of mineral deposits typically no more than two to three metres wide, with a dip exceeding 50 to 55 degrees (an angle at which broken ore can be expected to flow). Historically, conventional narrow-vein mining has been associated with high operating costs and low capital costs (Brewis 1995; Paraszczak 1992; Robertson, 1990).

Over the past 20 years there has been a general trend away from conventional small-scale narrow-vein mining methods towards mechanized mining methods. Longhole stoping is the dominant narrow-vein mining method in both Australia and Canada. While longhole stoping has lower mining costs per tonne and higher production rates than conventional mining methods, longhole stoping has been associated with increased dilution. Therefore, mining method selection is relevant to minimising narrow-vein dilution.
2.2 DILUTION ECONOMICS

Dilution is one of the most commonly quoted reasons for mine failure (Scoble and Moss, 1994; Miller et al. 1992). The level of dilution budgeted for a particular method of extraction is critical to the overall economics of a project (Pakalnis et al., 1995). Numerous authors have referred to the adverse effect of dilution on an operation’s economic performance (Manunen and Lahtela, 1984; Scoble and Moss, 1994; Pakalnis et al., 1995, Dunne and Pakalnis, 1996; Revey, 1998). Elbrond (1994) concludes from his economic modelling of the effect dilution has on the net present value of a hypothetical deposit, that dilution constitutes a severe constraint on a project by increasing costs as well as resulting in metal loss. Lane (1988) showed that operations with mill capacity limitations are most affected by dilution. Mining of waste dilution results in an opportunity cost, where waste is replaced by ore in the mill circuit. Lane (1988) showed that mill capacity limited operations experience opportunity costs due to grade bearing material being displaced by the waste or subgrade material. Due to small tonnages, it is relatively uncommon for small narrow-vein operations to be mill limited. Pakalnis et al. (1995) identify dilution as a source of additional cost associated with mucking, trucking, crushing, grinding and processing.

Reliable estimates of unplanned dilution are essential for prudent economic feasibility studies. However, predicting dilution is complex and requires considerable judgement. Site-specific factors are thought to contribute to dilution. In the early 1990’s it was demonstrated that dilution can be reduced at an operating mine by process improvement and quality control strategies. Since the mid 1990s there have been extensive attempts to improve dilution prediction through empirical study and modelling of the parameters believed to affect dilution. The parameters with significant potential to cause dilution in narrow-vein are discussed in Chapter 3 and Chapter 4.

2.2.1 Summary

Dilution greatly impacts on the economic viability of an operation and is a significant economic risk. Despite the importance of dilution prediction to the economic performance of an operation, dilution prediction is complex often requiring considerable engineering judgment with limited information.

There are no general rules about what is an acceptable level of dilution. What may be reasonable at one mine may be economically untenable at another. An acceptable level of dilution will depend upon economic factors such as waste rock grade, mucking, trucking and...
milling costs, as well as the cost benefit of decreasing dilution by decreasing sublevel spacings or using fill or pillars.

### 2.3 DEFINING DILUTION

Dilution can be defined as the contamination of ore by non-ore material during the mining process (Wright, 1983). Lappalainen and Pitkajarvi (1996) identified the following four types of dilution:

1. Uncertainty in the exact location of the ore boundary.
2. Technical dilution unavoidable with existing mining methods.
3. Waste material outside of the designed stope limit.
4. Possible internal waste (not included in the resource estimate).

Waste material outside of the stope design limit has been referred to as unplanned dilution (Pakalnis et al., 1995; Scoble and Moss, 1994; Villaescusa, 1998). Unplanned dilution has also been referred to as ‘outer dilution’ as opposed to ‘inner dilution’ which is inside the planned stope limits (Manunen and Lahtela, 1984). It is unplanned dilution that is addressed in this thesis. Figure 2.1 illustrates the concept of unplanned dilution. Planned dilution refers to material below the cut-off grade that lies within the designed stope boundaries. Unplanned dilution is derived from three possible sources; blast overbreak, sloughage or backfill (Scoble and Moss, 1994).

![Figure 2.1 – Planned and unplanned dilution](image-url)
Based on a survey of 22 mine operations, (Pakalnis, 1986) identified ten different definitions of dilution during his survey of open stoping in Canada. Definitions ranged from comparisons of diamond drillhole grade to draw point assay grade, through to definitions which focus on a linear measurement of overbreak.

According to Scoble and Moss (1994) the two most common methods for calculating dilution are based on tonnages according to Equation 2.1 and Equation 2.2:

\[
\text{Dilution} = \frac{\text{Waste Tonnes}}{\text{Ore Tonnes}} \quad \text{Equation 2.1}
\]

\[
\text{Dilution} = \frac{\text{Waste Tonnes}}{(\text{Ore} + \text{Waste})} \quad \text{Equation 2.2}
\]

However, Pakalnis et al. (1995) studied the sensitivity of these two definitions to mining width and demonstrated that Equation 2.2 is relatively insensitive to mining width. For this reason Pakalnis et al. (1995) recommend Equation 2.1 as a standard measure of dilution. Martin et al. (1999) suggests that unplanned dilution in a narrow-vein operation can be defined as per Equation 2.3. In the case of narrow-vein mining where almost all unplanned dilution is from the hangingwall and footwall and stope geometries are consistent along strike, Equation 2.2 and Equation 2.3 are approximately equivalent. Therefore, in the case of narrow-vein mining either Equation 2.2 or Equation 2.3 could be selected depending on the information available.

\[
\text{Dilution} = \frac{\text{Metres Off Footwall} + \text{Metres Off Hangingwall}}{\text{Planned Mining Width}} \quad \text{Equation 2.3}
\]

Lappalainen and Pitkajarvi (1996) define waste rock dilution (WRD) as a relative decrease in grade from in situ resource grade (\(G_{is}\)) to mill feed grade (\(G_{mf}\)) as per Equation 2.4. This method of dilution estimation was adopted at Lupin operations at a time when it was not possible to survey stope limits (Bullis et al., 1994).

\[
\text{WRD}\% = \frac{(G_{is} - G_{mf}) \times 100}{G_{is}} \quad \text{Equation 2.4}
\]
While dilution calculated as a percentage of planned tonnes or planned mining width is well suited to economic analysis and evaluation, it is not suited to objective empirical analysis of the parameters affecting dilution. For the purposes of empirical analysis of the factors contributing to dilution, Wang et al. (2002a) and Wang et al. (2002b) chose to use average metres of unplanned material that had sloughed or failed off the stope wall. The advantage of analysing average linear overbreak as the dependent variable in empirical analyses is that it is not dependent on mining width. Therefore, different mining widths can be analysed together without introducing bias due to mining width. Methods to determine average linear overbreak are discussed later in this chapter.

2.4 NARROW-VEIN MINING METHODS

In the Third edition of Peele’s Mining Engineer’s Handbook well over 50 pages of the section on underground mining are devoted to the working of narrow-vein mines (Brewis, 1995). Gemell (1989) provides a comprehensive summary of conventional narrow-vein mining methods employed in Australia. Recent developments in mechanised narrow-vein mining have increased the economic viability of narrow-vein mining (Brewis, 1995). Based upon a thorough review of narrow-vein mining, longhole stoping methods appears to be the most common type of narrow-vein mining method encountered in Australia. Therefore, for the purposes of this study, dilution minimisation strategies are directed towards mechanised stoping methods. However, conventional narrow-vein mining methods have been reviewed with a view to determining if conventional mining practices may offer dilution minimisation strategies that could be applied in mechanised narrow-vein mining.

2.4.1 Conventional Narrow-vein Mining Methods

Conventional narrow-vein mining methods refer to mining methods using conventional mining equipment such as handheld airleg drills, scrapers and air-powered mucking units. These methods are man-entry methods meaning that the miner enters the stope. The following mining methods are associated with conventional mining equipment and man-entry stoping:

- Shrinkage stoping as illustrated in Figure 2.2.
- Cut and fill stoping (also uses Jumbo drill rigs) as illustrated in Figure 2.3.
- Overhand open (stull) stoping as illustrated in Figure 2.4.
- Underhand open (stull) stoping as illustrated in Figure 2.5.
- Gallery stoping as illustrated in Figure 2.6.
Gemell (1989) provides a comprehensive description of each of these methods as well as summarising the advantages and disadvantages of each method. In terms of dilution minimisation shrinkage and cut and fill stoping minimise dilution by providing passive support to the stope hangingwall and footwall. When the rockmass can be described as competent the open stoping methods can be employed (Gemell, 1989).

Figure 2.2 – Shrinkage stoping, after Gemell (1989)

Figure 2.3 - Cut and fill stoping, after Brady and Brown (1993)
Figure 2.4 – Overhand open (stull) stoping, after Gemell (1989)

Figure 2.5 – Underhand open (stull) stoping, after Gemell (1989)

Figure 2.6 – Gallery stoping, after Gemell (1989)
From the perspective of dilution minimisation conventional mining methods are associated with well controlled dilution. The ability to support hangingwall and footwall using timber stulls, fill or broken ore means that effective spans can be reduced according to the ground conditions encountered. In addition, due to the relatively small size of mining equipment mining widths can be kept below one metre and this reduces the amount of dilution inherent in these methods. In contrast, minimum longhole stoping mining widths are rarely less than one metre and are often more than 1.5 metres.

Conventional mining methods have been associated with increased safety risks. In Canada, Lizotte (1991) found that the difficulty of hiring skilled miners for conventional narrow-vein mining and the inherent improved safety of longhole stoping, has made longhole narrow-vein stoping an attractive alternative to conventional methods.

Conventional mining methods are employed at some mines due to advantages associated with ore dip, dilution control and selectivity. Although statistics are not available, the trend away from conventional mining methods continues in Australia with relatively small numbers of mines employing conventional narrow-vein mining methods. Geological irregularity, high stresses, flat dipping orebodies and availability of cheap labour are some of the reasons why conventional mining methods have historically dominated narrow-vein mining in South Africa (Wills et al., 2001). However, conventional stoping in South Africa has been advanced with developments in drilling technology (Wills et al., 2001) and potential application of non-blasting rock breakage (Willis, 2001). In addition, competitive pressures and labour availability are forcing South African narrow-vein mines to adopt more mechanized mining methods (Wills et al. 2001).

Godin et al. (2001) found that in certain parts of the Mouska mine in Canada, shrinkage stoping produced better economic results than longhole stoping. When comparing the same tonnage at Mouska mine, Godin et al. (2001) determined that longhole stopes needed to be 1.85 wide or more to economically compete with shrinkage stoping. At the time Mouska mine was using air-powered drill rigs and mucking units, rather than the more productive hydraulic production drills and LHDs commonly used in longhole stoping. Between 1999 and 2001 CANMET undertook a survey of narrow-vein Canadian mines. Of the 17 mines surveyed, 11 mines were employing conventional narrow-vein mining techniques in some areas of the mine. Shrinkage stoping was particularly common, with six of the mines surveyed obtaining over half of their production from shrinkage stopes.
2.4.2 Alimak Stoping

Alimak narrow vein mining has been successfully employed at Placer Dome’s Dome mine in Canada (Robertson et al., 1990). Alimak stoping has also been successfully employed at the David Bell mine (Patton, 2001) and the Kiena mine (Emond, 2001) in Canada. Figure 2.7 illustrates the Alimak mining method in which stoping is conducted from a platform that is raised into the stope on a rail system (Robertson et al., 1990). In the mid 1990’s the Kundana Gold mine, Western Australia conducted the only known trial of Alimak stoping in Australia (Chadwick, 1995). Elsewhere in Australia, Alimaks have been used for developing rises, as opposed to stoping. Alimak stoping ceased at the mine following a fatal accident at the Kundana mine involving the Alimak stoping. In Australia, this experience has resulted in the belief that the method is associated with increased safety risks.

![Alimak stoping method, after Brewis (1995)](image)

2.4.3 Longhole Stoping

Lizotte (1991) defines longhole vein mining as longhole stoping applied to ore widths less than two metres, with drilling of parallel holes from sublevels, with no more than three holes per row and drill holes diameters not exceeding 80 mm. Longhole and blasthole stoping refer to the same general mining method. The term longhole stoping is more commonly used in Australia, while blasthole stoping is more commonly used in Canada. Brewis’s (1995) description of narrow-vein blasthole stoping includes a general description of narrow-vein stoping and includes mechanised stoping where load-haul-dump (LHD) bucket widths range from 1.6 metres to 2.5 metres wide. Allowing for clearance on either side, mechanised mining methods require between 2.5 metres to 3.5 metres minimum drive width.
Generally, the term bench stoping is used in eastern Australia to refer to longhole stopes where the full width of the orebody can be extracted in a single blast fired along the strike of the orebody. However, in terms of conventional stoping methods, benching traditionally refers to underhand benching of the stope floor as shown in Figure 2.5. Bench stoping usually refers to non-gold mines where the mining widths tend to be between two and six metres. Figure 2.8 through to Figure 2.11 illustrate four variations in longhole bench stoping (Villaescusa and Kaganathan, 1998). These variations are applicable to both narrow-vein and bench longhole stoping.

![Figure 2.8 – Continuous bench backfilling techniques, after Villaescusa and Kaganathan (1998)](image)

![Figure 2.9 – Non-recoverable permanent pillars and backfill, after Villaescusa and Kaganathan (1998)](image)
The move to longhole stoping of narrow-vein deposits has been associated with increased dilution. Decreased selectivity, increased drillhole diameters, drillhole deviation potential and increased minimum mining widths are some of the reasons cited for increased dilution from longhole stopes. LHD units are often wider than the vein. When ore is extracted via sill drives (as shown in Figure 2.8 to Figure 2.11), as opposed to crosscuts and drawpoints, the sill drive width needs to be sufficient for LHDs to operate. As a consequence, either stope width is designed to include significant dilution or stope walls are undercut by the sill drive. Lizotte (1991) notes in relation to longhole stopes that wall slough is the only source of dilution that
the ongoing mining operation can attempt to reduce. This appears to be a reference to the inherent selectivity issues associated with extracting narrow-vein deposits with the relatively large equipment associated with lower cost mechanised mining.

2.4.4 Resue Mining

Resue mining is an alternative to ordinary longhole open stoping where ore and waste are blasted in two separate blasts. Figure 2.12 illustrates the geometry of the resue mining trial at the Dome mine in Ontario (Robertson, 1986). Robertson (1986) reported that this trial was largely unsuccessful due to premature sloughage of the footwall resulting in production delays and ore dilution. The footwall sloughage is likely to have been attributable to temporary footwall overhang. No other documented cases of resue mining of mechanised narrow-vein mines were found in the literature. Gemell (1989) found that resue mining is distinctly unpopular in Australia, primarily because it is difficult to mechanise.

![Figure 2.12 – Resue mining at Dome mine, after Gemell (1989)](image_url)
2.4.5 Summary

Longhole stoping is the dominant narrow-vein mining method in Australia. Longhole stoping facilitates mechanisation of narrow-vein mining. While longhole mining costs are generally lower than conventional stoping methods, some operations have found a combination of mechanised and conventional methods can be beneficial even in the same stope.

Practical minimum widths required for LHD mucking and the associated potential for undercutting of either the hangingwall or footwall are technical limitations of mechanised narrow-vein longhole stoping methods. Both rib pillars and backfill are used to minimise dilution associated with longhole stoping. The effect of larger diameter drillholes and drillhole deviation may also contribute to increased dilution from longhole stopes. The potential effects of drill and blast parameters on dilution are discussed in Chapter 4.

While both resue mining and Alimak stoping eliminate the need for undercutting of stope walls, these methods have not proved popular. There appears to be significant scope to either develop new mining methods or equipment to minimise dilution.

2.5 QUANTIFYING DILUTION

Due to their narrow width narrow-vein stopes are particularly prone to high levels of dilution. For example, one metre of dilution in a 1 metre wide orebody represents 100 percent dilution, while 1 metre of dilution in a 20m wide orebody only represents 5 percent dilution. It is clear from this comparison that the method used to quantify dilution levels is very important.

2.5.1 Categorical Approach

Prior to the development of cavity monitoring survey (CMS) technology (Miller et al., 1992), unplanned dilution was quantified by reconciling planned stope tonnes to actual tonnes mucked, or in some cases planned grade to actual grade. One of the limitations of this approach is it assumes that there is no ore loss. Furthermore, it was uncommon to use these estimates of unplanned dilution in geotechnical studies of the factors affecting stope stability. Instead, geotechnical studies of stope stability and dilution were based on stope stability ratings or categories. The formulation of empirical stability charts is detailed in the following chapter. The stability zones or categories defined in the various charts are relevant to dilution measurement or quantification. The following stability zones or categories have been used to quantify stability:
Chapter 2 – Narrow-vein Dilution

− Potentially stable, potential unstable, potentially caving (Mathews et al., 1981).
− Stable, transitional, caved (Potvin, 1988).
− Potentially stable, potentially unstable, potential major collapse and potential caving (Stewart and Forsyth, 1995).
− Stable, failure, caving (Trueman et al., 2000, Mawdesley et al., 2001; Mawdesley, 2002).

Due to the subjectivity of categorical stability variables, there has been different interpretations and usage of these categories. Stewart and Forsyth (1995) highlight use of the term caved as an example of a term defined differently by several authors. Mathews et al. (1981) and Stewart and Forsyth (1995) define ‘caved’ as an excavation that will not stabilise until the void is full, while Potvin (1988) and Nickson (1992) define caved as well beyond the designed excavation limits. Using logistical analysis Mawdesley (2002) determined that the difference between failed and major failure case studies was not sufficient to justify separate stability zones. For this reason the Extended Mathews Stability Chart does not separate failures and major failures into different zones. However, Mawdesley (2002) determined that that the differences between stable and failure were significant and likewise the difference between failure and continuous caving case studies warranted a separate stability zone.

2.5.2 Equivalent Linear Overbreak (ELOS)

Miller et al. (1992) detail the development and evolution of the first cavity monitoring survey (CMS) system. The system is comprised of a commercial laser, a set of lightweight rods, an automatic scanning device, a hand held data logger and Autocad-based software (Miller et al., 1992). The Noranda Technology Centre conducted the first trials of CMS at the Gaspe mine, Canada. The three dimensional survey was detailed enough to identify a skin of ore, blasthole craters and broken ore at the bottom of the stope. Miller et al. (1992) present a further four case studies in this paper. In addition, Pakalnis et al. (1995), Germain et al. (1996), Milne et al. (1996a), Milne et al. (1996b), Germain and Hadjigeorgiou (1997), Henning and Mitri (1999), Dunne and Pakalnis (1996) and Suorineni et al. (2001) present Canadian examples of different mines where cavity monitoring surveys have been successfully used to evaluate stope overbreak. CMS equipment has been widely adopted in Australia, especially in the past five years. Most open stoping operations have their own CMS equipment (Buchanan, 2001).

Jarasz and Shepherd (2002) compared a CMS survey of a room with a classical survey of the same room. By comparing the classical survey with the CMS survey they were able to determine the instrumental error associated with survey. They found that the CMS equipment
used had an azimuth error of $0.506^\circ$, a inclination error of $-1.335^\circ$ and a rotation error of $-0.724^\circ$. Depending upon the distance involved, these instrument errors would have a significant effect of the precision of CMS survey results. For this reason Jarasz and Shepherd (2002) proposed that by determining an instrument’s error, it is possible to calibrate the instrument and greatly improve CMS precision.

The wide use of CMS surveys facilitates, with a certain level of precision, the difference between design surface and final excavation surface (Germain and Hadjigeorgiou, 1997). This enables dilution to be calculated based on Equation 2.1 and Equation 2.2. Clark and Pakalnis (1997) propose the use of equivalent linear overbreak/slough (ELOS) to estimate dilution as shown in Figure 2.13. ELOS is calculated according to Equation 2.5 (Clark and Pakalnis, 1997). Alternatively, ELOS could be calculated by dividing the difference between the designed stope volume and the final extracted stope volume by the stope surface area (excluding the floor surface area). ELOS is useful measure of dilution because it is independent of stope width. When dilution is quoted as a percentage of the designed stope width or design tonnes, it is less independent because it depends upon on the stope width and geometry.

\[ ELOS = \frac{VolumeOfSloughOrOverbreak}{(StopeHeight \times WallStrikeLength)} \]  

Equation 2.5

![Diagram of Equivalent Linear Overbreak/Slough (ELOS)](image)
Stewart and Forsyth (1995) define stable stopes as those with dilution less than 10 percent, unstable stopes are associated with dilution between 10 and 30 percent while major collapse/failure are associated with more than 30 percent dilution. Implicit in this approach to stability categories is that stability categories are based on percentage dilution and associated economic consequence. This contrasts with the ELOS approach where stability at one site can be directly compared to stability at another site without subjectivity associated with mining scale. Therefore, for the purpose of empirical modelling of the factors affecting dilution the ELOS approach is more objective. This is particularly important in the case of narrow-vein mining where percentage dilution is very sensitive to mining width.

Clark and Pakalnis (1997) used ELOS to define design zones in their ELOS design chart. The design zones indicate that ELOS less than 0.5 metres is considered blast damage only, ELOS between 0.5 and 1.0 metres is minor sloughing, 1.0 to 2.0 metres is moderate sloughing and greater than 2.0 metres is severe sloughing or possible wall collapse. Suorineni et al. (2001) quantitatively defined the stability zones associated with the Modified Stability Chart (Potvin, 1988). According to this definition, a stope is stable if the ELOS is less than or equal to 0.5 metres, unstable when ELOS is between 0.5 metres and 5 metres and caved when ELOS is greater than 5 metres.

2.5.3 Summary

ELOS is a measure of dilution independent of mining width. Because narrow-vein dilution is particularly sensitive to mining width the ELOS approach to quantifying dilution is well suited to narrow-vein stope stability analysis. Percentage based dilution is dependent on stope width. Therefore, it is not as amenable to geotechnical empirical analysis as ELOS, especially in the case of narrow-vein. There has been an increasing move away from dilution expressed as a percentage of stope width and towards ELOS (Clark and Pakalnis, 1997; Suorineni et al., 2001). The advantage of the ELOS based stability categories as defined by Clark and Pakalnis (1997) is that they are more objective than the subjective assessment required to discriminate between stability zones.

2.6 CONCLUSIONS

Stope instability and associated dilution have an adverse economic impact upon a mining operation. Unplanned dilution is the main source of dilution considered within the scope of this thesis. Other sources of dilution such as planned dilution associated with mining method
selection, geological control and backfill contamination are largely beyond the scope of this thesis.

There are many methods employed to calculate dilution. Equivalent linear overbreak is an index of dilution that is independent of mining width, and is therefore well suited to comparing case studies with varying mining widths. Therefore, equivalent linear overbreak or slough (ELOS) is the most objective index of dilution levels and will be adopted whenever possible.

Mining method selection involves a trade off between the unit cost and production rate benefits of mechanised mining against the costs of dilution associated with mechanised mining. Longhole stoping has emerged as the dominant narrow-vein mining method over the past 20 years. Longhole stoping mines use some combination of backfill, cablebolts and or pillars to control effective span in attempt to minimise dilution. Undercutting and blast related overbreak appear to be two causes of dilution associated with the longhole stoping. Dilution parameters are discussed in Chapter 4.

Dilution minimisation strategies can broadly be divided into three main approaches:

1. Stope design evaluation.
2. Quality control.

Stope designs should be evaluated in terms of dilution potential, as well as, production rate and mining costs. This requires adequate methods for dilution prediction. This thesis aims to develop improved methodologies for narrow-vein dilution prediction. Quality control and process improvement are dilution control measures that can be implemented as part of feedback loop between production and engineering personnel. Drillhole survey is an example of quality control, while explosive and drill pattern trials would be considered process improvement methods of dilution control. This thesis addresses quality control and process improvement issues when relevant. However, the primary aim is to focus on dilution prediction methodology as a means of minimising dilution through stope design evaluation. The next chapter considers the potential of existing stope design methods to minimise narrow-vein dilution as part of the stope design evaluation process.
Broadly speaking there are four main approaches to the design of stable stopes; mining methods selection, empirical stope design, analytical methods and numerical modelling. Mining method selection has been discussed in the previous chapter. Acknowledging that for various reasons conventional narrow-vein mining methods are generally on the demise, this chapter reviews and evaluates the applicability of empirical, analytical and numerical stope design methodologies with respect to narrow-vein longhole stoping. The ability of the empirical stability graph approach to capture both stress and kinematic stability mechanisms is a significant advantage of this approach. The stability graph approach has demonstrated general applicability to large-open stoping and can be used at the feasibility stage to predict stable stope dimensions. Analytical and numerical methods have the potential to compliment the stability graph approach in cases where mechanisms causing stope instability have been clearly identified and input parameters can be calibrated.

3.1 INTRODUCTION

There are two main approaches to dilution minimisation in the literature. The first approach focuses on stope design with and without cablebolt support. This approach considers geotechnical factors affecting stope stability and is the main approach to determining stable stope dimensions. The second approach focuses upon drilling and blasting strategies and technology, and is evaluated with respect to narrow-vein dilution in the next chapter. In this chapter the various stope design methods have been reviewed and evaluated with respect to applicability to narrow-vein stoping.

The design of stable stopes is a complex process taking into account mining method selection, stable dimensions, level spacing, support requirements and fill requirements. The aim of this thesis is to develop improved methods for predicting narrow-vein dilution so that optimal mining method, stope size, geometry and even mining equipment can be selected. When comparing stope design options it is important to be able to evaluate options not just in terms
of mining costs and production rates, but also with respect to stope stability and dilution potential. Poor stope stability is associated with dilution costs as well as risks to remote equipment, production rate and secondary breakage requirements.

Stope design in the context of geotechnical stability encompasses geometry, stable spans, support and fill requirements, pillar size and location, as well as extraction sequence. There are three main approaches to determining stable stope design.

1. Empirical stope design.
2. Analytical methods.

Empirical stope design is based on the principle that stoping experience at other mines provides valuable information upon which to base future design. Empirical design methods incorporate analytical and numerical methods in the formulation of their parameters. Numerous authors advocate calibrating these empirical techniques to site conditions or developing site-specific stability charts. While both numerical and analytical techniques have been developed for stope design, they have not been used as widely as empirical stope design methods.

3.2 EMPIRICAL STOPE DESIGN

The engineering properties of rock are difficult to quantify. In mechanical and construction engineering the engineering properties of materials used in designs are well defined and therefore, evaluation of the loading and strength characteristics of a particular design lends itself to analytical and numerical design methods. However, in the case of stope design the engineering properties of the rock are commonly affected by discontinuities (eg. faults, jointing and shear zones), isotropy (eg. bedding planes) and inhomogeneous properties (eg stiffness of intrusions relative to host rock). For this reason the most widely adopted stope design methods are based on empirical rock mass classification systems. The rock mass classification systems employed in the most common empirical stope design methods include:

- NGI Q System Classification (Q) (Barton et al. 1974).
- Rock Mass Rating classification (RMR) (Bienawski, 1974).
3.2.1 Mathews Stability Graph

Mathews et al. (1981) developed the first empirical stope stability graph – Figure 3.1. The Mathews Stability Graph method has gained wide acceptance and is used worldwide as a design tool (Potvin and Hadjigeorgiou, 2001). The method has been used in Australia, Canada, Africa, Europe and the United States (Potvin and Hadjigeorgiou, 2001). The scope of the original CANMET study was to determine the information required to predict stable spans for open stopes at mining depths below 1000 metres. The original stability chart (Mathews et al., 1981) was developed for open stope case histories at depths exceeding 1000 metres. However, the majority of case studies actually come from mines less than 1000 metres depth (Trueman et al., 2000). The original database was comprised of 50 case histories (Mathews et al., 1981). However, thousands of stability chart case studies would have been produced over the past 20 years.

![Mathews Stability Graph](image)

Figure 3.1 – Mathews Stability Graph, after Mathews et al. (1981)

Mathews et al. (1981) reviewed the applicability of rock mass classification systems to open stope design. In addition, an empirical relationship between rock mass properties, joint orientation, induced stress and stope surface dimensions was determined. Mathews et al. (1981) adopted hydraulic radius as measure of stope surface geometry. Hydraulic radius is defined as the ratio of the surface area to the surface perimeter (Laubscher and Taylor, 1976). Mathews et al. (1981) notes that as the aspect ratio increased to four, the hydraulic radius remains relatively constant and reflects one-way spanning situations.
Chapter 3 – Stope Design

Under Mathews et al. (1981) stope stability graph, selected geotechnical factors are combined to produce an index called the stability number (N). The stability number is calculated according to Equation 3.1, where A, B and C are factors taking into account induced stress acting parallel to the middle of the stope surface, joint orientation and gravity, respectively. Q’ is the NGI Q classification index value (Barton et al. 1974), with the SRF and joint water reduction factors set to 1.

\[ N = Q' \times A \times B \times C \]  

Equation 3.1

Stress factor A replaces SRF in the Q system and is based upon the ratio of the uniaxial compressive strength of intact rock to the induced compressive stress parallel to the surface under consideration (Mathews et al. 1981). Stress factor A is calculated from the ratio of the unconfined compressive strength of the intact rock (\( \sigma_c \)), to the induced compressive stress (\( \sigma_I \)), parallel to the stope face. Mathews et al. (1981) suggest that induced stress can best be determined using numerical analysis techniques. The rock stress factor is linearly related to \( \sigma_c/\sigma_I \) and ranges from 0.1 to 1.0. The value of the rock stress factor can be determined from the graph in Figure 3.2.

\[
\sigma_c \quad \text{Uniaxial compressive strength of intact rock} \\
\sigma_I \quad \text{Induced compressive stress}
\]

\( \sigma_c/\sigma_I \)

Figure 3.2 – Formulation of stress factor A, after Mathews et al. (1981)

Factor B considers the orientation of the most critical structure relative to the stope surface. The critical structure could be a joint set, bedding plane or foliation (Potvin and Hadjiigeorgiou, 2001). Figure 3.3 provides a guide for determining factor B.
The gravity adjustment factor $C$ is based upon the underlying assumption that the effect on stability of gravity on a horizontal surface is eight times that of a vertical surface. The gravity adjustment factor considers the effect of gravity on the stability of the stope surfaces from falling, slabbing and sliding (Mathews et al., 1981). The relationship between the gravity adjustment factor $C$ and the dip of the surface is defined as:

$$\text{Factor } C = 8 - 7\cos(\text{Angle of Dip})$$  \hspace{1cm} \text{Equation 3.2}$$

Mathews et al. (1981) devised the following three stability categories:

- Stable; the excavation will stand unsupported with occasional localised ground support.
- Unstable; the excavation will experience some localised caving but will tend to form a stable arch.
- Caving; the excavation will cave and will not stabilise until the void is full.
3.2.2 Modifications and Adjustments to the Mathews Stability Graph Method

Modified Stability Chart

The Modified Stability Chart is a modification of the original Mathews Stability Graph and is based upon 176 new case histories collected between 1986 and 1987 (Potvin, 1988). Figure 3.4 is the Modified Stability Chart (Potvin, 1988). The larger database improved confidence in the boundaries and broadened the applicability of the chart. Nevertheless, as noted by Trueman and Mawdesley (2003) about 100 of these case studies had acknowledged uncertainties with respect to parameters necessary to determine the stability number. (Potvin, 1988) proposed the use of slightly different A, B and C factors as well as new categories of stability. Figure 3.5 is the chart used to determine the modified joint orientation factor B as proposed by Potvin (1988).
Based upon an additional 13 unsupported case histories and using Mahalanobis distance statistical analysis (Seber, 1984), Nickson (1992) found good agreement between the modified stability chart transition zone proposed by Potvin (1988) and his statistically derived relationship between HR and $N'$. Equation 3.3 is the unsupported stable-caved (unstable) boundary derived by using Mahalanobis distance statistical analysis (Nickson, 1992).

$$HR = 10^{(0.573 + 0.338 \log(N'))}$$  \textit{Equation 3.3}

Stewart and Forsyth (1995) recommend that the original factors and stability number, N be adhered to, as opposed to the modified stability number $N'$. Trueman et al. (2000) suggest that the new factors imply a level of engineering rigour that is not necessarily justifiable. Furthermore, based upon a large data base from the Mt Charlotte mine, they found no evidence that Potvin (1988) modifications improved the predictive ability of method (Trueman et al., 2000). For this reason, Trueman et al. (2000) recommend that the easier to calculate original stability number N as described by Stewart and Forsyth (1995) be used in preference to $N'$. In their update of the stability graph method, Hadjigeorgiou et al. (1995) adhere to the modified
stability number and agree with Potvin's (1988) recommendation that when the ratio of the uniaxial compressive strength (UCS) to the induced stress is lower than two, factor A should be 0.1, as opposed to the zero value suggested by original Mathews factor A chart shown in Figure 3.2.

The Modified Stability Chart and modified stability number (N’) have been included in a number of textbooks (Hoek et al., 1995; Hutchinson and Diederichs, 1996) and for this reason N’ has gained broad acceptance as the industry and research standard (Potvin and Hadjigeorgiou, 2001). Following review of the input methodology, Hadjigeorgiou et al. (1995) propose an additional modification to determination of the gravity adjustment factor C. Hadjigeorgiou et al. (1995) found that the nomogram for sliding failure proposed by Potvin (1988), was inappropriate for the specific case when the critical joint is parallel to the footwall and when the dip of the critical joint is greater than the dip of the footwall. Based upon the results of kinematic tests Hadjigeorgiou et al. (1995) proposed a second curve suited to the aforementioned conditions.

**Fault Factor**

Suorineni et al. (2001a) propose a detailed procedure to evaluate a fault factor to take into account the following parameters; included angle between fault and stope surface, stress ratio (K), stope aspect ratio, fault shear strength, distance of fault from stope and stope dip. The fault factor was developed based on the premise that faults affect tensile stress distributions near stope wall thereby increasing ELOS. Using two-dimensional linear elastic modelling Suorineni et al. (1999) undertook a comprehensive parametric study of the aforementioned parameters’ effect on the extent of tensile stress development (Suorineni et al., 1999). Based upon the Coulomb criterion for slip along a fault and limit equilibrium analysis, Suorineni (1998) determined that the critical range for clamping stresses is about 0.01 MPa to 0.2 MPa for wedges from 0.5 metres to 3 metres. Based upon this criterion for failure, the ELOS attributed to the fault, \( ELOS_f \) was calculated as the increase in area inside the 0.1 MPa contour associated with introducing the fault to the two-dimensional model, divided by the stope height.

By relating \( ELOS_f \) to the ELOS contours plotted by Clark and Pakalnis (1997) a set of seven generic fault factor curves were produced (Suorineni et al., 1999). Suorineni et al. (1999) recommend that when stope conditions are significantly different from those indicated in
Figure 3.6 individual mines should develop their own fault factors according to the methodology proposed in Suorineni et al. (2001b).

To validate this methodology, Suorineni (1998) used faults factor curves to improve the separability of 112 case studies from the Kidd mine in Canada, and slightly improve
differentiation between 130 case studies collected from the Ashanti mine in Ghana (Suorineni, 1998). In terms of reviewing the general applicability of the fault factor approach it is important to consider the underlying assumptions.

Firstly, Suorineni (1998) states that the methodology is applicable to excavations in discontinuous (jointed or blocky) rock masses. It can be assumed that this is because it is under these conditions that low or tensile stress may have the potential to affect stability. However, the specific conditions in which low stress affects stability is in itself the subject of uncertainty. Furthermore, two-dimensional plane strain modelling overestimates the potential for the development of low or tensile stress when the stope length is less than four times the stope height (Pakalnis, 1991). In the case when the stope length equals the stope height the zone of relaxation as predicted by two-dimensional plane strain is as much as four times as large as the three-dimensional case (Pakalnis, 1991). Provided these limitations are considered, the fault factor methodology proposed by Suorineni et al. (2001a) could be a good index for the effect of faulting under rock mass conditions when low or tensile stress has potential to affect stability.

While the applicability of the fault factor may be limited to said conditions, the method used by Suorineni (1998) to determine the included angle between the stope surface and the fault (\(\xi\)) could be considered an excellent alternative method for assessing factor B using the original factor B chart shown in Figure 3.3. \(\xi\) takes into account the dip and strike of both critical joint and stope surface. Figure 3.7 illustrates the normal vector to the fault, stope wall or critical joint. Direction cosines are defined in Equation 3.4 to Equation 3.6 and are based on a right hand coordinate system.

\[
\cos \alpha = \frac{a}{\sqrt{a^2 + b^2 + c^2}} = a'
\]

\[
\cos \beta = \frac{b}{\sqrt{a^2 + b^2 + c^2}} = b'
\]

\[
\cos \gamma = \frac{c}{\sqrt{a^2 + b^2 + c^2}} = c'
\]
The direction cosines \((a', b' \text{ and } c')\) define the plane in question, whether it be the fault, \(a'f, b'f\) and \(c'f\) or stope wall, \(a's, b's\) or \(c's\). The direction cosines can be calculated from the dip \((\theta)\) and dip direction \((\omega)\) of the plane as per Equations 3.7 to 3.9. The direction cosine for the fault (or critical joint) and the stope wall are then used to calculate the angle between the two planes \((\xi)\) according to Equation 3.10.

\[
\sin \theta \sin \omega = a' \quad \text{Equation 3.7}
\]

\[
\cos \omega \sin \theta = b' \quad \text{Equation 3.8}
\]

\[
\cos \theta = c' \quad \text{Equation 3.9}
\]

\[
\xi = \cos^{-1}(a', a'_s + b', b'_f + c', c'_f)
\]

Deiderichs and Hutchison (1996) also suggest that the included angle be used to calculate Factor B.

**Stress Damage Factor**

The stress damage factor proposed by Sprott et al. (1999) and adjustment to factor A proposed by Kaiser et al. (1997) and Diederichs et al. (1999) for cases of stress relaxation are discussed in the following chapter as they relate directly to parameters believed to affect narrow-vein stability. However, Potvin and Hadjigeorgiou (2001) recommends that stope sequencing be used to minimise the risk of both stress damage and stress relaxation.
**Cablebolt Support**

Potvin (1988) was the first to empirically incorporate cablebolt support recommendations into stope design charts. The modified stability chart (Potvin, 1988) established a zone, between the stable zone and the caving zone that would be stable if supported with cablebolts. Assuming a stope surface can be stabilised with cablebolt support, Potvin (1988) developed the first empirical cablebolt support recommendations. It should be noted that these recommendations were developed for stope backs. The cablebolt support recommendations are based upon the ratio of RQD to the product of the joint set number, J_n and hydraulic radius, HR. Misra (1998) found that while the modified stability chart is frequently consulted to identify whether a stope surface could be stabilised using cablebolts, the cablebolt support density recommendations are rarely used to prepare the final design. Furthermore, Misra (1998) suggests that this chart is only intended for evenly distributed stope back support, and not for the design of hangingwall support of point anchor back support.

Nickson (1992) extended this work by adding 46 case histories including hangingwall data and conducted a statistical analysis to determine if modifications should be made to the modified stability chart supportable zone. Mahalanobis distance analysis was used to determine Equation 3.11. Equation 3.11 defines the relationship between hydraulic radius and modified stability number, N’, that best delineates the supportable zone. Figure 3.8 shows the original supportable zone and the modified supportable zone as determined in Equation 3.11. Using the combined databases of Potvin (1988) and Nickson (1992) cablebolt support density recommendations were also revised. Instead of predicting support density based upon the ratio of the joint set number (J_n) to the hydraulic radius (HR), Nickson (1992) statistically determined that the ratio of the modified stability number N’ to hydraulic radius, HR, was a better predictor of cablebolt support density requirements.

\[ HR = 10^{(0.872 + 0.17 \log N')} \]

Equation 3.11

Diederichs et al. (1999) have identified the following weaknesses in the approach to cablebolt limits proposed by Potvin (1988) and Nickson (1992):

- The proposed cablebolting limits represent the average performance of all the cablebolted stopes, and
- Do not take into account cablebolt density, length, orientation or quality of installation.
Diederichs and Kaiser (1996) use Potvin's and Nickson's databases to develop a different approach to cablebolt support recommendations. Diederichs and Kaiser qualify their recommendations by stating that they are only applicable to a rock mass that is well represented by the pseudo-continuous rock mass that may contain persistent and ubiquitous joints, but may not contain discrete faults, shears or localised delamination planes.


\[ E_{rm} = 25\log_{10}Q \]  

Equation 3.12

Support density recommendations are based upon a 20 tonne per strand yield capacity and the minimum self-supporting beam determined from Voussoir beam analysis (Diederichs et al., 1999). Cablebolt lengths are taken as twice the beam thickness plus two to three metres for anchorage (Diederichs et al., 1999).
There are two main approaches to cablebolt support recommendations. The approach of Potvin (1988) and Nickson (1992) is largely empirical, while the approach of Diederichs and Kaiser (1996) is primarily based upon analytical theory. A serious limitation of the analytical theory based recommendation is that it assumes a pseudo-continuous rock mass and may not be suited to large-scale structures commonly exposed by stoping. The voussoir beam analog assumes that the rock mass can be reinforced to create a self-supporting beam or plate. Therefore, the approach is somewhat limited in general applicability.

The empirical support density recommendations of Potvin (1988) and Nickson (1992) have been determined based upon a relatively small amount of data considering they are attempting to distinguish multiple levels of support. Nickson (1992) combined database contained only 59 case histories. It is also important to note that these empirical guidelines are based on plane strand cables and with modified strand cables and increases in grout thickness, cablebolt effectiveness is likely to be higher than indicated by the historical data collated by Potvin(1988) and Nickson (1992).

Cablebolts have been used to improve narrow stope stability (Kaiser et al., 2001; de Vries et al., 2003). However, both these mines had stope widths between 4 and 7 metres and therefore, would not be considered narrow-vein. There did not seem to be any cases of narrow-vein mines (less than two to three metres) using cablebolts for stope wall support. However, some narrow-vein mines use cablebolts for sill drive stability and these cables may have improved stope stability. The principle behind stope wall support is that by preventing unravelling and maintaining stope wall integrity the rock is reinforced and therefore, better able to be form a self-supporting beam (Diederichs et al., 1999). Alternatively, if a self supporting beam can not be formed, then the cablebolts need to be capable of holding the weight of the rock (Diederichs et al., 1999). The point anchor approach illustrated in Figure 3.9 is the pattern most widely adopted in narrow stoping. The point anchor cablebolt method is a reinforcement approach. In the case of tabular narrow-vein deposits it is uncommon to have development in the hangingwall, and therefore the hangingwall patterns shown in Figure 3.9 are usually uneconomic for most narrow-vein deposits.

The author installed hangingwall point anchor cablebolts at the Renison Tin mine (3-8 metre wide stopes) and designed cablebolts at the Mariners Nickel mine (2-4 metres wide stopes). Cablebolts were associated with decreased dilution at the Renison Nickel mine. Mariners cablebolt patterns were designed using the methodology proposed by Diederichs and Kaiser.
Cablebolting of stope walls in the upper areas of the Mariners Nickel mine proved effective in controlling dilution. However, in the deeper number 7 orebody at the Mariners mine the hangingwall was a low strength highly deformable ultramafic. The deformability of the rockmass appeared to affect cablebolt effectiveness by reducing cablebolt bond strength as observed by cablebolts failing at the bond between the rock and the grout (cables hanging from stope walls). In addition, mining induced stress changes may have affected radial stiffness. Reichert et al. (1992) provides a detailed discussion of the effect of radial stiffness on cablebolt effectiveness. Because narrow-vein stoping is usually associated with decreases in stress and cablebolts are installed prior to stoping, the propensity of cablebolts to decreases in capacity due to reduced radial stiffness warrants particular consideration in the case of narrow-vein stoping. Reichert et al. (1992) discusses the relationship between rockmass modulus and borehole stiffness, as well as, noting that mine induced stress reductions will decrease radial stiffness thereby reducing cablebolt bond strength. The problems associated with decreases in stress and stress relaxation can largely be avoided by using bulbed cables and plating cables (Deiderichs and Hutchison, 1996).

