An Integrated Field Mapping-Numerical Modelling Approach to Characterising Discontinuity Persistence and Intact Rock Bridges in Large Open Pit Slopes

by

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B.Sc., Queen’s University, 2008

Thesis Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Science

in the

Department of Earth Sciences
Faculty of Science

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SIMON FRASER UNIVERSITY

Fall 2012

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Abstract

Field investigations were undertaken at three open pit mines and one major natural rock slope. Modified geotechnical field mapping techniques and remote sensing methods including LiDAR and photogrammetry were adapted for each site, in an effort to improve characterisation of discontinuity persistence and intact rock bridges. A fracture network engineering approach is proposed for trace mapping, with intensity factors to describe intact rock bridge trace intensity, \( R_{21} \), and blast-induced fracture intensity, \( B_{21} \).

Results from the field investigations form the basis for a conceptual numerical modelling trial, where finite element, distinct element, and lattice-spring codes are used to investigate the role of persistence and rock bridges in large open pit slopes. A damage intensity approach is introduced to characterise the fracturing induced within a slope, \( D_{21} \). The results are used to make preliminary recommendations for improving field characterization and post-processing methods to assess discontinuity persistence and rock bridges in large open pits.

Keywords: slope stability; discontinuity persistence; intact rock bridges; open pit mining
Dedication

I dedicate this thesis to Sauron the Deceiver, Dark Lord of Mordor, Master of Barad-dûr, the One True Lord of the Rings.
Acknowledgements

I thank my senior supervisor Dr. Doug Stead; this thesis would not have been possible without his funding, academic support and mentorship. I also thank the members of my examining committee for their expert feedback; their help has been instrumental in improving this document: Dr. Erik Eberhardt of UBC Geological Engineering, Dr. Davide Elmo of UBC Department of Mining Engineering, and Dr. Matthieu Sturzenegger of Klohn Crippen Berger. I also thank my external examiner Dr. Wayne Barnett of SRK consulting, whose extensive experience at Jwaneng was of great help in my understanding of geological structure in the Jwaneng East wall.

I thank all past and current members of the Engineering Geology and Resource Geotechnics Research Group at SFU: thank you for helping to foster a productive and sociable research environment. I thank former students Gabe Hensold and Sung Lee, who helped me to acclimatize to graduate studies. I also specifically thank my former office mate Fuqiang Gao, without whose numerical modelling and FISH acumen I would never have been able to extract useful crack monitoring information from my models. I also thank Andrea Wolter, Janisse Vivas, Mohsen Havaej, Pooya Hamdi, Yabing Zhang, Kenneth Lupogo, Ryan Preson, Anne Buckingham and John Mayer. Our numerous discussions into intact rock bridges and discontinuity persistence will probably be burned into my mind forever. Thanks also to the faculty and department staff of the Department of Earth Sciences at SFU. In particular I thank the graduate secretary Glenda Pauls for helping me to navigate university and department procedures, Tarja Vaisanen for repeated assistance with my error-ridden expense reports, and Matt Plotnikoff and Rodney Arnold for assistance with field equipment, department vehicles and computing issues.

The investigation at Jwaneng was made possible with help from Jarek Jakubec and Peter Terbrugge of SRK Consulting; I also thank the geotechnical team at Jwaneng for their hospitality and extensive assistance in coordinating the field investigation, in particular Jacob Balesamang, Mmpoloki Kgotlhane, and Mmoloki Keitumetse.
The investigation at Diavik was made possible with help from Cameron Clayton and Sonia D’Ambra of Golder Associates; I also thank the geotechnical staff at Diavik for accommodation and transportation during the field investigation, in particular Sarah Greer, Richard LeBreton, and Carson Sutton.

The investigation at Highland Valley Copper was facilitated by Piteau Associates with the assistance of Nick Rose; thank you for the useful background information and coordination efforts. I also thank Sebastien Fortin of Teck Resources, who provided useful insights into the structure of the Upper West Wall, and provided useful maps and transportation around the mine during the investigation.

The use of the Slope Model code was made possible by Loren Lorig of Itasca, and the sponsors of the Large Open Pit (LOP) project. The project is funded by a consortium of international mining companies that currently includes Anglo American plc; AngloGold Ashanti; Barrick; BHP Billiton; Compañía Minera Doña Inés de Collahuasi SCM; De Beers; Newcrest; Newmont; Ok Tedi Mining Limited; RioTinto; Teck Resources Ltd; Vale; and Xstrata Copper. I also thank Dr. John Read of CSIRO, Australia for assistance in the provision of the Slope Model code. Funding for the research was provided through a British Columbia NRAS grant, an FRBC endowment and an NSERC Discovery Grant. Finally the I acknowledge support for software training through the Itasca Education Partnership Program.