3.2.3 Shape Factor

Both the original and modified versions of the stability graph approach determine shape factor (S) using hydraulic radius. While hydraulic radius has proved a useful parameter to take into account size and to some extent shape of an excavation, it does have important limitations when applied to irregular geometry (Milne et al. 1996a; Milne et al. 1996b; Potvin, 2001). Milne et al. (1996a) suggests that radius factor provides a more accurate assessment of the
distance to abutments and is therefore better suited to irregular shape geometry. Radius factor is half the harmonic radius which is calculated using Equation 3.13, where $r_0$ is the distance to the abutments measured from the surface centre.

$$RF = \frac{Rh}{2} = \frac{0.5}{\sum_{\theta=1}^{n} \frac{1}{r_\theta}}$$  \hspace{1cm} \text{Equation 3.13}

Milne et al.'s (1996a) comparison of hydraulic radius to radius factor for a 100 metre wide rectangle with increasing length is shown in Figure 3.10. It is clear from Figure 3.10 that radius factor is less sensitive to length than hydraulic radius once the length exceeds approximately four times the width. Based on Hoek et al. (1995) observations that support from a tunnel face is insignificant at 1.5 times the tunnel width, Milne et al. (1996a) suggests that radius factor assesses the effect of two way spanning more realistically than HR. Milne et al. (1996a) presents three case studies where back-analyses of the stability predictions using radius factor improved the reliability of stability assessments compared to the original incorrect stability assessment based on hydraulic radius.
In-stope or rib pillars are commonly used in narrow-vein mines either to improve stability or to avoid waste. Furthermore, in sublevel retreat narrow-vein stoping irregular geometries are often associated with the uneven retreat profile and rib pillars. Radius factor would be a useful alternative to hydraulic radius in these circumstances. The applicability of radius factor to account for irregular narrow-vein geometry is discussed further in Chapter 4.

3.2.4 Laubscher Design Graph

The Laubscher Design Graph is based on the Mining Rock Mass Rating Classification (MRMR) (Laubscher, 1977) and hydraulic radius - Figure 3.11. MRMR is based on RQD, joint spacing, intact rock strength, joint spacing, condition of joint and ground water. The MRMR value is then adjusted for weathering, field and induced stresses, changes in stress, joint orientation and blasting (Laubscher, 1977).

![Caving Stability Graph](image)

Figure 3.11 - Laubscher’s caving chart with 29 case histories from the 1994 and 1998 caving charts, after Laubscher (1994) and Bartlett (1998a).
The primary limitation of Laubscher Design Graph is the engineering judgement and practical experience necessary to decide upon the adjustments. For this reason application of Laubscher’s design graph to stope design has been primarily limited to use amongst geotechnical and mining engineers who have practical understanding of how to apply the adjustments factors. However, the Laubscher Design Graph has been used extensively worldwide in the fields of cavability prediction (Mawdesley, 2002).

### 3.2.5 Extended Mathews Stability Graph

Trueman et al. (2000), Mawdesley et al. (2001) and Mawdesley (2002) extended the original Mathews Stability Graph by increasing the number of case histories from 176 to 485. The 100 case studies appearing in Potvin’s (1988) modified stability graph that had acknowledged uncertainties were removed from the extended data base and significantly larger stopes were included. The Extended Mathews Stability chart only has two boundaries and is based on the original formulation of the N and HR. This is because Mawdesley (2002) determined using logistical analysis that failures were not significantly different from major failures and therefore, a failure major failure boundary could not be justified. However, Trueman (2000) and Mawdesley et al. (2001) have also adopted the modifications to stress factor A suggested by Potvin (1988) where A is assumed to equal 0.1 when the ratio of uniaxial compressive strength (σc) to the induced stress (σi) is less than two. In addition, Mawdesley et al. (2001) and Mawdesley (2002) used logistic regression to statistically delineate the boundary between stable and failed stope surfaces. Figure 3.12 is the Extended Mathews Stability Graph (Trueman and Mawdesley, 2003).

Trueman et al. (2000) and Mawdesley (2002) increased the range of data used to determine the stability graph boundaries and conducted rigorous delineation and testing of the significance of the Extended Mathews Stability chart boundaries. A considerable advantage of the Extended Mathews Stability chart over other stability charts is that it covers the largest range of conditions and stope sizes.

Existing empirical stope stability models do not take into account a number of the parameters believed to affect stope stability (Clark and Pakalnis, 1997). For example the stability chart method does not explicitly take into account any of the parameters shown in Figure 1.1. Despite this, stability charts are approximately 80 percent accurate in terms of predicting instability. This implies that for 80 percent of case studies in the Extended Mathews database
these parameters do not significantly affect stability. The aim of this thesis is to determine whether this is true in the case of narrow-vein stoping.

Figure 3.12 – The Extended Mathews Stability Graph, after Trueman and Mawdesley (2003)

3.2.6 ELOS Stability Chart

The ability of cavity monitoring survey (CMS) equipment to produce a three dimensional surveys of a stope enabled Clark and Pakalnis (1997) and Clark (1998) to develop a new stability chart based upon ELOS. ELOS dilution chart design zones are based solely on the ELOS values and were determined using logistic regression as well as engineering judgement (Clark and Pakalnis, 1997; Clark, 1998). The ELOS stability chart is shown in Figure 3.13.

Clark and Pakalnis (1997) acknowledge that the ELOS stability chart, with only 85 case histories, requires more data to give confidence to the design zones. At this stage the database is limited to hangingwalls and footwalls in a low or relaxed stress state with parallel structure being critical to stability (Clark and Pakalnis, 1997; Clark, 1998). In addition, Clark and Pakalnis (1997) also list as a limitation the bias in the existing database towards stopes with relatively small blasthole diameters less than 65 millimetres. Clark and Pakalnis (1997) recognised that although they were able to demonstrate statistically that both undercutting of
stope wall and stope life influences stope stability, the amount of data prevented them from quantifying these effects.

![Empirical Dilution Design Graph](image)

**Figure 3.13 - Empirical dilution design graph after (Clark 1998)**

### 3.2.7 Empirical Mathematical Dilution Model

Based upon 133 case histories from the Ruttan mine, Canada, Pakalnis (1986) developed an empirical relationship to predict stability in terms of dilution. Three equations were developed; one for isolated stopes, one for echelon stopes and one for rib stopes. Equations 3.14, Equation 3.15 and Equation 3.16 were used to empirically predict dilution for isolated, echelon and rib stopes, respectively, at the Ruttan mine, Canada (Pakalnis, 1986). These equations were derived using multivariate analysis. Pakalnis (1986) evaluated strength, RQD, spacing, joint condition, RMR, area, hydraulic radius, span, height, width, volume, span/width, depth, fill, exposure rate, mining rate, ITH versus conventional drill holes, mining sequence and blasting.

Isolated stopes:

\[
\text{Dilution} = 8.6 - 0.09(RMR) - 13.2(\text{ExposureRate}) + 0.0038(\text{AreaExpose}) \quad \text{Equation 3.14}
\]

\[R^2 = 0.79 \quad s=\pm/3\%\]
Chapter 3 – Stope Design

Echelon stopes;

\[
Dilution = 10.3 - 0.13(RMR) - 14.8(ExposureRate) + 0.003(AreaExposed) \quad \text{Equation 3.15}
\]

\( R^2=0.83 \quad s=\pm/-2\% \)

Rib stopes;

\[
Dilution = 15.8 - 0.18(RMR) - 7.7(ExposureRate) + 0.0026(AreaExposed) \quad \text{Equation 3.16}
\]

\( R^2=0.80 \quad s=\pm/-4\% \)

Pakalnis (1986) considered the dilution predicting equations to be unique to Ruttan mine and therefore, not generally applicable. However, this does not preclude the use of mathematical dilution modelling on a site-specific basis.

3.2.8 Site-specific Methods

Some authors have expressed concern about the general applicability of the Mathews method. Both Mathews et al. (1981) and Potvin (1988) noted that the method was originally developed for open stope mining methods in geological conditions similar to those encountered in the Canadian shield. Stewart and Forsyth (1995), whilst acknowledging there are some indications that the method may be generally applied, emphasised the potential bias in their more limited database and recommended users concentrate on collecting sufficient examples to define their own stability zones. Bawden (1993) also suggested that the analysis of stable versus failed stopes could be used to derive a stability boundary for a particular operation. From their experience of back-analysing a large database from the Mount Charlotte gold mine in Western Australia, Trueman et al. (2000) concluded that the model gave reasonable predictions of stope surface stability, at least for steeply dipping deposits in moderately good to good rock. Nevertheless, Trueman et al. (2000) followed the guidelines of Stewart and Forsyth (1995) and developed a site-specific graph for Mount Charlotte mine, Western Australia.

In the late 1990s there was significant interest in the development of site-specific stability graphs within the Australian metalliferous mining industry. However, Potvin and Hadjigeorgiou (2001) suggest that while site-specific calibration procedures are attractive, they are limited in that the calibrations would only be validated towards the end of mine life. This would be particularly true in the case of the relatively short mine life commonly associated with small narrow-vein gold mines.
Villaescusa (1996) developed a site-specific stability chart for bench stopes at the Mt Isa Lead mine. This was made possible because of large numbers of historical stope records that had been collected over years of stoping in the Mt Isa Lead mine. Figure 3.14 illustrates the bench stability chart developed by Villaescusa (1996). This chart has been developed to capture the parameters believed to affect stope stability in the Mt Isa Lead mine bench stopes. The rockmass is classified based on the number of bedding planes per metre. Stresses normal to the orebody are used as an index of stress and blasting practice is rated. This method continued to be used in the Mt Isa Lead mine until mining ceased in 2003.

![Figure 3.14 - Mt Isa Lead mine bench stability chart, after Villaescusa (1996)](image)

### 3.2.9 Summary

Stability graphs have demonstrated predictive ability in large open stoping and are used widely at both the feasibility and production stage of stope design (Potvin and Hadjigeorgiou 2001). The overall merit of developing site-specific stability charts seems to depend on the amount of time available to develop a chart before a mine ceases operations. In addition, there is uncertainty surrounding the amount of case studies required to develop a reliable site-specific chart.

There has been concern about the applicability and reliability of existing stability graphs and associated modifications and adjustments to narrow-vein stoping. These concerns relate to narrow-vein operating conditions such as; undercutting of stope walls, propensity to stress
relaxation and stress damage, irregular stope geometry, drill and blast blasting related damage and overbreak, as well as moving backfill abutments. The applicability of the stability graph approach to narrow-vein stoping is evaluated with respect to each of these parameters in the following chapter.

3.3 ANALYTICAL STOPE DESIGN

3.3.1 Beam and Plate Theory

Beam and plate theory is based on civil engineering principles of static analysis. Obert and Duval (1967) describe how plate and beam theory can be applied to the stability of rock excavations. Forces and moments are resolved based on the assumptions of beam properties that would frequently be invalid in a stoping environment. In particular, both beam theory and plate theory are based on the assumption that the rock beam or plate is continuous, homogenous, isotropic and linear elastic. In practice the rockmass surrounding a stope is commonly discontinuous with dominant joint sets generating isotropic behaviour. Both plate and beam theory assumes that the load is applied normal to beam or plate. However, narrow-vein hangingwalls and footwalls in Australia, Canada and the UK are commonly sub-vertical and therefore, the direction of loading would not be normal to the beam or plate being used to analyse the excavation. Narrow-vein stopes have small widths, and therefore back stability is not usually a significant source of dilution in narrow-vein mines. Backs refer to the roof of the stope. Therefore, it can be concluded that in general plate and beam analysis would have limited applicability in narrow-vein stope design.

3.3.2 Voissoir Beam Theory

Brady and Brown (1993) provide a detailed procedure for evaluating stability using voissoir beam theory. The voissoir beam forms in laminated or blocky ground when the tensile strength is reduced to zero in the radial or normal direction (Diederichs and Kaiser 1999). The voissoir beam forms a compressive arch which can fail by compressive crushing on the upper side of the beam or snap-through. The basis for the solution is to balance the moment generated at the abutment by weight of the half beam with the opposing moment generated by the reaction force at the middle of the beam (Diederichs and Kaiser, 1999). The voissoir beam solution is based on the assumption that the reaction forces and distributions acting parallel to the beam at the abutments are identical.
In the case of sub-vertical narrow-vein geometry the reaction forces and distributions associated with the weight of the beam would not be equal. For example, in the case of a vertical stope walls the reaction force at the top of the beam wall due to half the beams weight would be zero. Therefore, even from a purely analytical approach voussoir beam theory is not appropriate for stope walls which are not approximately horizontal and the assumption of equal reaction forces at the abutments is not reasonable. To summarise, voussoir beam theory does not take into account the effect of excavation dip and is essentially a horizontal surface analytical technique.

In practice, narrow-vein longhole stoping can only be used when the stope dip exceeds the angle of repose of the broken ore. When stope dip is less than the angle of repose of broken ore alternative mining methods are required. The alternative methods commonly use pillars. Voissoir beam theory does not allow for the effect of pillars. Therefore, voissoir theory is not applicable to flat-dipping or sub-vertical narrow-vein stoping.

3.3.3 Kinematic

Those involved in the development of empirical stope design note that in general, the number of variables to be considered in stope design is too high to permit other than empirical approaches to design (Mathews et al, 1981). However, they also note that analytical methods are often used to identify excessive stress conditions or excessive deformations. Grenon and Hadjigeorgiou (2003) propose that while the stability graph approach is the method of choice at the pre-feasibility and feasibility stages, once it is possible to collect data from underground more detailed kinematic analyses should be undertaken. They argue that while empirical rockmass classification systems are useful when information is limited, these classification indices can fail to capture the structural complexity. Once structural data becomes available from underground scanline mapping, Grenon and Hadjigeorgiou (2003) propose stope stability be assessed using a three-dimensional joint network. Germain et al. (1996) developed Stereoblock where joints are represented as disks and it is based on the work of Baecher et al. (1977), Villaescusa (1991), Lessard (1996), Grenon (2000) and others.

Once a three-dimensional network has been simulated, an excavation can be introduced into the model and all blocks formed at the surface of the excavation can be quantified. The factor of safety of every block is evaluated by applying the limit equilibrium approach proposed by Hoek and Brown (1980). In the case discussed, Coulomb failure criterion was applied where angle of friction and cohesion values are sampled from the normal distributions determined
from direct shear test mean and variance for each parameter. However, the supporting effect of compressive stress is not taken into account.

### 3.3.4 Summary

Analytical techniques on their own do not take into account all factors influencing narrow-vein stope stability. Beam, plate and voissoir beam theories are based on the assumption that the rockmass will behave as a homogenous, isotropic and linear elastic material. In addition, these theories are based on analysis of a horizontal surface without pillars. For these reasons the applicability of beam, plate and voissoir beam theories to narrow-vein stoping is very limited.

Kinematic analysis of stope stability using a three-dimensional joint model could be considered complimentary to the stability graph approach, and has the potential to improve stability predictions at an operating mine where stability is known to be structurally controlled. A serious limitation of the Stereoblock approach is that it does not take account the effect of confining stress on seemingly unstable blocks. The simple distinct element model discussed in the following section highlights how block stability is dependent on confining stress. However, that being said if wedge and block failures are a persistent cause of instability at an operating mine then it could be assumed that the confining stress state is such that kinematic analysis maybe warranted. Under these circumstances kinematic analysis could be valuable in locating areas with high potential for instability.

### 3.4 NUMERICAL METHODS

#### 3.4.1 UDEC and 3DEC Distinct Element Codes

*UDEC* and *3DEC* are distinct element codes that simulate the stability of an excavation by dividing the modelled rockmass into discrete polyhedra (Itasca, 2004). *UDEC* is two-dimensional and *3DEC* is three-dimensional. Relative motion along discontinuities is governed by linear and non-linear force-displacement relations for movement in both the normal and shear directions (Itasca, 2004). Therefore, the behaviour of the modelled rockmass is dependent upon the assigned shear strength properties needed to evaluate force-displacement relations for each block loaded under both gravity and in situ stress.

*UDEC* has been used by Kaiser and Maloney (1992a) to simulate failure due to abutment relaxation in tabular stopes with blocky or laminated rock mass. This simulation is illustrated
in Figure 3.15 and was the mechanism believed to drive instability in Winston Lake hangingwalls.

![Diagram](image)

**Figure 3.15 – UDEC simulation of failure induced by softening of lower abutment, after Kaiser and Maloney (1992b)**

In practice, it would be difficult to represent the structural complexity a stope wall rockmass in 3DEC or UDEC. Unlike the simple structural geometry depicted in Figure 3.15, the structural complexity and scale of many stope walls would make modelling cumbersome. Therefore, as shown in Figure 3.15 simplifications are necessary. In addition, discrete element modelling of block stability requires assumptions about the shear properties at the boundary between blocks. These assumptions would require calibration to underground observations. This is not possible at the feasibility stage of development.

### 3.4.2 FLAC and FLAC3D Finite Difference Modelling

FLAC and FLAC3D are two and three dimensional finite difference modelling software (Itasca 2004). FLAC and FLAC3D model the rockmass an equivalent continuum where the modelled rockmass behaviour can be assigned various properties. Rockmass behaviour is often modelled on empirical rock failure criteria.
Henning and Mitri (1999) used a non-linear FLAC modelling program to examine the influence of stope geometry, stress environment and rock material properties on hangingwall stability at the Bousquet 2 mine, Canada. Numerical modelling predictions were consistent with observations in stope stability. However, while low stress appeared to be a major influence on stability at this particular mine, the modelling results were not able to take into account structural failures due to wedges in the second stope.

Aydan (1997) suggests that the most serious limitation of the continuum numerical modelling approaches is the tendency to assume an equivalent continuum to represent the complex array of geological discontinuities of rock masses.

Wattimena (2003) and Mawdesley (2002) have undertaken a comprehensive review of the application of equivalent continuum models. Both Wattimena (2003) and Mawdesley (2002) have demonstrated significant problems with respect to mesh dependency, material properties and post-peak properties when attempting to predict the response of a rock mass. Some methods have been proposed to overcome mesh dependency. However, these methods are based on simplified geometry and loading conditions, and are themselves dependent on input parameters which are difficult to determine for rock masses (Wattimena, 2002). The accuracy of input parameters that significantly influence model results is of paramount importance to a model that attempts to predict stability from first principles (Mawdesley, 2002). At the feasibility stage in a mine life it is very difficult to determine accurate estimates of the model input parameters. Model input parameters such as friction angle, cohesion and tensile strength are often estimated from correlations between these parameters and the assigned rockmass classifications.

### 3.4.3 Boundary Element Modelling

Boundary element modelling can also be used for equivalent continuum analysis. Like the finite difference approach to stability assessment, excavation stability can be assessed by evaluating stability with respect to an appropriate failure criterion. In terms of linear elastic modelling, finite difference and boundary element models should give the same induced stress levels. The main difference between boundary element and finite difference is that boundary element is a static analysis where forces are resolved on the boundary of elements, while a finite difference formulation is based on a dynamic solution to equations of motion. Therefore, finite difference can be used to model a failure process dynamically, while boundary element determines a steady state solution.
Pakalnis et al. (1991) presents three case histories where PCBEM two-dimensional boundary element models were used to predict stope stability by comparing linear elastic stresses to the Hoek-Brown failure criterion. This approach is essentially an equivalent continuum approach. In all three cases numerical modelling results were augmented with empirical assessment. Pakalnis et al. (1991) found that the numerical modelling results when complimented with adverse jointing information enabled identification of zones of instability. Pakalnis et al. (1991) emphasised the importance of calibrating failure criterion parameters to underground observations.

### 3.4.4 Determination of Induced Stresses

Potvin (1988) used the BITEM boundary element program to undertake a parametric study of induced stresses for different stope geometries. The results of this study were used to produce estimates of induced stress as required in the determination of factor A. Since the early to mid 1990s three-dimensional modelling software has become widely available and consideration of induced stresses is part of a standard stope stability assessment. It should be noted that Kaiser and Maloney (1992b) found that elastic models, such as Map3d tend to over estimate stress relaxation but changes in stress can be observed. Kaiser and Maloney (1992b) suggests that non-linearity of the rock mass during unloading explains the unreasonably large modelled stresses. In the case of mid-stope stress determination required to evaluate factor A, linear elastic stress are used as an index of stress. As an empirical technique, the actual stress magnitudes are not as important as consistency in the method of estimating induced stresses.

Pakalnis (1991) undertook a parametric study of hangingwall stresses considering the effect of two-dimensional versus three-dimensional linear elastic modelling. Based on this study Pakalnis (1991) concluded that when the ratio of the stope length to stope height is less than four, two-dimensional plane-strain modelling was greater than 10 percent different from the three-dimensional result. Potvin et al. (1988) came to similar conclusions in their comparison of two and three dimensional boundary element modelling.

The original Mathews Stability Graph, the Modified Stability Graph and the Extended Mathews stability chart case studies were almost exclusively based on mid-stope stresses estimated by two-dimensional linear elastic modelling. In contrast to large open stopes, narrow-vein stopes often have stope height to length ratios exceeding four. In Potvin’s (1988) two-dimensional analyses of 89 relaxed case studies (large open stopes) he found no evidence that stress relaxation impacts on stability. However, two-dimensional plane strain modelling of
large open stopes may indicate low or tensile stresses in cases where three-dimensional modelling would not. Therefore, three-dimensional modelling may indicate that these case studies were not actually relaxed.

3.4.5 Summary

While numerical models can be used as part of a back analysis of stope stability, the input parameters required to predict instability are difficult to quantify without calibration against underground observations. In an operating mine where simulation input parameters can be calibrated to underground observations, this type of modelling may be useful when comparing the relative stability of alternative stope designs.

Two-dimensional plane-strain linear elastic stress estimates are significantly (10 percent) different from three-dimensional linear elastic modelling when the stope length to height ratio is less than four. This may have implications for the narrow-vein stoping where length to width ratios are often more than four, especially in relation to the impact of stress relaxation. The potential for stress relaxation to impact on narrow-vein stability is discussed further in the following chapter.

3.5 CONCLUSIONS

The stability graph approach is largely successful because it captures the two main stope failure mechanisms. Local failures due to structure are taken into account by block kinematics which is accounted for by factor B and factor C. However, when detailed scanline mapping is available, kinematic analysis carried out using three-dimensional joint networks may have the potential to improve stability prediction in some mining induced stress states. While failure mechanisms associated with the stress state and overall rock mass properties are taken into account by Q’ and stress factor, there is potential for narrow-vein geometries to be particularly susceptible to stress relaxation and discrepancies between two and three dimensional modelling.

There appears to be uncertainty surrounding the applicability and usefulness of site-specific or site-calibrated stability charts. While some authors continue to advocate site calibration of the stability graph method, two of the primary authors and developers of the stability graph approach see the value of site-specific stability charts as limited.
The advantage of the original Mathews stability chart over the modified stability chart is that it is easier to calculate. There do not appear to be any significant improvements in the predictive ability of the modified stability chart over the original Mathews method. However, the modified stability number is in wider use than the original method and has been published in several textbooks. The considerable advantage of the Extended Mathews Stability Chart, over the original Mathew’s Stability Graph and the Modified Stability Chart is increased range of case studies; both in terms of hydraulic radius and stability number. In addition, the Extended Mathews Stability Graph stable-failure boundary has been determined statistically using logistic regression.

The objectivity possible with the ELOS stability chart is a significant advancement in empirical stability charts, especially in relation to narrow-vein stoping. Due to the subjectivity of what could be termed stable, the same level of ELOS associated with a stable large open stope could result in uneconomic dilution levels at a narrow-vein mine. However, more data is required to validate the ELOS stability chart zones and increase the general applicability of the model. It is worth noting that the increasing availability of the objective continuous stability variable, ELOS is amenable to other empirical modelling techniques. The same data used to develop the ELOS stability chart could be used to develop mathematical models to predict dilution.

Numerical modelling and analytical methods general applicability is limited because both approaches only account for one failure mechanism. However, when employed in combination with empirical approaches which capture a broader range of stability parameters, both numerical and kinematic analyses have the potential to improve stability prediction in some circumstances. This is particularly true in the case of sites where scanline mapping has been undertaken and it is possible to evaluate stability using a three-dimensional joint network and limit equilibrium analysis. Provided input parameters are calibrated to reality, the equivalent continuum approach using a failure criterion has the potential to improve stability prediction at an operating mine. Neither analytical nor numerical methods in isolation are likely to provide reliable stope stability predictions at the feasibility stage of a project.

Existing empirical stope stability models do not take into account a number of the parameters believed to affect stope stability. Despite this, stability charts are approximately 80 percent accurate in terms of predicting instability. This implies that those factors taken into account are able to predict instability in 80 percent of cases. One of the objectives of this thesis is to assess
whether this level of accuracy applies to narrow-vein stability prediction. It has been hypothesised that some parameters excluded from the formulation of existing stability charts may have a greater influence on narrow-vein stability than on large open stope stability. For this reason there is some uncertainty associated with the accuracy of existing stability charts to narrow-vein stability prediction. In the following chapter, parameters believed to affect narrow-vein stope stability have been reviewed with the purpose of prioritising research objectives.
Seven parameters believed to have a high potential to affect narrow-vein stability and not generally incorporated into stability charts have been reviewed. The effect of one or more of these parameters has the potential to dominate stability at a narrow-vein mine and in this case explicit consideration of the dominant factor or factors would be warranted. Factors and methods to take into account moving backfill abutments, stress relaxation, stress damage, complex geometry and undercutting have been qualitatively reviewed. In most cases further validation is required to determine the applicability of these factors to narrow-vein dilution prediction. The effect of time, blast damage, stress relaxation, undercutting and stress damage are rock mass dependent and therefore, to some extent are taken into account by $Q'$. For this reason narrow-vein mines may be more sensitive to $Q'$ than large open stopes. If this is the case then the stability chart approach could be calibrated to narrow-vein conditions. The possibility of developing site-specific stability charts for narrow-vein mines is considered in the following chapter. Improved dilution prediction facilitates a more accurate comparison of total costs per tonne for longhole versus conventional mining methods.

4.1 INTRODUCTION

For various reasons including improved safety and economies of scale, longhole stoping is increasingly the mining method of choice for narrow-vein orebodies with dip exceeding the angle of repose of the ore. Incentives to increase the size of stopes, equipment and blasts relate to the economies of scale which have the potential to increase productivity and decrease costs (Scoble and Moss, 1994). However, as discussed in Chapter 2, the increasing employment of longhole stoping in narrow-vein mines has been associated with increased dilution. The following parameters are considered for their potential effect on narrow-vein dilution:

1. Drilling and blasting effects.
2. Backfill abutments.
3. Effect of time.
Chapter 4 – Narrow-vein Dilution Parameters

4. Stress relaxation.
5. Stress damage.
6. Complex geometry and in-stope pillars.
7. Effect of undercutting of stope walls.

The stability graph approach to stope design has been widely adopted for open stope design. The literature relating to stability graphs does not preclude its application to narrow-vein stoping. However, applicability to narrow-vein operating conditions could be adversely affected by a number of parameters not generally included in the stability graph approach. In this chapter, factors or methods proposed to take into account some of these parameters have been qualitatively reviewed. Based on the review of the potential effect of each of these parameters to narrow-vein dilution research priorities and strategies can be determined.

4.2 DRILLING AND BLASTING EFFECTS

Based on experience Scoble and Moss (1994) suggest that dilution due to blasting overbreak in longhole stopes is typically one metre, with more in narrow-vein orebodies. Clark and Pakalnis's (1997) ELOS dilution chart categorises ELOS less than 0.5 metres as blast damage only. Clark and Pakalnis (1997) note that the quantity of ELOS associated with blasting will depend on the quality of drilling and blasting. Potvin (1988) suggests that in most cases, blast induced dilution could not be isolated by the stability graph because blasting was not a dominant cause of instability. Within the context of large open stopes 0.5 to 1.0 metres blast related overbreak would probably not represent a failure. This explains why within Potvin (1988) database of relatively large stopes he could not isolate blast induced dilution as a stability issue. However, as discussed in Chapter 2, 0.5 to 1.0 metres ELOS would have a significant economic effect on narrow-vein mines.

The relationship between overbreak and damage appears to be rockmass dependent. Therefore, it could be argued that because damage is rockmass dependent (low Q’ stopes would have higher levels of blast damage than high Q’ stopes), blast damage is indirectly taken into account by the inclusion of Q’ in existing stability charts (Mathews et al., 1981; Potvin, 1988; Mawdesley et al., 2001). This could account for the reason why, even though blasting parameters are not included in the most commonly used empirical stability charts, the stability charts generally provide predictive accuracy of approximately 80 percent.
4.2.1 Overbreak and Blast Damage

Singh (1998) defines overbreak as the breakage of rock beyond the designed profile of an excavation. This definition distinguishes overbreak from other sources of dilution such as slough, wedge failures, slabbing and unravelling. Blast damage is defined as the creation, extension and/or opening of pre-existing geological discontinuities in the rock mass (Villaescusa et al., 2003). Blast induced damage weakens a rockmass, potentially leading to stability problems when the excavation size is increased (Villaescusa et al., 2003).

Clark (1995) summary of previous work on blast induced damage indicates that strain or shock energy is one of two blast damage mechanisms. In good quality rocks, strain or shock energy has the greatest effect on damage, while in the case of highly fractured poor rockmasses the gas energy dominates blast damage mechanisms (Clark, 1995). The most common method of identifying blast damage potential is the peak particle velocity (PPV) (Singh, 1998). The strain produced in the rock mass is proportional to PPV, which is a measure of the damage causing potential of a blast (Singh, 1998). However, Germain and Hadjigeorgiou (1997) refer to personal communications with Clark and Pakalnis (1997) indicating that attempts to link PPV with overbreak at two mine sites had produced conflicting and inconclusive results. Germain and Hadjigeorgiou (1997) suggests the reason for these inconclusive results may be that uncertainty in drilling and blasting parameters results in poor charge distribution.

Villaescusa et al. (2003) determined a site-specific Holmsberg-Persson critical PPV or damage threshold model for the Mt Isa lead mine bench stopes. Using the site-specific critical PPV their analysis predicted that the extent of blast damage of hangingwalls was of the order of 3.6 metres depending on hole location. Observed damage (down-hole video survey) ranged from 5 metres to 8 metres. The only discussion of the relationship between overbreak and damage was to note that the extent of blast damage generally extends beyond overbreak boundaries. In a similar study conducted at the Bousquet mine by Henning et al. (1997) and reported by Diakite (1998), higher blast vibrations in primary stopes were associated with higher maximum linear overbreak than the secondary stopes with lower vibrations levels.

It seems that because the relationship between blast damage and overbreak is rockmass dependent there is no generic model for predicting overbreak using blast damage modelling. However, individual mines could compare overbreak stability to damage modelling and use this information in blast design. The value of blast damage modelling is in the ability to compare the blast damage potential of alternative drill and blast designs.
4.2.2 Blast Design

Wang et al. (2002a) found that stopes drilled with parallel holes had more dilution than fanned holes. These conclusions were based on comparing the average difference between the actual and predicted ELOS for the two groups. The average difference between the actual and predicted ELOS for the fanned hole stopes was 0.19 metres, whilst the average difference between actual and predicted ELOS for the parallel hole stopes was 0.67 metres. Wang et al. (2002a) suggest that although parallel patterns give better explosive distribution and allow for wall control blasting, parallel holes are very sensitive to borehole deviation. In contrast, Clark and Pakalnis (1997) found a tendency for overbreak to increase when holes are fanned as opposed to drilled parallel to stope surfaces. The difference between Clark and Pakalnis (1997) and Wang et al. (2002a) findings is an example of the complexity of the issue of the effect of drilling and blast pattern on dilution and points to the difficulty of attempting to predict dilution based on blast design.

Pakalnis (1986) introduced a blast correction factor to his dilution prediction model. However, he noted that this value is difficult to estimate and recorded blast-related dilution as the damage observable after the slot was extracted (Pakalnis, 1986). Potvin (1988) hypothesised that the blast induced dilution observed by Pakalnis (1986) could be attributed to the unusually high charge weights per delay.

Germain and Hadjigeorgiou (1997) used CMS surveys of overbreak and underbreak from the Louvicourt mine, Canada to determine statistical correlations between stope performance and powder factor. Stope performance is defined in Equation 4.1.

\[\text{Stope Performance} = 1 - \frac{\text{Overbreak} + \text{Underbreak}}{\text{Planned break}}\]

\textbf{Equation 4.1}

The correlation coefficient for the relationship between stope performance and powder factor was \(-0.083\), with a standard deviation of +/-0.185. This indicates that at the Louvicourt mine, powder factor was not related to stope performance.

Scoble and Moss (1994) discuss the potential for blast damage at the slot to unravel from the hangingwall as the stope is mined out. The confinement related slot dilution would be analogous to the high confinement longhole rises which are often used to start a narrow-vein stope.
Clark and Pakalnis (1997) found, using scatter plots and neural network analysis that overbreak increases with blasthole diameter and blasthole length. This contrasts with the Ruttan data which indicated dilution was not sensitive to changes in blasthole diameter (151 mm ITH holes compared to 51 mm conventional holes) (Pakalnis, 1986). However, personal experience with drilling and surveying of ITH and conventional drill holes at the now closed Renison Tin mine in Tasmania leads the author to believe that the increased accuracy possible with ITH compared to standard longhole drilling may have countered the negative effect of higher blast damage potential.

Pageau et al. (1992) found that by implementing smooth wall blasting, including a pre-split, they were able to reduce dilution at the Richmont mines, Francouer mine Quebec from 10-15 percent to 5 percent. They were particularly critical of attempts to relate overbreak to powder factor and believed that pre-shearing is effective in controlling hangingwall damage due to shock energy and placed emphasis on provision of adequate free face (reduced burden) to prevent damage due to gas energy. As a consequence of drillhole placement design, more drillhole metres were required per cubic metre (Pageau et al., 1992). In terms of dilution minimisation, the resultant elevated powder factor was deemed less important than ensuring reduced burden. Similarly, Williams (2002) benchmarking study of 80 case studies from the Campbell mine, Canada found no trends between the average explosive energy density (pounds per ton) and stope stability.

At Inco’s Thompson mine in Manitoba, Canada a process improvement team was formed to reduce dilution (Revey, 1998). After analysing all the parameters influencing dilution, the process improvement team concluded that drilling accuracy was critical to dilution reduction (Revey, 1998). Aplin (1997) identified inaccurate longhole drilling as one of the causes of dilution at the Selebi North mine in Botswana. Both Cambell mine and Thompson mine are large open stoping mines.

4.2.3 Millisecond Blasting (Electronic Detonator Blasting)

According to the elastic theory developed by Ito and Sasa (1968) simultaneous detonation of adjacent holes results in greater maximum stress, which then results in larger tensile cracks between holes. By firing perimeter holes at the same time elastic theory indicates that there is a increased probability that the cracks generated between holes will join and this results in preferential splitting of the rock along the perimeter. For this reason it is a common practice in underground development or tunnelling to fire perimeter holes on the same delay number. This
is different from pre-split where the holes are fired as a separate blast before the main blast. Following from this theory, Ichijo et al. (1994) hypothesise that the large tensile cracks resulting from the improved simultaneity possible with millisecond blasting, will result in less damage to the rock mass and therefore, a smoother blast profile.

In order to validate this theory, Ichijo et al. (1994) and Yamamoto et al. (1999) conducted a well-controlled study of the effect of electronic delay detonators on tunnel overbreak and rock mass damage at the Kamaishi mine, Japan. Of the five rounds fired, three were deemed suitable for further analysis (Yamamoto et al., 1999). During each round, half of the tunnel was fired using electronic delays, while the other half was fired using pyrotechnic delays. The rock mass at the experimental site is described as very hard granodiorite with high UCS and a high Young’s modulus. Yamamoto et al.'s (1999) analysis of these results indicates that the use of millisecond delays resulted in a six percent difference in the ratio of actual to design cross-sectional area compared to pyrotechnic delays. A ratio of one indicates no overbreak, while greater than one indicates overbreak and less than one indicates underbreak. The difference of six percent, was obtained by setting all underbreak excavations ratios to one. This was done to account for underbreak removal in a tunnelling scenario (Yamamoto et al., 1999). However, because the pyrotechnic delays have a larger standard deviation than the electronic delay detonators, setting of underbreak excavation ratios to one has biased the statistical analysis. In addition, Yamamoto et al. (1999) calculated the standard deviation of the difference between the two data sets incorrectly. The standard deviations were subtracted, when the standard deviation should have been calculated according to Equation 4.2 (Devore 1991), where \( m \) and \( n \) are the respective sample sizes of the two data sets.

\[
\sigma_{\bar{X}-\bar{T}} = \sqrt{\frac{\sigma_1^2}{m} + \frac{\sigma_2^2}{n}} \tag{Equation 4.2}
\]

Statistical revaluation using Equation 4.2 and including the undercut values indicates no significant difference at the 95 percent confidence level, with a p value of 0.35. When the undercut values are set to zero, as suggested by Yamamoto et al. (1999), there was still no significant difference, with a slightly improved p value of 0.19. A p value of 0.19 means that there is a 19 percent probability that there was no difference between the two data sets, or in other words that data sets were not significantly different. It is usual to reject a hypothesis if the p value at either the 95 percent level (p value > 0.05) or the 90 percent level (p value > 0.1).
While direct evidence of overbreak has been shown to be inconclusive, Yamamoto et al. (1999) and Ichijo et al. (1994) have strong evidence to indicate millisecond blasting results in reduced damage to the rock mass. Both borehole trace analysis and seismic tomography results indicate that the region of damage was smaller when using millisecond delays compared to pyrotechnic delays (Yamamoto et al., 1999).

Yamamoto et al. (1999) extended these ideas to produce a finite element numerical simulation based upon fracture mechanics. These simulations appear to indicate that in cases where time errors are in the order of tens of μs or less, the stress waves arising result in a fracture surface on the line connecting holes. However, it should be noted that cracks between holes also occurred for holes initiation separated by 350 μs. The reported results (Yamamoto et al., 1999) do not cover the same time period so it is difficult to compare the results. Olsson and Bergqvist (1996) investigated the influence of scatter in delays timing through a series of trials carried out in a quarry in southern Sweden. Based upon these trials Olsson and Bergqvist (1996) concluded that instantaneous initiation by electronic detonators is the best way to achieve short cracks. Unfortunately, crack lengths results for simultaneous initiation were not reported so it is difficult to ascertain how these conclusions were reached.

Solomon and MacNulty (1999) cite the Chamber of Mines Research Organisation (Brinkman et al., 1987) as including blast damage control as an advantage of millisecond delay blasting. However, two summary papers reporting on the findings of this project by Brinkman et al. (1987) and Giltner and Brinkman (1990) do not make any conclusions in relation to blast damage control. It should be noted that all four references presenting evidence of the advantages of millisecond blasting have a commercial interest in the wider application of millisecond blasting (Yamamoto et al., 1999; Ichijo et al., 1994; Solomon and MacNulty, 1999; Olsson and Bergqvist, 1996). On the other hand, Brinkman et al. (1987) and Giltner and Brinkman (1990) with no vested commercial interest in the application of millisecond delays, make no mention of their effect on overbreak or rock mass damage.

### 4.2.4 Resue Mining using Millisecond Blasting

One application of millisecond blasting technology has been commercialised under the name of Selective Blast Mining, SBM, (Bock et al., 1998). It is a variation of resue stoping, and uses electronic or millisecond blasting to separate ore and waste during a single blast. SBM contrasts with the Dome resue mining experience where the waste was blasted in a separate blast after the ore had been bogged by LHDs (Robertson, 1986). Millisecond blasting may
present an opportunity to minimise dilution using a modern version of rescue mining. Application of millisecond technology at West Rand Shaft appeared to result in the successful separation of ore and waste in a single blast. Bock et al., (1998) claims that Selective Blast Mining results in an increase in mine call factor from 80 percent to 90 percent. Mine call factor is another name for mine recovery. However, this claim is not backed up by data, and appears to be a theoretical claim based on the assumption that lower powder factor involved in the ore fragmenting second stage results in reduced losses of fine gold liberated during blasting. It is conceivable that this technology could be applied in steeply dipping longhole stopes. At this stage the technology lacks independent evaluation.

4.2.5 Drill-split Tool System

A second important technological development in the fields of narrow vein mining is a conceptual design presented by Lombardi (1994) and Brewis (1995) of a drill-split narrow vein mining system. The system employs the US Bureau of Mines developed drill-split tool where a mechanical wedge and expanding feather mechanism breaks the rock in tension. An important feature of such technology is that the drill split system results in minimal wall rock overbreak (Brewis, 1995). Cessation of U.S. Bureau of Mines Research in 1996 has left further development of this technology to others.

4.2.6 Summary

Causes of blast related dilution are complex and difficult to quantify. Because the impact on dilution of blast related overbreak in large open stopes is generally considered relatively minor, the stability graph approach has not taken into account blasting factors. In contrast, narrow-vein mine blast related overbreak has the potential to significantly affect the economic viability of a mine. The parameters affecting blast overbreak are complex with variability associated with rock mass properties, quality control issues and equipment characteristics complicating empirical analysis of design parameters. Empirical studies of the effect of blasting parameters on overbreak have produced conflicting conclusions, with uncertainty surrounding the effect of fan drilling versus parallel drilling, charge weight per delay, powder factor and PPV.

The literature indicates that smaller drillhole diameters, millisecond detonation, decreased burden and/or confinement have the potential to reduce blast damage. Whether reduced blast damage translates into less overbreak or not, will depend on the original rockmass quality and degree of degradation associated with blast damage. For this reason Q may to some extent take into account blast overbreak potential.
Several authors have identified drill and blast quality control issues as critical to overbreak minimisation. Smooth wall blasting techniques including pre-split have proved successful in reducing stope dilution.

To date there is insufficient evidence to conclude that simultaneous millisecond blasting results in significantly less overbreak than the less accurate simultaneous pyrotechnic delay detonation. However, the results of borehole trace length analysis and seismic tomography at the Kamaishi mine trial do indicate reduced damage of the rock mass when millisecond delays are used. There has been no work done in relation to translating the effects of millisecond instantaneous initiation in a stoping environment.

4.3 BACKFILL ABUTMENTS

Existing empirical stability charts treat backfill abutments the same as solid rock abutments (Potvin, 1988; Hutchinson and Diederichs, 1996; Milne, 1996). Potvin (1988) back analysed 59 case histories from a single transverse stoping mine to examine the effect of backfill to determine how best to treat backfill abutments in terms of calculating hydraulic radius from stope span. He found that when the length was taken as the summation of adjacent stope spans along strike, the modified stability chart incorrectly predicts failure. Therefore, Potvin (1988) argues that the stope span should be taken as the unfilled span as these stopes plot correctly when the span is taken as the unfilled span. Potvin (1988) acknowledges that while the hypothesis was verified for this particular mine, monitoring instrumentation and more case histories are required to prove this hypothesis generally.

Pakalnis (1986) statistical examination of the effect of adjacent filled stopes on dilution found no correlation or sensitivity, at the 99 percent level of confidence for the five stopes affected by adjacent backfill. However, Pakalnis (1986) was of the opinion that the data was insufficient to be a reliable estimator. Hutchinson and Diederichs (1996) recommend that backfill abutments be taken into account by considering a range of HR values. This results in a range of HR values and gives some indication as to the risk associated with backfill abutments.

Milne (1996) distinguishes between the role of backfill in large open stopes and the role of backfill in continuous backfilling scenarios. Milne (1996) suggests that the mechanisms of support are invoked differently under the two mining scenarios. In large open stoping, a stope is only extracted when the adjacent stope has been completely backfilled. Milne (1996) explains that the hangingwall deformation profile is effectively frozen in place and this limits
deformation in the adjacent stope. However, in the case of a continuous mining and backfilling scenarios, such as modified Avoca, the hangingwall adjacent to the open stope surface would facilitate greater deformation than would be possible otherwise (Milne, 1996). Milne (1996) proposes that the effective final span converges to the sum of the opening after blasting plus the opening span after backfilling. However, due to insufficient data, Milne (1996) was unable to confirm this hypothesis. However, the results of extensive hangingwall monitoring (extensometers) at the Winston lake mine did suggest that the common practice of treating a moving backfill abutment like a rock abutment is overly optimistic (Milne, 1996).

4.3.1 Summary
The batch type filling sequence associated with transverse longhole stoping has not been associated with increased stope instability. Therefore, the original recommendation that a backfill abutment can be treated the same as a solid abutment appears reasonable. However, there is compelling evidence that continuous backfilling sequences such as those used in narrow-vein stoping result in what has been termed a moving backfill abutment, and that a moving backfill abutment can not be treated the same as the batch type fill. The proposal to use the average of the maximum and minimum open span is a logical method for taking into account the effect of moving backfill abutments. However further validation is required.

4.4 EFFECT OF TIME
Numerous authors report evidence of the time dependent properties of underground excavation stability (Bienawski, 1989; Bawden, 1993b; Carter and Miller, 1996; Tharp, 1997; Villaescusa et al., 2003). Potvin (1988) found that openings plotting within the transition zone were sensitive to the effects of blasting and time. In addition, time dependent properties have also been measured in the laboratory (Atkinson and Meredith, 1987; Martin et al., 1997; Shao et al., 1997; Linkov, 2000).

Figure 4.1 is a graph developed by Bienawski (1989) to empirically relate excavation stand-up times, minimum span and rock mass rating, RMR. However, Franklin and Palasi (1993) argue that Bieniawski’s upper and lower bounds are due to limitations of the data rather than real stability boundaries. Franklin and Palasi (1993) developed a slightly different chart to represent the same parameters and data. Clark and Pakalnis (1997) scatter plot analysis showed a correlation between equivalent linear overbreak (ELOS) and the number of stope blasts, and neural network analysis showed a correlation between ELOS and stope life. Pakalnis (1986)
empirical model to predict dilution at Ruttan mine incorporates excavation rate that is an indirect incorporation of the effect of time.

The effect of time on narrow-vein stope stability may not be as severe as in most large open stope walls where stope wall failure can contribute to dilution for several months while the stope is extracted. However, in the case of sublevel retreat (without fill) narrow-vein stoping an uneven retreat profile has the potential to allow dilution from levels above to become entrained in ore mucked on lower levels.

Figure 4.1 – Graph relating RMR and minimum span to stand-up time, after Bienawski (1989)

4.4.1 Summary
There appears to be significant evidence in the literature of the effect of time on stope stability, especially in the case of stopes that plot adjacent to the stable-failure boundary. Empirical relationships between rockmass quality and stand-up time indicate that the effect of time on stope stability is rockmass dependent and therefore, may be implicitly taken into account by Q’.

4.5 EFFECT OF STRESS RELAXATION
The literature refers in numerous cases to the negative effect of stress relaxation, low stress and or tensile stress (Hutchinson et al., 1992; Clark and Pakalnis, 1997; Paraszczak, 1992; Diederichs and Kaiser, 1999; Wang et al., 2002b). However, few authors have attempted to
quantify the magnitude of its effect or whether it should be taken into account in empirical stope stability charts.

Potvin (1988) postulates that the effect of relaxation is indirectly taken into account by the factor C and that this is the reason he found no difference between the relaxed data subset and the rest of his database. Clark and Pakalnis (1997) took issue with factor A being set to one in cases where the induced stress is low or tensile, citing the potential destabilising effect of low stress or tensile stress on stability. Clark and Pakalnis (1997) conclude that more work is required to fully understand the effect of stress on hangingwall and footwall stability.

Diederichs and Kaiser (1999) used a UDEC numerical modelling simulation to show that relaxation reduces the self-supporting capacity of the excavation. Diederichs and Kaiser (1999) propose an adjustment for relaxation that links voussoir analogues to the modified stability graph. The method uses an iterative procedure to locate ‘logical upper and lower parametric limits’ and assumes; A equals one (low stress), B equals 0.3 (surface parallel jointing) and C equals two (horizontal roof). Young’s modulus and UCS are then estimated based upon the empirical relations stated in Equation 4.3 and Equation 4.4.

\[ E_{\text{rockmass}} = 5\sqrt{Q'} \]  
\[ UCS = 20\sqrt{Q'} \]

Diederichs and Kaiser (1999) qualify the use of these formulae for a moderately relaxed setting by stating that these formulae are not used as engineering recommendations, but rather as reasonable baseline assumptions in order to create a generalised model to assess the effects of relaxation. Based upon these assumptions N’ is derived using the relation 150 times the cube of the thickness.

The Voussoir simulation was run using the relationships above to relate N’ to UCS, E, Q’ and thickness; the simulation was run using different hydraulic radius (Diederichs and Kaiser, 1999). The partition between stable and unstable has been calibrated using Potvin (1988) upper no-support limit (Diederichs and Kaiser, 1999). The hydraulic radii for critical square openings are uniformly 0.77 times that of critical tunnel spans for equivalent N values. The voussoir simulation predicts a square span is equivalent to a long tunnel with a span equal to 0.65 times.
the square span (Diederichs and Kaiser, 1999). Therefore, square spans are 0.77 times the HR of critical tunnel spans (Diederichs and Kaiser, 1999). This corresponds well with the 0.72 correction proposed by Milne (1996) using a radius factor based upon the harmonic average distance of the centre of the span to the perimeter.

Based upon the results of these simulations and calibrated against the Thompson mine data (Greer, 1989; Bawden, 1993a), Diederichs and Kaiser (1999) developed an adjustment to \(A\) for states of relaxation based on tensile stress (\(\sigma_T\)) – Equation 4.5.

\[
A = 0.9e^{11(\frac{\sigma_T}{ECS})}
\]

Equation 4.5

However, this calibration is based upon a data set affected by other operational conditions described by Greer (1989). Greer (1989) back analysed Thompson mine case histories to determine how well Potvin’s modified stability chart predicted stability at this particular site. Greer’s analysis found unstable hangingwall case histories were largely misclassified as stable (Bawden, 1993b). Although this particular case uses Potvin’s modified stability graph, the Extended Mathews stability chart would have predicted similarly erroneous stability predictions. Greer (1989) identified operational conditions such as blast damage, the effect of adjacent sand fill and delays to filling as likely causes for the misclassification. Early versions of the vertical crater retreat method were used during this period of the Thompson mine life. This method can result in significant blast damage due to inherently more blasting confinement and higher powder factors than long-hole stoping.

4.5.1 Summary

From a theoretical perspective stress relaxation has significant potential to decrease stope stability. However, there are limited stope stability case studies to support the theory that stress relaxation adversely affects stope stability. This may be because stress relaxation is more likely to be affected by fair or poor quality rock masses and that because rock quality is taken into account in \(Q'\), the effect of stress damage is implicitly taken into account. In addition, plain strain two-dimensional modelling may have indicated a relaxed state when in fact the stopes would not have appeared relaxed if three-dimensional modelling had been used.

Due to their tabular geometry and usually continuous extraction sequences, narrow-vein stopes are particularly susceptible to the development of low or tensile stress in stope hangingwalls.
and footwalls. This is especially true when the maximum principal stress is perpendicular to strike. Therefore, if stress relaxation does significantly affect stope stability, narrow-vein mines are more likely to be affected than wide large open stopes where there is less of a propensity for stress relaxation.

4.6 EFFECT OF STRESS DAMAGE

4.6.1 Stress Damage

In generic terms, rock is considered damaged when the strength of the rock is reduced. In fracture mechanics, damage or crack damage threshold refers to the onset of irreversible volumetric strain (Bawden, 2002). Volumetric strain is simply a measure of rock deformation. Wiles (2002) uses a post peak-strength damage definition. This approach is well suited to an empirical failure criterion based damage model. Irrespective of the damage definition, the most important consideration is that the damage model parameters are calibrated to underground observations. As shown in the stress-strain curve shown in Figure 4.2, the onset of stress damage (or yield) marks a change from linear elastic deformation to non-linear plastic deformation (Bawden, 2002). Up until the onset of stress damage, removal of the unloading stress-strain path follows the same linear path as loading. Prior to the onset of crack damage deformations are recoverable.

![Stress-strain curve for hard rock](image)

**Figure 4.2 – Stress-strain curve for hard rock**
4.6.2 Predicting Stress Damage

Deviatoric stress has been calibrated to underground stress damage observations. A deviatoric stress based damage criterion is expressed in Equation 4.6 where $\sigma_1$ and $\sigma_3$ are the maximum and minimum principal stresses, respectively and $\sigma_{ci}$ is the in situ crack initiation stress (Kaiser, 1994; Castro et al., 1996; Martin and Read, 1996).

$$\sigma_1 - \sigma_3 \cong \sigma_{ci} \quad \text{Equation 4.6}$$

The input parameter, $\sigma_{ci}$ can be estimated from the short-term UCS, which is usually available. The AECL Mine-by experiment indicates that the in situ crack initiation stress occurs at about 0.3 times the UCS (Martin and Read, 1996). While the deviatoric stress approach has been calibrated to rock masses (Castro et al., 1996; Martin and Read, 1996), the calibration process was limited to the massive or moderately jointed rock mass observations at the AECL’s Underground Research Laboratory experimental mine (Martin and Read, 1996) and the Sudbury neutrino observatory cavern (Castro et al., 1996). This calibration also indicated that damage in massive and moderately jointed rock can be directly linked to the lab tested crack initiation threshold. Laboratory testing of rock specimens predicts crack initiation occurs when the deviatoric stress is between 0.25 and 0.5 times the UCS (Bawden, 2003a).

4.6.3 Effect of Stress Damage on Stope Stability

While post-stoping stresses are taken into account in all variants of the Stability Graph approach (Mathews et al., 1981; Potvin, 1988; Clark and Pakalnis, 1997; Mawdesley et al., 2001), the stability graph approach does not take into account the stresses experienced by stope walls prior to stoping. However, Sprott et al. (1999) propose an adjustment to the stability graph approach to account for pre-mining stresses. Formulation of Sprott et al. (1999) stress damage factor $D$ is based on changes in deviatoric stress. The stress factor $D$ formulation is based on the extra stress deviator. The extra stress deviator is the difference between the pre-stoping deviatoric stress, ($\sigma_1-\sigma_3$) and the in situ deviatoric stress ($P_1-P_3$), as calculated in Equation 4.7. The stress damage factor is then determined using Figure 4.3.

$$\text{ExtraStressDeviator} = (\sigma_1 - \sigma_3) - (P_1 - P_3) \quad \text{Equation 4.7}$$

Sprott et al. (1999) stress damage adjustments were successfully applied at the three large open-stoping Hemlo operations in Canada to predict stope stability and evaluate alternative
extraction sequences. However, this approach does not consider the full stress history experienced by the stope wall, only the pre-stoping stress.

Figure 4.3 – Stress damage graph, with calibration case studies shown after Sprott et al. (1999)

4.6.4 Summary
Stress damage has been demonstrated to affect stope stability. Deviatoric stress can be used to predict the onset of stress damage. However, the deviatoric stress damage criterion was calibrated in massive or moderately jointed rock masses and its applicability to fair and poor rock masses remains to be validated. Stress factor D is based on case studies from three Hemlo operations in Canada and therefore its applicability may be limited to similar rock mass conditions as those encountered at these mines. Both large and narrow-vein stopes are extracted in multiple blasts, which could result in short-term stress concentration above pre-mining stress levels. Retreat longhole narrow-vein stoping produces high stress concentration at the brow. Because the brow is extracted in small increments along strike there is potential for stress damage associated with the brow to affect large areas of hangingwall and footwall. Therefore, stress damage has the potential to affect narrow-vein stability especially as narrow-vein mining depths increase.

4.7 COMPLEX GEOMETRY AND IN-STOPE PILLARS
The literature contains three main approaches to taking into account the size of an underground excavation. The first approach was minimum span. Minimum span was developed in the context of civil engineering tunnels where the second dimension, the ends of the tunnel, are far enough apart to not provide significant support to the tunnel. Recognising that in the case of
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stope surfaces, the second dimension is such that two-way spanning can be expected, Laubscher (1977) introduced the concept of hydraulic radius to underground excavation empirical design. Hydraulic radius is defined as the area divided by the perimeter. As a mathematical function it is characterised by convergence to half the minimum span, such that at nine times the minimum span, the hydraulic radius is within 90 percent of its maximum value (Milne, 1996). Milne (1996) notes that this property implies the far ends of the excavation provide significant support even when length is nine times the minimum span. Milne (1996) draws upon Brady and Brown (1993) analytical conclusion that the restraint offered by the proximity of the face in an advancing tunnel is negligible at a distance of 1.5 times the tunnel span to suggest that hydraulic radius should approach its maximum value at approximately three times the span as opposed to nine times. To illustrate this point he uses an example of 10 metre by 100 metre surface having the same hydraulic radius as an 18.2 metre by 18.2 metres surface, suggesting that the two surfaces are unlikely to have the same stability.

Based upon analysis of logistic regression statistics Mawdesley et al. (2001) found that hydraulic radius is a very good predictor of stope stability and slightly better than minimum span. These conclusions are based upon the 485 case histories contained in the Extended Mathews Stability Chart (Mawdesley et al., 2001). However, it should be noted that this database is largely comprised of large open stoping data which are usually associated with aspect ratios less than three.

In the case of narrow-vein mining irregular geometry is encountered relatively frequently. The presence of in-stope pillars presents geometry for which it is particularly difficult to estimate a hydraulic radius (Li, 2000). Milne (1996) discusses the difficulty in estimating hydraulic radius for stopes with irregular geometries due to post pillars, raises and irregular wall geometries. Often there is more than one way of calculating hydraulic radius (Milne, 1996). As discussed in Chapter 3 Milne et al. (1996a), Milne et al. (1996b) and McHaughty (1994) developed the concept of harmonic radius to take into account irregular stope geometry. The term harmonic radius was suggested to Milne et al. (1996b) through personal communications as a means of estimating the average distance to abutments. A harmonic average is used to ensure that the large distances do not overly influence the calculated average (Milne et al., 1996b).

Milne et al. (1996b) proposed radius factor, which is half the maximum harmonic radius, as it closely approximates the hydraulic radius of a surface (Milne et al., 1996b). The effective
radius factor (ERF) is the harmonic radius at any point on the surface (Milne et al., 1996b). The maximum harmonic radius occurs at the centre of a surface (Milne et al., 1996b). Milne et al. (1996b) applied the radius factor concept to the following two mines; Detour Lake mine, Ontario, Canada and Brunswick mine, New Brunswick, Canada.

The Detour Lake mine is a longhole sub-level retreat mine and the extraction sequence is illustrated in Figure 4.4. Operators of this mine were most interested in overbreak with potential for entrainment in the ore on the mucking/extraction levels (Milne et al., 1996b). Milne et al. (1996b) found that the effective radius factor, ERF was useful for evaluating the stability of the zone of hangingwall near the retreating mining front. Figure 4.4 shows the area with an ERF greater than 12 and greater than one metre of predicted overbreak. The overbreak associated with the area shown as crosshatched in Figure 4.4 could be expected to contribute to dilution. Milne et al. (1996b) found that understanding which areas of the hanging-wall produce overbreak greater than 1 metre, enabled mine design engineers to adjust the mining retreat front and relocate pillars to minimise mucking overbreak with ore. Noranda Technology Centre, Montreal developed a program that automatically calculates effective radius for any point on a surface or grid pattern (Milne et al., 1996b).

![Figure 4.4 – Sub-level retreat stope at Detour Lake mine showing effect on ERF on stability, after Milne et al. (1996b)](image-url)
Milne et al. (1996b) demonstrated a clear trend linking effective radius factor and both strain and movement detected an extensometer installed in the hangingwall of Brunswick mine 16 N stope. The trend in the detection of increased movement and strain was demonstrated as subsequent stope blasts resulted in an effective radius factor of 2.6, 8.5 and 12.4, respectively. Figure 4.5 demonstrates the relationship between effective radius factor and strain and movement. Increases in strain and movement can both be taken as indicators of decreased hangingwall stability.

The concept of harmonic radius and effective radius were used by Trueman and Doktan (1999) to calculate effective radius factor for the sublevel retreat stopes at Kundana Gold mine, Western Australia. In-stope or rib pillars were left every 15 to 17 metres along strike and made determination of a hydraulic radius ambiguous and difficult. For this reason, Trueman and Doktan (1999) proposed that empirical stope stability analysis be undertaken using effective radius factor in place of hydraulic radius.

Figure 4.5 – Deformation detected by hangingwall extensometer at Brunswick mine 18 N stope, after Milne et al. (1996b)
4.7.1 Summary

Hydraulic radius has been shown to be a good predictor of stability for stope surfaces with regular geometry and aspect ratios less than three. Effective radius factor has been validated as a useful predictor of stability, especially for complex stope geometry. It has been demonstrated that for simple geometry with aspect ratios less than three, hydraulic radius and radius factor are approximately equal. Radius factor can be used in place of hydraulic radius for complex stope geometries with respect to empirical stope stability prediction.

4.8 UNDERCUTTING OF STOPE WALLS

Mining of ore widths less than the minimum sublevel or sill drive width of 2.5 metres results in undercutting of either the hangingwall or footwall (Brewis, 1995). Clark and Pakalnis (1997) found that stopes walls with a modified stability number less than five, are very sensitive to undercutting. Clark and Pakalnis (1997) found that overbreak equal to or greater than the undercut should be anticipated for stopes with stability numbers less than five. Germain and Hadjigeorgiou (1997) note that a hangingwall will be less stable and more sensitive to blast vibrations if it is undercut by too much development work.

Based on the premise that reduced clamping stresses associated with a relaxed hangingwall results in instability in large hangingwalls, Wang et al. (2002b) developed the following undercutting factor (UF) to take into account the effect of undercutting (and overcutting on the level above) on hangingwall stability – Equation 4.8.

\[
UF = \frac{l_o + l_u}{2 \times (L + H)} \times \frac{d_o + d_u}{2}
\]

Equation 4.8

where,

- \(l_o\) = Drift length where undercutting occurs on the overcut sill drive
- \(l_u\) = Drift length where undercutting occurs on the undercut sill drive
- \(L\) = Stope strike length
- \(H\) = Stope height (up dip)
- \(d_o\) = Average depth of undercutting along the overcut drift length \(l_o\)
- \(d_u\) = Average depth of undercutting along the undercut drift length \(l_u\)

The effect of UF on dilution was studied empirically using 146 case studies from Trout Lake mine, Canada and 45 case studies from Callinan mine, Canada. The difference between
dilution predicted using Clark and Pakalnis (1997) Dilution Graph and measured ELOS was compared to UF. This study confirmed that undercutting was a contributing factor in measured ELOS being 0.5 metres greater than predicted.

While Wang et al. (2002b) have demonstrated the adverse effect of undercutting and developed a parameter to quantify the extent of undercutting, they have not proposed how UF could be incorporated into Dilution Graph ELOS predictions. While undercutting has been demonstrated as a cause of dilution, the stress relaxation associated with undercutting is not the only mechanism which could have resulted in UF being associated with increased dilution. The kinematic effect of exposing the hangingwall to increased gravity effects is an alternative underlying mechanism for why undercutting affects dilution. If there were case studies which were primarily only affected by overcutting then these case studies could be compared to undercut case studies primarily affected by undercutting. This comparison would test whether stress relaxation or the increased effect of gravity is the underlying cause of increased instability associated with UF. If the primarily undercut case studies had significantly more dilution compared to predicted values than the overcut case studies then this would indicate that stress relaxation is not the underlying mechanism. If stress relaxation were the underlying mechanism then it could be expected that the two groups’ average dilution above predicted values would be similar.

4.8.1 **Summary**

Mechanisation of narrow-vein stoping has increased the number of stope walls that are undercut. Undercutting of stope walls has been demonstrated to increase dilution. If stress relaxation can be demonstrated to be the underlying mechanism for increased instability associated with undercutting then dilution predictions need to take into account both the effect of undercutting and overcutting in the sill drive above the stope wall in question. The undercutting factor UF appears to be a good index for the extent of stope wall undercutting. However, UF has not been directly related to dilution prediction.

4.9 **CONCLUSIONS**

Drill and blast parameters have potential to result in significant narrow-vein dilution. Blasting related dilution has been associated with both the design stage and the production stage. In terms of drill and blast design peak particle velocity has been shown to be a good index of blast damage potential. However, the extent to which blast damage affects overbreak remains
to be quantified. At the production stage, tighter control on quality control issues such as drillhole deviation has been shown to reduce blast related overbreak

While tight backfilled abutments have not been associated with increased instability, moving backfill abutments associated with continuous mining sequences have been linked to increased dilution. It has been proposed that the sum of the opening after blasting plus the opening span after backfilling be used when calculating hydraulic radius to take into account the effect of moving backfill abutments. However, further validation of this approach is required.

Due to the small incremental extraction of narrow-vein mines the potential for stress damage to affect adjacent hangingwall and footwalls is high, especially as mining depths increase. For this reason this thesis examines the effect of stress damage on narrow-vein stability. The stress damage factor D is not applicable when the extra stress deviator is negative.

Six out of the seven parameters considered are rock mass dependent and therefore, to some extent are taken into account by Q’. For this reason narrow-vein mines may be more sensitive to Q’ than large open stopes. If this is the case then the stability chart approach could be calibrated to narrow-vein conditions. The possibility of developing site-specific stability charts for narrow-vein mines is considered in the following chapter.

Alternatively, one or more of these parameters may have the potential to dominate stability at a narrow-vein mine and in this case explicit consideration of the dominant factor or factors is required. In this chapter, factors and methods to take into account moving backfill abutments, stress relaxation, stress damage and undercutting have been qualitatively reviewed. In all cases further validation is required to determine the applicability of these factors to narrow-vein dilution prediction. Review and evaluation of these factors and methods is undertaken throughout this thesis depending upon data availability.
Narrow-vein underground stope design engineers in Australia have expressed significant interest in the requirements and methodology for developing site-specific stability charts. This is largely due to concern that existing stope stability charts inadequately take into account factors influencing narrow vein stope stability. This chapter presents conclusions from a comprehensive statistical examination of case history requirements for a site specific stability chart and critiques literature arguments for site-specific charts. A new statistical analysis technique enabled the quantification of case history requirements. In addition, any site-specific effects the model exhibits have been analysed. The analysis indicates that a reliable stable-failure boundary requires at least 150 case histories, of which a minimum of 10 percent should be unstable stope surfaces. Marginal site-specific effects were observed for the operating conditions captured within the database. It has been concluded that the apparent site-specific effects contained in previous literature are attributable to operating conditions inadequately represented in the database. Such operating conditions could induce erroneous stability predictions at any site, and therefore, are not truly site-specific. The operating conditions relate directly to the parameters identified in Chapter 4 as having a high potential to affect narrow-vein stope stability. Chapter 5, Chapter 6 and Chapter 7 empirically investigate, evaluate and quantify the effect of these operating conditions on narrow-vein dilution. Following from these conclusions, a methodology for taking into account operating conditions particularly relevant to narrow vein mining has been proposed.

5.1 INTRODUCTION

Some concern has been expressed by a number of authors about the general applicability of the Mathews method. Mathews et al. (1981) and Potvin (1988) noted that the method was originally developed for open stope mining methods in geological conditions similar to those encountered in the Canadian shield. Stewart and Forsyth (1995) whilst acknowledging that there are some indications that the method may be generally applied, emphasised the potential bias in their much more limited database and recommended users concentrate on collecting
sufficient examples to define their own stability zones. Bawden (1993) also suggested that the analysis of stable versus failed stopes can be used to derive a stability boundary for a particular operation. From their experience of back-analysing a large database from the Mount Charlotte gold mine in Western Australia, Trueman et al. (2000) concluded that the model gave reasonable predictions of stope surface stability, at least for steeply dipping deposits in moderately good to good rock. Nevertheless, Trueman et al. (2000) followed the guidelines of Stewart and Forsyth (1995) and developed a site-specific graph for Mount Charlotte. In late 1990’s, 2000 and 2001 there was significant interest in the development of site-specific Mathews Stability Graphs within the Australian metalliferous mining industry.

The original Mathews Stability Graph for open stope design (Mathews et al., 1981) had only 50 case histories. The collection of more data by Potvin et al. (1989), Stewart and Forsyth (1995) and Trueman et al. (2000) increased the case history data which resulted in changes to the stability boundaries. Trueman et al. (2000) from their experience at the Mount Charlotte mine estimated that at least 100 case histories would be needed to determine a reliable site specific Stability Graph. However, this estimate was not arrived at from a rigorous analysis. An indication of how many case histories would be required to determine reliable stability boundaries is a pre-requisite for the development of a site-specific variant.

The qualitative nature of the stability category has meant that, until now, the relative reliability of a site-specific line to the ‘generic’ line, involved comparing the number of misclassified points until the design engineer was confident the site-specific line was more reliable than the generic stability line. In cases where site-specific effects appear to be affecting the reliability of the generic stable-failure boundary, this approach seems reasonable. However, inherent in this approach is the assumption that the site-specific database is sufficiently large to justify changing the stable-failure boundary. If the database is inadequate, the design engineer may make changes to the stable-failure boundary attributable to short-term operational conditions rather than any real site-specific factor. Furthermore, the site-specific chart may not be applicable in a new area of mine with different operating conditions. Operating conditions refers to equipment and mining method parameters, and does not include rock mass conditions. This chapter sets out to answer the question of how many case histories are required to set a reliable stability boundary, what proportion must be of a different stability classification and how site-specific is the stability graph approach. Use is made of a logistical regression technique to quantify these. Until now, there has been no way of rigorously determining
database requirements for a stable-failure boundary. This question has been answered by analysing variance in logit model parameters.

Once the database requirements were quantified in terms of number and type of points, the author could statistically examine whether site-specific model parameters are significantly different from the generic database. In this chapter, a site-specific stability chart has been analysed with respect to the statistical significance of site-specific effects. The sites to be analysed include the Mt Charlotte mine in the eastern goldfields of Western Australia and the Cannington mine in Queensland. Site visits to Mt Charlotte present no obvious site-specific parameters (Trueman et al., 2000). Logit model parameters and comparative statistics have been used to determine whether there is a significant difference between a stable-failure boundary developed specifically for Mt Charlotte and a generic stable-failure boundary. There are no apparent operational conditions to explain why the site-specific line would be significantly different to the generic stable-failure line. Therefore, if there were a significant difference, it would be evidence of the existence of site-specific effects. Ideally it would have been preferable to have investigated the same hypothesis with respect to other sites. However, Mt Charlotte is the only site that meets database requirements.

5.2 SITE-SPECIFIC EFFECTS

There is substantial anecdotal evidence that site-specific effects may result in erroneous stability predictions. Greer (1989) back analysed Thompson mine data to determine how well Potvin’s modified stability chart predicted stability at this particular site. Greer’s analysis (Bawden, 1993) found unstable hangingwall points were largely misclassified as stable. Although this particular case uses Potvin’s modified stability graph, the extended Mathews stability chart would have predicted similarly erroneous stability predictions. Greer identified operational conditions such as blast damage, the effect of adjacent sand fill and delays to filling as likely causes for the misclassification. As noted previously, early versions of the vertical crater retreat method were used during this period of the Thompson mine life. This method can result in significant blast damage due to inherently more blasting confinement and higher powder factors than long-hole stoping. Additional examples of inaccurate stability prediction are the Winston Lake hangingwalls (Milne, 1997). The assumption moving backfill abutments provides the same abutment characteristics as solid rock could be a source of erroneous stability prediction for the Winston Lake mine. Milne concluded, from comprehensive hangingwall monitoring at the Winston Lake mine, that treating moving backfill limits the same as rock abutments is overly optimistic. The Thompson and Winston
Lake mines are both examples of sites where apparent site-specific effects may have impacted adversely upon the ability of a generic stability graph to predict stability. However, it could be argued that these effects are not truly site-specific. Similar operational conditions would have resulted in erroneous predictions at any mine site and are therefore not truly site-specific.

5.3 THE EXTENDED MATHEWS STABILITY CHART

The Mathews Stability graph method was originally developed in 1980 as part of a CANMET report into stope stability in deep Canadian mines (Mathews 1981). As discussed in Chapter 2, a number of authors have added data (Potvin 1988; Trueman et al. 2000) and proposed modifications to the way the stability number is calculated (Potvin 1988; Sprott et al. 1999; Diederichs and Kaiser 1999; Kaiser et al, 2001). As discussed in Chapter 3 several authors suggest retaining the original formulation of the stability number proposed by Mathews (Stewart and Forsyth, 1995; Trueman et al., 2000). However, the modified stability number is the version most widely used both in industry and by researchers (Potvin and Hadjigeorgiou, 2001). Due to the absence of information required to determine the modified factor B, it has not been possible to apply nor test the validity of these modifications with respect to the Extended Mathews stability database of 485 case studies. For this reason, the original Mathews Stability graph formulation of N has been retained.

As discussed in Chapter 2, the Extended Mathews Stability chart stable-failure boundaries were determined using logistic regression. The advantages of delineating the stable failure boundary mathematically, as opposed to by eye, include; increased objectivity and the ability to quantify variance in the stable-failure boundary. Analysis of variance in the stable-failure boundary model parameters has facilitated a study to determine the database requirements for a reliable site-specific stability chart.

The logistic regression line defining the stable-failure boundary is defined by Equation 5.1 and Equation 5.2 (DeMaris, 1992; Mawdesley et al., 2001), where P(z) is the logit value. The logit value is analogous to the response variable in a linear regression model and is determined for each data point based upon the stability number N, the hydraulic radius S and the stability. Stability is the categorical response variable and is assigned a value. In ordinary binomial logit models, the categorical response variable would be assigned a value of 1 or 0. However, in the case of the stability graph, there are four categorical response variables; stable, failure, major failure and caving (Mawdesley, 2002). In order to incorporate these four response levels, the following values were assigned; stable points were set to 1, failures were set to 0.6 while major
failures and caving points were set to 0.3 and 0, respectively as noted by Mawdesley. The logit values are calculated using a MATLAB routine developed by Holtsberg (1998).

\[ z = B_1 \ln(N) + B_2 \ln(S) + B_3 \]  
\[ P(z) = \frac{1}{1 + e^{-z}} \]

To evaluate the stable-failure logistic regression line; \( P(z) \) is set to the logit value at the intersection of the cumulative probability function for stable points and the inverse cumulative probability function for failures. This represents the logit value that separates stable points and failures with the least amount of error. Using Equation 5.2, it is then possible to evaluate the prediction, \( z \) and substitute this value into Equation 5.1 to determine the stable-failure logistic regression line. The logit model parameters \( B_1, B_2 \) and \( B_3 \) are determined using the maximum likelihood function contained within the Matlab procedure logitfit (Holtsberg 1998).

In the case of the generic stable-failure logistic regression line 81 percent of stable points correctly reported to the stable zone, while 84 percent of the unstable points correctly report below the stable-failure boundary. The extended Mathews Stability Graph is best thought of as a three-dimensional probability surface, where the probability of a case history being stable, or unstable, is defined by its position in the two-dimensional graph space. It is possible to evaluate the probability of stability or failure at any point on the graph.

5.4 ANALYTICAL TECHNIQUE

The logit model framework used to determine the Extended Mathews stability boundaries has been used to determine the database requirement for a site-specific stability chart. By analysing variance in logit model parameters it has been possible to examine the effect that the number of case studies and type of case studies has on the stable failure boundary reliability. Appendix A contains the Extended Mathews Stability Graph database used for this study.

5.4.1 Previous Limitations

The logit model predicts a three dimensional surface of probabilities, not a two-dimensional boundary. In the case of logistic regression there is no agreed upon technique to quantify the quality of the logistic regression line (Whiten, 2001). Logistic regression does not have a
technique equivalent to the analysis of variance approach that would be used for a least squares regression model (Devore, 1991). A new statistical technique has therefore been developed to analyse the effect of changing database parameters on model reliability.

The MATLAB procedure logitfit (Holtsberg, 1998) evaluates confidence intervals for model parameters $B_1$, $B_2$ and $B_3$. However, logitfit does not provide confidence intervals for the logit models defined by Equation 5.2. The author attempted to produce confidence intervals for the stable-failure logit boundary using the Monte Carlo simulation function in the @Risk computer program (Palisade, 1996). The Monte Carlo simulation uses random combinations from the probability distributions of each model parameter to produce a stochastic model. The results of the Monte Carlo simulation produced impossibly large stability numbers. The impossibly large stability numbers produced by the model indicates the existence of mathematical dependencies between $B_1$, $B_2$ and $B_3$. Extensive plotting of model parameters indicates model parameters are mutually dependent. It was therefore concluded that due to dependencies between the model parameters, $B_1$, $B_2$ and $B_3$, a Monte Carlo simulation could not be used to determine confidence limits for the stable-failure boundary.

5.4.2 Quantifying Logit Model Reliability

Defining the relationships between the model parameters was beyond the scope of the research project. Therefore, an alternative analytical technique was required. For this reason, an analysis technique to facilitate a parametric study of the effect of database parameters on logit model reliability was developed. Model parameter variance has been analysed for the purpose of quantifying logit model reliability. The technique analyses the variance of logit model parameters, $B_1$, $B_2$ and $B_3$ for different database scenarios.

Ten random samples were taken from the generic database to represent each database scenario. The mean and standard deviation was calculated for each set of ten random samples. Trends in the variance were analysed using normalised standard deviation. Standard deviation has the same units as the mean, and is therefore normalised by division by the mean. By definition, minimising the variance in model parameters means further increases to the size of the database will not improve the reliability of the stable-failure boundary. When the gradient of normalised standard deviations of $B_1$, $B_2$ and $B_3$ level out, there are no further gains to be made by increasing the size of the database. This analytical technique provides a method for comparing the reliability of various database scenarios.
5.4.3 Assumptions

The technique assumes the generic database, from which random samples were taken, is sufficiently large to not invoke trends in the variance. For example, if the database had been comprised of only 250 points and ten sets of 200 samples were taken, one would expect the variance to be smaller than if ten sets of 100 random samples were taken. This assumption was tested. The results obtained were consistent with the hypothesis that the variability in $B_1$, $B_2$ and $B_3$ will decrease at a greatly reduced rate once the database size reached a critical size. The much smaller ongoing decreases in variability with increasing sample size can be attributed to sample size approaching that of the database. This indicates the generic database was sufficiently large to observe the hypothesised relationships between model parameter variance and database sample scenario. If the database had been too small, the levelling out of the normalised standard deviation would not have been apparent and the technique would not have enabled any clear results to be obtained. A second assumption of the technique is that the generic 485 case history database is a representative sample of the population of all stope surfaces from which data could, theoretically, have been collected. The generic database comprises of data from 35 mines, including the broad range of open stoping variations used in Australia, Canada and the United Kingdom and from a broad range of geotechnical environments and stope dimensions. However, it is not possible to quantify, in absolute terms, whether the generic database represents the total stope surface population. Nevertheless, the range of data and variety of sources of data contained in the generic database ensures that it is a reasonable representation.

The logistic regression line variance is also affected by the value of $p$, as defined in Equation 5.1 and Equation 5.2. As the value of $p$ is not independent of $B_1$, $B_2$ and $B_3$, it was necessary to check the magnitude of $p$ value variance. The variance of $p$ was found to be less than 5 percent over all database scenarios. Therefore, it was possible to analyse the variance in $B$ with respect to database size independently of $p$. The variance in $p$ would be very small provided there is an even distribution of data. In particular, that there are sufficient points close to the stable-failure boundary.

5.5 RESULTS

5.5.1 Parametric Study of Database Requirements

Two database requirements were examined, the number of case histories in the database and the proportion of unstable points in the database. Figure 5.1, 5.2 and 5.3 illustrate trends in the
normalised standard deviation for \( B_1 \), \( B_2 \) and \( B_3 \), respectively. There is a clear change in gradient as random sample size levels out rapidly when random sample size is approximately 150 case histories. Thus, analysis of variance in logit model parameters indicates approximately 150 case histories are required to minimise variance in model parameters. The ongoing small decrease in parameter variance beyond 150 case histories can be attributed to the sample size approaching the size of the database, such that the normalised standard deviation reaches zero when the sample is the database.

A similar analysis examined the effect of the proportion of unstable points, as a percentage of the total number of points, on model parameter variance. Each point in Figure 5.4 represents statistics for the logistic regression results of ten random samples of 200 case histories. Trends in the normalised standard deviation indicate that between 10 and 12 percent unstable points are required to minimise variance in model parameters. Minimised normalised standard deviations range between 7 percent and 10 percent for each of the model parameters. In this case, the normalised standard deviation approaches a semi-asymptote. Parameter normalised standard deviations fluctuate about this semi-asymptote due the interdependence of model parameters.

![Figure 5.1 – Effect of sample size on normalised standard deviation of \( B_1 \)](image-url)
Figure 5.2 – Effect of sample size on normalised standard deviation of $B_2$

Figure 5.3 – Effect of sample size on normalised standard deviation of $B_3$
Chapter 5 – Site-specific Effects

Figure 5.4 – Effect of proportion of unstable points on normalised standard deviation

An important observation was made when plotting variously sized random database logistic regression lines. It was found that the variance in the model parameters $B_1$, $B_2$ and $B_3$ has greatest effect for regions of the chart beyond the range of the data. Based upon this observation, it is recommended that stable-failure boundaries should not be extended beyond the range of the data. In addition, as shown in Figure 3.12 the Extended Mathews stable-failure boundary is characterised by excellent data coverage. It is therefore believed that an additional database requirement is data coverage.

5.6 MT CHARLOTTE – CASE STUDY

The generic Extended Mathews Stability Graph (Trueman et al, 2000; Mawdesley et al, 2001; Trueman and Mawdesley, 2003) gives reasonable predictiveness for site-specific data, although not quite as reliable as a site-specific logistic regression line. Table 5.1 compares the reliability of the site-specific stable-failure boundary to the reliability of two variations of the generic stable-failure boundary. The generic boundary including Mt Charlotte is the logistic regression boundary for the entire 485 case history database, while the generic boundary excluding Mt Charlotte is the result of logistic regression of the remaining 272 case histories. Mt Charlotte was excluded from the database to remove potential bias in the generic database. Sensitivity is defined as the percentage of stable points that report correctly to the stable zone, while specificity is defined as the number of case histories that report correctly to the unstable zone (Parker and Davis, 1999). The site-specific boundary resulted in a 3.7 percent increase in
the sensitivity and a 3.9 percent increase in the specificity compared to the generic boundary including Mt Charlotte. In the case of the generic boundary excluding Mt Charlotte data, the increase in sensitivity was 8.0 percent and the increase in specificity was 8.6 percent.

Figure 5.5 illustrates that there is a small difference between the two generic regression lines. Comparative statistics analysis was utilised to determine if this difference is statistically significant. In the case of the Mt Charlotte model, comparative statistics indicated, with 95% confidence, that no significant difference between Mt Charlotte, B₁ and B₃ values and the generic case, B₁ and B₂ values. However, there is a 98 percent probability Mt Charlotte model parameter B₂ is significantly different from the generic logit model parameter B₂.

<table>
<thead>
<tr>
<th></th>
<th>Mt Charlotte Boundary</th>
<th>Generic Boundary -Including Mt Charlotte data</th>
<th>Generic Boundary -Excluding Mt Charlotte data</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Sensitivity</strong></td>
<td>83.3%</td>
<td>79.6%</td>
<td>75.3%</td>
</tr>
<tr>
<td>(Percentage of Mount Charlotte stable case histories correctly reporting to stable zone)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Specificity</strong></td>
<td>73.1%</td>
<td>69.2%</td>
<td>64.5%</td>
</tr>
<tr>
<td>(Percentage of Mount Charlotte unstable case histories correctly reporting to unstable zone)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Figure 5.5** - Mt Charlotte site-specific stable-failure boundary compared to the generic stable-failure boundaries
Chapter 5 – Site-specific Effects

Table 5.2 summarises the results of the comparative statistical analysis. Due to the significant difference detected for parameter $B_2$, it was decided to investigate further. An investigation was made as to whether this level of significant difference could be obtained between a sample and the source database. If so, then this would be evidence that the $B_2$ 98 percent significance test result could have been a type I error. A type I error occurs when the difference appears to be significant when it is not (Walpole and Myers, 1990). Table 5.2 contains the results of three comparative statistical analyses for three random samples of 150 case studies from the generic database excluding Mt Charlotte data. These results show that at the 90 percent level of confidence, there are three type I errors out of the nine comparisons conducted. We know they are type I errors because the samples come from the database and there should be no significant difference.

<table>
<thead>
<tr>
<th>Model Parameter</th>
<th>Probability sample of 150 cases from generic database excluding Mt Charlotte is significantly different to Generic excluding Mt Charlotte</th>
<th>Probability Mt Charlotte database is significantly different to Generic excluding Mt Charlotte</th>
</tr>
</thead>
<tbody>
<tr>
<td>$B_1$</td>
<td>79%</td>
<td>93% 93% 83%</td>
</tr>
<tr>
<td>$B_2$</td>
<td>98%</td>
<td>55% 63% 61%</td>
</tr>
<tr>
<td>$B_3$</td>
<td>72%</td>
<td>91% 86% 75%</td>
</tr>
</tbody>
</table>

Based upon these results it can be concluded that the 98 percent result for $B_2$ was a type I error and therefore does not indicate a significant difference between Mt Charlotte and the generic database. It is concluded that the apparent site-specific effects contained in previous literature are attributable to operating conditions inadequately represented in the database and are therefore not truly site-specific.

5.7 CANNINGTON MINE – CASE STUDY

The aim of this case study is to illustrate the effect of an inadequate database on the stable-failure boundary. At the time of this study the Cannington mine stoping database contained 85 case histories, of which only 8 percent were unstable. Therefore, Cannington mine had insufficient data to meet the database requirements. The total number of case histories is insufficient, as is the proportion of unstable case histories. The site-specific logistic regression
model for the Cannington mine data did not improve the misclassification compared to the generic stable-failure boundary. In fact, the site-specific boundary produced one more misclassification than the generic stable failure boundary. Figure 5.6 compares the Cannington mine site-specific stability boundary to the generic boundary. If the database requirements had been met, there may have been an improvement in the reliability of the site-specific stable-failure boundary. However, with an inadequate database it is impossible to make any conclusions about the site-specific effects at this site. However, the site-specific stability boundary does appear to predict that unrealistically high rock mass strengths are required for larger stope dimensions.

![Figure 5.6 – Cannington mine stable-failure boundary compared to generic boundary](image)

**5.8 NARROW-VEIN OPERATING CONDITIONS**

As discussed in Chapter 4 there are a number of parameters or narrow-vein operating conditions with the potential to make the use of existing empirical models problematic for narrow-vein stope design. A number of parameters have been hypothesised to have a significantly larger influence upon narrow stope stability than on large open stope stability. For the purpose of clarity these parameters have been termed narrow-vein operating conditions. The following narrow-vein operating conditions have the potential to significantly affect the reliability of stability graphs in predicting narrow-vein dilution:

- Drilling and blasting.
- Moving backfill abutments.
− Stress relaxation.
− Stress damage.
− Complex geometry and in-stope pillars.
− Undercutting of stope walls.

In an effort to address the issue of narrow-vein operating conditions affecting stability chart reliability, some sites have attempted or considered developing their own stability graphs. However, the time required to collate 150 case studies for a reliable site-specific stability graph would result in a substantial delay. Furthermore, at the feasibility stage, estimates of dilution are frequently required prior to the possibility of developing a site-specific stability graph.

Based on the results of this study apparent site-specific effects can be attributed to operating conditions inadequately represented in stability chart databases, and are therefore not truly site-specific. Based upon these findings and the practical difficulty of collecting 150 case histories for a site-specific chart, development of site-specific stability charts are unlikely to be a practical solution to the issue. This is true even if the six parameters identified in Chapter 4 as being to some extent implicitly taken into account by Q’ could be calibrated to narrow-vein conditions by developing a site-specific stability chart. In Chapter 7 the validity of the implicit inclusion theory is discussed with respect to 115 narrow-vein case studies from the Barkers mine.

To overcome these issues, it has been proposed that rather than attempt to develop site-specific stability charts a more practical solution is to develop a chart or methodology to take into account narrow-vein operating conditions. The underlying premise for this strategy is that existing stability charts are in fact problematic for narrow-vein stability prediction. In Chapters 6, 7 and 8 the narrow-vein operating conditions hypothesised to affect the reliability of stability charts for narrow-vein are investigated empirically.

5.9 CONCLUSIONS

A new technique that analyses trends in the variances of three logit model parameters has enabled the author to define database requirements that minimise the variance in the Mathews Stability Graph method stable-failure boundary.