Finally I thank my parents and my sister for letting me returning to the nest for 2.5 years, and for always tolerating my frequent rants about brittle fracture frustrations.
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1. Introduction and Research Objectives

1.1. Introduction

Despite over four decades of research into rock slope stability and many technological advances in surveying, monitoring and numerical modelling, there is still no standard recommended methodology for characterising discontinuity persistence and intact rock bridges in rock slopes. Economic factors and improvements in mining equipment are leading to the development of larger, deeper open pits, requiring that geotechnical researchers and engineering practitioners understand the role of discontinuity persistence and intact rock bridges in slope stability.

Open pit mine stakeholders now frequently demand rigorous, risk-based assessments of slope designs. For large slopes, designers must consider the potential for stress-induced brittle failure of intact rock bridges. Studies of large naturally occurring rockslides show that large scale instabilities can involve interaction between pre-existing discontinuities ranging from metre-scale joints to kilometre-scale faults, and brittle failure of intact rock bridges. The kinematics of large-scale instabilities may therefore comprise a complex combination of translation, rotation, and internal shearing.

This thesis presents observations from field investigations undertaken at three open pit mines and one natural rock slope. Field methods and post-processing techniques were adapted according to site-specific conditions, with the objective of improving understanding of discontinuity persistence and intact rock bridge content. Results from the field investigations were used to formulate a conceptual numerical modelling study, where three different codes were used to simulate the influence of intact rock bridges in slope failures ranging from the bench-scale to overall slope-scale.

Based on a synthesis of observations from field mapping and numerically modelling, a set of preliminary recommendations are provided, which aim to address common sources of uncertainty and potential means for improving methods to characterize discontinuity persistence and intact rock bridge content in large open pit slopes.
1.2. Research Objectives

The main objectives of the research include:

1. To assess the applicability of terrestrial remote sensing methods for quantifying intact rock bridge content and discontinuity persistence, especially for large-scale features not encompassed by current ISRM (1978) suggested persistence classification.
2. To investigate the feasibility of field estimation of intact rock bridge content and discontinuity persistence, for a range of rock masses with variable geology and blast-induced damage conditions.
3. To investigate finite element and discrete element numerical methods for simulating brittle failure of intact rock bridges or rock mass bridges slopes (Chapter 7).
4. To investigate the influence of persistence and intact rock bridge content on rock mass dilation in large bi-planar slope failures.
5. To create a set of preliminary recommendations for characterizing discontinuity persistence and intact rock bridges in open pit mines, to better capture their influence on rock mass shear strength and deformability.

Secondary objectives of the research include:

- To develop a field-based methodology for trace mapping of persistence, rock bridges, and blast-induced fractures
- To develop methods of characterising step-path geometry in rock slopes based on variation in 3-D step paths in a foliated rock mass
  - To introduce a volumetric rock bridge measure for considering dispersed rock bridges within a volume of rock mass
- To apply remote sensing methods to characterise fractographic features of major discontinuity surfaces
- To apply the principles of discrete fracture network engineering to the field characterization of rock bridges, blast-induced fractures and simulated rock mass damage in numerical models
- To develop the UDEC Voronoi technique for characterizing the influence of rock bridge content on internal slope deformation in large open pit slopes
• To explore the use of the lattice-spring method to model brittle rock mass fracturing in a bi-planar open pit slope failure

1.3. Thesis Structure

This thesis is divided into eight chapters:

• Chapter 1 presents the introduction, purpose, objectives and thesis structure.
• Chapter 2 presents a review of rock mechanics related to rock mass and discontinuity characterisation, and brittle failure of intact rock.
• Chapter 3 introduces the field methodologies and post-processing methods applied to study sites, and presents observations from a pilot investigation at a natural rock slope near Squamish, BC.
• Chapter 4 presents the results of field investigations at Jwaneng diamond mine, Botswana, in a foliated metasedimentary rock mass.
• Chapter 5 presents the results of field investigations at Diavik diamond mine, NWT Canada, in a massive granite rock mass.
• Chapter 6 presents the results of field investigations at Highland Valley Copper mine, BC Canada, in a highly fractured granodiorite rock mass.
• Chapter 7 presents results from a conceptual numerical modelling investigation using finite element, distinct element, and lattice-spring codes.
• Chapter 8 presents a synthesis of observations from the field investigations and numerical modelling, and provides concluding remarks and recommendations for future work.
2. Literature Review

2.1. Introduction

This chapter reviews historical literature and recent research into rock mass characterisation, with emphasis on investigations concerning the influence of discontinuity persistence and intact rock bridges on rock slope stability. The following section presents a brief discussion of rock mass characterization and relevant rock mechanics concepts. Methods for incorporating the influence of non-persistent discontinuities and intact rock bridges into stability analysis are then discussed. The last section of the chapter introduces a selection of numerical methods that have been used to simulate step-path failure and brittle failure mechanisms in slopes.