The parametric study into the database requirements indicates 150 case histories, of which at least 10 percent must be unstable, are required to minimise the variance in a stable-failure
boundary. These requirements can be used as a guide for developing site-specific stable-failure boundaries and as a check that sufficient case histories have been collected for a generic model. These requirements apply only within the range of the current database. In terms of validation of these conclusions, the Cannington mine case study illustrated the importance of meeting the database requirements (minimum number of case histories and stope failures) before a reliable site-specific boundary can be delineated.

The Mt Charlotte mine case study suggests that the Extended Mathew Stability Graph method predictive capability may be slightly improved by using a site-specific graph that meets the database requirements. However, despite the small improvement in predictive capability noted for a Mount Charlotte site-specific variant, comparative statistics indicated no significant difference between the Mt Charlotte and generic logit models. Therefore, the Mt Charlotte case study indicates that site-specific effects may be insignificant. Further case studies are required to confirm this conclusion.

When operating conditions are significantly different from those captured in the current generic database (as appears to be the case for longhole narrow-vein operating conditions), an operating condition specific stability chart or alternative methodology may provide a method for capturing these effects. In Chapters 6 though to 8, the possibility of developing an operating condition specific stability chart for longhole narrow-vein stoping has been evaluated.
Stress relaxation is one of the narrow-vein operating conditions hypothesised to adversely effect narrow-vein stope stability prediction using existing stability chart methods. While many authors refer to the adverse effect of stress relaxation on excavation stability, some authors present compelling empirical evidence indicating that stress relaxation does not have a significant effect. Establishing clear definitions of stress relaxation was critical to understanding and quantifying stress relaxation effects. Three types of stress relaxation have been defined: partial relaxation, full relaxation and tangential relaxation. Once clear definitions were determined, it became clear that the theoretical arguments and empirical evidence presented by various authors to support their respective cases are not contradictory; rather the different conclusions can be attributed to different types of stress relaxation. In particular, when the minor principal stress is negative the intermediate principal stress has been identified as significantly affecting jointed rock mass behaviour.

The aim of the study was to review and evaluate existing methods of quantifying the effect of stress relaxation around underground excavations and, if necessary, propose a new set of recommendations. A new set of guidelines to account for the effect of stress relaxation within the stability chart approach has been proposed based on back-analysis of 55 case histories of stress relaxation.

6.1 INTRODUCTION

Due to high length to width aspect ratios, narrow-vein stopes have a propensity to low or tensile induced stresses, especially when the principal stress is perpendicular to strike. For this reason stress relaxation is one of the narrow-vein operating conditions with significant potential to reduce the reliability of the stability chart for narrow-vein stopes. While it can be argued that stress relaxation may to some limited extent be taken into account implicitly by Q’, the high proportion of narrow-vein stopes with relaxed hangingwalls and footwalls justifies the separate consideration of stress relaxation for narrow-vein stopes. In this chapter stress
relaxation has been investigated with the view to establishing a methodology for explicit consideration of stress relaxation effects.

Several authors refer to the adverse effect of stress relaxation on excavation stability (Bawden, 1993; Kaiser et al., 1997; Diederichs et al., 1999; Kaiser et al., 2001; Suorineni et al., 2001). While stress relaxation is commonly recognised as having the potential to affect excavation stability, the conditions under which stress relaxation impacts stability appear to be less well understood. Furthermore, some empirical evidence suggests stress relaxation may not have a significant effect on excavation stability (Potvin, 1988; Tyler et al., 1993; Wang et al., 2002).

This study of relaxation investigates the hypothesis that the discrepancies between empirical evidence and existing two-dimensional analyses can be attributed to the influence of the intermediate principal stress, and or, minimum principal stress direction. Therefore, a three-dimensional method of analysis was required that could consider the effect of different types of stress relaxation.

For the first time three types of stress relaxation have been analysed; partial relaxation, full relaxation and tangential relaxation. Partial relaxation has been defined as excavation surfaces where linear elastic three-dimensional modelling of $\sigma_3$ is less than 0.2 MPa, while $\sigma_2$ and $\sigma_1$ both exceed 0.2 MPa. Full relaxation has been defined as excavation surfaces where linear elastic three-dimensional model estimates of $\sigma_3$ and $\sigma_2$ are both less than 0.2 MPa. Tangential relaxation has been defined as excavation surfaces where at least one of the modelled principal stresses is less than 0.2 MPa and the associated stress direction diverges less than 20 degrees from parallel to the excavation wall in a three-dimensional analysis. It is important to note that in the case of three-dimensional analysis, the angle between the associated stress direction and the stope surface must be determined both with respect to the stope surface dip and the stope surface strike. If the angle subtended by the stress direction and the stope surface dip or strike is less than 20 degrees, then this stress direction has been considered ‘tangential’.

Mathews et al. (1981) distinguished between ‘partial relaxation’, when $\sigma_3$ is less than zero, and ‘full relaxation’, when both $\sigma_3$ and $\sigma_2$ are less than zero. Prior to the wide availability of the computer capacity required to run three-dimensional elastic models, Potvin (1988) and Pakalnis (1986) refer to relaxation generally, without distinguishing between partial and full relaxation. This was also the case with Suorineni (1998). On the other hand, Kaiser et al. (2001) use the three-dimensional linear elastic boundary element computer program Map3d to
analyse stress relaxation. The impact of switching between two-dimensional and three-dimensional stress modelling on the definition of relaxation has been considered in this paper.

6.2 STRESS RELAXATION

6.2.1 Theoretical Basis
Coulomb’s (1776) shear strength criterion provides a strong theoretical basis for the adverse effect of stress relaxation on excavation stability. As illustrated in Figure 6.1, Coulomb’s shear strength criteria is a two-dimensional analysis in which the shear strength \( \tau \) is determined from the normal stress, \( \sigma_n \) (obtained by transforming \( \sigma_1 \) and \( \sigma_2 \)), cohesion (c) and friction angle (\( \phi \)):

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

Equation 6.1

Figure 6.1– Mohr-Coulomb shear strength criterion

Suorineni (1998) undertook a parametric study to examine the critical clamping stresses for a range of wedge types and friction angles using the Mohr-Coulomb slip failure criterion. The parametric study examined rhombohedral, rectangular and symmetrical prism block shapes, with varying joint dip and joint spacing. Suorineni (1998) determined that the critical range of clamping stresses is approximately 0.01 to 0.2 MPa for wedges ranging from 0.5 metres to 3 metres. Based upon this analysis, stress relaxation has been taken to have potentially destabilising effects when the induced stress is less than 0.2 MPa.
6.2.2 Types of Stress Relaxation

Diederichs and Kaiser (1999) illustrate eight different causes of relaxation. It is possible to separate the causes of relaxation illustrated by Diederichs and Kaiser, (1999) into those prone to elastic relaxation, and those with a high potential for inelastic relaxation. Figure 6.2 illustrates scenarios with a high potential for elastic relaxation.

Figure 6.2 - Elastic stress relaxation a) adverse stress ratio b) stress shadow c) adverse stress ratio d) intersection e) concave geometry, after Deiderichs and Kaiser (1999)

Figure 6.3 illustrates scenarios with high potential for inelastic stress relaxation. In each of the scenarios represented in Figure 6.3, there is high potential for the large strains associated with the inelastic displacement mechanisms such as; unravelling, delamination, crushing and splitting. Due to the absence of sufficient case studies where actual displacements and stresses were measured, relaxation attributable to inelastic displacement has not been considered in this thesis.
Chapter 6 – Stress Relaxation

6.3 METHODS OF ANALYSING THE EFFECTS OF STRESS RELAXATION ON EXCAVATION STABILITY

Numerical, analytical and empirical approaches have in the past been used to examine the effect of stress relaxation on excavation stability.

6.3.1 Distinct Element Model

There are two numerical approaches that can be used to estimate the effect of stress relaxation on an excavation. The first approach is based upon a distinct element type model where the stability of discrete blocks is a function of resolved stresses and discontinuity frictional properties (Voegele et al., 1978; Kaiser et al., 2001; Brown, 2002; Jing, 2003). For example, Voegele et al., (1978) used the two-dimensional distinct element approach illustrated in Figure 6.4 to show how low lateral stresses enable blocks to fall under the influence of gravity (Brown, 2002). Three-dimensional distinct element modelling appears well suited to an analysis of the effect of the different forms of relaxation on excavation stability. However, validation of such a model against real case studies requires assumptions about the shear strength of discontinuities (Brown, 2002), boundary conditions (Kaiser et al., 2001), and joint persistence. As discussed in Chapter 3 the impact of those assumptions on model results can be large.
6.3.2 Equivalent Continuum

The second numerical approach is based upon an empirical failure criterion and assumptions of an equivalent continuum. In this approach, empirical failure criteria are used within a numerical modelling package to predict the onset of rockmass failure.

In order to investigate the effect of full relaxation, using this approach, a three-dimensional empirical failure criterion would be required. However, three-dimensional rock failure criteria do not exist in the tensile quadrant (Sheory, 1997). Therefore, it is not possible to analyse the effect of full relaxation, where $\sigma_2$ is considered independently of $\sigma_3$, using an empirical failure criterion within an equivalent continuum type numerical model.
Brady and Brown (1993) highlight that because available data (intact rock testing) indicates that $\sigma_2$ has less influence on peak strength than the minor principal stress, $\sigma_3$, all of the criteria used in practice do not take into account $\sigma_2$. The back-analyses in this paper suggest that in jointed rock masses, $\sigma_2$ may have an impact on rockmass behaviour when $\sigma_3$ is tensile.

6.3.3 Voussoir Beam Analogy

Deiderichs and Kaiser (1999) used the voussoir beam analogy to develop a theoretical mechanistic model to examine the effect of stress relaxation on excavation stability. The theory is based upon the premise that rock bridges provide residual tensile strength that facilitates arching to the abutments. Diederichs and Kaiser (1999) propose that discrepancies between failure criterion predicted failure and actual failure is due to the tensile load carrying capacity of the rock mass. The voussoir beam analogy is also a two-dimensional analysis and therefore, can not be used to investigate the effect of full relaxation. As discussed in Chapter 3 the voussoir beam theory assumes that reaction moment at the abutments due to the half weight of the beam is equal at either end. However, this would not be true when the beam is vertical or sub-vertical. Therefore, from a mechanistic point of view it is inappropriate to use this voussoir beam theory as the underlying mechanism for stress relaxation related instability in longhole narrow-vein stopes. However, mechanistically the voussoir beam may be appropriate for relatively flat dipping stopes where the assumption of even reactions at the abutments is reasonable.

6.3.4 Empirical Stability Charts

The stability chart approach was chosen as the method of analysis due to the availability of case studies, and also due to the fact that the calculation of stability number, $N$ effectively quantifies other parameters that may influence stability such as stope size, orientation and the orientation of major discontinuities. If these parameters are not taken into account, it is difficult to ascertain the true cause of instability. Therefore, if a stope plots incorrectly according to the stability graph a possible reason could be stress relaxation. It follows that if the number of failed stopes plotting above the stable-failure boundary is significantly higher than the rest of the case studies for a certain type of stress relaxation there must be some explanation. The analysis conducted in this paper investigates whether misclassification of failed stopes could be due to a particular type of stress relaxation.

Figure 3.2 is used to determine the stress factor $A$ for a given strength to stress ratio (Mathews et al., 1981). In the original Mathews Stability Graph method, Mathews et al. (1981)
recommend that where the ratio of the induced stress, \( \sigma_I \) to the vertical stress, \( \sigma_V \) is negative, \( \sigma_I \) be set to zero. Furthermore, Mathews et al. (1981) recommend that, as the ratio of \( \sigma_C \) to \( \sigma_I \) is greater than 10 for these cases, the stress factor \( A \) should be set to one. Despite this recommendation, Mathews et al. (1981) note that horizontal joints intersecting the hangingwall will open as the induced stress at the centre of the hangingwall span is tensile. However, Mathews et al. (1981) suggest that when the ratio of \( \sigma_C \) to \( \sigma_I \) is greater than 10 any failure is related to movement on defined structures only, and for these cases \( A \) is set to 1. Despite noting that hangingwall and footwall overbreak was found to occur predominantly within the de-stressed zone, Mathews et al. (1981) suggest that rock mass quality is probably the main control and rock stress only a minor factor.

### 6.3.5 Modifications to the Stress Factor to Account for Stress Relaxation

Deiderichs and Kaiser (1999) related their version of the two-dimensional voissoir beam model to the Modified Stability number, \( N' \) by assuming;

\[
Q' = \frac{N'}{0.6} \quad \text{Equation 6.2}
\]

when \( A \) equals one in cases of low stress and \( C \) equals two in the case of back or roof of excavation, Young’s modulus,

\[
E_{\text{rockmass}} = 5\sqrt{Q'} \approx 6.5\sqrt{N'} \quad \text{Equation 6.3}
\]

and

\[
N' = 150 \times (\text{BeamThickness})^3 \quad \text{Equation 6.4}
\]

Diederichs and Kaiser (1999) acknowledge that their calibration procedure is somewhat subjective. Diederichs and Kaiser’s calibration procedure uses the apparent shift in the Modified Stability number, \( N' \) associated with the undocumented tensile stresses shown in Figure 6.5 to derive the following adjustment for \( A \) in cases of stress relaxation;

\[
A = 0.9e^{11(\sigma_T/\text{UCS})} \quad \text{when } \sigma_T < 0 \quad \text{Equation 6.5}
\]
where $\sigma_T$ is the induced stress at the centreline of the excavation. Although not specified in the paper, the induced stress referred to in Equation 6.5 appears to relate to a two-dimensional stress analysis.

Diederichs and Kaiser (1999) calibrated their modification to the stress factor $A$ using data from the Thompson vertical crater retreat mine (Greer, 1989; Bawden, 1993) where stope surfaces would be predicted to be relaxed using a two-dimensional stress analysis. Diederichs and Kaiser (1999) highlighted that stope surface stability was poorly predicted for this mine using the Mathews stability graph approach and concluded that this was due to stress relaxation which was not accounted for in the original formulation of the stress factor $A$. They demonstrated that by using Equation 6.5 to account for stress relaxation the prediction of stope stability was significantly improved for this mine using the Mathews stability graph method. Diederichs and Kaiser (1999) used the analysis to both justify their modifications to the Mathews stability graph formulation and to illustrate that stress relaxation was damaging to excavation stability. However, Greer (1989) stated that the over prediction of hangingwall stability for the Thompson mine using the Mathew’s stability graph approach was due to factors such as blast damage of stope walls, the supporting effect of sand fill in an adjacent block, or possibly the stopes remained open for too long. This mine utilised an early version of the vertical crater retreat mining method that was characterised by high confinement and high...
powder factors. This suggests that there are alternate explanations, aside from stress relaxation, for the poor correlation between Thompson mine hangingwall case studies and the Modified Stability graph. Therefore, the apparent shift in $N'$ used by Diederichs and Kaiser (1999) to calibrate their adjustment for stress relaxation may be attributable to the operating conditions at the Thompson mine, rather than stress relaxation. Stewart and Trueman (2001) have noted the impact of differing operating conditions on the predictive capability of stability graphs.

The conclusions of Diederichs and Kaiser (1999) concerning the effect of stress relaxation on stope stability were in direct contrast to the analyses carried out by Potvin (1988) using the Mathews stability graph approach. Potvin (1988) concluded from back-analyses of stope predicted to be relaxed using a two-dimensional stress analysis, that stress relaxation does not affect stope stability and therefore, the stress reduction factor $A$ should be set at 1.

From the above discussion it can be concluded that there remains controversy relative to the effect of relaxation upon open stope stability. A database of relaxed stope surfaces was back-analysed using the framework of the Extended Mathews stability chart as will be discussed in subsequent sections.

The stability chart approach was used as the framework for analysis because this framework takes into account other factors affecting stability and this enables the effect of stress relaxation to be detected with less case studies than may have been required if stress relaxation had been considered in isolation. The Extended Mathews Stability chart was selected over other stability charts for the following reasons:

1. More than double the number of case studies of any of the other stability charts
2. The boundaries between stability categories were determined using and objective logistical method.
3. Furthermore, the information to determine the modified factor $B$ (Potvin, 1988a) was unavailable.

6.4  **CASE STUDIES OF RELAXED STOPE SURFACES**

The relaxation case studies analysed in this paper were collated from the literature (Mathews et al., 1981; Pakalnis, 1986; Potvin, 1988; Pakalnis et al., 1991; Pine et al., 1992; Dunne et al., 1996). The database includes stope walls where the minimum principal stress was less than 0.2 MPa when modelled using the three-dimensional linear elastic boundary element package
Chapter 6 – Stress Relaxation

Table 6.1 contains a summary of the relaxation case studies analysed. Appendix A contains the relaxed case studies database. Some case histories cited in the literature as relaxed were omitted from the database either because the stope surface was supported with cablebolts, or because three-dimensional modelling suggested that the stope was not relaxed.

<table>
<thead>
<tr>
<th>Mine</th>
<th>Number of Case Studies</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ruttan mine -isolated stopes (Pakalnis, 1986; Potvin, 1988)</td>
<td>42</td>
</tr>
<tr>
<td>Ruttan mine 800-26J stope (Pakalnis et al., 1991)</td>
<td>2</td>
</tr>
<tr>
<td>Detour Lake mine (Pakalnis et al., 1991)</td>
<td>2</td>
</tr>
<tr>
<td>South Crofty mine (Pine et al., 1992)</td>
<td>1</td>
</tr>
<tr>
<td>Cobar mine (Mathews et al., 1981)</td>
<td>8</td>
</tr>
</tbody>
</table>

When modelled in three dimensions, only 25 out of the 55 case studies are tangentially relaxed; i.e. the minimum principal stress is parallel to the stope surface and less than 0.2 MPa in magnitude. However, it is important to note that all 55 case studies are tangentially relaxed when modelled in two dimensions. Similarly, when modelled in three dimensions only 20 of the 55 case studies are fully relaxed; i.e. at least two of the principal stresses are less than 0.2 MPa. When evaluating the potential for stress relaxation, the choice of three-dimensional modelling will impact upon the modelled state of relaxation.

The model framework in the Mathews method was developed using two-dimensional stress analysis. However, this may not be adequate when dealing with stress relaxation. For some stope geometries a two-dimensional stress analysis will predict that the rock mass in the vicinity of an excavation is relaxed, but it may not be when a three-dimensional stress analysis is performed. In such a case, the stope surface will not be truly relaxed. Therefore, the recommendations put forward in this paper to predict the effect of stress relaxation are relevant only to cases of stress relaxation identified using three-dimensional linear elastic modelling.

6.5 STRESS MODELLING

The author undertook both two-dimensional (Phase 2) and three-dimensional numerical modelling (Map3d) software were used to estimate linear elastic stresses in the centre of each of the 55 case studies’ stope walls (Appendix B). Stopes were modelled as isolated stopes. The two-dimensional finite element package Phase 2 was used to evaluate $\sigma_3$ in both vertical
and horizontal planes for all 55 case studies. Figure 6.6 is an example of Phase 2 model for the 320 15H stope at Ruttan mine, and illustrates $\sigma_3$ is tensile stress in both the hangingwall and footwall. The three-dimensional boundary element program Map3D was used to estimate linear elastic stresses at the mid-point of stope walls. Figure 6.7 is an example of a Map3D model of South Crofty mine, and illustrates how $\sigma_3$ is tensile stress in the hangingwall. The two-dimensional finite element package was used to estimate linear elastic stresses in both the vertical and horizontal planes. Linear elastic stresses were evaluated for all 55 case studies using both Map3D and Phase2. Comparison of Map3D and Phase2 results demonstrated the previously observed (Potvin et al., 1988; Pakalnis, 1991) large differences between two-dimensional and three-dimensional modelling for aspect ratios less than five.

While the stress normal to an excavation is equal to zero, this does not mean that the $\sigma_3$ is always normal to an opening. While closed form analytical linear elastic stress solutions always show $\sigma_3$ being equal to 0 and normal to an opening surface, numerical solutions shows that $\sigma_3$ is not always perpendicular to a excavation boundary. Linear elastic stresses are used as an index of stress levels in the wall of the stope. In reality stress levels are likely to be have lower absolute values than indicated by linear elastic modelling. Actual stress levels in stope walls would be lower because movements within the rockmass facilitates dissipation of stress so that the magnitude of both tensile and compressive stress is less than indicated by linear elastic modelling.

A limitation of linear elastic modelling is that it assumes that the Young’s modulus is constant even after the rock mass would has failed. Non-linear modelling means that stresses can be redistributed once a rock mass have failed. While non-linear modelling packages are available, these packages require some type of model for defining the stress levels under which the rock mass will fail. The input parameters for the various rock mass failure criteria such as Hoek-Brown require estimates of rock mass parameters e.g. $m_b$ and $s$. Due to the uncertainty associated with estimating rock mass failure criterion parameters, it has become common practice in mining rock mechanics empirical models to use linear elastic stresses as index of stress (e.g. Q System and Stability Graph Methods databases). An important exception is the case of an operating mine where failure criterion input parameters are calibrated to underground observations of rock mass damage.

All modelling undertaken for this thesis assumes the rock mass is homogenous and behaves isoptropically. This can affect the accuracy of stress estimates on a case-by-case basis.
Chapter 6 – Stress Relaxation

However, it can be expected that across each of the databases analysed the errors are unbiased and random, and are therefore unlikely to affect the results and conclusions even if the stress magnitudes for individual stopes are not accurate.

Figure 6.6 – Example of Phases2 modelling results, Ruttan mine Stope 320 15H (in this case $\sigma_3$ is parallel to the stope wall)

Figure 6.7 – Map3d modelling South Crofty mine, illustrates $\sigma_3$ stress distribution for typical narrow-vein geometry prone to stress relaxation (grey areas $\sigma_3<-2$)
6.6 ANALYSIS OF RESULTS

The framework of the Extended Mathews stability chart was used to investigate the effect of three different types of stress relaxation on stability. The following three types of stress relaxation were investigated; partial relaxation, full relaxation and tangential relaxation. The misclassification statistics, sensitivity and specificity were used to examine the effect of each type of stress relaxation. Sensitivity is defined as the probability that a true case will be correctly classified (Parker et al., 1999). Therefore, with respect to the Extended Mathews stable-failure boundary, sensitivity refers to the proportion of stable case studies that correctly plot above the stable-failure boundary. Conversely, specificity is the probability that an unstable case study will correctly plot below the stable-failure boundary. Parker and Davis (1999) define the sum of sensitivity and specificity as the accuracy of the test classification.

6.6.1 Partial Stress Relaxation

The effect of partial relaxation has been analysed using both misclassification statistics and statistical analysis of logit model parameters.

**Misclassification Statistics**

Figure 6.8 illustrates trends in misclassification statistics for the partially relaxed case studies plotted on the Extended Mathews stability chart. The analysis suggests partial stress relaxation, when quantified in terms of minimum principal stress, is a poor predictor of stability. The significant number of stable partially relaxed case studies provide evidence of this. In total, 25 out of 55 partially relaxed case studies are stable. Therefore, partial relaxation, when considered in isolation from other parameters, does not appear to be a good predictor of instability. In terms of a two-dimensional analysis, arching to the abutments facilitated by the residual tensile strength of the rock mass as proposed by Diederichs and Kaiser (1999) is a compelling explanation as to why a loss of confinement in one direction does not necessarily result in excavation failure.

However, Figure 6.8 illustrates that when the minimum principal stress drops below –0.5 MPa, the specificity of the Extended Mathews stable-failure boundary appears to decrease markedly. In practical terms, this suggests that significant numbers of partially relaxed failures are incorrectly plotting in the stable zone. At first appearance this seems to indicate that partial relaxation below –0.5 MPa has the potential to cause instability in stopes that would otherwise have been stable.
Figure 6.8 - Effect of minimum principal stress on sensitivity and specificity

Figure 6.9 plots relaxed case studies with a minimum principal stress less than \(-0.5\) MPa with respect to the Extended Mathews stable-failure boundary. While only one out of the five failures plots correctly below the stable-failure boundary, two case studies are very close to correctly plotting as failures.
Due to the small number of case studies with minimum principal stress less than −0.5 MPa, specificity is very sensitive to these two case studies. If these two case studies had plotted slightly lower in the failure zone, then the specificity would have been 60 percent which is considerably better than 20 percent specificity obtained when these points plot above the line. Therefore, although the initial analysis of misclassification statistics shown in Figure 6.8 appears to indicate a decrease in specificity with decreasing minimum principal stress, the observed trend is very sensitive to the two case studies plotting just above the stable-failure boundary as shown in Figure 6.9. These two case studies plot in the same position and therefore, appear as a single point in Figure 6.9.

**Logistic Regression**

The Extended Mathews logit model framework has been used to test the significance of normalised tensile stress to stope stability using a new model termed the relaxation logit model. Tensile stress was normalised by dividing the minimum principal stress by the uniaxial compressive stress and incorporated into the relaxation logit model as follows:

\[
z = B'_1 \ln(N) + B'_2 \ln(S) + B'_3 \ln\left(\frac{\sigma_3}{\sigma_c}\right) + B'_4
\]

Equation 6.6

Only those case studies where the minimum principal stress is less than zero have been analysed. To avoid the non-real numbers produced by taking the log of a negative number, minimum principal stress is multiplied by negative one. The logit model fitting procedure requires real numbers. The model was run using the *MATLAB* procedure ‘logitfit’ (Holtsberg, 1998). The output from the logit procedure includes 95 percent confidence intervals for each of the coefficients, B’1, B’2, B’3 and B’4. If the 95 percent confidence interval for the coefficient B’3 passes through zero, then this is an indication the ratio of \(\sigma_3\) to \(\sigma_c\) is not significant to the stability outcome. Conversely, if the 95 percent confidence interval for B’3 does not pass through zero, this is an indication the ratio of \(\sigma_3\) to \(\sigma_c\) is significant to the stability outcome.

The 95 percent confidence interval for the coefficient B3 ranged from −0.8001 to 0.5536. As the 95 percent confidence interval for the coefficient B3 passes through zero, this is an indication the ratio of \(\sigma_3\) to \(\sigma_c\) is not significant to the stability outcome. However, this could in part be due to an insufficient number of case studies. The relatively high variance suggests there may have been insufficient case studies to gain a result that is statistically reliable. The 55 case studies analysed is considerably less than the 150 case studies required to determine a reliable stable-failure boundary – as determined in Chapter 5. Table 6.2 contains a comparison of the
variances of the Extended Mathews logit model and the relaxation logit model. It is clear the
variances of the relaxation logit model coefficients are significantly larger than those
determined for the 485 case history based Extended Mathews logit model. This is strong
evidence there was insufficient data to make conclusions about the significance of normalised
tensile stress to excavation stability using the logit model framework.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Relaxation Logit Model Coefficients</th>
<th>Variance</th>
<th>Extended Mathews Logit Model Coefficients</th>
<th>Variance</th>
</tr>
</thead>
<tbody>
<tr>
<td>ln(N)</td>
<td>B’1</td>
<td>-1.65</td>
<td>B’1</td>
<td>-1.44</td>
</tr>
<tr>
<td>ln(S)</td>
<td>B’2</td>
<td>0.74</td>
<td>B’2</td>
<td>0.79</td>
</tr>
<tr>
<td>ln(σ3/σc)</td>
<td>B’3</td>
<td>-0.12</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### 6.6.2 Full and Tangential Stress Relaxation

Full relaxation requires that at least two principal stress directions are less than 0.2 MPa.
Tangential relaxation is defined as a stress state where the minimum principal stress magnitude
is less than 0.2 MPa and the direction is less than 20 degrees from parallel to the excavation
surface. The relaxation database contains a subset of 20 cases of full relaxation and subset of
25 cases of tangential relaxation. It is important to note that 18 of the fully relaxed case studies
are also tangentially relaxed. Table 6.3 contains the sensitivity and specificity obtained for
each type of relaxation.

<table>
<thead>
<tr>
<th>Type of Relaxation</th>
<th>Sensitivity</th>
<th>Specificity</th>
<th>Accuracy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Not relaxed (Extended Mathews database)</td>
<td>81.3 %</td>
<td>83.6 %</td>
<td>1.65</td>
</tr>
<tr>
<td>Partial stress relaxation</td>
<td>83.3 %</td>
<td>79.1 %</td>
<td>1.62</td>
</tr>
<tr>
<td>Full stress relaxation</td>
<td>90.9 %</td>
<td>44.4 %</td>
<td>1.35</td>
</tr>
<tr>
<td>Tangential stress relaxation</td>
<td>85.7 %</td>
<td>45.5 %</td>
<td>1.31</td>
</tr>
</tbody>
</table>

As shown in Table 6.3, partial relaxation has misclassification statistics very similar to those
obtained for the non-relaxed Extended Mathews case studies. The misclassification statistics
contained in Table 6.3 were obtained with stress factor A equal to one. This suggests partial
relaxation does not affect nor cause excavation instability. By contrast, the low specificity obtained for cases of full relaxation indicates full relaxation has an adverse impact upon excavation stability. The practical consequence of this result is that in cases of full relaxation, where at least two principal stresses are less than 0.2 MPa, existing stability charts will frequently incorrectly predict a stable condition. In fact, the specificity obtained for cases of full relaxation suggests that the stability graph approach will be correct in less than half of cases. Similarly poor specificity was obtained for cases of tangential relaxation. In this case the specificity was 45.5 percent. This means that less than half the tangential relaxation failures correctly plotted in the failure zone.

There are 14 stable tangentially relaxed case studies. Therefore, while relaxation tangential to the stope wall increases the probability of failure, it does not necessary result in failure. As noted previously, arching to abutments provides a mechanistic explanation of why relaxation in one direction parallel to the excavation need not result in instability as might be suggested by an equivalent continuum type tensile failure criterion approach. As illustrated in Figure 6.10, arching facilitates stability even when there is minimal confinement perpendicular to the arch.

![Roman stone wall at Tarsus](image)

Figure 6.10 - Roman stone wall at Tarsus

18 out of the 20 fully relaxed case studies had either $\sigma_2$ or $\sigma_3$ sub-parallel to the stope wall. Therefore, 18 case studies belonging to the fully relaxed subset also belong to the tangentially relaxed subset. There were only six case studies of tangential relaxation that were not fully
relaxed. The sensitivity and specificity for these six case studies were both 67 percent. Due to the vast majority of case studies being both tangentially and fully relaxed, it is not possible to make conclusions regarding the underlying mechanism for increased instability associated with both tangential and fully relaxed case studies. More case studies would be required to consider the differences between full and tangential relaxation.

The effect of stress directions on the 20 fully relaxed case studies was also investigated. The sensitivity and specificity for the five fully relaxed stopes with both \( \sigma_2 \) and \( \sigma_3 \) sub-parallel to the stope wall were 100 percent and 33 percent, respectively. This compares to a sensitivity of 89 and specificity of 50 percent for the 15 cases of full relaxation with only one relaxed principal stress sub-parallel to the stope wall. While the specificity for full relaxation with both \( \sigma_2 \) and \( \sigma_3 \) sub-parallel to the stope wall was 17 percent lower than for cases where only one of the relaxed stresses is parallel to the stope wall, the small number of case studies makes it difficult to assess whether this is a significant difference.

### 6.7 EVALUATION OF EXISTING METHODS TO QUANTIFY THE EFFECT OF STRESS RELAXATION

Back-analysis has been used to evaluate the two existing approaches to quantifying the effect of stress relaxation using the Mathew’s stability graph framework. The first approach is to set \( A \) equal to one (Mathews et al., 1981; Potvin, 1988) Secondly, Diederichs and Kaiser’s (1999) adjustment to \( A \) defined by Equation 6.4 has also been applied to the relaxation case studies. Both two-dimensional (\textit{Phase2}) and three-dimensional (\textit{Map3D}) stress estimation packages were used to estimate the induced stress values required to calculate Diederichs and Kaiser’s adjustments to \( A \) for all 55 case studies.

Misclassification statistics were used to evaluate each method for the three types of relaxation. The results of the evaluation are contained in Table 6.4. In the case of partial relaxation, using stress factor \( A \) equal to one has a slightly better accuracy than Diederichs and Kaiser’s adjustment to \( A \) when using three-dimensional modelling (\textit{Map3D}) to estimate induced stresses. However, in the case of full and tangential relaxation, Diederichs and Kaiser’s adjustment to \( A \) provide similar levels of accuracy to stress factor \( A \) equal to one. The stress factors determined using three-dimensional modelling results ranged from 0.80 to 0.93. However, when two-dimensional induced stresses are used to calculate Diederichs and Kaiser’s adjustment to \( A \), as implied by these authors’ original work, the accuracy is very poor due to a sensitivity of zero percent. Two-dimensional modelling (\textit{Phase2}) produced minimum
induced stresses ranging from -17 MPa to -9 MPa, with a mean of -12.6 MPa. Using these induced stresses and uniaxial compressive strengths for each case study, stress factor A ranged from 0.02 to 0.30. In practical terms, this means that if Diederichs and Kaiser’s adjustment had been used in accordance with the implicitly suggested two-dimensional modelling none of the stable stopes would have correctly plotted above the stable-failure boundary.

Table 6.4 – Misclassification statistics for existing methods of quantifying the effect of stress relaxation

<table>
<thead>
<tr>
<th>Method to quantify effect of stress relaxation</th>
<th>Partial relaxation</th>
<th>Full relaxation</th>
<th>Tangential relaxation</th>
</tr>
</thead>
<tbody>
<tr>
<td>A=1 (Mathews et al., 1981, Potvin, 1988)</td>
<td>Sensitivity</td>
<td>90.0 %</td>
<td>90.9 %</td>
</tr>
<tr>
<td></td>
<td>Specificity</td>
<td>92.3 %</td>
<td>44.4 %</td>
</tr>
<tr>
<td></td>
<td>Accuracy</td>
<td>1.82</td>
<td>1.35</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Map3d</td>
<td>Phase2</td>
</tr>
<tr>
<td>$A = 0.9 \exp^{11x(\sigma_T/UCS)}$</td>
<td>Sensitivity</td>
<td>88.9 %</td>
<td>0 %</td>
</tr>
<tr>
<td>(Diederichs et al., 1999)</td>
<td>Specificity</td>
<td>85.7 %</td>
<td>94.1 %</td>
</tr>
<tr>
<td></td>
<td>Accuracy</td>
<td>1.74</td>
<td>0.94</td>
</tr>
</tbody>
</table>

6.8 NEW EMPIRICAL ADJUSTMENT FOR FULL AND TANGENTIAL RELAXATION

The poor specificity obtained for cases of full and tangential relaxation necessitates the development of new adjustments for these types of stress relaxation. Three separate approaches were examined. The first approach was to empirically develop an adjustment based upon the hypothesis that the adjustment magnitude would be related to the normalised tensile stress. The second approach was to evaluate the Hoek-Brown tensile failure criterion as a predictor of instability. The final approach was to empirically determine an adjustment to factor A by optimising the accuracy of the stable-failure boundary for cases of tangential and full relaxation.

In the case of partial relaxation, no adjustment was required as the accuracy obtained with either A set to one or A calculated using Diederichs et al. (1999) adjustment produced accuracy comparable with that obtained with the generic Extended Mathews stability chart.

6.8.1 Normalised Tensile Stress

An attempt to relate stability to normalised tensile stress proved unsuccessful. Figure 6.11 and Figure 6.12 illustrate the relationship between normalised tensile stress and the stability of
fully relaxed and tangentially relaxed case studies. Tensile stress was normalised with respect to uniaxial compressive strength. The log of the absolute value of normalised tensile stress has been plotted to facilitate the large range in values obtained when UCS is divided by tensile stress values approaching zero. Figure 6.11 and Figure 6.12 indicate that when normalised tensile stress is considered in isolation from other factors affecting excavation stability, it is difficult to use empirical back-analysis to determine an adjustment for stress relaxation. It should be noted that the seven case studies with slightly compressive $\sigma_3$ were excluded from this analysis.

An alternate reason for the lack of trend between normalised tensile stress and stability is that the effect of relaxation on stability occurs at some threshold stress state, and that further decreases in the stress state do not affect stability. This threshold stress state would be specific to a particular rockmass. Triaxial rock testing of 14 commonly occurring rock types showed that the rock is weak in tension, with none of the rock types withstanding more than 0.2 MPa tensile load (Hoek and Brown, 1981). Therefore, rockmasses are incapable of withstanding the magnitude of tension indicated by linear elastic modelling. It could be interpreted that once the threshold stress state is reached, further reductions in the modelled stress state would not affect stability. In other words, Figure 6.11 and Figure 6.12 provide evidence that relaxation occurs at some threshold value, and once this value is exceeded further reducing stress levels does not affect stability.
Figure 6.12 - Effect of normalised tensile stress and hydraulic radius on the stability of tangentially relaxed case studies

6.8.2 Hoek-Brown Tensile Failure Criterion

An attempt to use the Hoek-Brown tensile failure criterion as predictor of excavation stability for 20 cases of full relaxation produced an accuracy of only 1.1, or 55 percent. The low accuracy obtained could be attributable to the inaccuracy of the assumed Hoek-Brown rock property constants, \( m_i \) and \( s \). In the absence of measured values for the rock property constants, Hoek-Brown rock property constants were estimated based on the values provided by Hoek et al. (1997) for various rock types. Rock mass strength parameters, \( m_b \) and \( s \) were then estimated from Geological Strength Index, GSI as follows (Hoek et al., 1997):

\[
m_b = m_i \exp\left(\frac{GSI - 100}{28}\right)\quad \text{Equation 6.7}
\]

\[
s = \exp\left(\frac{GSI - 100}{9}\right)\quad \text{Equation 6.8}
\]

If laboratory test values for \( m_i \) had been available, the predictive ability of the Hoek-Brown failure criterion may have been significantly higher. However, in the case of the South Crofty...
mine Hoek-Brown stability back-analysis (Pine et al., 1992) the stability was incorrectly predicted despite using a laboratory value for m_i.

### 6.8.3 Adjustment to Factor A

An adjustment to factor A based upon empirical back-analysis has been proposed. Back-analysis within the stability chart framework removed variability due to other parameters affecting excavation stability. By accounting for the variability associated with rock mass characteristics, stope size, stope orientation and joint orientation, it is possible to quantify the effect of stress relaxation on stability with fewer case studies. The high number of fully and tangentially relaxed failures plotting in the stable zone (low specificity) is shown in Figure 6.13 and Figure 6.14.

A threshold value based relaxation effect (threshold value = 0.2MPa) makes sense given that the adverse effect of relaxation appears to occur at some threshold value, and once this value is reached further reducing stress levels would not affect stability.

In the case of both full relaxation and tangential relaxation, experimentation (iterative procedure) with a series of adjustments resulted in an optimal stability prediction (highest accuracy) being achieved when A is assigned a value of 0.7. Factor A was set to a range of values between 0.4 and 0.8. As shown in Table 6.5, setting A equal to 0.7 gave the best result in terms of accuracy of stability predictions (lowest misclassification rate,). In the case of 63 partially relaxed case studies setting A equal to one has a slightly higher level of accuracy than setting A equal to 0.7. However, in the case of full and tangential stress relaxation the accuracy improved by 0.36 (26 percent) and 0.29 (22 percent) respectively (Figure 6.15 and Figure 6.16 respectively). Therefore, it is recommended that in cases of full and tangential relaxation, as defined in this chapter, Factor A should be set to 0.7 to account for the destabilising effect of these types of stress relaxation.
Chapter 6 – Stress Relaxation

Figure 6.13 – Misclassification of fully relaxed case studies (A = 1)

Figure 6.14 - Misclassification of tangentially relaxed case studies (A = 1)
Table 6.5 – Factor A and misclassification statistics

<table>
<thead>
<tr>
<th>Factor A</th>
<th>Misclassification Statistics</th>
<th>Partial Relaxation</th>
<th>Full Relaxation</th>
<th>Tangential Relaxation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Sensitivity</td>
<td>83.3 %</td>
<td>90.9 %</td>
<td>85.7 %</td>
</tr>
<tr>
<td></td>
<td>Specificity</td>
<td>76.9 %</td>
<td>44.4 %</td>
<td>45.5 %</td>
</tr>
<tr>
<td></td>
<td>Accuracy</td>
<td>1.60</td>
<td>1.35</td>
<td>1.31</td>
</tr>
<tr>
<td>0.4</td>
<td>Sensitivity</td>
<td>45.8 %</td>
<td>27.3 %</td>
<td>35.7 %</td>
</tr>
<tr>
<td></td>
<td>Specificity</td>
<td>92.3 %</td>
<td>88.9 %</td>
<td>90.9 %</td>
</tr>
<tr>
<td></td>
<td>Accuracy</td>
<td>1.38</td>
<td>1.16</td>
<td>1.27</td>
</tr>
<tr>
<td>0.6</td>
<td>Sensitivity</td>
<td>70.8 %</td>
<td>72.7 %</td>
<td>71.4 %</td>
</tr>
<tr>
<td></td>
<td>Specificity</td>
<td>89.7 %</td>
<td>88.9 %</td>
<td>81.8 %</td>
</tr>
<tr>
<td></td>
<td>Accuracy</td>
<td>1.61</td>
<td>1.62</td>
<td>1.53</td>
</tr>
<tr>
<td>0.7</td>
<td>Sensitivity</td>
<td>75.0 %</td>
<td>81.8 %</td>
<td>78.6 %</td>
</tr>
<tr>
<td></td>
<td>Specificity</td>
<td>84.2 %</td>
<td>88.9 %</td>
<td>81.8 %</td>
</tr>
<tr>
<td></td>
<td>Accuracy</td>
<td>1.59</td>
<td>1.71</td>
<td>1.60</td>
</tr>
<tr>
<td>0.75</td>
<td>Sensitivity</td>
<td>75.0%</td>
<td>81.8 %</td>
<td>78.6%</td>
</tr>
<tr>
<td></td>
<td>Specificity</td>
<td>84.6%</td>
<td>77.8%</td>
<td>72.7%</td>
</tr>
<tr>
<td></td>
<td>Accuracy</td>
<td>1.60</td>
<td>1.60</td>
<td>1.51</td>
</tr>
<tr>
<td>0.8</td>
<td>Sensitivity</td>
<td>83.4%</td>
<td>90.9%</td>
<td>85.7%</td>
</tr>
<tr>
<td></td>
<td>Specificity</td>
<td>82.1%</td>
<td>66.7%</td>
<td>63.6%</td>
</tr>
<tr>
<td></td>
<td>Accuracy</td>
<td>1.66</td>
<td>1.58</td>
<td>1.49</td>
</tr>
</tbody>
</table>

Figure 6.15 – Misclassification of fully relaxed case studies (A = 0.7)
Chapter 6 – Stress Relaxation

6.9 APPLICATIONS

Based upon the empirical examination of stress relaxation presented in this chapter it is recommended that in cases of full and tangential stress relaxation stress factor A should be set to 0.7. These recommendations are based upon three-dimensional linear elastic modelling of induced stresses where the induced stress was taken at the mid-point of the excavation wall. The reason for using three-dimensional stress modelling is that under some circumstances a two-dimensional stress model will predict that the stope surface is relaxed, but in actuality it is not. However, two-dimensional stress analysis can be used provided the aspect ratio of the stope surface exceeds five. Table 6.6 summarises these recommendations. While these adjustments were developed within the Extended Mathews Stability Graph framework, there is no apparent reason why these adjustments would not be applicable to the Modified Stability Chart (Potvin, 1988) and the ELOS Dilution Graph (Clark et al., 1997).

The Extended Mathew’s stability chart approach to predicting cavability uses stress factor A, as shown in Figure 3.2, to take into account the effect of moderate and high tangential stress (Mawdesley, 2002; Trueman et al., 2003). However, potential increased cavability due to low stress is not currently factored into the Extended Mathews stability chart approach for predicting caving (Mawdesley, 2002; Trueman et al., 2003). In the model framework to assess
cavability outlined by Trueman and Mawdesley (2003) stress relaxation may have an adverse effect upon cavability because the lower stresses induced in the vicinity of the cave back would increase the stress adjustment factor A and thereby, the stability number. However, in this paper, evidence has been presented to demonstrate the potential destabilising effect of both tangential and full relaxation. Therefore, in cases where three-dimensional linear elastic modelling indicates a state of full or tangential relaxation in the block cave crown, the empirical evidence presented in this paper suggests that the stress factor A should be set to 0.7.

### Table 6.6 – Stress relaxation adjustments to A

<table>
<thead>
<tr>
<th>Type of Stress Relaxation</th>
<th>Factor A</th>
</tr>
</thead>
<tbody>
<tr>
<td>Partial relaxation: One principal stress &lt; 0.2 MPa$^1$</td>
<td>1.0</td>
</tr>
<tr>
<td>Full relaxation:</td>
<td></td>
</tr>
<tr>
<td>At least two principal stress &lt; 0.2 MPa$^1$</td>
<td>0.7</td>
</tr>
<tr>
<td>Tangential relaxation:</td>
<td></td>
</tr>
<tr>
<td>At least one principal stress &lt; 0.2 MPa$^1$ and &lt; 20 degrees from parallel to stope wall$^2$.</td>
<td>0.7</td>
</tr>
</tbody>
</table>

Note:
1. Induced principal stress estimated at mid-stope span using three-dimensional linear elastic modelling. Two-dimensional analysis can be used provided the aspect ratio exceeds five.
2. Consider both the angle between the stress direction and the stope surface strike, and the stress direction and stope surface dip.

### 6.10 CONCLUSIONS

Two dimensional stress analyses are insufficient to determine if an excavation surface is relaxed unless the aspect ratio of the excavation surface is 5 or greater. If the aspect ratio is less than 5, a three-dimensional stress analysis is required to confirm that the excavation surface is in actuality relaxed. Because narrow-vein stope length to width ratios are almost always greater than five, two-dimensional plain strain modelling can usually be used to confirm stress relaxation in the case of narrow-vein mining.

Three forms of stress relaxation have been defined; partial, full and tangential. Partial relaxation refers to stope surfaces where one of the induced principal stresses has a value less than 0.2 MPa, but that stress is greater than 20 degrees from being parallel to the excavation surface. Full relaxation refers to the situation where more than one of the induced principal stresses has a value of less than 0.2 MPa. Tangential relaxation refers to the situation where at least one of the induced principal stresses is less than 0.2 MPa and is within 20 degrees of
being parallel to the excavation surface. The back-analysis in this paper indicates that in jointed rock masses, a compressive $\sigma_2$ can have a stabilising effect even when the minimum principal stress is tensile.

Analysis of misclassification statistics using the Extended Mathews stability graph indicates that partial stress relaxation is a poor predictor of stability. However, tangential relaxation and full relaxation were found to have an adverse effect on excavation stability. Back-analysis of partial relaxation case studies, modelled in three-dimensions, found that Diederichs and Kaiser’s adjustment for cases of stress relaxation achieved slightly lower levels of accuracy to the original approach of setting the stress adjustment factor $A$ equal to one in the Mathews stability graph. In the case of full and tangential stress relaxation there was little difference between the two methods. Using a stress factor equal to one and the adjustment factor proposed by Diederichs and Kaiser resulted in accuracy significantly less than for non-relaxed stope surfaces. However, if two-dimensional induced stresses are used, the predictive capability of the Mathews method was significantly reduced by using Diederichs and Kaiser’s recommended stress adjustment.

There were insufficient case studies to investigate full and tangential stress relaxation separately. Similarly, there were insufficient case studies to compare full relaxation with two relaxed stresses parallel to the excavation, to those with only one relaxed stress parallel to the excavation. However, it can be inferred from the back-analyse reported in this chapter that when the minor principal stress is tensile, the intermediate principal stress has an impact on rock mass behaviour in jointed rock masses. Further work is required to quantify the impact of the intermediate principal stress on the behaviour of jointed rock masses.

A new adjustment for the stress factor $A$ that significantly improves the accuracy of stability prediction for cases of full and tangential stress relaxation has been proposed. This adjustment is an explicit method for taking into account the destabilising effect of full and tangential stress relaxation. For cases of partial stress relaxation, a stress adjustment factor $A$ equal to one, as would be the case in the original Mathews method gives the best predictive capability to the model. Because narrow-vein geometries are prone to stress relaxation effects, the adjustment for stress relaxation presented in this chapter is an important contribution towards improved narrow-vein dilution prediction and will form part of the methodology for improved dilution prediction. A threshold value based relaxation effect (threshold value has been determined to be 0.2MPa) makes sense given that the adverse effect of relaxation appears to occur at some
threshold value, and once this stress limit is reached further reducing stress levels would not affect stability.
In this chapter the predictive ability of the stability graph is reviewed with respect to 115 narrow-vein stope case studies from the Barkers mine in Western Australia. The following narrow-vein operating conditions have been analysed:

1. Tight backfill abutments.
2. Drilling accuracy (indirect analysis of stope height).
3. Drill and blast pattern.
4. Undercutting of footwall.

A poor correlation was found between stope stability and both the Mathews stability number, $N$, and hydraulic radius, $HR$. Given that both $N$ and $HR$ correlate well with stability in the vast majority of stability chart case studies, this suggests there is an overriding influence on stability at Barkers not accounted for in the Mathews method. Using comparative statistics, drill and blast issues were isolated as the most likely cause of this poor correlation. Blast pattern was found to have a statistically significant affect on overbreak. In terms of the drill and blast patterns used at the mine, the in-line 3 pattern performed significantly better than both the staggered and dice 5 patterns for the vein width at the time. Undercut footwalls were found to behave in a similar manner to non-undercut hangingwalls. There was no evidence that tight backfill abutments (not continuous moving) behave differently from solid rock abutments in terms of determination of stable stope dimensions. Drillhole accuracy was indirectly examined by considering the effect of stope heights within the limits of 13 metres to 20 metres. Within these limits, stope height did not affect the magnitude of overbreak.

The findings from this chapter support the thesis hypothesis that narrow-vein stope stability is poorly predicted by existing stability graph methods and justifies further explicit consideration of narrow-vein operating conditions.
7.1 INTRODUCTION

As discussed in Chapter 4 a number of narrow-vein operating conditions have been hypothesised to have a significantly larger influence upon narrow-vein stope stability than they do on large open stope stability. In Chapter 5 the limited potential of site-specific charts to calibrate to narrow-vein operating conditions was demonstrated, and pointed to the need for explicit consideration of narrow-vein operating conditions. In this chapter the hypothesis that existing stability charts poorly predict narrow-vein stope stability is investigated by back-analysing the predictive ability of stability charts at the Barkers narrow-vein mine. Analysis of the Barkers narrow-vein case studies have been undertaken in two stages. In the first stage the predictive ability of the stability chart approach has been examined. In the second stage comparative statistics have been used to back-analyse the parameters affecting dilution in the 115 Barkers case studies. The following narrow-vein operating conditions have been analysed:

1. Tight backfill abutments.
2. Drilling accuracy (indirect analysis of stope height).
3. Drill and blast pattern.
4. Undercutting of footwall.

This chapter contains the results of the first set of Barkers case studies collated between in August 2001 and October 2001. In the following chapter (Chapter 8) a second set of 410 case studies from the Barkers mine has been analysed with the express purpose of considering the effect of stress damage on dilution at the mine. Both datasets are new datasets collected as part of this thesis project. Because the second set of data was collected with the primary purpose of conducting analysis of the effect of stress damage due to stress concentration at the brow, the second database does not contain the same fields as the first database. In addition some fields were determined in a different manner. Therefore, the two databases have not been merged and the analysis contained in this chapter has been undertaken separately from the second set of 410 case studies presented in Chapter 8.

7.2 BARKERS MINE, KUNDANA GOLD OPERATIONS

7.2.1 Location

Kundana Gold Operations is located 25 kilometres west-northwest of Coolgardie, which is approximately 595 km east of Perth in Western Australia as shown in Figure 7.1. Placerdome’s Kundana Gold Operations ceased underground production in May 2004. Mining
of the Barkers and Strzelecki ore bodies formed one underground mining operation. All case studies referred to in this chapter come from the Barkers orebody.

![Diagram of the Yilgarn Craton and surrounding gold mines and shear zones]

**Figure 7.1 – Location of Kundana Gold Operations within the Archean Greenstone belt of the Yilgarn Craton, Western Australia, after Slade (2004)**

### 7.2.2 Stratigraphy

The Kundana Mining lease contains a sequence of rocks generally striking AMG 300° to 330° and dipping steeply west. Mineralisation is constrained within a deep crustal shear zone known as the Zuleika Shear (Slade, 2004). The general stratigraphy is comprised of a sequence of mafic to intermediate volcanics and derived sediments. The Kundana sequence is interpreted to form part of an upright isoclinal anticline between the east and west synclines (Hadlow, 1990). The dominant sub-vertical foliation associated with the Zuleika Shear Zone trends from 320° to 350°. The bulk of the mineralisation is in the form of thin planar laminated quartz veins which dip moderately to steeply to the west with strike lengths up to 600 metres (Hadlow, 1990).

The Barkers orebody mineralisation occurs within a laminated quartz vein at the sheared contact between the footwall western facies of the Gabbro intrusion and a hangingwall felsic volcanic sediment (Reid et al., 2001). The average vein dip is 70 degrees with width ranging...
from 0.2 metres to 0.7 metres. Shearing extends up to 2.5 metres either side of the vein. The hangingwall zone consists of two major rock units consisting of dolerite in the northern section of the orebody, and volcanic/sediment units in the southern section of the orebody.

The orebody is relatively tabular with only gradual changes in orientation along strike and along dip. For this reason the sill drives are relatively straight and designed stope walls relatively planar along both strike and dip. However, there is a four to six metre fault induced offset in the orebody, approximately 110 metres from the end of the northern extent of the orebody. This fault zone is clearly seen as a lateral ‘kink’ in the orebody in the northern section of the ore drives. The fault zone has resulted in localised degradation in rock mass properties and has been allocated a separate geotechnical domain (Brunton and Trueman, 2001). This fault zone affects approximately 12 metres along strike. The footwall consists of gabbro, which can be divided into two distinct zones; one immediately adjacent to the orebody and the other further into the footwall (Brunton and Trueman, 2001).

### 7.2.3 Rockmass Characterisation

Table 7.1 contains a summary of laboratory geomechanical properties of the Barkers ore and host rocks. The values shown are global estimates of each rock type based on sampling from limited exploration drill holes and data obtained from hollow inclusion cell stress measurements.

**Table 7.1 – Laboratory geomechanical properties of Barkers ore and host rocks, after Slade (2004)**

<table>
<thead>
<tr>
<th>Location</th>
<th>Number</th>
<th>UCS (MPa)</th>
<th>Static Young’s Modulus Av. (GPa)</th>
<th>Static Young’s Modulus Av. (GPa)</th>
<th>Static Poisson’s Ratio Av.</th>
<th>Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Barkers FW Gabbro</td>
<td>Mean</td>
<td>142</td>
<td>-</td>
<td>64</td>
<td>0.26</td>
<td>2.7</td>
</tr>
<tr>
<td></td>
<td>St.Dev.</td>
<td>40</td>
<td>-</td>
<td>6</td>
<td>0.04</td>
<td>-</td>
</tr>
<tr>
<td>Barkers HW Volcanic Sediments</td>
<td>Number</td>
<td>n.a</td>
<td>n.a.</td>
<td>n.a.</td>
<td>n.a.</td>
<td>n.a.</td>
</tr>
<tr>
<td></td>
<td>Mean</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>St.Dev.</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Quartz vein</td>
<td>Number</td>
<td>3</td>
<td>3</td>
<td>n.a.</td>
<td>n.a.</td>
<td>75</td>
</tr>
<tr>
<td></td>
<td>Mean</td>
<td>130</td>
<td>8</td>
<td>-</td>
<td>-</td>
<td>2.71</td>
</tr>
<tr>
<td></td>
<td>St.Dev.</td>
<td>6</td>
<td>1</td>
<td>-</td>
<td>-</td>
<td>0.15</td>
</tr>
</tbody>
</table>
The stopes analysed for this study were limited to the panels for which detailed rockmass characterisation had been undertaken and where geotechnical domains showed high vertical consistency. Brunton and Trueman (2001) undertook detailed rockmass characterisation and scanline mapping in the 5990, 6055 and 6070 sill drives. Scanline mapping data was analysed using JointStats stereographic software and joint sets identified (Brunton and Trueman, 2001). JointStats was developed by the JKMRC to analyse discontinuity data collected from scanline and window mapping. RQD was estimated using Palmstrom’s (1982) volumetric joint count. The locations of the hangingwall and footwall structural domains are shown in Figure 7.2 and Figure 7.3, respectively. Stereographic interpretation of the hangingwall joint sets resulted in identification of the five structural domains shown in Table 7.2.

![Figure 7.2 – Location of hangingwall structural domains on the 6070 m level, after Brunton and Trueman (2001)](image-url)
Chapter 7 – A Narrow-vein Case Study – Barkers Mine

Table 7.2 – Summary of structural domains in the hangingwall (6055 m and 6070 m Levels), after Brunton and Trueman (2001)

<table>
<thead>
<tr>
<th>Structural Domain</th>
<th>Location</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hangingwall North</td>
<td>Northern section of orebody.</td>
<td>The rock mass consists of three joint sets (two sub-vertical dipping to the SE and SW, and one sub-horizontal).</td>
</tr>
<tr>
<td>(HWN)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hangingwall Fault</td>
<td>Fault zone northern section of orebody.</td>
<td>A number of faults in this zone cut and displace the orebody in this area.</td>
</tr>
<tr>
<td>(HWF)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hangingwall Central</td>
<td>Central section of orebody south of fault zone.</td>
<td>The rock mass consists of sub-vertical jointing which is common to both the HWN and HWS domains.</td>
</tr>
<tr>
<td>(HWC)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hangingwall South</td>
<td>Southern section of orebody.</td>
<td>The rock mass consists of three joint sets (two sub-vertical dipping to the SE and SW, and one sub-horizontal). The dip direction of the sub-horizontal joint set differs to that encountered in HWN and HWS domains.</td>
</tr>
<tr>
<td>(HWS)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 7.3 illustrates the location of the footwall structural domains. Footwall Zone 1 (FZ1) is characterized by strong foliation similar to that seen in the hangingwall. From the limited exposures mapped, this zone appears to widen as the orebody widens, and has a general width of 0.5 to 2.0 metres (Brunton and Trueman, 2001). For the purpose of Barkers stope stability analysis, the FW1 structural domain has been used for all footwall case studies.

Figure 7.3 – Two footwall zones located along strike of the orebody, after Brunton and Trueman (2001)
Table 7.3 contains the $Q'$ rockmass classification (Mathews et al., 1981). Domain boundaries between the 6055 m and 6070 m levels were found to be sub-vertical in nature (Brunton and Trueman, 2001). Therefore, it has been possible to extrapolate rock mass characterisation results from the 6055 m and 6070 m levels to 6120 m, 6105 m, 6090 m and 6040 m levels. The Barkers stope stability database contains case studies from levels 6040 m through to 6120 m and therefore, some of the rock mass characterisation contained in the Barkers stope stability database has been extrapolated from vertically adjacent levels. The high vertical consistency of geotechnical domains meant that it was reasonable to assume domain consistency up to two levels away (30 metres) when necessary.

<table>
<thead>
<tr>
<th>Domain</th>
<th>RQD</th>
<th>$J_a$</th>
<th>$J_r$</th>
<th>$J_s$</th>
<th>$Q'$</th>
</tr>
</thead>
<tbody>
<tr>
<td>HWN</td>
<td>97</td>
<td>9</td>
<td>1.0</td>
<td>1.0</td>
<td>10.8</td>
</tr>
<tr>
<td>HWC</td>
<td>94</td>
<td>9</td>
<td>1.0</td>
<td>1.0</td>
<td>10.4</td>
</tr>
<tr>
<td>HWS</td>
<td>95</td>
<td>12</td>
<td>1.0</td>
<td>1.0</td>
<td>7.9</td>
</tr>
<tr>
<td>HWS1</td>
<td>86</td>
<td>6</td>
<td>1.25</td>
<td>1.0</td>
<td>17.9</td>
</tr>
<tr>
<td>HWS2</td>
<td>87</td>
<td>9</td>
<td>1.0</td>
<td>1.0</td>
<td>9.7</td>
</tr>
<tr>
<td>FWN</td>
<td>100</td>
<td>12</td>
<td>3.25</td>
<td>1.0</td>
<td>27.1</td>
</tr>
<tr>
<td>FWS</td>
<td>100</td>
<td>12</td>
<td>1.25</td>
<td>1.0</td>
<td>10.4</td>
</tr>
<tr>
<td>FZ1</td>
<td>100</td>
<td>12</td>
<td>1.25</td>
<td>1.0</td>
<td>10.4</td>
</tr>
</tbody>
</table>

7.2.4 In situ Stresses

Pascoe, (2001) undertook hollow inclusion cell stress measurements on the 6025 level (319 metres depth). The case studies discussed in this chapter come from the 6040 metre level to the 6120 metre level. The Barkers 6025 stress measurement was located 80 metres from the orebody. Stress measurements undertaken by Australian Mining Consultants (2001) are summarised in Table 7.4. The Barker 6025 stress measurement was rated as a good site (Pascoe, 2001).

<table>
<thead>
<tr>
<th>Magnitude (MPa)</th>
<th>Bearing (º)</th>
<th>Plunge (º)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_1$</td>
<td>26.6</td>
<td>20</td>
</tr>
<tr>
<td>$\sigma_2$</td>
<td>17.2</td>
<td>112</td>
</tr>
<tr>
<td>$\sigma_3$</td>
<td>15.5</td>
<td>221</td>
</tr>
</tbody>
</table>
7.2.5 Mining Method

The mining method was a combination of the bottom-up modified Avoca method using development waste as fill and longhole open stoping with small rib pillars. Figure 7.4 is a schematic representation of the Barkers mining method. Longhole open stoping with small rib pillars were always used on the top levels of the panels because there is no access for filling. Backfill was tight filled against the brow with 10 to 30 metre spans opened prior to the next backfill cycle. The orebody was accessed via horizontally offset crosscuts that access the orebody approximately half way along strike.Stoping proceeded from both ends of the orebody on multiple levels retreating to a central access. Typically, levels were spaced at 15 metre intervals. Rib pillars were two metres along strike and the sill pillars separating panels were between one and four metres high.

![Figure 7.4 – Schematic representation of the Barkers mining method (long-section)](image)

The Barkers orebody employs the modified Avoca mining method with tight backfilling to the brow. Therefore, the Barkers extraction and filling sequence is different from that employed at Winston Lake mine, in that there is no backfill lag and therefore the adjacent stope is completely filled prior to the next stage of stoping. Therefore, the Barkers stopes are not
subject to the effect of moving backfill abutments. In this paper the effect of backfill abutments has been evaluated in terms of the number of backfill abutments.

7.3 BARKERS STABILITY DATABASE

The 115 case studies analysed in this chapter come from panels located between the 6120 metre sill drive and the 6040 metre sill drive. Appendix C contains the database of Barkers case studies. The Kundana mine geology staff recorded stope stability data including stope geometry and visual estimation of overbreak from the vein to the final stope walls in Stope Records Sheets. Collation of this raw data in conjunction with extensive rock mass characterisation work and the development of a linear elastic stress model to estimate induced stresses have facilitated the stope stability analysis contained in this report.

7.3.1 Estimation of Induced Stresses

Induced stresses are required for the estimation of stress factor, A. Induced stresses were estimated using the linear elastic boundary element program Map3d. Three panels have been incorporated in the model. The AMC (2001) stress measurements in Table 7.4 were used in the model. Figure 7.5 is an image depicting the elastic stresses at stage 5 of panel mining. The model was simplified by treating the entire top panel as a single block. None of the case studies considered in this thesis are located in the top panel, so this simplification does not affect the stopes analysed. All the stope surfaces contained in the Barkers stability database come from the second and third panel. Figure 7.5 shows two stress resolution grids at stage 5 of orebody extraction: one shows stress contours parallel to the footwall and the other shows stress contours perpendicular to the footwall.

Stress modelling was undertaken for 15 different stages of extraction. The sequence shown is an approximate sequence based on knowledge of the bottom-up shrinking central pillar sequence. The various stages of extraction shown in Figure 7.5 correspond to the various colours shown. Induced stresses tangential to the centre of stope walls ranged from 10 MPa to 50 MPa, with an estimated accuracy of +/-10 MPa. Depending on the proximity of the nearest grid, induced stresses were extrapolated up to 10 metres along strike. There was very little variation in stresses along strike so this extrapolation is likely to be associated with error in the order of 2 to 5 MPa.
7.3.2 Stope Geometry

For the purpose of stope stability analysis, backfill abutments are treated as though they are solid rock. Therefore, the stope span is taken to be the distance from the stope brow to the backfill, pillar or endwall as the case may be. As there is no access to the top of each panel, pillars are used in place of backfill to achieve reduced stope spans. A two to four metre high sill/crown pillar is left between panels. The 6120 m and 6070 m levels are top level stopes. Stope record sheets recorded ring numbers and sill drive name. The location of the stopes was determined from a long-section showing ring location.

7.4 STOPE STABILITY ANALYSIS

The Barkers stability database overbreak records are based upon an estimate of overbreak from the vein. Because the vein width is less than a practical minimum mining width, it was necessary to adjust these values to take into account overbreak with respect to some optimum design width. For the purposes of this stope stability analysis footwall overbreak has been corrected to take into account a minimum practical stope design width of 1.2 metres (or planned dilution on the footwall side). Therefore, corrected overbreak refers to overbreak outside of the designed stope limits. The correction is not applicable to hangingwalls because
there is no planned dilution on the hangingwall side. Throughout this chapter and chapter 8 footwall overbreak refers to corrected footwall overbreak.

Two approaches to stope stability analysis have been undertaken. Firstly, stope stability has been examined within the framework of the Extended Mathews Stability Chart (Trueman and Mawdesley, 2003; Mawdesley et al., 2001; Trueman et al., 2000). Secondly, stability has been statistically evaluated with respect to a number of parameters not taken into account by existing stability chart methods. The effect of undercutting of the stope wall has been taken into account by experimenting with the gravity adjustment factor, C until stable and unstable misclassifications are minimised.

7.4.1 Barkers Stope Stability and the Extended Mathews Stability Chart

In Figure 7.6, Figure 7.7 and Figure 7.8 the Barkers case studies have been plotted with respect to the Extended Mathews Stability Graph. Each of the plots categorises stable and failed stope surfaces according to a different corrected overbreak cut-off. Clark and Pakalnis (1997) define a failure as an equivalent linear overbreak greater than 0.5 metres. Figure 7.6 uses the 0.5 metre cut-off for categorising stopes as stable, whilst Figure 7.7 and Figure 7.8 contain cut offs of 0.65 metres and 1.0 metres, respectively. Almost all Barkers stope data falls above the Extended Mathews stable-failure boundary, in the stable zone. Therefore, according to the Extended Mathews stability chart, all but one point should have been stable.

The possibility that inaccuracies or inconsistencies in measuring overbreak are the reason for the poor separation of stable and unstable points is unlikely. The type of error introduced by this source would most likely produce random classification errors through which some separation of stable and failures should still be evident. Based upon Figure 7.6, Figure 7.7, and Figure 7.8 there is no evidence of a separation of stable and unstable points with respect to the stability number, N. As discussed in Chapter 3 the 485 generic Extended Mathews stability case studies were found to have a good correlation to stability, with an accuracy greater than 80 percent. This suggests that a dominant effect on stope stability at Barkers is not taken into account by parameters captured by the stability number, N, or stope size.

As shown in Figure 7.7 and Figure 7.8 there are no failures (corrected overbreak <0.65 metres) below a hydraulic radius of 3.2 metres. This suggests that within the range of N values captured in the database, overbreak of less than 0.65 metres can be expected for stopes with a hydraulic radius below 3.2 metres.
Figure 7.6 – Barkers stability data with stable corrected overbreak cutoff < 0.50 metres

Figure 7.7 – Barkers stability data with stable overbreak cutoff <0.65m
In summary, the Barkers case studies provide evidence indicating that the hypothesis that narrow-vein stability is poorly predicted by stability charts is true. In the following section additional narrow-vein case studies providing additional evidence supporting the hypothesis of this thesis are discussed.

7.5 TROUT LAKE AND CALLINAN CASE STUDIES

Wang et al. (2002b) present 146 case studies from Trout Lake and Callinan narrow-vein mines. 85 percent of stope widths ranged from 4 to 12 metres (Wang, 2005), and could be considered relatively narrow. These case studies provide further evidence of poor correlation of narrow stope stability to existing stability charts. Figure 7.9 plots the difference between actual and predicted metres of hangingwall slough.

Over 50 percent of the case studies had more than 0.5 metres more slough than predicted. Wang et al. (2002b) refer to Clark and Pakalanis’s (1997) original list of factors either ignored or poorly accounted for as possible explanations for the poor predictive ability. These factors include; irregular wall geometry, undercutting of stope walls, blast design and stope life. In this thesis irregular wall geometry, undercutting of stope walls and blast design have been
identified as narrow-vein operating conditions that predispose narrow-vein stope stability prediction to inaccuracy.

![Diagram of Modified Stability Number N' vs HR (m) with different ELOS (act-pred) thresholds: ELOS |act-pred|<0.5, 0.5<|act-pred|<1.0, 1.0<|act-pred|<2.0, |act-pred|>2.0]

Figure 7.9 – Trout Lake and Callinan narrow-vein case studies showing difference between actual and predicted dilution, after Wang et al. (2002b)

### 7.6 Statistical Analysis of Causes of Barkers Dilution

Because overbreak has been recorded as a continuous variable it is possible to undertake statistical analysis of the possible causes of dilution at Barkers mine. When appropriate (normal distributions and equal variances) the simple two-sample t-test has been applied. There is a general consensus amongst statisticians that the two-sample t-test should be used when the assumptions of normality and equal variances are satisfied (DeVore, 1991). However, in cases where assumptions of normality were not reasonable alternative non-parametric tests were performed. However, it should be noted that these tests are more likely to incorrectly accept the null hypothesis (Type II error). In cases where an assumption of non-equal variance was not reasonable a heteroscedastic t-test was performed.

#### 7.6.1 Effect of Tight Backfilling

The effect of tight backfilling abutments has been studied as one of the possible causes of dilution at Barkers mine. In this thesis tight filling refers to filling sequences where the adjacent stope is completely filled prior to stoping. Because backfill is tight filled at Barkers
the abutments are not moving backfill abutments. Potvin’s (1988) analysis of the effect of backfill abutments on stope stability indicated that tight backfilling does not affect stope stability and a backfill abutment can be treated as a solid rock abutment.

The effect of backfill abutments on Barkers overbreak has been examined by investigating the effect of the number of backfill abutments on stope stability. For example, a stope is said to have one backfill abutment if one out of the four stope wall abutments is fill. Depending on the position of a stope wall within a panel and whether a pillar or backfill was used to shorten stope spans, a stope may have between zero and two backfill abutments.

Comparative statistics were conducted to compare overbreak for each of the three backfill scenarios. Table 7.5 contains the results of statistical comparisons between one and two backfill abutments, zero and one backfill abutments and zero and two backfill abutments. A standard t-test could not be used because the F-statistic was significant in each case indicating that the requirement for equal variances does not hold true. Therefore, heteroscedastic t-tests were conducted to accommodate the non-equal variances of each comparison. Heteroscedastic tests provided no evidence the mean overbreaks were different. However, it is possible to draw statistically significant conclusions about the probability of obtaining a difference in means greater than some value by conducting an analysis of statistical power. Power is defined as 1-β, where β is the probability of accepting a false hypothesis (type II error) (Walpole et al., 1990). In this case an acceptable level of power was set at 0.90. Power analysis was conducted using the computer program Statistica using a pooled variance. The difference between means was adjusted by trial and error until the probability of a type II error was less than 10 percent (power > 0.90). Using power analysis, it was possible to make the following conclusions regarding the effect of backfill abutments:

1. There is a 90 percent probability that the difference in mean overbreaks between one and two backfill abutments is less than 0.47 metres.
2. There is a 90 percent probability that the difference in mean overbreaks between zero and two backfill abutments is less than 0.37 metres.
3. There is a 90 percent probability that the difference in mean overbreaks between one and two backfill abutments is less than 0.20 metres.

Although histograms of each backfill configuration appeared normal, it was decided that a non-parametric Mann-Whitney U test was required to confirm the findings of the heteroscedastic t-test. The results of these comparisons are contained in Table 7.5 and confirm
that the number of backfill abutments does not significantly affect overbreak. These results confirm Potvin’s (1988) recommendation for treating backfill abutments the same as solid rock in the case where the adjacent stope is completely filled prior to recommencing stoping.

Table 7.5 – Effect of number of backfill abutments on overbreak

<table>
<thead>
<tr>
<th>Comparison</th>
<th>Test</th>
<th>t-test Power</th>
<th>P Value</th>
<th>Conclusions</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Fill Abut. compared to 2 Fill Abut.</td>
<td>Heteroscedastic t test (normal but unequal variances)</td>
<td>For $\sigma_p=0.426$ $\mu_1-\mu_2&gt;0.47m$ p=0.90</td>
<td>0.528</td>
<td>90% confident the difference between 1 and 2 fill abutments is $&lt;0.47m$</td>
</tr>
<tr>
<td>0 Fill Abut. compared to 1 Fill Abut.</td>
<td>Heteroscedastic t test (normal but unequal variances)</td>
<td>For $\sigma_p=0.325$ $\mu_0-\mu_2&gt;0.37m$ p=0.90</td>
<td>0.269</td>
<td>90% confident difference between 0 and 1 fill abutments is $&lt;0.37m$</td>
</tr>
<tr>
<td>0 Fill Abut. to 2 Fill abut.</td>
<td>Heteroscedastic t test (normal but unequal variances test)</td>
<td>For $\sigma_p=0.337$ $\mu_0-\mu_2&gt;0.20m$ p=0.90</td>
<td>0.626</td>
<td>90% confident difference between 0 and 2 fill abutments is $&lt;0.20m$</td>
</tr>
</tbody>
</table>

The effect of unconsolidated backfill on excavation stability has been analysed using strain-stiffening curves for goaf waste rock and Q system support requirements. According to the Q system support requirements by Hoek and Brown (1980), the Barkers hangingwalls require approximately 100 kPa support pressure, while the Barkers footwalls require approximately 50 kPa. According to Thin (1994), the relationship describing the goaf strain-stiffening response, 2.5 percent compaction corresponds to 100 kPa support pressure. Stope closure of 3.8 centimetres corresponds to 2.5 percent compaction of the backfill. Assuming the unconsolidated backfill used at Barkers has similar strain-stiffening properties to the goaf waste tested by Thin (1994), this analysis suggests the support pressure associated with 2.5 percent closure of the unconsolidated waste backfill can be expected to provide similar support to a solid rock abutment. In this scenario the stope closure reaches equilibrium with the backfill reaction pressure. The ability of unconsolidated backfill to provide adequate support pressure is consistent with the empirical evidence presented in Table 7.5 and the observations of Potvin (1988).

As noted previously, the Barkers case studies are different from Milne’s (1997) Winston Lake case studies where the backfilling was not tight filled and there was a lag in the backfill front
(moving backfill abutment). In this case, Milne (1997) presented evidence that treating the moving backfill abutment the same as fill was overly optimistic, and proposed that the effective span could be calculated as the sum of the maximum and minimum open span.

### 7.6.2 Effect of Stope Height

There are two reasons why the effect of stope height on corrected overbreak was investigated. Firstly, site mining and geotechnical personnel had been considering the implications of increasing stope heights in order to reduce both production development and capital development costs. Secondly, drillhole deviation could be expected to increase with stope height and comparing stopes of difference heights may indirectly indicate whether drillhole deviation could be a cause of dilution at Barkers mine. Figure 7.10 is a scatter plot of overbreak versus stope height. The correlation coefficient for overbreak versus stope height was –0.105. This suggests that within the range of stope heights collated, stope height does not impact upon stability.

![Figure 7.10 – Corrected overbreak versus stope height](image)

Figure 7.11 is a comparison of cumulative frequency distributions for stable and unstable stopes. A comparison of these distributions indicates that stope stability is not improved by reducing the stope height to less than 19 metres. Furthermore, within the range of stope heights...
analysed there is no evidence that that the increased drillhole length associated with increased stope height is associated with increased overbreak.

![Cumulative Height Chart for Stable and Failed Stopes](image)

**Figure 7.11 – Cumulative height chart for stable and failed stopes**

### 7.6.3 Effect of Drill and Blast Parameters

The Barkers database includes information on the drill and blast pattern used for each stope. Three different drill and blast patterns are contained in the database; Dice 5, Staggered and In-line 3. Figure 7.12 illustrates the layout of each of the three blast patterns with respect to stope design width and the vein.

Summary statistics for the effect of blast pattern on stope stability are shown in Table 7.6. The comparison of the three blast patterns contained in the Barkers database indicates that in-line 3 results in less overbreak than both the dice 5 and staggered. This difference becomes even more pronounced for overbreak greater than 0.65 metres. In the case of 0.65 metres overbreak, only 4.5 percent of in-line 3 stopes have failed at the 0.65 metres corrected overbreak cutoff. At the 0.50 metres corrected overbreak cutoff for failures 18.2 percent of in-line 3 stopes had failed compared to 42.5 percent and 37.1 percent, respectively for dice 5 and staggered.
Figure 7.12 - Drill-hole patterns (plan view)

Table 7.6 – Stability statistics for blast patterns

<table>
<thead>
<tr>
<th>Blast Pattern</th>
<th>Number of Case Studies</th>
<th>Mean Corrected Overbreak (m)</th>
<th>Corrected Overbreak Variance (m)</th>
<th>Proportion of Case Studies Overbreak &gt; 0.50 m</th>
<th>Proportion of Case Studies Overbreak &gt; 0.65 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dice 5</td>
<td>31</td>
<td>0.43</td>
<td>0.15</td>
<td>42.5 %</td>
<td>25.8 %</td>
</tr>
<tr>
<td>Staggered</td>
<td>62</td>
<td>0.39</td>
<td>0.19</td>
<td>37.1 %</td>
<td>19.4 %</td>
</tr>
<tr>
<td>In-line 3</td>
<td>22</td>
<td>0.25</td>
<td>0.07</td>
<td>18.2 %</td>
<td>4.5 %</td>
</tr>
</tbody>
</table>

A t-test was conducted to test the significance of the difference between overbreak for each of the blast patterns. A t-test enables the difference between the means of two independent data sets to be compared provided the two data sets are normally distributed and the variances are equal. Comparisons of staggered to dice 5 and dice 5 to in-line 3 were shown to meet these assumptions. However, in the case of the comparison of Staggered to in-line 3, the variances were found to be considerably different as evidenced by the F-test of variance equality (p value of 0.0112). Therefore, a heteroscedastic t-test was used for this comparison. Table 7.7 contains results of a statistical comparison of overbreak for each of the blast patterns illustrated in Figure 7.12. Stopes blasted using in-line 3 had significantly less (at the 90 percent confidence level) overbreak than both staggered and dice 5 blast patterns. Overbreak for Staggered and Dice 5 were not significantly different. While there was no record of in-line 3 causing problems with bridging it could be considered a higher risk drill pattern. It is clear that blast pattern has the potential to significantly impact upon overbreak in narrow vein stopes.
Table 7.7 – Effect of blast pattern on overbreak

<table>
<thead>
<tr>
<th>Comparison</th>
<th>N₁, N₂</th>
<th>μ₁, μ₂</th>
<th>Difference in means</th>
<th>Test</th>
<th>Power t-test</th>
<th>P value</th>
<th>Conclusions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Staggered to In-line 3</td>
<td>62, 22</td>
<td>0.396, 0.248</td>
<td>0.148</td>
<td>Heteroscedastic t test</td>
<td>0.39</td>
<td>0.063</td>
<td>Mean overbreak significantly different @ 90 percent confidence level</td>
</tr>
<tr>
<td>Dice 5 to In-line 3</td>
<td>31, 22</td>
<td>0.434, 0.248</td>
<td>0.186</td>
<td>t-test</td>
<td>0.53</td>
<td>0.054</td>
<td>Mean overbreak significantly different @ 90 percent confidence level</td>
</tr>
<tr>
<td>Dice 5 to Staggered</td>
<td>31, 62</td>
<td>0.434, 0.396</td>
<td>0.038</td>
<td>Heteroscedastic t test</td>
<td>0.12</td>
<td>0.670</td>
<td>No evidence mean overbreaks are difference</td>
</tr>
</tbody>
</table>

7.7 EFFECT OF UNDERCUTTING FOOTWALLS

Under the original Mathews stability graph method a footwall is assigned a stope orientation factor, C of 8. However, due to the undercutting of footwalls associated with equipment minimum width requirements, it was deemed that a C factor of 8 is likely to be too high. For this reason, a number of trials were carried out experimenting with the stope orientation factor. Figure 7.13, Figure 7.14 and Figure 7.15 show the C values trialed.

![Figure 7.13 – Stope orientation factor for footwalls: C = 8](image-url)
Figure 7.14 – Stope orientation factor for footwalls: C=1.0

Figure 7.15 – Stope orientation factor for footwalls as for hangingwalls: C=4.9

Figure 7.13 demonstrates that when a stope orientation of 8 is applied to the footwall data, N values are high in comparison to their stability classification. Considering that the footwall is just as likely as the hangingwall to experience overbreak greater than 0.5 metres, engineering judgement suggests that this data is plotting unrealistically high compared to the hangingwall
data. Figure 7.14 shows the data with a stope orientation factor of 1.0. A low value was adopted to reflect the perceived decrease in stability associated with two kinematic degrees of freedom (two free faces). However, a stope orientation factor of 1.0 was found to be too low. In Figure 7.15, a stope orientation factor the same as the hangingwall was applied. On the basis of engineering judgement, this result was deemed to be the most realistic and was applied to all other representation of the data in terms of N.

As discussed in Chapter 4, Wang et al. (2002b) propose an undercutting factor (UF). UF is defined in Equation 4.8 and is based on the premise that undercutting and overcutting of stope walls results in increased dilution due to increased stress relaxation. Based on the empirical study of stress relaxation presented in Chapter 5, the underlying premise for UF could be limited in that it does not take into account different types of stress relaxation. Like the fault factor proposed by Suorineni (1998), the undercutting factor assumes that the destabilising effects of stress relaxation are dependent on stress relaxation as defined by the minimum principal stress. As determined in Chapter 5 partial relaxation does not affect stope stability. However, both full and tangential stress relaxation were demonstrated to affect stope stability. Therefore, both the fault factor and the undercutting factor may better correlate to stability if they were based on full and tangential stress relaxation. It has not been possible to evaluate UF with respect to the Barkers case studies because the parameters required to determine UF were not recorded.

7.8 CONCLUSIONS

Barkers stope stability case studies indicated that the stability number, N is a poor predictor of dilution at Barkers. In the case of stope shape and size as quantified by the term hydraulic radius there was not as strong a correlation to stability with the Barkers data than with the Mathews type generic stability database. However, stopes with a hydraulic radius less than 3.4 were found to perform better than larger stopes with respect to overbreak for the operating conditions encountered.

An additional 146 narrow-vein case studies from the Callinan and Trout Lake mine were also poorly predicted by existing stability charts. The Barkers, Callinan and Trout Lake narrow-vein case studies support the hypothesis that existing stability charts provide poor predictive ability in the case of narrow-vein stoping.
Statistical analysis of narrow-vein operating conditions not captured by the stability number $N$ were examined as possible causes of dilution at the mine. Issues pertaining to drill and blast were isolated as a most likely source of this overriding dominant influence on stability. There was no evidence that the number of backfilled abutments influenced stope stability. This implies that for the tight backfilling conditions at Barkers it is reasonable to treat the backfill abutment as a solid rock. An analysis of the backfill support pressure associated with 2.5 percent compaction indicated that the backfill could develop sufficient support pressure for the rock mass conditions encountered. Therefore, provided backfill stiffening due to compaction occurs at reasonably low stope closure and the support pressure available is adequate for the rock mass in question, a tight backfill abutment can be treated as a solid rock abutment.

Within the range of the Barkers database, stope height did not appear to affect stope stability. For the vein widths captured in the Barkers stability database, the In-line 3 blast pattern performed significantly better than both the Staggered and Dice 5 in terms of overbreak. In the case of the undercut footwalls it was found that adopting a stope surface orientation factor $C$ equal to that of the hangingwall provided a realistic stability number outcome. If a $C$ factor of 8 is used as recommended under the original Mathews recommendations, the footwall data sits unrealistically high above the hangingwall data. In engineering terms an undercut footwall appears to behave in a similar manner to a non-undercut hangingwall.

Overall, the Barkers stope stability back-analysis provides validation of the need to address narrow-vein operating conditions such as drill and blast design explicitly. While adopting an in-line drill and blast pattern instead of a dice five pattern has the potential to reduce dilution by an average of 0.19 metres, the large number of narrow-vein case studies with dilution in excess of stability chart predictions suggests that additional causes of dilution are probable. In the following chapter the effect of stress damage on dilution on narrow-vein dilution has been investigated as an additional possible cause of dilution at the Barkers mine.
In this chapter, the effect of stress damage associated with a stope wall’s stress history is examined. In the case of narrow-vein mining, the incremental extraction of long-hole rings has the potential to result in a moving high stressed zone at the stope brow. This results in the hangingwall and footwall experiencing a spike in the stress to strength ratios as the brow passes. In some cases, the stress to strength ratio may be high enough to result in fracturing or damage to the rock mass.

The aim of the study described in this chapter was to investigate whether stress damage in advance of the excavation results in a significant increase in dilution. The study involved analysis of overbreak from 412 case studies from the Kundana Gold mine in Western Australia. Site personnel had already undertaken a calibration of the stress levels that result in rock mass damage. This calibration, in conjunction with numerical modelling of a 32 month extraction sequence demonstrated, with 94 percent confidence, that stress damaged stope walls at this mine had an average 0.27 metres more overbreak than stope walls where stresses had not exceeded the damage criterion. Assuming a designed mining width of approximately 1.5 metres, and both hangingwall and footwall were affected by stress damage, this represents 36 percent dilution. The potential for stress damage related overbreak should therefore be considered as part of any assessment of narrow-vein dilution.

8.1 INTRODUCTION

The effect of a stope wall’s stress history is one of the narrow-vein operating conditions hypothesised to contribute to narrow-vein dilution. Stress history considers the full stress history of a stope wall while Stress Factor A considers the mid-stope induced stress post-stoping. Stope walls adjacent to the brow experience a spike in stress as the brow passes, which in the case of a shrinking central pillar extraction sequence potentially results in stress damage to the adjacent hangingwall and footwall. Narrow-vein stope extraction involves relatively small mining increments along strike and therefore brow stresses have the potential
to affect large sections of hangingwall and footwall. Narrow-vein small incremental extraction contrasts with large open stoping blasting which is blasted in much larger increments. Therefore, in the case of large open stoping the area of stope wall surface exposed to a stress spike will usually be much less than in narrow-vein stoping. Figure 8.1 illustrates how brow stresses can affect large sections of hangingwall and footwall.

Cooper (2002) noted the potential for stope failure to be increased by poor stope sequencing. Understanding the circumstances in which this form of stress damage affects dilution enables operators to minimise dilution within the limits of economically practicable mining. Many of the causes of unplanned dilution involve significant cost to reduce their impact. In contrast, the potential to avoid stress damage related dilution through combined geotechnical and mining engineering teamwork is high. In addition, the strategies employed to reduce stress concentration may also improve mineability and reduce ore losses as discussed by Beck and Sandy (2002), and Beck and Sandy (2003).
Chapter 8 – Effect of Stress History on Dilution

Post-stoping stresses are taken into account in all variants of the Stability Graph approach (Mathews et al., 1981; Potvin, 1988; Clark et al., 1997; Mawdesley et al., 2001) but the stability graph approach does not take into account the stresses experienced by stope walls prior to stoping. As discussed in Chapter 4, Sprott et al. (1999) propose an adjustment to the stability graph approach to account for pre-mining or virgin stresses. Sprott et al. (1999) stress damage adjustments were successfully applied at the three large open-stopping Hemlo operations in Canada to predict stope stability and evaluate alternative extraction sequences. However, this approach does not consider the full stress history experienced by the stope wall, only the pre-mining stress. Therefore, this method does not take into account spikes in the stress history which may exceed the pre-mining stress.

The aim of this chapter is to examine the effect of stress damage on dilution, as well as investigate how extraction sequence impacts on brow stresses in narrow-vein mines. This has been achieved by back-analysing the stress path (32 months) of 412 stope walls from the Barkers orebody. All modelling results presented in this thesis were done by the author as part of this thesis project. Appendix D contains the second Barkers database of 412 case studies. Chapter 7 contains details of the Barkers geology and mining method. The stress path was obtained from linear elastic modelling of a 32-step extraction sequence using Map3D boundary element software. Each stage in the sequence corresponds to one month. Appendix E contains contour plots of $\sigma_n$ for each stage in the extraction sequence.

8.2 STRESS DAMAGE

In generic terms, rock is considered damaged when the strength of the rock is reduced. In fracture mechanics, damage or 'crack damage threshold' refers to the onset of irreversible volumetric strain (Bawden, 2002b). Volumetric strain is simply a measure of rock deformation. However, Wiles (2002) uses a post peak-strength damage definition. This approach is well suited to an empirical failure criterion based damage model. In practice it doesn’t really matter, provided the damage model parameters are calibrated to underground observations. As shown in the stress-strain curve shown in Figure 8.2, the onset of stress damage (or yield) marks a change from linear elastic deformation to non-linear plastic deformation (Bawden, 2002b). Up until the onset of stress damage, removal of the unloading stress-strain path follows the same linear path as loading. Prior to the onset of crack damage deformations are recoverable.
Following are four methods that could be used to estimate the potential for stress damage to occur in the brow region shown in Figure 8.1. All four methods use linear elastic numerical modelling to estimate stress levels.

1. Empirical failure criterion for rock (shear based) (eg Hoek et al., 1980).
2. Deviatoric stresses (empirical fracture mechanics based criterion) (eg Kaiser, 1994; Castro et al., 1996; Martin et al., 1996).
3. Empirical pillar yield charts (factor of safety based on strength to stress ratio) (Hedley et al., 1972; Martin et al., 2000; Lunde, 1994).
4. Site calibrated damage criterion.

A site calibrated damage criterion based upon the stress normal to strike has been used as a basis for the Barkers damage study reported in this chapter. The alternative methods for predicting stress damage have been discussed for completeness and in recognition that prior to mining it is not possible to calibrate to underground observations. Therefore, predicting stress damage related dilution requires a method for assessing stress damage potential prior to the possibility of calibrating to underground observations.

8.2.1 Empirical Failure Criterion

Ideally, empirical failure criterion should only be used when the opportunity to calibrate model parameters to underground observations exists. Wiles (2002) quantifies stress damage in terms
of ‘excess stress’ and discusses how determining an appropriate stress path to failure is an important consideration when attempting to quantify the extent of stress damage. In the case of pillar failure, Wiles (2002) suggests an increasing load type stress path. While the brow can be considered a pillar, at the early stages in the extraction sequence the edge of the very elongated pillar could be considered an abutment. Wiles (2002) suggests that in the case of an abutment, changes in shear stress would be an appropriate stress path.

### 8.2.2 Deviatoric Stress Based Damage Criteria

A deviatoric stress based damage criterion is expressed in Equation 8.1 where $\sigma_1$ and $\sigma_3$ are the maximum and minimum principal stresses respectively and $\sigma_{ci}$ is the in situ crack initiation stress (Kaiser, 1994; Castro et al., 1996, Martin et al., 1996).

$$\sigma_1 - \sigma_3 \cong \sigma_{ci}$$ 

Equation 8.1

The advantage of the deviatoric stress approach to damage estimation is its simplicity and availability of input parameters. The input parameter, $\sigma_{ci}$ can be estimated from the short-term UCS, which is usually available. The AECL Mine-by experiment indicates that the in situ crack initiation stress occurs at about 0.3 times the laboratory determined UCS (Martin et al., 1996). While the deviatoric stress approach has been calibrated to rock masses (Castro et al., 1996; Martin et al., 1996). The calibration process was limited to the massive or moderately jointed rock mass observations at the AECL’s Underground Research Laboratory experimental mine (Martin et al., 1996) and the Sudbury neutrino observatory cavern (Castro et al., 1996). This calibration also indicated that damage in massive and moderately jointed rock can be directly linked to the lab tested crack initiation threshold. Laboratory testing of rock specimens predicts crack initiation occurs when the deviatoric stress is between 0.25 and 0.5 times the UCS (Martin, 1994). The crack initiation threshold may not correspond to the onset of damage in all rock masses, especially those that are not ‘massive or moderately jointed’.

As discussed in Chapter 4, Sprott et al.’s (1999) adjustment for stress damage factor $D$, is based upon the difference between the in situ deviatoric stress and the pre-stopping deviatoric stress. It is important to note that the deviatoric stress damage criterion was calibrated in massive or moderately jointed rock masses and its applicability to fair and poor rock masses remains to be validated. Stress factor $D$ is based on case studies from three Hemlo operations in Canada and therefore its applicability may be limited to similar rock mass conditions as those encountered at these mines.
The stress factor D formulation is based on the extra stress deviator. The extra stress deviator is the difference between the pre-stoping deviatoric stress, \((\sigma_1 - \sigma_3)\) and the in situ mining deviatoric stress \((P_1 - P_3)\), as calculated in Equation 4.7. The stress damage factor is then determined using Figure 4.3. Stress factor D does not consider the full stress history experienced by the stope wall, only the pre-stoping stress. Therefore, this method does not take into account spikes in the stress history that may exceed the pre-stoping stress. The relationship between pre-stoping deviatoric stress and overbreak is discussed further in Section 8.4.4.

8.2.3 Empirical Pillar Yield Charts

The brow region is effectively at the edge of an elongated pillar. Therefore, empirical pillar yield charts based on the ratio of pillar strength to the pillar stress could be applied to predict the onset of brow stress damage. Martin et al. (2000) provide a comprehensive review and comparison of empirical pillar strength formula and charts. Brow stress damage would correspond to category three for pillar yield (fracturing in pillar walls) and the unstable region of the Confinement Formula Stability Graph (Lunde, 1994). The main advantage of this approach over the deviatoric stress approach is the very large database of rockmass conditions allowing better prediction of stress damage when there is no opportunity to calibrate to underground observations. The confinement formula stability graph is widely used in Canada and has been shown to work quite well for pillar design (Bawden, 2002a). It should be noted that the case studies plotted on the graph are limited to pillar width to height ratios less than 3. Due to the horizontal rather than vertical loading direction of vertical narrow-vein stope pillars, the ‘width’ corresponds to the bench height and the ‘height’ corresponds to the stope width. Therefore, a two metre wide stope with a 12 metre high bench has a width to height ratio of six.

8.2.4 Barkers Damage Criterion

The best method for predicting the location of areas likely to be affected by stress damage is a site calibrated stress damage model. This could be based on an empirical failure criterion, deviatoric stress, pillar stress or some other stress or strain parameter that correlates well with observed damage. At the Barkers mine, geotechnical engineers in consultation with underground operators and shift supervisors calibrated the observations of stress and strain (Figure 8.3 and Figure 8.4) with linear elastic stress modelling (Slade, 2004). This calibration process indicated that when the stress normal to strike exceeded 125 MPa rockmass damage was observed. This damage criterion corresponds to the uniaxial compressive strength of 142
Chapter 8 – Effect of Stress History on Dilution

MPa. Using stress normal to strike, site geotechnical engineers utilised numerical models to predict regions of high stress damage potential (Slade, 2004). Wiles (2002) suggests that rock mass damage is observable at approximately 10 percent strain. However, as shown in Figure 8.2 degradation of the rock mass through macro-cracking occurs when strain is only 1 percent (Wiles, 2002).

![Diagram](image1)

Figure 8.3 – Progressive deformation of the rockmass surrounding a sill drive in Strzelecki, after Luke (1999)

![Diagram](image2)

Figure 8.4 – Rock failure, reinforcement loading and failure indicators about an excavation under high stress conditions, after Beck and Sandy (2003)
8.3 STRESS PATH MODELLING

8.3.1 In Situ Stresses
AMC (2001) originally undertook hollow inclusion cell stress measurements at 319 metres depth. Since this time there have been two additional series of stress measurements undertaken at Barkers, one at 505 metres and one at 602 metres depth. Using linear regression the three measurements were incorporated into a linear regression model of in situ stresses. However for this study, it was decided that the first measurement (319 metres depth) would give a better indication of in situ stresses because the measurement was effectively in the middle of the panels considered in this study. Table 8.1 contains the in situ stress measurements at 319 metres. The stopes analysed in this study range in depth from 205 metres to 380 metres below surface. While these depths seem quite moderate by international standards, the high horizontal to vertical stress ratio means that the maximum principal stress is over three times the weight of overburden. In addition, the shrinking central pillar extraction sequence results in even higher induced stresses.

Table 8.1 – Barkers in situ stress measurement at 319 metres depth

<table>
<thead>
<tr>
<th></th>
<th>Magnitude (MPa)</th>
<th>Bearing (°)</th>
<th>Plunge (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_1$</td>
<td>26.6</td>
<td>20</td>
<td>16</td>
</tr>
<tr>
<td>$\sigma_2$</td>
<td>17.2</td>
<td>112</td>
<td>6</td>
</tr>
<tr>
<td>$\sigma_3$</td>
<td>15.5</td>
<td>221</td>
<td>72</td>
</tr>
</tbody>
</table>

As a general indicator of stress damage potential Martin et al. (1996) suggests, based on the AECL Mine-by experiment and Hoek et al. (1980) underground practical experience in massive brittle rocks, that when the ratio of the far field maximum principal stress ($\sigma_1$) to the short-term uniaxial compressive strength (UCS) exceeds approximately 0.2, crack induced damage weakens the rock. The ratio of far field in situ stress $\sigma_1$ to the UCS on the top level of the panels analysed for this study is 0.14, while on the bottom level the ratio is 0.25. This implies that the case studies considered for this study are in the range at which stress damage could be expected.

8.3.2 Extraction Sequence
As detailed in Chapter 7 the Barkers mining method is a combination of the bottom-up modified Avoca method using development waste as fill and longhole open stoping with small rib pillars. Figure 7.4 provides a schematic representation of the Barkers mining method. The
stopes considered in this chapter come from the four panels located between the 5960 metre sill drive and the 6135 metre sill drive in the Barkers mine. All bar one of the panels has three sill drives.

A central pillar extraction sequence has the advantage of a single access being sufficient to produce ore from two stope. From a mining point of view it makes sense to reduce the number of accesses not just to reduce capital costs but also for logistical reasons such as reduced tramming time for drilling equipment, less services to run and maintain etc. However, a possible limitation of this extraction sequence is that a shrinking central pillar is created and that as mining retreats and the pillar becomes smaller, stress related problems increase. Figure 8.1 illustrates how with each successive stope blast the brow retreats a small amount and a new section of stope wall experiences a spike in stress levels. This chapter evaluates the potential for this stress spike to cause stress damage related overbreak.

The 32 months of stoping have been modelled as a 32 step extraction sequence. The sequence was reconstituted from the dates recorded in the 206 Stope Report Sheets that were also the source of overbreak estimates for this study. Appendix E contains 30 Map3d long sections. Each long section depicts one month in the extraction sequence between December 2000 and February 2003. Stope Report Sheets were used by site geologists to record linear overbreak as well as ground conditions, generally. In most cases the geologist used a handheld laser electronic distance measuring device to measure the stope width. The geologist would stand on the ledge at the top of the stope in the top sill drive and take a measurement across the width of the stope. Because the stope walls tended to break to a plane this method of stope width has been estimated to have an accuracy of approximately, +/-0.25 metres over the length of the stope. However, in some cases it was not possible(safe) to make a measurement, and in these cases a estimate was made by eye standing on the lower sill drive at a safe distance from the brow. Stope Report Sheets covered approximately 80 percent of the stope in the four panels examined in this study. It was necessary to estimate the extraction sequence for the remaining 20 percent where dates were unknown.

**8.3.3 Displacement Discontinuity Model**

Linear elastic estimates of normal and deviatoric stresses have been estimated by creating a Map3d model of Barkers stoping. The estimates were then analysed with respect to the stress damage criterion and observed overbreak. The Barkers mine was modelled as a displacement
discontinuity plane as shown in Figure 8.5. Appendix E contains Map3d long sections showing $\sigma_n$ contours for each of the 32 months modelled.

The aim of the stress modelling was to capture the entire stress history for each of the 206 stopes in the database. Stresses were logged at a point corresponding to the brow for each month in the sequence. The point selected corresponded approximately (+/-10 metres) with the mid-stope span post stoping and was located half an element width from the brow. Due to this limitation in accuracy, some stopes have the same stress history. For example within the accuracy of the monthly increments modelled, the 203-204 and 205-206 stopes have effectively the same stress path history. The stope numbers correspond to the number assigned when collating the database. Odd numbers correspond to hangingwalls and even numbers correspond to footwalls. It is also worth noting that a stope was considered to be a different stope each time the stope was retreated more than five metres. Approximately, 40 percent of the stopes have walls that partially overlap with other stopes. Selecting an element size of approximately three to four metres meant that the point selected was approximately 1.5 metres to 2 metres from the edge of the brow at approximately the mid-stope height. The advantage of using a relatively large grid size meant that the stress was averaged over and along strike distance similar to a single blast. This had an averaging effect over the area of interest rather than selecting a small highly localised area of very high stress.

**Figure 8.5 – Map3d model of stress normal to strike: Barkers mine (January 2002)**
The reason for examining the full stress history was to consider whether extended periods of high stress or multiple stress spikes impacted on observed overbreak. Stress path histories were plotted for all stopes. Figure 8.6 is a typical stress path history. The peak normal stress shown in Figure 8.6 occurs at the brow. The typical stress path shown in Figure 8.6 produced a relatively slow increase in stress then spikes as the brow passes before dropping off to zero following mining. Over 80 percent of the 206 stopes analysed had stress paths similar to the typical stress path shown in Figure 8.6. Modelled normal stresses at the brow were up to five times the in situ stress levels. As shown in Figure 8.7, modelling suggests some stopes experienced moderate increases in stress up to six months prior to the brow reaching that point. These increases can be attributed to the effect of adjacent mining. Figure 8.8 illustrates a stope that experienced sustained high stress when production from this level was delayed for several months. Appendix E contains the full database of stress histories modelled for this study.

Figure 8.6 – Stress path history for the 131-132 stope (5975 to 5990 level) illustrating typical stress path for Barkers case studies
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Figure 8.7 – Stress path history for the 163-164 stope (6020 to 6040 level) illustrating moderately increasing stress associated with adjacent mining.

Figure 8.8 – Stress path history for the 203-206 stopes (6005 to 6020 level) illustrating sustained high stress.
8.4 ANALYSIS OF RESULTS

8.4.1 Effect of Normal Stress

As discussed earlier, stress and strain damage was observed in sill drives at sites where the modelled normal stress exceeds 125 MPa stress. In the panels studied none of the 206 stopes modelled had peak normal stresses greater than 125 MPa. However, back-analysis of case studies with peak normal stresses greater than 100 MPa were found to have on average 0.27 metres more overbreak than the 400 stope surfaces with peak normal stresses less than 100 MPa. Peak normal stress below 100 MPa did not affect overbreak. As discussed earlier, it is probable damage (decrease in rock mass strength) would have occurred at peak normal stress less than 125 MPa. With reference to the stage at which stress and strain damage is observable (Figure 8.2), it could be expected that stress damage effects on overbreak could occur at stress levels below the observable stress damage limit. This explains why the 100 MPa cut off for stress damage related overbreak is less than the 125 MPa criterion for observable damage.

The average overbreak for the ten case studies with normal stresses greater than 100 MPa was 0.77 metres, compared to 0.50 metres average overbreak for the 400 case studies with peak normal stresses less than 100 MPa. The t-test result indicated 94 percent confidence in this result. The validity of the t-test depends on equal variances and a normal distribution for both data sets. Figure 8.9 and Figure 8.10 are histograms of the two data sets. The distribution of the ten case studies with peak normal stresses greater than 100MPa appears to be skewed slightly to the left. This may be a function of the small number of case studies with a peak stress greater than 100 MPa.

![Figure 8.9 – Histogram of 400 case studies with peak normal stress less than 100 MPa](image-url)
Due to these concerns about the normality assumption, a Monte Carlo simulation was conducted to manually evaluate the probability that the difference between the samples was a random event. Using the random number generator function in Excel, 100 samples of ten were selected from the 412 case studies and the average overbreak evaluated for each of the 100 samples. Six of the samples had average overbreak equal to or greater than 0.77 metres. This result indicates that there is a six percent chance that the difference in overbreak was a random event and therefore confirms the t-test result of 94 percent confidence.

8.4.2 Other Factors Affecting Overbreak at Barkers

An important consideration when reviewing a statistical result is to ensure that there is not another explanation for the result, especially as there are only ten case studies in the group with peak normal stress greater than 100 MPa. In other words, is there some underlying effect that could have biased the result? This is especially important when there are only ten case studies upon which the result is based. To answer this question, comprehensive back-analyses of other stability factors was conducted.

Effect of Northern Domain

On average, the 83 northern domain case studies had 0.24 metres more overbreak than the database average of 0.5 metres even though Q values were similar. Slade (2004) notes that kinematic instability associated with the relief planes that formed due to the mobilisation/slip of healed quartz/carbonate veins following stoping was probably the cause of this. Most of these veins were not included in scanline mapping or Q classification that occurred prior to stoping.
Effect of Pillars on Overbreak
Leaving pillars resulted in 0.16 metres more overbreak than stopes with filling along strike. One hypothesis for this result is that the longhole rising required to restart the stope after each pillar could be responsible for the increased overbreak. Scoble et al. (1994) suggests based on their experience that damage initiated at the slot (or in this case rise) is likely to be greater due the higher powder factors and higher confinement. However, stopes at the ends of the orebody are also started by longhole rises and they had lower dilution than either the filled or pillar stopes.

Effect of Blasting Pattern on Overbreak
Previous back-analysis of Barkers stope stability discussed in the previous chapter demonstrated that blasting pattern significantly affected stability. The In-line pattern had 0.19 metres less overbreak on average than the Dice 5 pattern and 0.15 metres less overbreak than the Staggered pattern.

Factors that did not Affect Overbreak at Barkers
Within the range of spans and rockmass conditions collated at Barkers, overbreak was not correlated to either the stability number N, nor the hydraulic radius, S. Mid-stope stresses, as quantified by the mid-stope maximum induced stress and used in the formulation of Factor A, did not affect overbreak. Neither stress relaxation (full, partial and tangential) nor stope height (ranging from 10 to 20 metres) significantly affected overbreak at Barkers. As shown in Figure 8.11, the Extended Mathews Stability chart predicts that all 412 case studies should be stable (less than 0.5 metres overbreak). However, 144 out the 412 stope surfaces had overbreak exceeding 0.5 metres. The effect of stress damage and leaving pillars accounts for 83 of the 144 case studies with overbreak exceeding 0.5 metres. Appendix D contains the stability chart parameters for the second Barkers database.
8.4.3 Potential Impact of Other Factors on Stress Damage Conclusion

Eight out of ten of the case studies (80 percent) with peak normal stress greater than 100 MPa had a rib pillar abutment along strike, compared to 42 percent of the 400 case studies with peak normal stress less than 100 MPa. Therefore, the higher proportion of rib pillar case studies could be a source of bias and may account for some of the 0.27 metre difference between the two groups. Recalling that the average effect of a rib pillar abutment on overbreak is 0.16 metres, the effect of rib pillars can be calculated by subtracting 0.16 from four of the case studies and recalculating the average for the group. Using this averaging method, the effect of rib pillar abutments accounted for 0.06 metres of the 0.27 metres difference between the two groups. Taking into account the effect of rib pillars, the ten case studies with peak normal stresses greater than 100 MPa have an average 0.21 metres more overbreak than case studies below 100 MPa.

Similarly, 50 percent of the 10 case studies with peak normal stress great than 100 MPa are in the northern domain compared to 21 percent of the remaining 400 case studies. Therefore, the higher proportion of northern domain stopes is a possible source of bias. Taking into account that the northern domain has on average 0.24 metres more overbreak than the database average reduces the difference between the two groups by 0.07 metres.
Unfortunately, it was not possible to check whether the ten case studies with peak normal stresses exceeding 100 MPa were mined with the dice-five or the staggered patterns as blasting records were not available for the panels considered in this study.

After adjusting for the effect of rib pillars and northern domain faults the effect of stress damage on overbreak reducing from an average 0.23 metres to an average 0.10 metres. However, while 0.10 metres seems quite small it is important to recall that in a two metre wide stope 0.1 metres of overbreak from both the hangingwall and the footwall represents 10 percent dilution.

In summary, the small number of cases of high stress coupled with several factors acting simultaneously reduces confidence in the stress damage effect. Assuming that there are no other parameters which could have biased the stress damage conclusion, it is reasonable to conclude that stress damage was a significant cause of overbreak at Barkers.

### 8.4.4 Effect of Deviatoric Stress

Deviatoric stress is considered to be a good predictor of stress damage and has been used by consultants to evaluate stress damage potential at the feasibility stage when there is no possibility of stress damage calibration (Beck et al., 2003). This approach is based on the AECL’s Mine-by experiment (Martin et al., 1996) and observations in the Sudbury neutrino observatory cavern (Castro et al., 1996) which indicate that rock mass damage occurs when \( \sigma_1 - \sigma_3 \) exceeds 0.3 times the uniaxial compressive strength. Only four of the 206 Barkers stopes experienced deviatoric stresses exceeding 0.3 times the UCS (37.5 MPa). The average overbreak for these four stopes was actually less than the average overbreak for the rest of the database. However, with only four stopes exceeding the damage criterion this result was not significant.

### 8.5 STRATEGIES TO MINIMISE STRESS DAMAGE POTENTIAL

In addition to the potential for stress damage related dilution quantified in this chapter and by Sprott et al. (1999), a shrinking central pillar sequence has also been associated with ore loss related to impracticable mining conditions (Beck et al., 2002; Beck et al., 2003). Therefore, when there is a significant risk of either stress damage or seismicity, the consequences of selecting a shrinking central pillar sequence need to be evaluated carefully. Avoiding a shrinking central pillar sequence is likely to reduce stress concentration. However, in terms of maximising the NPV of an operation or project, the possible production difficulties and
increased capital costs associated with alternative extraction sequences may outweigh the benefits of reduced ore loss and dilution. In such cases where a shrinking central pillar extraction is chosen to maximise overall NPV, the potential exists for production, planning and geotechnical personnel to work together to reduce stress concentration as far as practical by evaluating extraction sequences as part of the short-term mine planning process. Modelling a detailed extraction sequence facilitates attempts to analyse the effect of alternative extraction sequence on stress concentration.

Extraction strategies which exacerbate stress concentration associated with a shrinking central pillar sequence were highlighted by ‘stepping’ through the 32 month model extraction sequence. At this point it is important to note that the principal stress at Barkers is parallel to strike and that the effect of a central pillar extraction sequence on stress concentration would be more severe if the maximum principal stress had been perpendicular to strike.

The following is a summary of the observations made when stepping through the stress history for Barkers between September 2000 and August 2002:

- The brow stress can be up to five times the in situ maximum principal stress. The typical Barkers stress path at the brow starts with an in situ stress around 20 MPa which then ramps up to an average peak normal stress of 54 MPa.
- Maintaining a relatively even retreat between levels reduces brow stresses.

Figure 8.12 and Figure 8.13 illustrate how the uneven profile created when the bottom level is retreated 28 metres ahead of the middle level increases stress concentration at the brow on the middle level.

Typically, rib pillars reduce modelled brow stress by 10 MPa provided they are within 15 metres of the brow. Underground observations indicate that within 15 metres of the brow pillars had not failed. Therefore, it is reasonable to assume that pillars were in reality bearing loads close to that predicted by linear elastic modelling. This may be a useful strategy if peak brow stresses are predicted to be close to the damage threshold. However, leaving a two metre pillar would result in 12 percent ore loss over a 17 metre strike length.
Chapter 8 – Effect of Stress History on Dilution

Figure 8.12 – Map3d contour plot of $\sigma_n$ showing uneven retreat between 6040 level and 6055 level (July 2001)

Figure 8.13 – Map3d contour plot of $\sigma_n$ showing even retreat 6040 level and 6055 level (August 2001)
Beck et al. (2002) refer to the potential of sequencing layered orebodies to manage stress concentration. While some operations extract the uppermost orebodies first to shadow lower orebodies, there is the risk that shadowing of weak ground could result in increased dilution due to stress relaxation (Beck et al., 2002). The empirical analysis of stress relaxation presented in Chapter 6 indicated that for cases of full and tangential relaxation stress factor A should be set to 0.7 when applying the various stability graphs. Therefore, alternative extraction sequences can be evaluated for both stress damage and stress relaxation potential.

By increasing the length of strike extracted in each production blast the amount of hangingwall and footwall exposed to high stresses would decrease, thereby decreasing stress damage potential. The extent to which production blasts can be lengthened along strike will depend on vein geometry, rockmass conditions and drill and blast conditions. In particular, whether longer strike length blasts result in more blast damage.

8.6 CONCLUSIONS

Narrow-vein retreat stoping is conducted in relatively small increments and depending on the extraction sequence this may result in extensive areas of the stope walls adjacent to the brow experiencing very high stress prior to stoping. Linear elastic modelling of 32 months of stope extraction at the Barker mine between 205 metres and 380 metres depth indicated that stresses normal to the brow can be up to five times the in situ stress. On average stresses normal to the brow were 54 MPa or approximately twice the in situ maximum principal stress. The stress concentration effect could be expected to be significantly higher if the maximum principal stress had been perpendicular to strike.

Stopes with peak normal stress exceeding 100 MPa had on average 0.27 metres more overbreak per stope wall than stopes with peak normal stress less than 100 MPa. The effect of stress damage on overbreak was significant at a normal stress 25 MPa less than the underground observable stress and strain damage criterion of 125 MPa. After taking into account possible bias due to the effect of rib pillars and kinematic instability in the northern domain, the average difference in overbreak reduced to 0.10 metres per stope wall. In a 1.5 metre wide stope, 0.1 metres overbreak from both the hangingwall and footwall corresponds to 13 percent dilution.

Peak stress normal to the brow has been demonstrated to be a useful parameter for assessing the potential for stress damage related overbreak. At Barkers, peak stress normal to brow
greater than 100 MPa was associated with overbreak and corresponds to a magnitude equal to 0.7 times the UCS. It remains to be seen whether a criterion of peak normal stress greater than 0.7 times the UCS for stress damage related overbreak would be applicable at other narrow-vein sites.

Four methods for evaluating stress damage potential at the brow have been proposed. The selection of a method for evaluating stress damage potential will depend on rock mass conditions as well as knowledge of the rock mass at that stage in the project’s life.

In situations where a shrinking central pillar sequence is unavoidable, the possibility of mitigating stress concentration through collaboration between production, planning and geotechnical personnel exists. Maintaining a relatively even retreat profile has the potential to reduce stress concentration. In addition, a linear elastic model of the mine would enable alternative sequences to be routinely evaluated as part of the mine planning process.
Adjustments have been proposed based on the effects of narrow-vein parameters quantified in this thesis. By examining the effect these adjustments have on the sensitivity and specificity of the stability graph predictions it possible to assess the magnitude of any increase in predictive accuracy. Adjusting for blast pattern improved stability graph accuracy from 64.0 percent to 78.9 percent, while adjusting for stress damage and leaving pillars improved accuracy from 64.9 percent to 67.3 percent. Theoretically, the combined effect of these adjustments would improve stability graph accuracy from 65 percent to 82 percent. Using a stress factor equal to 0.7 did not improve the accuracy of stability graph predictions for the Barkers case studies. A possible explanation for this result is that well inside the stable zone overbreak is not continuously related to N and HR.

Barkers, Callinan and Trout Lake mine case studies suggest that overbreak is not continuously related to N and HR, and that the relationship between HR and overbreak, and N and stability is best described by a logistical relationship. Based on this result it was concluded that the caused overbreak is unlikely to be related to geotechnical parameters if a stope plots well inside the stable zone. And therefore, in this scenario reducing stope size would not reduce dilution. This interpretation implies that the causes of narrow-vein dilution can be separated into two unrelated causes:

1. Geotechnical instability (stope size dependent).
2. Blast overbreak (unrelated to stope size).

9.1 INTRODUCTION

The premise for this thesis is that narrow-vein dilution can be minimised by improved narrow-vein stope design. The hypothesis that existing stope design techniques inadequately take into account the operating conditions affecting dilution in narrow-vein mines was supported by the
case studies presented in Chapter 7. In Chapter 7 the stability of 261 narrow-vein case studies from Barkers mine, Trout Lake mine and Callinan mine were shown to be poorly predicted by existing stability charts providing evidence supporting the thesis hypothesis and justifying further quantification of the causes of narrow-vein dilution.

In Chapters 6 through to 8, six out of the seven parameters hypothesised to affect narrow-vein dilution more than large open stoping were investigated. Figure 9.1 highlights the parameters evaluated as part of this thesis. As shown in Figure 9.1 there are additional parameters that could affect narrow-vein dilution. However, there was no reason or basis to believe that these parameters would affect narrow-vein more than large open stopes. As discussed in Chapter 3 existing stability charts are approximately 80 percent accurate. This level of accuracy is considered reasonable given the uncertainty in input parameters and possible affect of the remaining parameters in Figure 9.1. Therefore, this thesis did not attempt to address the remaining 20 percent and focused instead on the narrow-vein operating conditions shown in Figure 9.1 with the potential to greatly affect the reliability of stability charts for narrow-vein mines.

Figure 9.1 – Stope dilution parameters not explicitly considered by existing stability chart methods (those highlighted in bold are parameters evaluated in this thesis)
Table 9.1 lists parameters not explicitly considered by existing stability charts and hypothesised to have a significantly more effect on narrow-vein stability than on large open stoping stability – as depicted in Figure 9.1.

### Table 9.1 – Narrow-vein vein dilution parameters

<table>
<thead>
<tr>
<th>Narrow-vein parameter</th>
<th>Implicitly taken into account by ( Q' )?</th>
<th>Effects empirically quantified in this thesis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Undercutting of stope walls</td>
<td>Possibly</td>
<td>Undercut footwalls similar stability to hangingwalls.</td>
</tr>
<tr>
<td>Backfill abutments</td>
<td>No</td>
<td>The number of backfilled abutments does not affect narrow-vein stability. Tight filled abutments have no effect on stability.</td>
</tr>
<tr>
<td>Blast damage and overbreak</td>
<td>Possibly</td>
<td>In-line 3 pattern had on average 0.19 metres less overbreak than a dice 5 pattern. In-line 3 had on average 0.13 metres less overbreak than a staggered pattern.</td>
</tr>
<tr>
<td>Stress relaxation</td>
<td>Possibly</td>
<td>Partial relaxation does not affect stope stability. Full relaxation (( \sigma_2 ) and ( \sigma_3 ) both less than 0.2 MPa) affects stability to the extent that case studies best fit the stability chart when ( A=0.7 ). Tangential stress relaxation (( \sigma_3 ) less than 0.2 MPa with a stress direction not more than 20 degrees from parallel), also affects stability to the extent that case studies best fit the stability chart when ( A=0.7 ).</td>
</tr>
<tr>
<td>Stress damage</td>
<td>Possibly</td>
<td>Stress damage caused by the retreating brow was associated with an average increase in overbreak of at least 0.10 metres, possibly as much as 0.22 metres.</td>
</tr>
<tr>
<td>In-stope pillars</td>
<td>No</td>
<td>Leaving pillars resulted in 0.16 metres more overbreak than stopes with filling along strike. Pillar stopes did not have lower stability numbers nor are they larger or more stressed than filled stopes (statistical checks were done).</td>
</tr>
<tr>
<td>Site-specific effects</td>
<td>Possibly</td>
<td>Only very marginal site-specific effects were observed for a large open stope mine. It has been proposed that apparent site-specific effects are not truly site-specific, but are actually operating condition specific eg. narrow-vein operating conditions such as those listed in this table. It was not possible to develop a site-specific stability chart for the Barkers mine even though the minimum requirements determined in this thesis were met (&gt;150 case studies with at least 10 % unstable). The poor separation of stable and unstable case studies was attributed to narrow-vein operating conditions, in particular the effect of blast pattern, rib pillars and stress damage.</td>
</tr>
</tbody>
</table>
As discussed in Chapter 4 a number of authors suggested that because some of these parameters are rockmass dependent, they may be implicitly taken into account by $Q'$. Therefore, in Chapter 5, the possibility of site-specific charts being capable of accounting for these parameters by calibration to site rockmass conditions was evaluated. The analysis indicated that at least 150 case studies of which at least 10 percent are unstable are required to establish a reliable site-specific stability chart. This data requirement was a significant practical limitation of the site-specific stability chart option.

The possibility of developing a site-specific stability chart for the Barkers case studies was ruled out because of poor separation of stable and unstable case studies. It was not possible to separate stable and unstable case studies even when the overbreak cut off for failure was varied. This was despite the fact that there were sufficient numbers of case studies and a sufficient number of them were unstable. The Barkers case studies present compelling evidence that the development of a site-specific chart to calibrate the rockmass dependent parameters to site conditions was inadequate. This was further evidence of the need to quantify explicitly the effect the parameters contained in Table 9.1 have on narrow-vein dilution.

The effects of some of the parameters listed in Table 9.1 were quantified in Chapter 6, Chapter 7 and Chapter 8. These parameters have been termed narrow-vein operating conditions. The effect of these narrow-vein operating conditions are summarised in the last column of Table 9.1.

9.2 IMPLICATIONS OF NARROW-VEIN OPERATING CONDITIONS ON STABILITY CHART ACCURACY

As shown in Table 9.1 empirical analysis of the two Barkers databases and the relaxation database has enabled the effect of blast pattern, leaving rib pillars, stress damage, full and tangential relaxation to be quantified. A series of narrow-vein operating condition adjustments have been proposed based on these findings. In the case of blast pattern, leaving rib pillars and stress damage the mean differences shown in Table 9.1 are applied as adjustments. In the case of full and tangential stress relaxation factor $A$ is set to 0.7. As discussed in Chapter 6, setting factor $A$ to 0.7 for cases of full and tangential stress relaxation resulted in the highest level of accuracy for the case studies considered. By examining the effect these adjustments have on the sensitivity and specificity of the stability predictions of these case studies it possible to assess the magnitude of any increase in predictive accuracy.
9.2.1 Blast Pattern, Leaving Pillars and Stress Damage

All Barkers case studies except one, plotted in the stable region of the Extended Mathews stability chart. According to Clark and Pakalnis (1997), 0.5 metres of ELOS corresponds to blast damage only, while between 0.5 metres and 1.0 metre ELOS is considered minor sloughing. As discussed in Chapter 2, 0.5 metres of ELOS from the hangingwall and footwall of a stope has a significant economic consequence for a narrow-vein mine. Therefore, case studies with overbreak exceeding 0.5 metres have been classified as failures.

In the case of the Barkers 1 database, 41 case studies that plotted in the stable zone had overbreak greater than 0.5 metres. This corresponds to a sensitivity of 64.0 percent (recalling that sensitivity is the percentage of case studies that correctly plot in the stable zone). Similarly, in the case of the Barkers 2 database 144 case studies with overbreak in excess of 0.5 metres plotted in the stable zone – corresponding to a sensitivity of 64.9 percent. Only sensitivity has been considered because all except one case study plot in the stable zone, and specificity is the proportion of case studies that correctly plot as failures in the failure zone.

Table 9.2 shows the effects of applying adjustments for blast pattern, stress damage and rib pillars on sensitivity. The sensitivity of the 114 case studies plotting in the stable zone increased from 64.0 percent to 78.9 percent after adjusting for the effect of either a dice 5 pattern or a staggered pattern. There was no adjustment for the in-line 3 blast pattern as this pattern was considered the base case with only four of the 22 case studies with overbreak greater than 0.5 metres.

The sensitivity of the 410 Barkers 2 case studies improved from 64.9 percent to 67.3 percent after applying adjustments for the effect of leaving pillars and stress damage. The 0.16 metres adjustment was based on comparing rib pillar case studies to continuous filling case studies. This represents a 2.4 percent improvement in accuracy. The reason why the adjustments resulted in only a small improvement in accuracy can be attributed to the fact that only 10 of the 77 rib pillar case studies with overbreak greater than 0.5 metres changed from failure to stable after the 0.16 metre adjustment was applied. Three of these case studies had also been adjusted for stress damage. Therefore, the three stress damage case studies that changed from failure to stable had also been adjusted for leaving a rib pillar.
Table 9.2 – Effect of adjusting for stress damage and leaving rib pillars on the specificity of the second set of Barkers case studies.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Average effect on overbreak (m)</th>
<th>Database</th>
<th>No.# of case studies</th>
<th>No.# of case studies with overbreak &gt; 0.5 m before adjustment</th>
<th>No.# of case studies with overbreak &gt; 0.5 m after adjustment</th>
<th>Improvement to database sensitivity due to adjustment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Staggered blast pattern</td>
<td>0.13 m</td>
<td>Barkers 1</td>
<td>62</td>
<td>23 Sensitivity 62.9 %</td>
<td>12 Sensitivity 80.6 %</td>
<td>(23-12)/114 = 9.6 %</td>
</tr>
<tr>
<td>Dice five blast pattern</td>
<td>0.19 m</td>
<td>Barkers 1</td>
<td>31</td>
<td>14 Sensitivity 54.8 %</td>
<td>8 Sensitivity 74.2 %</td>
<td>(14-8)/114 = 5.3 %</td>
</tr>
<tr>
<td><strong>Barkers 1 Total</strong></td>
<td></td>
<td><strong>Barkers 1</strong></td>
<td><strong>115</strong></td>
<td><strong>41 Sensitivity 64.0 %</strong></td>
<td><strong>24 Sensitivity 78.9 %</strong></td>
<td><strong>78.9%-64% = 14.9%</strong></td>
</tr>
<tr>
<td>Stress damage</td>
<td>0.10 m to 0.27 m</td>
<td>Barkers 2</td>
<td>10</td>
<td>6 Sensitivity 40.0 %</td>
<td>3 Sensitivity 70 %</td>
<td>(6-3)/410 = 0.7 %</td>
</tr>
<tr>
<td>Leaving rib pillar³</td>
<td>0.16 m</td>
<td>Barkers 2</td>
<td>174</td>
<td>77 Sensitivity 55.7 %</td>
<td>67 Sensitivity 61.5 %</td>
<td>(77-67)/410 = 2.4 %</td>
</tr>
<tr>
<td><strong>Barkers 2 Total</strong></td>
<td></td>
<td><strong>Barkers 2</strong></td>
<td><strong>410</strong></td>
<td><strong>144 Sensitivity 64.9 %</strong></td>
<td><strong>134 Sensitivity 67.3 %</strong></td>
<td><strong>67.3%-64.9% = 2.4 %</strong></td>
</tr>
</tbody>
</table>

Notes:
1. 0.27 m adjustment was applied.
2. Three of the 10 rib pillar case studies were also adjusted for stress.
3. The adjustment for leaving a rib pillar is based on the difference between leaving a pillar and continuous filling.

Should additional data from the mine been available, it would have been very worthwhile to validate these adjustments for case studies from the last panels mined at Barkers. Theoretically, the combined effect of applying the adjustments for blast pattern, leaving rib pillars and stress damage could improve stope stability accuracy from approximately, 65 percent to 82 percent.

This improvement assumes that the mine continued to use blast patterns and rib pillars in the same proportions that they did in the Barkers 1 and Barkers 2 database, respectively. The number of case studies affected by stress damage may have been higher in the deeper panels. Furthermore, the findings presented in Chapter 7 and reported to the mine (Stewart and Trueman, 2001) may have affected the number of stopes blasted using either a dice 5 or staggered pattern. A letter summarising the findings of Chapter 7 and Chapter 8 and requesting access to blasting records was sent to Placerdome regional management in July 2004.

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Unfortunately, the blasting records were not available. However, Tombs (2004) made the following comments in his email reply to the letter:

“I also believe that blasting is affecting overbreak .......... The drill patterns at Barkers are normally holes drilled in line at 0.5 metre centres, this was arrived at through undocumented trial and error by the engineers and operators at Kundana over the years.”

These comments confirm the findings from the study of blast patterns reported by Stewart and Trueman (2001) and suggest the Barkers 2 case studies may have been largely blasted using an in-line pattern. However, without access to the actual records it was not possible to undertake more detailed analyses.

9.2.2 Full and Tangential Relaxation

In Chapter 6 the effect of full, tangential and partial stress relaxation on stope stability was examined. Based on this study it was recommended that the stability prediction for case studies with full or tangential stress relaxation is improved by setting stress factor A to 0.7. Table 9.3 illustrates how applying a stress factor of 0.7 to the relaxed database (Appendix B) improves the specificity by 44.5 percent in the case of full relaxation and by 36.3 percent in the case of tangential stress relaxation.

### Table 9.3 – Effect of stress relaxation adjustments on specificity, sensitivity and accuracy

<table>
<thead>
<tr>
<th>Type of Relaxation</th>
<th>Adjustment</th>
<th>Number of Case Studies</th>
<th>Change in Sensitivity</th>
<th>Change in Specificity</th>
<th>Change in Accuracy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Full stress relaxation</td>
<td>A=0.7</td>
<td>20</td>
<td>81.8 % - 90.9 %</td>
<td>88.9%-44.4 %</td>
<td>1.71-1.35</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>= -9.1 %</td>
<td>= 44.5 %</td>
<td>= 0.36 (18%)</td>
</tr>
<tr>
<td>Tangential stress relaxation</td>
<td>A=0.7</td>
<td>25</td>
<td>78.6 % - 85.7 %</td>
<td>81.8%-45.5 %</td>
<td>1.60-1.31</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>= - 7.1 %</td>
<td>= 36.3 %</td>
<td>=0.29 (14.5%)</td>
</tr>
<tr>
<td>Partial stress relaxation</td>
<td>na</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In attempt to validate the stress relaxation adjustments proposed in Chapter 6, the effect of applying the full and tangential stress relaxation adjustments on the accuracy of the Barkers 2 case studies was examined. Appendix C indicates which of the Barkers 2 case studies were fully or tangentially relaxed. Stress factor A was set to 0.7 for the 126 case studies that were either fully or tangentially relaxed. All 126 case studies continued to plot in the stable zone.
This means that setting stress factor $A$ equal to 0.7 had no effect on the accuracy of stability predictions.

Table 9.4 contains the average overbreak for each type of stress relaxation in the Barkers 2 database. t-tests indicated that the differences between overbreak for each type of stress relaxation were not significant. As shown in Table 9.4 partially relaxed case studies had on average 0.09 metres more overbreak than the six tangentially relaxed case studies and had on average 0.04 metres more overbreak than the fully relaxed case studies. However, t-test revealed that these results were not significant and therefore, do not contradict the stress relaxation conclusions reached in Chapter 6.

| Table 9.4 – Average overbreak for different types of stress relaxation: Barkers 2 case studies |
|-------------------------------------------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| Number of case studies                          | Not Relaxed     | Partially       | Fully            | Tangentially     | Fully &          |
|                                                 |                 | Relaxed         | Relaxed          | Relaxed         | Tangentially     |
| Average corrected overbreak (m)                 | 0.42            | 0.51            | 0.47            | 0.42            | 0.54            |

As discussed in Chapter 4 some authors suggest stress relaxation primarily affects weak rockmasses. The $Q'$ values for the Barkers 2 case studies ranged from 7.9 to 10.8, and indicates that rockmass would be classified as fair to good. One possible explanation for full and tangential stress relaxation not affecting stability at Barkers could be that relaxation does not affect the stability of relatively competent rockmasses. This possibility has been investigated by re-evaluating the full and tangential relaxation case studies presented in Chapter 6 with respect to $Q'$. According to the Q system (Loset, 1997) a Q value less than four represents a poor rock mass and a Q value greater than four represents a fair rockmass. Only five of the 20 fully relaxed case studies had $Q'$ values less than 4, and only seven of the 25 tangentially relaxed case studies had $Q'$ values less than 4. Table 9.5 compares the misclassification statistics for weak ($Q'<4$) and competent ($Q'>4$) full and tangential relaxation case studies.
Chapter 9 – Applicability of Stability Graphs to Narrow-Vein Dilution Prediction

### Table 9.5 – Comparison of misclassification statistics for weak and competent relaxation case studies

<table>
<thead>
<tr>
<th>Type of relaxation</th>
<th>Rockmass relaxation</th>
<th>Number of case studies</th>
<th>Stress Factor A</th>
<th>Misclassification statistics</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Fully relaxed</td>
<td>Relatively competent (Q' &gt; 4)</td>
<td>15</td>
<td>0.7, 1.0</td>
<td>Sensitivity: 80%, Specificity: 80%, Accuracy: 80%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Weak (Q' &lt; 4)</td>
<td>5</td>
<td>0.7, 1.0</td>
<td>Sensitivity: 100%, Specificity: 100%, Accuracy: 100%</td>
<td></td>
</tr>
<tr>
<td>Tangential relaxation</td>
<td>Relatively competent (Q' &gt; 4)</td>
<td>18</td>
<td>0.7, 1.0</td>
<td>Sensitivity: 83.3%, Specificity: 71.4%, Accuracy: 71.4%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Weak (Q' &lt; 4)</td>
<td>7</td>
<td>0.7, 1.0</td>
<td>Sensitivity: 50%, Specificity: 100%, Accuracy: 50%</td>
<td></td>
</tr>
</tbody>
</table>

As shown in Table 9.5 the accuracy of weak rockmass relaxation case studies was not improved by adjusting the stress factor from 1.0 to 0.7. Furthermore, the accuracy of the competent rockmass case studies appeared to improve by using the lower stress factor of 0.7. If relaxation did not affect competent rockmasses, then one would not expect the improvements in accuracy shown in Table 9.5. Using a lower stress factor for weak rockmasses does not appear to improve the accuracy of either the five fully or the seven tangentially relaxed case studies. In both cases the sensitivity and specificity remained constant irrespective of the A value assigned. However, this result is likely to be a function of the lack of data. Overall, this analysis appears to indicate that relaxation is not rockmass dependent.

#### 9.3 LOGISTICAL NATURE OF STOPE STABILITY

An alternative explanation for why full and tangential relaxation stresses did not affect overbreak at Barkers could be that within the range of spans captured in the database overbreak was not significantly affected by geotechnical parameters. In other words, if case studies had plotted closer to the stable-failure boundary then overbreak (or sloughage in the case of stress relaxation) may have been affected by full or tangential stress relaxation. In practical terms, if the spans had been large enough for geotechnical stability to be an issue, then perhaps full and tangential relaxation would have been correlated to overbreak. This indicates that from a mathematical perspective geotechnical stability is more accurately modelled as a logistical relationship rather than a continuous problem as implied by the ELOS Dilution Graph with different levels of overbreak corresponding to empirically determined contours (Figure 3.13). Further evidence of the truly logistical nature of geotechnical stope
stability is suggested by the poor correlation of N and HR to overbreak within the stable zone of the stability chart. Within the stable zone of the stability chart, overbreak was not continuously related to HR or N. This contrasts with the ELOS dilution chart approach to dilution prediction that implies that overbreak is continuously related to N and HR.

Figure 9.2 compares a logistic regression model to a linear regression model. The S shaped logistical distribution reflects that the dependent variable is relatively insensitive (flat section of S curve) to model parameters for a range of values before becoming highly sensitive to model parameters (steep section of S curve) over a small range of model parameters, and then reverting to being relatively insensitive again (flat section of S curve). Figure 9.2 depicts a single independent variable X.

As detailed in Chapter 5 and shown in Equation 5.1 the Extended Mathews stability chart has two independent variables, N and HR. Therefore, if the probabilities of the Extended Mathews stability chart were similarly plotted, the graph would be three dimensional, with the probability plotted as a surface. The insensitivity of overbreak to N and HR in the stable zone can be related to the flat section at the top of the S shaped curve shown in Figure 9.2. Similarly, the fact that within the stable zone Barkers overbreak was unaffected by full or
tangential stress relaxation suggests that stress relaxation is not continuously related to overbreak.

If stope stability is logistically related to N and HR as analysis of Barkers case studies suggests, then decreasing HR for stope wall plotting inside the geotechnically stable zone of the stability chart is unlikely to result in significant reductions in overbreak or slough. In the following sections this hypothesis is investigated further.

9.3.1 Effect of Hydraulic Radius on Dilution in the Stable Zone

In Chapter 8 the effect of independent variables HR and N on the dependent variable overbreak was examined by using the linear model measure of the extent of the relationship between two variables referred to as the correlation coefficient. Walpole and Myers (1990) define correlation coefficient as the linear association between two variables. Therefore, if overbreak is not linearly related to neither N, nor HR then the poor correlation coefficients obtained (0.08 and -0.13, respectively) could be a function of non-linearity rather than being a true indication that hydraulic radius does not affect dilution at Barkers.

Figure 9.3 is a plot of overbreak versus HR for the Barkers 1 database. As discussed in Chapter 7, this database suggests that case studies with hydraulic radii less than 3.5 metres incur less overbreak than the database average.

In contrast, Figure 9.4 is a plot of overbreak versus HR for all of the Barkers 2 case studies. In this case there is no indication that overbreak is related to hydraulic radius. In order to examine the data at the same resolution of Figure 9.3, the Barkers 2 case studies were plotted at the same scale as Figure 9.3. This resulted in all case studies with hydraulic radii greater than 10 metres being excluded from Figure 9.5. However, even after improving resolution, the Barkers 2 case studies did not demonstrate any relationship between hydraulic radius and overbreak.

In summary, while the Barkers 1 case studies seemed to suggest some relationship between hydraulic radius and overbreak, the Barkers 2 case studies did not. Therefore, further analysis was required to understand the difference in result between the two databases.
Figure 9.3 – Overbreak versus hydraulic radius for the Barkers 1 database

Figure 9.4 – Corrected overbreak versus hydraulic radius for Barkers 2 case studies
A simple sub-setting analysis has been undertaken to investigate the nature of any relationship between HR and corrected overbreak for both the Barkers 1 database and the Barkers 2 database. Figure 9.6 and Figure 9.7 examines the effect of HR on corrected overbreak at Barkers by breaking the data into the subsets shown. The aim of the analysis was to establish whether HR can be related to overbreak inside the stable zone of a stability chart.

The 20 case studies with hydraulic radii less than three had average overbreak of 0.24 metres. This represents 0.16 metres less overbreak than the 0.41 metres mean overbreak for the case studies with hydraulic radii greater than 3. A non-equal variance heteroscedastic t-test determined that this difference is significant (p value 0.0056 on one-tail basis). Because the Barkers 2 case studies did not demonstrate the same effect of HR (Figure 9.7), the possibility that this result could be a result of bias due to another parameter was investigated.

Of the 20 case studies with hydraulic radii less than 3, eight were blasted using a dice 5 pattern (40 percent) and the remaining 12 were blasted with a staggered pattern (60 percent). None were blasted with the in-line pattern which was associated with the least overbreak. In comparison, the 95 case studies with a hydraulic radius greater than three had 23 dice five, 50
staggered and 22 in-line case studies. The average effect of blast pattern for the case studies with hydraulic radii less than three was 0.154 metres. In contrast, the average effect of blast pattern for the case studies with hydraulic radii greater than three was 0.114. An adjustment for possible bias was made by subtracting the difference between these two averages (0.04 metres). After adjusting for possible bias due to blast pattern the difference is 0.12 metres. Therefore, the difference while not large remains significant.

**Figure 9.6 - Effect of HR on overbreak for the Barkers 1 database**

**Figure 9.7 – Effect of HR on overbreak for the Barkers 2 database**
In summary, while the Barkers 1 case studies indicate that HR has a small effect (0.12 metres) on dilution the much larger Barkers 2 database shows no relationship between HR and dilution. Therefore, it could be said that there is some evidence that maintaining a HR less than three could reduce dilution by 0.12 metres on average. This effect was not observed for the 410 Barkers 2 case studies. It has not been possible to isolate a reason for the difference in results. These conflicting results have been interpreted as follows: Decreasing HR to less than three metres may have the potential to decrease dilution by 0.12 metres. However, there is no evidence to suggest that HR is continuously related to dilution within the stable zone of stability charts as suggested by the ELOS Dilution Chart.

### 9.3.2 Effect of the Stability Number N on Dilution in the Stable Zone

The effect of the stability number (N) on corrected overbreak has also been analysed. Figure 9.8 and Figure 9.9 are plots of corrected overbreak versus N for the Barkers 1 database and Barkers 2 databases, respectively. Neither plot suggests that corrected overbreak decreases as N increases.
Figure 9.10 and Figure 9.11 represent the results of a sub-setting analysis that was conducted to determine if the Barkers case studies demonstrated any relationship between N and dilution. The case studies were grouped as shown in Figure 9.10 and Figure 9.11. The sub-setting analysis did not show any evidence to suggest that overbreak decreases as N increases. Therefore, the Barkers case studies suggest that within the stable zone dilution is not related to N. All but one of the Barkers case studies plot in the stable zone. The 525 Barkers case studies provide compelling evidence to suggest that within the stable zone of the stability graph N is not related to dilution. Furthermore, the poor correlation of the Callinan mine and Trout Lake mine case studies to the ELOS dilution chart provides further evidence that dilution is not continuously related to N and HR within the stable zone of a stability chart. Therefore the analyses suggest that the logistical representation of the relationship between stope stability and the stability graph parameters N and HR is more realistic within the stable zone. Under this interpretation the top flat sections of the S shaped logit model shown in Figure 9.2 represents the insensitivity of overbreak to N and HR in the stable zone. Due to absence of data below the stable failure boundary it has not been possible to test the sensitivity of N and HR closer to the stable failure boundary, not below the stable-failure boundary.
In practical terms this means that a mine where stopes continue to plot well inside the stable zone and yet incur significant dilution (and no undercutting and adjustments for relaxation have been made) then the cause of unplanned dilution is unlikely to be related to geotechnical parameters captured by the stability chart approach. This means that reducing the span is unlikely to reduce dilution.

**Figure 9.10 - Effect of N on overbreak for the Barkers 1 database**

**Figure 9.11 – Effect of N on overbreak for the Barkers 2 database**
9.4 IMPLICATIONS FOR STABILITY CHART STABLE ZONES

In terms of predicting narrow-vein dilution the analysis conducted in the previous section indicates that dilution in the stable zone is not significantly dependent on either HR or N. For example, in the case of the second Barkers case studies reducing the HR did not generally affect dilution levels. If this is true, then which initiatives are likely to reduce dilution for stopes plotting in the stable zone? Essentially the stability number captures geotechnical parameters whether they are stress related or kinematic stability related. The stability chart relationship between N and HR empirically demonstrates that these geotechnical parameters as quantified by N are stope size dependent. In contrast dilution associated with drill and blast parameters such as drill pattern, drill-hole deviation, explosive type and confinement would not be stope size dependent.

9.5 GEOTECHNICAL STABILITY VERSUS BLAST OVERBREAK

In the case of large open stoping it has been argued that blast damage is implicitly taken into account by Q’ because less competent rockmasses are more sensitive to blast damage mechanisms. While this is most probably true of the blast damage mechanisms discussed in Chapter 3, implicit consideration of blasting parameters does not seem to be sufficient for the case of narrow-vein stoping. As statistically demonstrated for the Barkers case studies, blast pattern directly affects dilution.

Blast pattern needs to be distinguished from the blast damage mechanisms discussed in Chapter 3. From this point forward dilution related to blasting parameters such as blast design, drill-hole deviation and quality control has been termed ‘blast overbreak’. Under this definition blast overbreak is quite literally breakage of the rock over the limits of the design. Dilution caused by blast overbreak is not necessarily a damage mechanism but a direct effect of the inherent difficulty of mechanised mining of narrow-vein orebodies as discussed in Chapter 2.

9.6 MINIMUM PRACTICAL STOPING WIDTH

In Chapter 2 the effect of mechanised mining on narrow-vein dilution was discussed and found to be an important parameter for narrow-vein dilution prediction. Throughout this thesis the emphasis has been on minimising unplanned dilution (outside the minimum practical mining width) whether related to geotechnical parameters or blast overbreak. While this is very important and is within the realms of control for an operating mine, it is also important to address the issue of mining method selection so that comparisons between mining methods can
be made with a more accurate understanding of the effect mechanised mining methods may have on dilution.

Figure 9.12 and Figure 9.13 are plots of overbreak from the vein at Barkers mine. Overbreak from the vein includes material inside the minimum practical blast design width (planned dilution). At Barkers the minimum practical mining width was estimated to be 0.5 metres wider than the vein (on the footwall side). Therefore, subtracting 0.5 meters from the footwall overbreak from the vein gave the corrected overbreak referred to elsewhere in this thesis. All planned dilution was on the footwall side.

In Figure 9.12 and Figure 9.13 the uncorrected overbreak from vein was plotted with the aim of examining the distribution of overbreak from the vein. A very simple but important observation can be made upon examining Figure 9.12 and Figure 9.13. There are fewer case studies with overbreak of less than 0.2 metres than there are case studies with overbreak between 0.2 metres and 0.4 metres or overbreak between 0.4 metres and 0.6 metres. This is compelling evidence that using a minimum practical mining width is valid and reflects the inherent level of dilution associated with common mechanised narrow-vein longhole stoping methods. If minimum practical stoping width were only theoretical then it would be expected that the group with the highest frequency would be the group closest to the vein. While this may seem obvious it is important to demonstrate the inherent level of planned dilution associated with mechanised mining as part of any attempt to predict narrow-vein dilution.

Minimum practical mining width is the baseline from which best-practice and substandard practice can be measured. This is the case whether the cause be geotechnical or blasting related. Therefore, determining minimum practical mining width is an essential element of predicting narrow-vein dilution and will be discussed further in the following chapter.
9.7 CONCLUSIONS

In this chapter it has been demonstrated that after taking into account blasting pattern, stress relaxation and stress damage the accuracy of stability predictions for narrow-vein case studies
were similar to the generic stope stability databases i.e. 80 percent sensitivity. The high levels of accuracy achievable after making adjustments for stress relaxation, blast pattern and stress damage suggests that the stability graph approach can be used to predict the geotechnical causes of narrow-vein dilution provided consideration of the aforementioned effects is made.

In all literature to date dealing with the subject of dilution and stope stability, the terms overbreak and stope stability are represented as being intrinsically related with the same set of causes. However, the evidence presented in this chapter suggests an alternate interpretation of the causes of narrow-vein dilution. This interpretation suggests that the parameters causing narrow-vein dilution can be separated into two independent groups:

1. Geotechnical instability.
2. Blast overbreak.

Geotechnical stope stability parameters include the stope size dependent geotechnical parameters that are well quantified by N. In practice, blast overbreak can be distinguished from geotechnical causes of dilution by analysing whether dilution occurs independently of stope size. For example, for the range of hydraulic radii captured in the two Barkers databases dilution was generally (apart from the Barkers 1 case studies with hydraulic radii less than three) unrelated to stope size. In practical terms this means that generally speaking for hydraulic radii plotting well inside the stable zone, dilution occurred independently of the distance between backfilling cycles.

As discussed in Chapter 2, prudent narrow-vein economic modelling requires accurate dilution estimates. Estimates of both planned and unplanned dilution are required. In the following chapter, Barkers 1 case studies have been used to estimate practical benchmark stoping widths. These benchmark practical stoping widths can be used as follows:

1. Estimate total dilution.
2. Provide benchmark stoping width from which unplanned dilution due to geotechnical and/or blasting causes can be assessed.
In this chapter a probabilistic benchmarking method is used to estimate benchmark stoping widths for three commonly used narrow-vein longhole blast patterns. The benchmark stability stoping width for each pattern define realistic planned dilution limits. These limits provide the basis from which true unplanned dilution can be assessed. In addition, the probabilistic overbreak model has been used to predict average stope widths for each of the patterns. Benchmark average stoping width, in conjunction with vein or ore width, can be used to estimate total dilution (planned and unplanned).

Maximum likelihood methods were used to develop the probabilistic overbreak model for the Barkers I case studies. This model was used to estimate a benchmark stoping widths for each blast pattern. The Extended Mathews Stability chart accuracy is approximately 80 percent. This accuracy has been used as the basis for selecting a probability of 80 percent for the limits of the narrow-vein benchmark stability stoping width. In other words, the maximum likelihood model has been used to estimate the stoping width incorporating 80 percent of case studies.

It has been assumed that benchmark stoping widths are not site-specific and can be considered operating condition specific. In this case, operating condition specific refers to standard narrow-vein longhole mining methods and equipment such as those used at the Barker mine. In other words, benchmark stoping widths are primarily a function of the longhole stoping method, and not the geotechnical parameters that are responsible for unplanned dilution. Based on this assumption, the benchmark stoping widths determined for the Barkers mine are applicable to similar narrow-vein longhole stoping mines. The benchmark average stoping width and benchmark stability stoping width form the basis of the narrow-vein stope design and dilution prediction method proposed in the following chapter.
10.1 INTRODUCTION

The previous chapter demonstrated that narrow-vein dilution is a function of geotechnical stability, blast overbreak and mining method. In this thesis adjustments to the stability chart approach have been proposed to improve geotechnical stability prediction for narrow-vein stopes. Adjustments have been proposed for taking into account the effect of stress damage, stress relaxation and undercutting of footwalls. In addition, the effects of blast pattern and leaving rib pillars on blast overbreak have been quantified. Evidence has been presented suggesting a practical stoping width is an important parameter when attempting to predict or analyse narrow-vein dilution. For this reason, this chapter considers the effect of blast pattern on practical stoping width for mechanised longhole narrow-vein stoping. The practical stoping width determined for the Barkers mine has been considered a benchmark from which other narrow-vein mines can predict and analyse dilution. For these reason, the practical stoping widths determined for Barkers have been termed ‘benchmark stoping widths’.

Barkers mine is considered an excellent benchmark mine because as shown in Chapter 9, dilution (linear overbreak) at the mine is unaffected by rockmass conditions and dilution is independent of stope span (within the limits of the database). While stress damage had a significant affect on 10 cases studies in the deeper Barkers 2 database, the shallower Barkers 1 database is highly unlikely to be affected by stress damage and is the database that has been used to determine benchmark stoping widths. Rockmass classification based on scanline mapping of sill drives indicates that the rockmass for the Barkers 1 database ranges from fair to good. It is reasonable to expect that mines with poor rockmass conditions would experience higher levels of blast overbreak than that incurred at Barkers. Further case studies would be required to establish the effect of a poor rockmass on blast overbreak.

Later in this chapter benchmark stability stoping width for various vein widths have been used as the benchmark from which true unplanned dilution can be estimated. For the reasons discussed in Chapter 2, this thesis has focused on minimising dilution in narrow-vein longhole stoping mines. Therefore, the benchmark stoping width referred to in this chapter refers to narrow-vein longhole stoping.

Two types of benchmark stoping width have been defined:

1. Benchmark stability stoping width is for geotechnical assessments of unplanned dilution.
2. Benchmark average stoping width is for predicting average dilution and provides a baseline for analysing the causes of dilution (independent of stope design outline).

While benchmark stability stoping width can be used as the basis for analysing geotechnical stability with respect to practical stoping widths, benchmark stability stoping width does not indicate average stoping widths. An estimate of an average stoping width enables total dilution to be predicted for stopes plotting in the stable zone of a stability graph. Furthermore, benchmark average dilution also provides a measure of dilution that is independent on biases associated with trends in setting the stope design outline. An average value is equivalent to the mean or expected value of a normal distribution. However, hangingwall and footwall overbreak is not normally distributed. Therefore, expected total dilution for each blast pattern has been evaluated using the parameters which best define the overbreak distributions for each of the three blast patterns.

Benchmark average stoping width facilitates realistic dilution estimates for mechanised narrow-vein stoping. This enables realistic cost comparisons between mechanised and conventional mining methods and integral part of mine method selection.

10.2 BENCHMARK STABILITY STOPING WIDTH

As defined in Chapter 1 and Chapter 2, this thesis is primarily concerned with minimising dilution by improved prediction and quantification of the causes of narrow-vein dilution. More accurate dilution prediction facilitates improved mining method selection. True unplanned dilution measures dilution from a practical stoping width. For example, there is no point measuring unplanned dilution from the vein when it is not possible in practice to only mine the vein without dilution. Minimising unplanned dilution at an operating mine requires a reasonable baseline or benchmark from which assessments and reconciliations can be conducted. Setting a reasonable benchmark is particularly important if the experience at one mine is to be related to another mine, as is the case when applying empirical methods.

Because the outcomes and applications proposed as a result of this thesis project are empirical it is very important that a standardised approach to estimating the boundary between planned and unplanned dilution be determined. For example, if one mine measures their unplanned dilution from a vein that is on average 0.8 metres wide and another mine measures their unplanned dilution from a standard 1.3 metre wide stoping width then the possibility of...
relating the experience obtained at one mine to another mine is limited unless a common baseline is adopted.

In the following sections, Barkers case studies have been used to estimate a set of benchmark stability stoping widths for various vein widths and blast patterns.

10.3 BENCHMARK AVERAGE STOPING WIDTH

Predicting dilution at the feasibility stage requires an estimate of expected(average) stoping width from which dilution can be predicted.

10.3.1 Stopes Plotting in the Stable Zone

For stopes plotting in the stable zone, an estimate of average stoping width for each blast pattern can be used as the basis for predicting narrow-vein dilution. If stopes are designed in the unstable zone of a stability chart, then stoping widths will on average exceed the expected stoping for a particular blast pattern. Dilution for stopes plotting in the stable zone can be estimated from vein width and benchmark average stoping width as follows:

\[
Total\ Dilution = \frac{(Expected\ Stope\ Width - Vein\ Width)}{Vein\ Width}
\]

Equation 10.1

Deposits with more complex geometry and structural influences are likely to incur more unplanned dilution than a simple tabular vein with gradual changes in strike and dip. The Barkers average stoping width is a benchmark from which planned dilution in more complex geological formations can be adjusted.

10.3.2 Stopes Plotting in the Unstable or Failure Zone

For stopes plotting in the unstable zone of a stability chart it can be expected that unplanned dilution due to geotechnical factors will be higher. As demonstrated by the Extended Mathews isoprobability contours (Mawdesley et al., 2001) shown in Figure 3.12, the probability of instability increases the further below the stable-failure boundary the case study plots. Therefore, the percentage of stopes exceeding the benchmark stability stoping width is likely to exceed 20 percent.
In Chapter 9 evidence was presented suggesting that the volume of dilution or ELOS is not continuously related to the stability number and hydraulic radius as suggested by the ELOS Dilution Chart. Based on the evidence presented in Chapter 9 two stope case studies could plot in the same position on the stability graph and yet incur different volumes of unplanned dilution. Trout Lake mine and Callinan mine case studies shown in Figure 7.9 showed poor correlation to the ELOS Dilution Chart. Therefore, it is suggested that considering failure mechanisms on a case-by-case basis best assesses unplanned dilution for stopes plotting in the failure zone. For example, a small block size rock mass with full relaxation could be expected to unravel less dramatically over time. However, if large pervasive structures are likely to dominate stability then large failures could result in large volumes of dilution.

10.4 ESTIMATION OF BENCHMARK STABILITY STOPING WIDTHS

The benchmark stability stoping widths determined for various vein widths in this thesis are only applicable to narrow-vein longhole open stoping. It has been assumed that benchmark stoping widths are operating condition specific and not site-specific. For this reason, the benchmark stoping widths determined in this chapter are generally applicable to sites with similar operating conditions and geological formation.

The vein width ranged from approximately 200mm to 400mm for the levels contained in the Barkers 1 database. It is important to note that staggered and in-line patterns were generally used for the narrower veins widths while dice-5 was generally used for the wider veins. However, because vein width was not recorded an average of 0.3 metres has been applied for all three blast patterns.

The probabilistic approach used to determine benchmark stoping widths also facilitate consideration of the effect of blast pattern on ore loss potential.

The Barkers blasting practices are described in the following section. The Barkers drill and blast patterns are typical of narrow-vein longhole stoping practices.

10.4.1 Barkers Longhole Blasting Practices

The Barkers 1 database has been used as the basis for estimating benchmark practical stoping widths for a range of vein widths captured in the Barkers database (0.2 metres to 0.4 metres). The benchmark narrow-vein stoping widths proposed in this chapter are based on the blast patterns shown in Figure 10.1. All holes were drilled at 64 mm diameter using an Atlas Copco
H157 longhole rig and were less than 15m long. Hangingwall holes were blow-loaded with a low impact ANFO product (polystyrene balls added to a product called Sanfold 50). However, when blasting against fill the first two holes were loaded with 100 percent ANFO. Holes were double primed with one booster 2.5 metres from the toe and one booster halfway between the collar and the first booster. Blow-loading refers to the use of high pressure air to blow and compact ANFO or low impact ANFO into the hole. Blow-loading increases the density of the explosive in the hole compared to pour-loading (larger diameter down-holes). Blow-loading is used to load ANFO into up-holes. Footwall and in-line drillholes were blow-loaded with ANFO. All holes were initiated with long delay non-electric detonators. The UCS of the quartz vein (ore) was 130 MPa (standard deviation of six from three tests). The UCS of the stope walls was 142 MPa.

The conditions and blast practices described would be considered standard practice for Australian subvertical tabular narrow-vein gold mines. Therefore, the results of the benchmarking study would be applicable in mines with similar conditions employing similar blasting practices and with similarly narrow veins. In this study only vein width has been considered. However, discontinuities in the orebody such as offsetting caused by faulting, pinching and swelling of the orebody as well as sudden changes in strike and dip of the orebody will affect the practical stoping width for a particular orebody.

10.4.2 Ore Loss Potential

By analysing the overbreak distributions for different blast patterns it is possible to evaluate ore loss potential as well as dilution. Although an in-line pattern may have a smaller practical stoping width than staggered and dice-5 patterns (refer to Chapter 7 for a description of these
patterns), in some cases the vein width may be such that adopting an in-line pattern could result in ore-loss. Depending on the grade, ore loss can have a worse economic implication than dilution. Therefore, determining a benchmark practical stoping width needs to take into account ore loss, as well as unplanned dilution. By analysing the overbreak distribution with respect to vein width it is possible to examine the effect of blast pattern on both the probability of dilution and probability of ore loss.

10.5 FITTING A DISTRIBUTION TO BARKERS OVERBREAK DATA

10.5.1 Selecting Appropriate Distribution Functions

Table 10.1 contains the overbreak values for each of the three blast patterns. Figure 10.2, Figure 10.3, and Figure 10.4 are histograms of overbreak for hangingwall case studies. It is clear from the histograms that the hangingwall overbreak follows a distribution that is centred on small distances, while the footwall overbreak follows a distribution that is centred at some finite distance. The small numbers of observations make the precise definition of the distributions difficult. Figure 10.5, Figure 10.6, and Figure 10.7 are histograms of footwall overbreak for the Barkers 1.
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<th>Staggered</th>
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Figure 10.2 – Overbreak from vein for Barkers 1 hangingwall case studies with Dice 5 pattern

Figure 10.3 – Overbreak from vein for Barkers 1 hangingwall case studies with Staggered pattern

Figure 10.4 – Overbreak from vein for Barkers 1 hangingwall case studies with an In-line pattern
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Figure 10.5 – Overbreak from vein for Barkers 1 footwall case studies with Dice 5 pattern

Figure 10.6 – Overbreak from vein for Barkers 1 footwall case studies with Staggered pattern

Figure 10.7 – Overbreak from vein for Barkers 1 footwall case studies with In-line pattern
Because the data sets are small, the simplest possible distributions have been chosen. For the hangingwall distribution, the exponential density provides a single parameter distribution that is heavy near the origin and also reflects that drillholes were offset from the hangingwall contact. In contrast, as shown in Figure 10.1 dice 5 and staggered patterns both result in drillholes being positioned beyond the footwall contact in the footwall. Therefore, planned dilution for staggered and dice 5 patterns was largely taken on the footwall side of the stope and this explains the difference between hangingwall and the footwall distributions. Therefore, the most probable distance of overbreak in the footwall will be finite and the overbreak will be distributed around the most probable distance. The logistic distribution provides a simple symmetric probability density of this type. The exponential probability density is given by:

\[
p(x) = \frac{1}{\lambda} \exp \left( -\frac{x}{\lambda} \right) \quad x > 0; \lambda > 0 \quad \text{Equation 10.2}
\]

and has the shape given in Figure 10.8. The exponential distribution has an average or expected value of \( \lambda \) and a standard deviation of \( \lambda \). This is a property the exponential distribution. The logistic probability density is given by:

\[
p(x) = \frac{1}{\lambda} \frac{\exp \left( -\left( \frac{x - x_0}{\lambda} \right) \right)}{\left[ 1 + \exp \left( -\left( \frac{x - x_0}{\lambda} \right) \right) \right]^2} \quad -\infty < x, x_0 < \infty; \lambda > 0 \quad \text{Equation 10.3}
\]

\( \lambda \) provides the distance scaling value and is a uniform measure of the distribution spread. The logistic density is illustrated in Figure 10.9. The mean or expected value of the logistic distribution is \( x_0 \). The corresponding cumulative distribution functions are:

\[
P(x) = 1 - \exp \left( -\frac{x}{\lambda} \right) \quad \text{Equation 10.4}
\]

\[
P(x) = \frac{1}{1 + \exp \left( -\frac{x - x_0}{\lambda} \right)} \quad \text{Equation 10.5}
\]
10.5.2 Maximum Likelihood Method for Hangingwall Distribution

Maximum likelihood is a probabilistic analysis method that uses the probability density function to provide a method of estimating the parameters of the distribution. While the probability density indicates the probability of observing a particular value of the random variable, given the form of the probability density and the parameter values, the likelihood function provides likelihood that the parameters of the distribution have particular values, given the form of the distribution and the observed data.

Figure 10.8 – Exponential probability density and cumulative distribution functions

Figure 10.9 – Logistic probability density and cumulative distribution functions
For the exponential density, when the $N$ observations, $x_i$, are independent, the likelihood is written as:

$$L(\lambda | \mathbf{x}) = \prod_{i=1}^{N} \left( \frac{\Delta x}{\lambda} \exp\left( -\frac{x_i}{\lambda} \right) \right)$$ \hspace{1cm} \text{Equation 10.6}$$

where $\Delta x$ is a narrow interval containing each of the observed values $x_i$. To find the best value of the parameter $\lambda$, the likelihood is maximised with respect to the value of the parameter. To determine whether the exponential density really provides an adequate description of the data, a goodness-of-fit test is necessary.

Mathematically it is more convenient to deal with the natural logarithm of the likelihood, the log-likelihood (LL). For the exponential, the LL is given by;

$$\Lambda = N \ln(\Delta x) - N \ln(\lambda) - \frac{1}{\lambda} \sum_{i=1}^{N} x_i$$ \hspace{1cm} \text{Equation 10.7}$$

Maximising the value of the likelihood is equally well achieved by maximising the value of the LL, as the logarithm is a monotonic function. In relation to maximising the LL with respect to the value of $\lambda$, the value of $\Delta x$ is not material. Maximisation can be carried out by differentiating and setting the derivative to zero and solving the resulting equation. The result is:

$$\hat{\lambda} = \frac{1}{N} \sum_{i=1}^{N} x_i$$ \hspace{1cm} \text{Equation 10.8}$$

The value of the LL at the maximum is then:

$$\Lambda_{ML} = -N \left( 1 + \ln\left( \hat{\lambda} \right) \right)$$ \hspace{1cm} \text{Equation 10.9}$$

where the arbitrary term in the interval width has been omitted.
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For the logistic density, the LL is given by:

\[
\Lambda = N \ln(\Delta x) - N \ln(\hat{\lambda}) + \sum_{i=1}^{N} \ln \left( \frac{f(x_i)}{[1 + f(x_i)]^2} \right)
\]

Equation 10.10

where,

\[
f(x_i) = \exp \left[ \frac{x_i - x_0}{\hat{\lambda}} \right]
\]

Equation 10.11

The most effective means of estimation of \( x_0 \) and \( \hat{\lambda} \) is direct maximisation of the LL with respect to the two parameters using an appropriate numerical algorithm. The Monte Carlo method has been used to estimate the confidence interval for the two parameters.

10.5.3 Goodness of Fit Analysis

To determine the extent to which it is possible to state that the exponential density is a reasonable 'fit' to the data, it is necessary to know the statistical distribution of \( \Lambda_{ML} \) over many sets of data with \( N \) data points, all of which are drawn from an exponential distribution having parameter \( \hat{\lambda} \). The most effective means of finding this distribution is by direct simulation (Monte Carlo method). The determination of the goodness-of-fit is then a simple matter of comparing the value of \( \Lambda_{ML} \) for the data at hand with the distribution of the simulated values. If \( \Lambda_{ML} \) is too negative in relation to the simulated values, then the fit is deemed poor and it is necessary to investigate the use of a different density function to describe the data.

It is also important to find a confidence interval for the parameter estimate. There are three related means of finding a confidence interval. First, when the data set is large enough, the ML method indicates that the log-likelihood ratio follows a chi-squared distribution with the number of degrees of freedom equal to the number of parameters that have been estimated. This ratio can also be used to find a confidence interval for a single parameter of the distribution by holding all but the parameter of interest constant at the ML estimate. The convergence of the log-likelihood ratio to a chi-squared distribution further implies that the parameter distribution will be normal.
When the data set does not have a sufficient number of observations in it, the likelihood ratio will not be chi-squared and Monte Carlo methods will need to be used. Using Monte Carlo methods, the most effective means of determination of confidence intervals for the parameter estimates is to generate them from the Monte Carlo process itself. This method is limited in practical applicability to problems having less than three parameters.

The third method finding a confidence interval can be applied when the data set is large and the LL function not too complex in form. In such a case, the Cramer-Rao theorem can be used to calculate an analytical expression for the minimum variance of the parameter estimate and the fact that the LL ratio follows a chi-squared distribution can be used to show that the parameter estimate follows a normal distribution. In the case of the exponential density, the result for the variance of the estimator is:

\[
\sigma_\lambda^2 = \frac{\lambda^2}{N}
\]

\textit{Equation 10.12}

In the analysis of each data set, the smallest set will be the most critical in terms of validation of the method of confidence interval determination. The hangingwall data with an in-line blast pattern is the smallest with 12 observations of overbreak.

The visual assessment of the fit for the hangingwall distribution, illustrated in Figure 10.10, appears to be satisfactory. To quantify the goodness-of-fit, a Monte Carlo analysis of the problem was made and the distribution of the log-likelihood function was determined for a data set of 12 observation drawn from an exponential density having the fitted parameter value. The LL distribution is given in Figure 10.11 which also shows the distribution of the parameter estimates derived from the Monte Carlo calculations.

The LL value from the fitting is 1.03. The distribution function indicates that given that the data are drawn from an exponential distribution, the probability of observing a LL value lower than 1.03 is approximately 0.47. Such a result indicates that the exponential density provides a good description of the observed data. The normal score plot of the parameter values from the Monte Carlo study indicates that the distribution of the parameter estimate is almost normal for this size of problem. On a normal score plot, data forming a straight line are normally distributed. The slight departure from the straight line in Figure 10.11 indicates that the distribution of the parameter values has some slight skewness to the right compared to a
normal distribution. Use of a normal distribution to calculate a confidence interval will be acceptable as long as intervals beyond 95 percent are not considered. The results of the Monte Carlo calculation indicates that for data sets larger than 12 values, the assumption of normality of the parameter estimate is reasonable given that the fit passes the goodness of fit test. The variance of the parameter estimate can be taken to be provided by Equation 10.12.

Figure 10.10 – Comparison of the observed data with the fitted exponential density for the in-line blasting pattern for hangingwall overbreak. The fitted value is $\lambda = 0.338$
The goodness-of-fit determinations for the other two hangingwall data sets show almost identical levels of goodness-of-fit of the exponential density to the observations. The results of the goodness-of-fit are provided in Table 10.2. Figure 10.12 and Figure 10.13 provide a visual indication of the fit to the data.
Figure 10.12 – Comparison of the observed data with the fitted exponential density for the staggered blasting pattern for hangingwall overbreak (\( \lambda = 0.456 \))
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Figure 10.13 – Comparison of the observed data with the fitted exponential density for the ‘dice five’ blasting pattern for hangingwall overbreak ($\hat{\lambda} = 0.380$)

For the footwall data, the smallest data set has only 10 observations. The goodness-of-fit of the logistic density to the data is very acceptable. Figure 10.14 shows the distribution of the LL for the data set structure and a scatter diagram of the two estimates over the 1000 Monte Carlo calculations. While there is no reason to expect that the two parameter estimates will be uncorrelated, the scatter diagram shows this to be true.
The correlation coefficient between these two estimates using the Monte Carlo results is -0.038 which is effectively zero. Similar results are found for the two other footwall data sets. The estimate of the intercept, \( \hat{x}_0 \) is found to be normally distributed, but the estimate of \( \hat{\lambda} \) is again slightly skewed to higher values. Figure 10.15, Figure 10.16, Figure 10.17, Figure 10.18, Figure 10.19, and Figure 10.20 illustrate the fit of the distribution to the data. The results for the analysis of the footwall data are also provided in Table 10.2.
Figure 10.15 – Comparison of histograms of raw data and fits to the distributions for in-line

Figure 10.16 – Comparison of histograms of raw data and fits to the distributions for staggered

Figure 10.17 – Comparison of histograms of raw data and fits to the distributions for dice five patterns
Figure 10.18 – Comparison of distribution functions of raw data and fits to the distributions for in-line pattern

Figure 10.19 – Comparison of distribution functions of raw data and fits to the distributions for staggered pattern

Figure 10.20 – Comparison of distribution functions of raw data and fits to the distributions for 'dice five' pattern
The final statistical calculation to be made is to determine whether the footwall overbreak models can be applied to the hangingwall data and vice versa. A goodness of fit test were carried out using the parameters of the opposite model to each data set. In no case was there more than a 1 percent probability that the opposite model could explain the data. One may conclude from this result that the distributions of overbreak for the hanging wall and the footwall are different and distinct.

10.6 DERIVATION OF BENCHMARK STABILITY STOPING WIDTHS USING DISTRIBUTION FUNCTIONS

It is clear from the analysis conducted in the previous section that overbreak distributions for the hangingwall and footwall differ in that for the footwall, the most probable overbreak distance is finite and is represented by the parameter $x_0$ while for the hangingwall, the most probable overbreak is zero. The expected value of the overbreak for the hanging wall is $\hat{\lambda}_{hw}$ and that for the footwall is again $x_0$. The standard deviation of the hangingwall overbreak around the mean is $\hat{\lambda}_{hw}$ and that for the footwall is $1.64\hat{\lambda}_{fw}$.

The practical objective of the data analysis is to define a practical stoping width for common narrow-vein blasting patterns. Conceptually, the benchmark stability stoping width is the width of the vein to be recovered plus regions of planned dilution that must be reasonably expected for a given blasting pattern. The difficulty lies in making a definition of the planned dilution widths. To be reasonable, the definition of the width of the region of damage must be one that will not be exceeded with an excessive probability. The definition of 'excessive' is necessarily subjective. The stability graph accuracy of 80 percent is considered reasonable. This implies that stopes are generally designed assuming a 20 percent probability of overbreak or failure significantly exceeding design. On this basis, it has been assumed that a practical narrow-vein benchmark stability stoping width should include 80 percent of case studies.

It is proposed that the benchmark stability stoping width be based on a overbreak distance $x_{80}$, the stope width that will not be exceeded in 80 percent of blasts. The benchmark stability stoping width (BSW) is then defined as:

$$BSW_{0.3} = \text{vein width} + \left(x_{fw} + x_{hw}\right)_{80}$$

Equation 10.13

with all distances measured in meters.
The quantity \( (x_{fw} + x_{hw})_{80} \) is the 80 percent point on the distribution function of the sum of the footwall and hangingwall overbreaks. To determine this point, it is necessary to determine the distribution function for the sum of the footwall overbreak, which follows the logistic density and the hangingwall overbreak, which follows an exponential density. Mathematical statistics indicates that the density function for the sum of the overbreaks is:

\[
p(x_i) = k \frac{x_i}{\lambda_1 \lambda_2} \exp \left[ - \frac{x_i - u}{\lambda_1} \right] \frac{f(u)}{(1 + f(u))^2} \, du \quad x_i > 0 \quad \text{Equation 10.14}
\]

where

\[
f(u) = \exp \left[ - \left( \frac{u - x_0}{\lambda_2} \right) \right] \quad \text{Equation 10.15}
\]

\[
k^{-1} = \frac{1}{\lambda_2} \int_{-\infty}^{0} \frac{f(u)}{(1 + f(u))^2} \, du \quad \text{Equation 10.16}
\]

The density cannot be expressed in closed form and must be calculated by numerical techniques. Note that \( \lambda_1 \) is the value associated with the hangingwall overbreak density and \( x_0 \) and \( \lambda_2 \) are the parameters for the footwall overbreak density. The densities and distribution functions for the three blasting patterns are provided in Figure 10.21, Figure 10.22, and Figure 10.23. The similarity between the dice five and staggered patterns is evident; the in-line distribution is narrower. The 80% points of the distributions are provided in Table 10.2.

While the staggered and dice five patterns can be adjusted to accommodate veins of increasing width, the in-line pattern is drilled into the centre of the vein. There is potential for loss of ore if the in-line pattern is used in a vein of substantial width. To retain symmetrical definitions relating to practical stoping width and ore loss, it is reasonable to define the critical vein width as that which ore loss will become significant when the vein width is wide enough to lead to ore loss in 20 percent of the blasts. The cumulative distribution curve for in-line blasting in Figure 10.23 indicates that total overbreak is less than 0.66 metres 20 percent of the time. Taking account 0.3 metres nominal vein width, this result indicates that when vein width exceeds 0.96 metres, there will be a 20 percent chance of underbreak with an in-line pattern. The curve can be used to estimate ore loss probability as a function of vein width. Using
Figure 10.23 there is a 10 percent probability of oreloss if the vein width exceeds 0.8 metres and there is a 5 percent probability of oreloss if the vein width exceeds 0.7 metres.
Figure 10.23 – Probability densities and distribution functions for the total overbreak on in-line pattern

Table 10.2 – ML parameter fits, calculated critical widths and predicted benchmark stability stoping width for a 0.3 metres wide vein.

<table>
<thead>
<tr>
<th>Pattern</th>
<th>$\lambda_{hw}$ (m)</th>
<th>$\lambda_{fw}$ (m)</th>
<th>$x_0$ (m)</th>
<th>$\left(x_{fw} + x_{hw}\right)_{80}$</th>
<th>0.3 m vein</th>
</tr>
</thead>
<tbody>
<tr>
<td>In-line</td>
<td>0.338</td>
<td>0.153</td>
<td>0.650</td>
<td>1.30</td>
<td>1.60</td>
</tr>
<tr>
<td>Staggered</td>
<td>0.456</td>
<td>0.255</td>
<td>0.778</td>
<td>1.73</td>
<td>2.03</td>
</tr>
<tr>
<td>Dice five</td>
<td>0.380</td>
<td>0.277</td>
<td>0.979</td>
<td>1.85</td>
<td>2.15</td>
</tr>
</tbody>
</table>

10.7 BENCHMARK AVERAGE STOPING WIDTH FOR LONGHOLE NARROW-VEIN STOPING

The benchmark stability stoping width determined in the previous section provides a realistic benchmark width for longhole narrow-vein mines. Realistic has been taken to mean that 80 percent of narrow-vein case studies will be less than 1.6 metres, 2.0 metres and 2.1 metres for the cases of in-line, staggered and dice 5 patterns, respectively. Benchmark stability stoping width can therefore be used to assess unplanned dilution. However, benchmark stability stoping width cannot be used to estimate total dilution. Total dilution can be estimated from expected total overbreak. Expected total dilution can be estimated from benchmark average stoping width using probabilistic techniques.

Because the footwall and hangingwall overbreaks are considered to be independent, random variables in this analysis, the expected total overbreak is simply the sum of the two expected values:
Chapter 10 – Predicting Dilution and Benchmark Narrow-Vein Stoping Widths

\[
E\left[ x_{hw} + x_{fw} \right] = \lambda_{hw} + x_0 \quad \text{Equation 10.17}
\]

and the standard deviation of the total overbreak is determined from the sum of the variances of the overbreak:

\[
SD\left\{ x_{fw} + x_{hw} \right\} = \sqrt{1.64^2 \sigma_{fw}^2 + \lambda_{hw}^2} \quad \text{Equation 10.18}
\]

The estimated means and standard deviations of the total overbreak are given in Table 10.3. Note that confidence intervals on the total overbreak should be determined from the distribution functions of total overbreak plotted in Figure 10.21, Figure 10.22, and Figure 10.23.

<table>
<thead>
<tr>
<th>Pattern</th>
<th>Mean (m)</th>
<th>SD (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>In-line</td>
<td>0.99</td>
<td>0.39</td>
</tr>
<tr>
<td>Staggered</td>
<td>1.23</td>
<td>0.56</td>
</tr>
<tr>
<td>Dice five</td>
<td>1.36</td>
<td>0.52</td>
</tr>
</tbody>
</table>

Table 10.3 – Estimated mean and standard deviation of the total overbreak

As noted previously, while the staggered and dice five patterns can be adjusted to accommodate veins of increasing width, the in-line pattern is drilled into the centre of the vein. There is potential for loss of ore if the in-line pattern is used in a vein of substantial width. Furthermore, in some rockmasses the competence may be such that an in-line pattern presents a significant risk of freezing or bridging of the stope. In such a situation a staggered or dice-5 pattern may be necessary. Table 10.4 contains the benchmark average stoping width for each blasting pattern. These average (expected) stoping widths for each blast pattern were calculated using Equation 10.17. In the following chapter a method for estimating total dilution using benchmark average stoping width for each blast pattern has been proposed. Benchmark average stoping width can be used in conjunction with vein width to predict total dilution for stopes plotting in the stable zone.
Table 10.4 – Benchmark average stoping width for each blast pattern

<table>
<thead>
<tr>
<th>Pattern</th>
<th>Average(expected) total overbreak from vein (m)</th>
<th>Benchmark Average stoping width (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>In-line</td>
<td>0.99</td>
<td>1.3</td>
</tr>
<tr>
<td>Staggered</td>
<td>1.23</td>
<td>1.5</td>
</tr>
<tr>
<td>Dice 5</td>
<td>1.36</td>
<td>1.7</td>
</tr>
</tbody>
</table>

10.8 UNCERTAINTIES IN THE RESULTS

The analysis up to this point has not considered the uncertainties in the data; only one test has been made to ensure that the hangingwall overbreak distribution shape is distinct from the logistic density used for the footwall. Given the size of the data sets, one can anticipate that the confidence intervals for the fitted parameters will be wide. Table 10.5 indicates the 95 percent confidence intervals for the fitted parameters and Figure 10.24 illustrates the intervals graphically. Significant overlap of the intervals indicates a lack of statistical significance in the difference between the parameter values. The greatest overlap is present in the data for the hangingwall parameters. The overbreak distributions for all three patterns are quite similar. However, the $x_0$ values for the in-line and dice five patterns are almost certainly significantly different.

Table 10.5 – 95 percent confidence intervals for the fitted parameters (table entries are the lower and upper bounds of the interval)

<table>
<thead>
<tr>
<th>Pattern</th>
<th>Hangingwall</th>
<th>Footwall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\lambda$ (m)</td>
<td>$\lambda$ (m)</td>
</tr>
<tr>
<td>In-line</td>
<td>0.147</td>
<td>0.071</td>
</tr>
<tr>
<td></td>
<td>0.528</td>
<td>0.235</td>
</tr>
<tr>
<td>Staggered</td>
<td>0.296</td>
<td>0.177</td>
</tr>
<tr>
<td></td>
<td>0.617</td>
<td>0.333</td>
</tr>
<tr>
<td>Dice five</td>
<td>0.188</td>
<td>0.161</td>
</tr>
<tr>
<td></td>
<td>0.572</td>
<td>0.393</td>
</tr>
</tbody>
</table>
Figure 10.24 – Graphical representation of the 95 percent confidence intervals for the fitted parameters

10.9 APPLICATION OF BENCHMARK STOPING WIDTHS TO BARKERS 1 CASE STUDIES

Ideally, an independent data set would have been used to validate the benchmark stoping widths determined in this chapter. In the absence of a validation set of data to check the general validity of the benchmark stoping widths, the validity of the methodology has been checked using the original data set to check the modelling methodology. As defined in Equation 10.13 the benchmark stability stoping width is given by the quantity \( (x_{fw} + x_{hw})_{80} \), which is the 80 percent point on the distribution function. Therefore, it follows that percentage of stopes with widths greater than the benchmark stability stoping width should be approximately, 20 percent. Table 10.6 contains the stoping width assuming a 0.3 metre vein width.
Table 10.6 – Barkers 1 case studies Stope Widths (assuming vein width = 0.3m)

<table>
<thead>
<tr>
<th>Dice 5 Stope Widths (m)</th>
<th>Staggered Stope Widths (m)</th>
<th>In-line Stope Widths (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>1.1</td>
<td>1.3</td>
</tr>
<tr>
<td>2.4</td>
<td>1.05</td>
<td>1.0</td>
</tr>
<tr>
<td>1.35</td>
<td>1.2</td>
<td>1.3</td>
</tr>
<tr>
<td>1</td>
<td>0.7</td>
<td>1.6</td>
</tr>
<tr>
<td>1.4</td>
<td>2.45</td>
<td>1.8</td>
</tr>
<tr>
<td>1.45</td>
<td>1.4</td>
<td>0.8</td>
</tr>
<tr>
<td>1.6</td>
<td>1.2</td>
<td>0.8</td>
</tr>
<tr>
<td>0.7</td>
<td>1.25</td>
<td>1.05</td>
</tr>
<tr>
<td>1.9</td>
<td>1.3</td>
<td>1.3</td>
</tr>
<tr>
<td>1.6</td>
<td>1.7</td>
<td>1.3</td>
</tr>
<tr>
<td>1.6</td>
<td>2.4</td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>1.6</td>
<td></td>
</tr>
<tr>
<td>0.9</td>
<td>1.9</td>
<td></td>
</tr>
<tr>
<td>2.15</td>
<td>2.8</td>
<td></td>
</tr>
<tr>
<td>0.8</td>
<td>2.55</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.2</td>
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<td></td>
<td>1.5</td>
<td></td>
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<td></td>
<td>1.5</td>
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<td></td>
<td>1.5</td>
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<tr>
<td></td>
<td>1.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>29</td>
<td>10</td>
</tr>
</tbody>
</table>

2/15 = 13% failures 5/29 = 17% failures 2/10 = 20% failures

87% accuracy 83% accuracy 80% accuracy

The stoping width was calculated as the sum of the hangingwall overbreak from the vein, the footwall overbreak from the vein and an assumed 0.3 metre average vein width. The number of case studies falling inside the benchmark stability stoping widths for in-line, staggered and dice patterns were, 87 percent, 83 percent and 80 percent, respectively. Given the limited number of case studies in each group these values agree reasonably well with the 80 percent
probability assigned to the benchmark stability stoping width. These results indicate that the
distribution fitting and probabilistic modelling methods used are reasonable.

10.10 CONCLUSIONS

Barkers mine is a typical mechanised narrow-vein mine employing typical blasting practices. According the stability graph approach all bar one of the 115 Barkers 1 case studies should plot in the stable zone. By assuming the 80 percent accuracy of the stability graph approach is applicable to the Barkers 1 case studies, it is possible to estimate benchmark stoping widths for each of the three blast patterns used at Barkers. These benchmark widths could be applied at narrow-vein mines with similar operating conditions and drill and blast practices. The benchmark stability stoping width will facilitate improved empirical analysis of narrow-vein stope case studies. Unfortunately, despite concerted efforts over a number of years to source data from additional narrow-vein mines it has not been possible to validate these findings at another mine.

The in-line blast pattern benchmark stability stoping width was 1.6 metres. In the case of the staggered blast pattern the overbreak distribution analysis indicated a benchmark stability stoping width of 2.0 metres. The dice-five pattern benchmark stability stoping width was 2.1 metres. These benchmark stability stoping widths are used as a basis for the narrow-vein design method proposed in the following chapter.

Narrow-vein dilution for stopes plotting in the unstable zone can be assessed using the benchmark average stoping widths estimated for each blast pattern. Benchmark average stoping width is effectively an average stoping width from which expected average dilution can be estimated. The expected stoping width estimated from probabilistic modelling of overbreak for in-line, staggered and dice-5 patterns are 1.3 metres, 1.5 metres and 1.7 metres, respectively. The expected or average stoping widths are used in the following chapter to predict dilution in narrow-vein longhole stopes.

Application of the benchmark stoping widths to the Barkers 1 case studies suggests the modelling methodology is valid. However, the validity of the benchmark stoping widths at mines with similar operating conditions to Barkers remains to be validated. That being said, given that benchmark stoping widths are largely a function of operating conditions such as equipment and blast pattern, it has been proposed that benchmark stoping widths can be considered operating condition specific and not site-specific. On this basis it is reasonable to
apply both benchmark stability stoping width and benchmark average stoping widths to similar narrow-vein operations where stopes plot on or above the stable-failure boundary of a stability graph. Irregular geology will impact on expected dilution and adjustments to expected dilution should be made accordingly.
NARROW-VEIN DILUTION METHOD

This chapter applies the research findings presented in this thesis. A new narrow-vein dilution minimisation methodology for both greenfield sites at the feasibility stage and brownfield operating mines has been proposed. The narrow-vein dilution methodology (NVD method) is a tool for predicting narrow-vein dilution based on benchmark average stoping width. In addition to dilution prediction, the NVD method also includes recommendation and strategies for narrow-vein dilution minimisation generally, including; filling, cablebolting, stress relaxation, stress damage and blast overbreak. The premise for the dilution minimisation strategies is that narrow-vein mines record stope dilution and use the stoping history to analyse the underlying causes of dilution at a particular mine. However, in the absence of sufficient stoping history it is still possible to assess the likelihood of some causes of dilution. Once the underlying causes of dilution at a particular mine have been identified, it is then possible to focus dilution management strategies accordingly.

11.1 INTRODUCTION

This thesis has demonstrated the limitations of applying existing stability chart methods to narrow-vein stope design and dilution prediction. In particular, evidence and analysis has been undertaken which demonstrates that the following limitations and conditions should be considered in narrow-vein stope design:

1. Development of a site-specific stability charts to capture narrow-vein operating conditions require at least 150 case studies, of which at least 10 to 15 percent must be unstable case studies.
2. Full and tangential stress relaxation affects stope stability. Due to their tabular geometry narrow-vein stopes have a propensity to stress relaxation. In cases of full and tangential stress relaxation stress factor A should be set to 0.7.
3. Blast pattern and associated benchmark average stoping width are essential elements in predicting planned dilution. The benchmark stability stoping width provides a practical
stope width (design outline) from which unplanned dilution due to geotechnical or overbreak can be assessed. This is a fundamental component of narrow-vein dilution assessment.

4. Stress damage and leaving pillars (re-slotting) were shown to significantly affect narrow-vein dilution. Therefore, narrow-vein dilution assessment should consider the potential for these factors to contribute to unplanned dilution.

5. 261 narrow-vein case studies showed no evidence that dilution is continuously related to N and HR. This is an important result because it means that for stopes plotting well inside a stability chart stable zone, reducing the stope size will not decrease dilution.

6. Dilution for case studies plotting well above the stable-failure boundary zone could largely be attributed to blast pattern. In fact, after adjusting for blast pattern related dilution, the narrow-vein case studies plotted with almost the same level of accuracy as the generic Extended Mathews case studies.

7. Blast overbreak related dilution was unrelated to N or HR. Therefore, the causes of narrow-vein dilution can be separated into two groups: Geotechnical stability (stope size dependent) and blasting overbreak (not stope size dependent).

The narrow-vein dilution method (NVD method) proposed in this chapter draws upon these finding to develop a tool which can be used to assess both assess dilution potential and reduce dilution at an operating mine. The NVD method also incorporates improvements to the stability graph method reviewed in Chapter 3. In particular, radius factor for complex geometry (Milne, 1996a) and updated cablebolt recommendations (Diederichs et al. 1999). These improvements are particularly relevant to narrow-vein stoping. Multiple lift retreat stoping and the leaving of rib pillars often results in complex geometry. The cablebolting recommendations of Diederichs et al., (1999) are particularly relevant to narrow-vein stoping because they include separate recommendations for cases of stress relaxation.

11.2 NARROW-VEIN DILUTION METHOD (NVD METHOD)

The NVD method addresses geotechnical and blasting overbreak related stability separately. In addition, the method attempts to address issues associated with different types and levels of data available at different stages in a mine life. This has been achieved by defining two processes for stope design:

1. Prefeasibility/Feasibility stage
2. Operating mine
Chapter 11 – Narrow-Vein Dilution Method

The proposed NVD method is comprised of three tables. Table A contains the assumptions and limitation of the NVD method. Table B contains the prefeasibility/feasibility NVD method. Table C contains the operating mine dilution minimisation method.

While many of the recommendations and strategies relate directly to findings of research presented in this thesis, there are a significant number of recommendations and strategies listed that are based on findings from the review of literature relating to narrow-vein dilution presented in Chapters 1 through to 4.

11.3 CONCLUSION

A new method has been proposed to improve narrow-vein stope design and dilution prediction. This method draws upon the findings of this thesis in addition to the results of literature pertaining to narrow-vein dilution reviewed in Chapter 1 through to 4 of this thesis.

The dilution prediction method for mines at the feasibility stage can be used to assess the appropriateness of the mining method and determine the likely filling and/or cablebolting requirements if narrow-vein longhole stoping is the preferred mining method. The dilution prediction tools proposed in this chapter have been developed primarily for mechanised narrow-vein longhole stoping. However, elements of the NVD method will be applicable to narrow-vein mining not using this method. The NVD method is based upon a list of assumptions and limitations. It is important that the recommendations and strategies proposed in this method are at all times quoted in context of these assumption and limitations.

In the case of an operating mine a method of back analysis of existing stope data has been proposed that enables a mine to first identify whether dilution is most likely caused by geotechnical or blast overbreak related. Once it can be established whether the primary cause of dilution is geotechnical or blasting overbreak related a number of strategies have been suggested to minimise dilution at a narrow-vein mine. These recommendations draw upon the research findings presented in Chapters 5 through to 10 of this thesis, as well as literature reviewed in Chapters 2 through to 4 of this thesis.
**Table A – NVD method: pre-feasibility/feasibility stage**

<table>
<thead>
<tr>
<th><strong>A1. Geotechnical Stability</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>A1.1 Stability graph assessment¹.</td>
</tr>
</tbody>
</table>

A1.2 Stress relaxation potential assessed for tabular stope geometries; especially if principal stress is perpendicular to strike. In cases of full and tangential stress relaxation set stress factor $A = 0.7$.

A1.3 If geometry is irregular or complex (e.g. pillars or multiple lift retreat) then hydraulic radius can be estimated from radius factor².

<table>
<thead>
<tr>
<th><strong>A2. Fill Requirements</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>A2.1 Tight filling³. Treat fill as solid rock</td>
</tr>
</tbody>
</table>

A2.2 Continuous filling with lag between stoping and fill⁴.

$$HR = HR_{after\ blasting} + HR_{after\ backfilling}$$

<table>
<thead>
<tr>
<th><strong>A3. Cablebolting</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>A3.1 Assess if stope is supportable using Modified Stability chart.¹</td>
</tr>
</tbody>
</table>

A3.2 Stable HR can be increased by pattern cablebolting. Modified stability chart can be used to assess possible increase HR achievable with pattern cablebolting⁵.

A3.3 Consider effect of backfilling on production cycle. Cablebolts have potential to improve productivity by decreasing delays associated with regular filling requirements⁶.

A3.4 Cablebolts can be used for areas with localised instability potential.
### Chapter 11 – Narrow-Vein Dilution Method

#### Table A (cont.) – NVD method: pre-feasibility/feasibility stage

<table>
<thead>
<tr>
<th>A4. Dilution Prediction</th>
</tr>
</thead>
<tbody>
<tr>
<td>A4.1 Total dilution for stopes plotting in the stable zone</td>
</tr>
<tr>
<td>Benchmark total dilution for stable stopes can be estimated as follows.</td>
</tr>
<tr>
<td>Total dilution = (Benchmark average stoping width(^{10}) – Vein width)/Vein width</td>
</tr>
<tr>
<td>The benchmark average stoping widths for common narrow-vein blast patterns(^{10}):</td>
</tr>
<tr>
<td>In-line = 1.3 m</td>
</tr>
<tr>
<td>Staggered = 1.5 m</td>
</tr>
<tr>
<td>Dice-5 = 1.7 m</td>
</tr>
<tr>
<td>For vein widths &gt;1.2m, an allowance of approximately 0.6m of dilution is required to ensure the probability of ore loss is less than 5% (from cumulative overbreak probability distributions). This estimate is based on the average dilution allowance required for a 0.7m vein using an in-line pattern i.e. 1.3m-0.7m requires a 0.6m dilution allowance to ensure ore loss probability is less than 5%. It is reasonable to expect that a similar allowance would be required for wider veins.</td>
</tr>
</tbody>
</table>

| A4.2 Dilution assessment for stopes plotting in the unstable or failure zone of a stability chart. |
| Isoprobability contours can be used to estimate the percentage of stopes expected to exceed the benchmark stability stoping width. \(^{7,8}\) |
| e.g. The probability of unplanned dilution for a stope plotting on the stable-failure boundary is approximately, 20%. \(^{12}\) |
| Benchmark stability stoping widths\(^{11}\) |
| Vein width < 0.7m\(^{12}\) using an in-line pattern |
| Vein width > 0.7 m\(^{12}\) using either staggered or dice-5 pattern |
| 1.6 m |
| 2.1 m |

| A4.3 Undercutting of stope walls |
| For stope walls with N’ or N < 5 empirical evidence suggests that undercutting will lead to stope wall failure (Potvin, 1988). |
| For stope walls with N’ or N > 5 the HR of undercut stope walls can be estimated by assuming that the stope height is infinite. \(^{13}\) |

| A4.4 Extraction sequence |
| Evaluate potential for pre-stopping stress history to cause dilution. Possible methods include: pillar stability chart\(^7\), \(\sigma_n > 0.8 \times \text{UCS}\)\(^8\) and \(\sigma_1-\sigma_3 > 0.3\)\(^9\) |
| If stress damage potential is high then increase dilution by between 0.1 to 0.3 m. \(^{14}\) |
| Layered orebodies should be assessed for stress relaxation potential. |

**Notes:** Table C – Assumptions and limitations contains notes as per superscript numbers.
Table B – NVD Method: minimising dilution at an operating mine

B1 Geotechnical Stability

B1.1 Determine stable HR for geotechnical domains as per A.1 to A.3.
Stable HR predictions should be based on revaluation of Q’ based on underground mapping and observations of rock mass behaviour. Observe any time dependent rockmass properties.

B1.2 Site-specific chart (if necessary)
If necessary, develop site-specific stability chart using at least 150 case studies, of which at least 10% are unstable. A site-specific chart would be considered necessary if the geotechnical stability is consistently poorly predicted by stability graphs. For example the Mt Isa bench stability chart takes into account bedding spacing and blasting parameters (Villaescusa, 1996).

B1.3 Kinematic analysis
If wedge and block failure appears to be dominating stability, a more detailed kinematic stability assessment is warranted. Kinematic stability can be assessed using scan-line mapping results in conjunction with some type of 3D joint network model e.g. Stereoblock (Grenon and Hadjigeorgiou, 2003).

B2 Minimising Dilution

There is significant evidence that dilution can be minimised by ensuring that all personnel involved in stoping and ore development are aware of its importance to a mines economic performance. For example, mine productivity can be measured in terms of weight of metal mined.

B2.1 Stope database
Maintain stope dilution database and plot dilution versus stope span. This will provide a good indication of whether dilution is caused by geotechnical causes or drill and blast causes. Empirical evidence suggests that reducing stope span will only reduce dilution if the causes of dilution are geotechnical.
If dilution is independent of stope span, then dilution is unlikely to be caused by geotechnical factors. Stress damage is the only geotechnical cause of dilution that would not be span dependent.
If dilution is span dependent, then dilution is likely to be related to geotechnical factors.
### Table B (cont.) – NVD Method: minimising dilution at an operating mine

<table>
<thead>
<tr>
<th>B2.3 Minimising geotechnical causes of dilution</th>
</tr>
</thead>
<tbody>
<tr>
<td>More detailed structural analysis using scanline mapping and stereonets.</td>
</tr>
<tr>
<td>Assess relaxation potential.</td>
</tr>
<tr>
<td>Cost benefit analysis for cablebolting and/or stope span reduction including fill cycle times.</td>
</tr>
<tr>
<td>Ore drive development under good geological control including appropriate drive profile.</td>
</tr>
<tr>
<td>Stress damage related dilution can be minimised by evaluation of extraction sequence against damage criterion, increasing number of rings fired per blast and where practical avoiding shrinking central pillar extraction sequence.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>B2.4 Minimising drill and blasting related dilution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Select appropriate blast pattern. An in-line pattern for veins &lt; 0.7 m. For vein widths greater than 0.7 m there is a 5 % risk of oreloss so in this case a staggered or dice 5 pattern should be selected. However, in some ground conditions an in-line pattern may encounter issues associated with bridging.</td>
</tr>
<tr>
<td>Survey drillholes and analyse results in a systematic manner.</td>
</tr>
<tr>
<td>Drill and blast trials should be randomised trials.</td>
</tr>
<tr>
<td>Blast damage minimisation.</td>
</tr>
<tr>
<td>Smoothwall blasting.</td>
</tr>
</tbody>
</table>

**Notes:** Table C – Assumptions and limitations contains notes as per superscript numbers.
### Table C – NVD limitations and assumptions.

1. Use either the Extended Mathews Stability graph (Mawdesley et al., 2000) approach or the Modified Stability chart (Potvin, 1988). The Extended Mathews has most data coverage and N is simpler to evaluate. The Modified Stability chart is in widest use and has a supportable zone delineated.

2. Radius factor provides a more accurate assessment of the distance to abutments and is therefore better suited to irregular shape geometry (Milne et al., 1996a). Radius factor \( RF \) is half the harmonic radius \( Rh \) that is calculated using equation 3.11, where \( r_0 \) is the distance to the abutments measured from the surface centre.

\[
RF = \frac{Rh}{2} = \frac{0.5}{n} \sum_{\theta=1}^{n} \frac{1}{r_\theta}
\]

3. Tight filling means that there is no volume between the fill and the next rings/slot to be fired.

4. Continuous filling (as opposed to tight fill) means there is a gap between the fill and the next rings to be fired. Milne (1996) proposes that the effective final span converges to the sum of the opening after blasting plus the opening span after backfilling. However, due to insufficient data, Milne (1996) was unable to confirm this hypothesis. However, the results of extensive hangingswall monitoring (extensometers) at the Winston Lake mine did suggest that the common practice of treating a moving backfill abutment like a rock abutment is overly optimistic (Milne 1996).


6. However, cablebolt effectiveness can be reduced by low rock mass stiffness and stress relaxation (Diederichs et al. 1999).

7. Brow stress damage would correspond to category three for pillar yield (fracturing in pillar walls) and the unstable region of the Confinement Formula Stability Graph (Lunde, 1994). The main advantage of this approach over the deviatoric stress approach is the very large database of rockmass conditions incorporated into the database means better prediction of stress damage when there is no opportunity to calibrate to underground observations.

8. Further validation required.

9. May have effect on dilution below this criterion damage limit e.g. Barkers case studies.
Table C (cont.) – NVD limitations, assumptions and notes

10. It has been assumed that benchmark average stoping width is not site-specific, and can be considered operating condition specific. In this case operating condition specific refers to standard narrow-vein longhole mining methods and equipment. In other words, benchmark stability stoping width is primarily a function of mining method and equipment, and not the geotechnical parameters that are responsible for unplanned dilution. Based on this assumption, the benchmark stability stoping widths determined for Barkers mine are applicable to similar narrow-vein longhole stoping mines. It should be noted that rockmass classification based on scanline mapping of sill drives indicates that the rockmass for the Barkers 1 database ranges from fair to good. It is reasonable to expect that mines with poor rockmass conditions would experience higher levels of blast overbreak than that incurred at Barkers.

For stopes plotting on the stable-failure boundary the probability of a stope width exceeding the benchmark stability stoping width is approximately, 20\%^{12}. This estimate assumes parallel holes and standard longhole drill and blast practices.

Standard practice;

i. Low impact explosives used on the hangingwall

ii. Blast pattern suited to the vein width

iii. 51mm to 64mm diameter holes <15m long.

iv. Hangingwall hole stand-off distance appropriate to the rockmass.

v. Some drill-hole surveys.

vi. Irregular reporting of dilution and or stope reconciliation.

vii. Some blast trials

viii. Ore drive development under geological control

Implementation of best practices and improved quality control has the potential to achieve narrower stoping widths. In these cases the probability of a stope width exceeding the stable stoping width would be significantly less than 20 percent.
Table C (cont.) – NVD limitations, assumptions and notes

Each of the practices listed has been proven to be effective in dilution minimisation at least one mine:

i. Regular and systematic drill-hole survey and analysis.

ii. Stope survey and stope reconciliation.

iii. Reporting dilution as a key performance indicator (KPI)

iv. Drill and blast continuous improvement projects based on properly design randomised drill and blast trials. Design modifications based on statistically significant results obtained from stope reconciliations.

v. Blast damage modelling based on analysis of PPV

vi. Tight geological and mining control on ore-drive development to minimise undercutting.

vii. Cablebolting of localised instability

viii. Smooth wall blasting e.g. pre-split

ix. Revaluation/calibration of feasibility study stable stope dimensions

Sub-standard practice;

i. No drill-hole surveys

ii. In appropriate blast pattern selected for vein width

iii. Only reporting tonnes as KPI

iv. No evaluation or reporting of dilution

v. No geotechnical mapping to reassess feasibility study design

11. Like benchmark average stoping width, it has been assumed that benchmark stability stoping width is not site-specific and is a function of mining method and equipment. An in-line pattern can be used for vein widths less than 0.7 m. If an in-line pattern is used for a vein width of 0.7 m then the probability of ore loss is approximately 5%. However, depending on geological and rockmass conditions an in-line pattern can result in bridging.
12. Inside the stable zone the probability of unplanned dilution decreases from 20% the further a stope plots inside the stable zone and conversely, inside the failure zone the probability of unplanned dilution increase from 20% the further below the stable-failure boundary. The Extended Mathews Isoprobability contours (Mawdesley et al., 2000) can be used to estimate the probability of unplanned dilution at a particular position on the Extended Mathews Stability graph. While the probability of failure is affected by distance from the stability graph stable-failure boundary, the amount of unplanned dilution in narrow-vein stopes does not appear to be related to position on the stability graph.

13. This recommendation is based on the assumption that an undercut stope wall cannot obtain support from arching to the abutments and therefore, assuming the stope height is infinite is a good approximation of the effect of removing lower abutment support. This recommendation is based on engineering assessment of the destabilising effect of undercutting and has not been validated with case studies. However, in the absence of alternatives this approach seems to be reasonable, and is likely to be more realistic than assuming that undercutting has no effect at all. Comment: in one of your chapters undercut footwalls were related to non undercut hangingwalls

14. Stress damage related unplanned dilution ranged from 0.1 m to 0.3 m and is based on data from one mine (Barkers mine) and therefore requires further validation. However, the upper limit of 0.3 m could be a function of mineability of ore drives. Barkers mine ore drives had high levels of support including shotcrete and cablebolting. Based on these levels of support it is reasonable to infer that mineability was moderately difficult. If stress related conditions in the ore drives were more difficult than Barkers, then stress damage related dilution could exceed 0.3m. However, more case studies are required from other mines to validate this proposition.

15. Ideally, a narrow-vein stope dilution database should record the following information;
   a. Stope dimensions
   b. Undercutting
   c. Failure mechanism
   d. Drill and blasting patterns including explosive types and initiation sequence.
   e. Support of stope walls

16. Drillhole inaccuracy is a significant cause of dilution (Aplin, 1997; Revey, 1998).
Table C (cont.) – NVD limitations, assumptions and notes

17. Randomised trials mean that changes to drill and blast parameters are undertaken randomly without consideration for any other parameters. The data collated from randomised drill and blast trials is less likely to be affected by systematic bias. This is the basis of experimental design.

18. It seems that because the relationship between blast damage and overbreak is rockmass dependent there is no generic model for predicting overbreak using blast damage modelling. However, individual mines could compare overbreak stability to damage modelling and use this information in blast design. The value of blast damage modelling is in the ability to compare the blast damage potential of alternative blast designs. Peak particle velocity (PPV) is a good predictor of blast damage (Singh, 1998; Villaescusa et al., 2003). Blast damage contributes to dilution (Henning et al., 1997). Blast damage modelling involves monitoring blast vibration to determine site-specific constants $k$ and $\alpha$. Once $k$ and $\alpha$ have been determined PPV can be estimated as follows:

$$PPV = k \alpha$$

Once the PPV limit for blast damage has been determined it is possible to design blast that do not exceed the PPV limit. However, scatter in delay timings and inconsistencies in rockmass attenuation properties can significantly affect the reliability of actual PPV versus design PPV.

19. Pageau et al. (1992) found that by implementing smooth wall blasting, including a pre-split, they were able to reduce dilution at the Richmont mines, Francouer mine Quebec from 10-15 percent to 5 percent.
CHAPTER 12

CONCLUSIONS

Dilution has an adverse economic impact upon mining operations. Both unplanned dilution and planned dilution have been studied in this thesis. Other sources of dilution such as the geological form of the deposit and backfill contamination were largely beyond the scope of this thesis. Mining method selection involves a trade off between the unit cost and production rate benefits of mechanised mining against the costs of dilution associated with mechanised mining. Longhole stoping has emerged as the dominant narrow-vein mining method over the past 20 years. Longhole stoping mines use some combination of backfill, cablebolts and or pillars to control effective span in attempt to minimise dilution. Stope designs should be evaluated in terms of dilution potential, as well as, production rate and mining costs. This requires adequate methods for dilution prediction. This thesis has developed improved methodologies for narrow-vein dilution prediction.

Quality control and process improvement are dilution control measures that can be implemented as part of a feedback loop between production and engineering personnel. Drillhole survey is an example of quality control, while explosive and drill pattern trials would be considered process improvement methods of dilution control.

Broadly speaking, there are four main approaches to the design of stable stopes; mining methods selection, empirical stope design, analytical methods and numerical modelling. Mining method selection has been discussed in Chapter 2. Acknowledging that for various reasons conventional narrow-vein mining methods are generally on the demise, a review was carried out to evaluate the applicability of empirical, analytical and numerical stope design methodologies with respect to narrow-vein longhole stoping. The ability of the empirical stability graph approach to capture both stress and kinematic stability mechanisms was concluded to be a significant advantage of this approach. The stability graph approach has demonstrated general applicability to large-open stoping and can be used at the feasibility stage to predict stable stope dimensions. Analytical (kinematic analysis) and numerical methods have the potential to compliment the stability graph approach in cases where
mechanisms causing stope instability have been clearly identified and the opportunity to calibrate input parameters exists. This is particularly true in the case of sites where scanline mapping has been undertaken and it is possible to evaluate stability using a three-dimensional joint network and limit equilibrium analysis. Numerical modelling and analytical methods general applicability is limited because both approaches only account for one failure mechanism. However, when employed in combination with empirical approaches that capture a broader range of stability parameters, both numerical and kinematic analyses have the potential to improve stability prediction.

Existing stability charts do not take into account a number of the parameters believed to affect stope stability. Despite this, stability charts are approximately 80 percent accurate in terms of predicting open stope stability generally. This implies that those factors taken into account are able to predict instability in 80 percent of cases. It was hypothesised that some parameters excluded from the formulation of existing stability charts may have a greater influence on narrow-vein stability than on large open stope stability. Factors and methods to take into account moving backfill abutments, drill and blast parameters, stress relaxation, stress damage, complex geometry and undercutting were qualitatively reviewed. It was concluded from this review that there was considerable uncertainty associated with the accuracy of existing stability charts when applied to narrow-vein stability prediction.

A review of the applicability and usefulness of site-specific or site-calibrated stability charts indicated that while some authors advocate site calibration of the stability graph method, two of the primary authors and developers of the stability graph approach see the value of site-specific stability charts as limited. Narrow-vein stability parameters including blast damage, stress relaxation, undercutting and stress damage are rock mass dependent and therefore, to some extent are taken into account by Q’. For this reason it was hypothesised that narrow-vein mines may be more sensitive to Q’ than large open stopes. A new technique that analyses trends in the variances of three logit model parameters (Extended Mathews Logistical model) has enabled the author to define database requirements. The analysis indicates that a reliable stable-failure boundary requires at least 150 case histories, of which a minimum of 10 percent should be unstable stope surfaces.

Marginal site-specific effects were observed. It was concluded that the apparent site-specific effects contained in previous literature are attributable to operating conditions inadequately represented in the database. Such operating conditions could induce erroneous stability
predictions at any site, and therefore, are not truly site-specific. It was concluded that by developing methods to take into account narrow-vein operating conditions it would be possible to improve the predictive ability of stability charts for narrow-vein operating conditions.

Stress relaxation was one of the narrow-vein operating conditions hypothesised to adversely effect narrow-vein stope stability prediction using existing stability chart methods. Establishing clear definitions of stress relaxation was critical to understanding and quantifying stress relaxation effects. Once clear definitions were determined, it was concluded that the theoretical arguments and empirical evidence presented by various authors to support their respective cases are not contradictory; rather the different conclusions can be attributed to different types of stress relaxation. Three types of stress relaxation have been defined; partial relaxation, full relaxation and tangential relaxation. A new set of guidelines to account for the effect of stress relaxation within the stability chart approach has been proposed based on back-analysis of 55 case histories of stress relaxation. It was also concluded that two-dimensional stress analyses are insufficient to determine if an excavation surface is relaxed unless the aspect ratio of the excavation surface is 5 or greater. If the aspect ratio is less than 5, a three-dimensional stress analysis is required to confirm that the excavation surface is in actuality relaxed. Because narrow-vein stope lengths to width ratios are almost always greater than five, two-dimensional plain strain modelling can usually be used to confirm stress relaxation in the case of narrow-vein mining.

Analysis of misclassification statistics using the Extended Mathews stability graph indicates that partial stress relaxation is a poor predictor of stability. However, tangential relaxation and full relaxation were found to have an adverse effect on excavation stability. In the case of full and tangential stress relaxation there was little difference between the two methods.

However, it can be inferred from the back-analyses that when the minor principal stress is tensile, the intermediate principal stress has an impact on rockmass behaviour in jointed rock masses. Further work is required to quantify the impact of the intermediate principal stress on the behaviour of jointed rock masses.

Setting Stress Factor A equal to 0.7 significantly improves the accuracy of stability prediction for cases of full and tangential stress relaxation. This adjustment is an explicit method for taking into account the destabilising effect of full and tangential stress relaxation. For cases of partial stress relaxation, a stress adjustment factor A equal to one, as would be the case in the
Chapter 12 – Conclusions

The original Mathews method gives the best predictive capability to the model. Because narrow-vein geometries are prone to stress relaxation effects, the adjustment for stress relaxation presented is an important contribution towards improved narrow-vein dilution prediction and forms part of the methodology for improved narrow-vein dilution prediction (NVD method).

Back analysis of 115 narrow-vein case studies from the Barkers mine in Western Australia indicated a poor correlation between stope stability and both the Mathews stability number, N, and hydraulic radius, HR. Given that both N and HR correlate well with stability in the vast majority of stability chart case studies, this suggests there is an overriding influence on stability at Barkers not accounted for in the Mathews method. Drill and blast issues were isolated as the most likely cause of this poor correlation. Blast pattern was found to have a statistically significant affect on overbreak. In terms of the drill and blast patterns used at the mine, the in-line 3 pattern performed significantly better than both the staggered and dice 5 patterns for the vein geometries at the time. There was no evidence that tight backfill abutments (not continuous moving) behave differently from solid rock abutments in terms of determination of stable stope dimensions. Therefore, it was concluded that for tight backfill abutments the fill can be treated as solid rock.

The stability of 146 narrow-vein case studies from the Callinan and Trout Lake mine were also poorly predicted by existing stability charts. It was concluded from the Barkers, Callinan and Trout Lake narrow-vein case studies that existing stability charts provide poor predictive ability in the case of narrow-vein stoping.

For the vein widths captured in the Barkers stability database, the in-line 3 blast pattern performed significantly better than both the staggered and dice 5 in terms of overbreak. The Barkers stope stability back-analysis provided validation of the need to address narrow-vein operating conditions such as drill and blast design explicitly. While adopting an in-line drill and blast pattern instead of a dice five pattern has the potential to reduce dilution by an average of 0.19 metres, the large number of narrow-vein case studies with dilution in excess of stability chart predictions suggests that additional causes of dilution are probable.

In the case of narrow-vein mining, the relatively small incremental extraction of long-hole rings has the potential to result in a moving high stressed zone at the stope brow. This results in the hangingwall and footwall experiencing a spike in the stress to strength ratios as the brow passes. In some cases, the stress to strength ratio may be high enough to result in fracturing or
damage to the rock mass prior to excavation. It was concluded from a study involving analysis of overbreak from 412 case studies from the Barkers mine that stress damage contributes to dilution in narrow-vein mines. Modelling of a 32 month extraction sequence demonstrated, with 94 percent confidence, that stress damaged stope walls at this mine had an average 0.27 metres more overbreak than stope walls where stresses had not exceeded the damage criterion. Assuming a designed mining width of approximately 1.5 metres, and both hangingwall and footwall affected by stress damage, this represents 36 percent dilution. After adjustment for possible sources of bias the difference reduced to an average 0.10 metres per stope wall, representing 13 percent dilution for the mining width under consideration. Therefore, it has been concluded that the potential for stress damage related overbreak should be considered as part of any assessment of narrow-vein dilution. Furthermore, extraction sequences can be modelled as part of the mine planning process to ensure that the risk of stress damage related dilution has been quantified and alternative strategies considered in view of balancing costs, production requirements and dilution minimisation. By increasing the length of strike extracted in each production blast the amount of hangingwall and footwall exposed to high stresses would decrease, thereby decreasing stress damage potential. The extent to which production blasts can be lengthened along strike will depend on vein geometry, rockmass conditions, seismicity considerations and drill and blast parameters. At Barkers, peak stress normal to brow greater than 100 MPa was associated with overbreak and corresponds to a magnitude equal to 0.7 times the UCS. Further validation work is required to determine whether a criterion of peak normal stress greater than 0.7 times the UCS for stress damage related overbreak would be applicable at other narrow-vein sites.

A number of adjustments have been proposed to explicitly take into account narrow-vein operating conditions. An adjustment for full and tangential stress relaxation has been proposed. In addition blast pattern, stress damage and leaving pillars were all found to have a significant effect on overbreak. The difference in overbreak was used as an adjustment to assess whether stress damage, blast pattern and leaving pillars on the sensitivity and specificity of the stability predictions of these case studies. Adjusting for blast pattern improved stability graph accuracy from 64.0 percent to 78.9 percent, while adjusting for stress damage and leaving pillars improved accuracy from 64.9 percent to 67.3 percent. Theoretically, the combined effect of these adjustments could improve stability graph accuracy from 65 percent to 82 percent.

The high levels of accuracy achievable after making adjustments for stress relaxation, blast pattern and stress damage suggests that the stability graph approach can be used to predict the
geotechnical causes of narrow-vein dilution provided consideration of the aforementioned effects is made.

Using a stress factor equal to 0.7 did not improve the accuracy of stability graph predictions for the Barkers case studies. Further analysis indicated that this result could be explained by a lack of continuity between stability and N and HR inside the stable zone. Barkers, Callinan and Trout Lake mine case studies suggest that overbreak is not continuously related to N and HR, and that the relationship between HR and overbreak and N and overbreak is best described by a logistical relationship. Based on this result it was concluded that the cause of dilution is unlikely to be related to geotechnical parameters if a stope plots well inside the stable zone. Therefore, in this scenario, reducing stope size would not reduce dilution. Based on this interpretation it can be concluded that the causes of narrow-vein dilution can be separated into two unrelated causes:

1. Geotechnical instability (stope size dependent).
2. Blast overbreak (unrelated to stope size).

In practice, blast overbreak can be distinguished from geotechnical causes of dilution by analysing whether dilution occurs independently of stope size. For example, for the range of hydraulic radii captured in the two Barkers databases dilution was generally (apart from the Barkers 1 case studies with hydraulic radii less than three) unrelated to stope size. In practical terms this means that generally speaking for hydraulic radii plotting well inside the stable zone, dilution occurred independently of the distance between backfilling cycles.

A probabilistic benchmarking method has been used to estimate a benchmark stoping width for three commonly used narrow-vein longhole blast patterns. The benchmark stoping width for each pattern define realistic planned dilution limits. These limits provide the basis from which true unplanned dilution can be assessed. In addition, the probabilistic overbreak model has been used to predict expected stope widths for each of the patterns. Expected stoping width, in conjunction with vein or ore width, can be used to estimate total dilution (planned and unplanned).

It has been assumed that benchmark stoping width and expected stoping width are not site-specific and can be considered operating condition specific. In this case, operating condition specific refers to standard narrow-vein longhole mining methods and equipment such as those
used at Barkers mine. In other words, benchmark stoping width and expected stoping width are primarily a function of the longhole stoping method, and not the geotechnical parameters that are responsible for unplanned dilution. Based on this assumption, the benchmark stoping widths determined for Barkers mine are applicable to similar narrow-vein longhole stoping mines. The benchmark stoping width will facilitate improved empirical analysis of narrow-vein stope case studies as it provides a benchmark from which unplanned dilution can be assessed. Irregular geology will require adjustment to both benchmark stoping width and expected stoping width.

Application of the benchmark stoping widths to the Barkers 1 case studies suggests the modelling methodology is valid. However, the validity of the benchmark stoping widths at mines with similar operating conditions to Barkers remains to be validated. That being said, given that benchmark stoping width and expected stoping width are largely a function of operating conditions such as equipment and blast pattern, it has been concluded that benchmark stoping widths can be considered operating condition specific and not site-specific. On this basis it is reasonable to apply both benchmark stoping width and expected stoping widths to similar narrow-vein operations where the stopes plot on or above the stable failure boundary. The benchmark stoping widths and expected stoping widths are the basis of the narrow-vein design method proposed in this thesis.

A benchmark stoping width of 1.6 metres was determined for the in-line blasting pattern and 2.1 metres for staggered and dice 5 patterns. The in-line blasting pattern was found to lead to a 5 percent probability of ore loss for vein width of greater than 0.7 metres. Expected stoping widths can be used to estimate narrow-vein dilution for stopes plotting in the stable zone of a stability chart. The expected stoping widths for in-line, staggered and dice 5 blast patterns are 1.2 metres, 1.5 metres and 1.7 metres, respectively. Improved dilution prediction will facilitate a more realistic comparison of the total costs per tonne for longhole versus conventional mining methods.

The research findings presented in this thesis in conjunction with literature reviewed throughout the course of this research project have been applied to develop a new narrow-vein dilution minimisation methodology for both greenfield sites at the feasibility stage and operating mines. The narrow-vein dilution methodology (NVD method) is a tool for predicting narrow-vein dilution based on the benchmark stoping width and expected stoping width determined for mechanised narrow-vein stoping generally. In addition to dilution prediction,
the NVD method also includes recommendation and strategies for narrow-vein dilution minimisation generally, including; filling, cablebolting, stress relaxation, stress damage and blast overbreak. The premise for the dilution minimisation strategies is that narrow-vein mines record stope dilution and use the stoping history to analyse the underlying causes of dilution at a particular mine. However, in the absence of sufficient stoping history it is still possible to assess the likelihood of some causes of dilution without reference to stoping history. Once the underlying causes of dilution at a particular mine have been identified, it is then possible to focus dilution management strategies accordingly. The narrow-vein dilution method in conjunction with dilution minimisation strategies presented in this thesis represent a significant improvement in the tools available for narrow-vein dilution minimisation.
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APPENDICES

Appendix A – Extended Mathews Database
Appendix B – Relaxed Database
Appendix C – Barkers 1 Database
Appendix D – Barkers 2 Database
Appendix E – Modelled Stress Histories
Appendix F – Publications

Appendices are contained electronically on the attached CD.