2.2. Rock Mass Characterization

The term rock mass is used in engineering to describe the two-part system of an in situ volume comprising unfractured blocks of intact rock, and the fractures, faults or other discontinuities that intersect it (Figure 2 - 1). Rock mass behaviour is a function of the interaction between imposed stresses, and the inherent strength and deformability of both intact rock and discontinuities.
In engineering applications, the stability of a rock mass is usually assessed by phenomenological methods, which are based on large-scale observations of rock mass behaviour (e.g. Eberhardt, 1998). Often, rock mass stability is characterized with respect to either the peak strength of the rock mass, or with respect to the maximum amount of deformation that can be accommodated before irreversible yield occurs. If discontinuity-controlled slope failure is expected, then block stability is commonly characterised with respect to the peak shear strength of the adverse discontinuities that define the failure surface.

For open pit slope design, rock mass characterization is undertaken with the aim of evaluating the stability of a proposed slope configuration. In order to assess the influence of intact rock properties and pre-existing discontinuities, field observations may be combined with laboratory testing of rock samples, discontinuities, slope movement records, and interpretation of numerical modelling or limit equilibrium assessments.
2.2.1. Influence of Intact Rock Strength

Common models for representing the strength of geological materials consider three separate elements: cohesion, friction and tensile strength. Intact rock contributes the greater part of cohesion and tensile strength to a rock mass, arising from individual grain strength and inter-grain bonds (Eberhardt, 1998; Diederichs et al., 2004, 2007). For several decades researchers have recognized that the mobilization of cohesion and friction is strain-dependent:

“The deformation resistance of the material bridges takes effect at much smaller deformations than the joint friction: this joint friction makes partly up for lost strength.”

(Müller, 1966, in Barton, 2011)

During intact rock failure, cohesive strength is mobilized before deformation is kinematically feasible. Once cohesive strength is destroyed, shear displacement can occur and frictional strength is mobilized.

At macroscopic scales, the peak shear strength of intact rock is usually orders of magnitude larger than the shear strength of discontinuities, with the possible exception of highly weathered rock masses with weak intact rock and strongly cemented or mineralized discontinuities (Barton, 2011). As a consequence of the shear strength disparity between intact rock and discontinuities, a small proportion of intact rock bridges can be of significant benefit to slope stability.

Researchers including Diederichs (1999), Eberhardt (1998), Hajiabdolmajid (2001), Martin (1994) and others have observed that at microscopic scales brittle failure of intact rock is characterized by the initiation and propagation of new inter- and intra-grain tensile cracks (Figure 2 - 2); in slopes, the development of brittle failure may eventually lead to crack interaction, coalescence and the formation of a continuous failure surface (Stead et al., 2006). Even if the cumulative result of intact rock failure is a macroscopic shear surface, the pre-peak behaviour may in some cases be controlled by microscopic cracking in tension (Diederichs, 2007).
Although tensile cracking tends to dominate under low confining stresses (Lajtai, 1969a, b, c), there are in fact three modes of brittle crack growth recognized in fracture mechanics (Figure 2 - 3). Mode I cracks grow by opening, when tensile stress is applied perpendicular to the direction of crack growth; Mode II cracks grow by in-plane shearing, with shear stress applied parallel to the direction of crack growth; Mode III cracks grow by out-of-plane shearing or tearing, with displacement orthogonal to the direction of crack growth.
2.2.2.  Influence of Discontinuities

In near-surface excavations where stresses are usually low relative to the strength of intact rock, discontinuities can be the primary determinant of stability. The orientation, size, shear strength, and spatial distribution of discontinuities directly influence rock mass blockiness and the susceptibility of discrete blocks to sliding or toppling.

Empirical methods for rock mass classification such as the Geological Strength Index (GSI; Hoek et al., 2002), Rock Mass Rating system (RMR; Bieniawski, 1989), RMI system (Palmström, 1995), and the Norwegian Geotechnical Institute’s Tunnelling Quality Index (Q; Barton et al., 1974) all incorporate adjustments for rock mass strength that depend largely on the properties of discontinuities. The following sections briefly introduce the role of different discontinuity parameters that are influential to rock slope stability.

2.2.2.1. Orientation

Orientation determines kinematic freedom for stability with idealized, fully-persistent joints. Dip angle relative to horizontal, and dip direction are usually the first parameters recorded in geotechnical logging of discontinuities. Once the orientation of major discontinuity sets is recorded, simple kinematic assessment of discontinuity-controlled failure is often the first stage in rock slope stability assessment.

In kinematic assessment, the dip, dip direction and spatial distribution of pre-existing discontinuities are evaluated with respect to slope geometry, in order to assess the potential to cause simple rotational and translational failure of discrete blocks. Discrete block failures are commonly grouped into three categories: wedge failure, planar sliding failure, and toppling failure (Figure 2 - 4).

Low-level analyses may use the assumption of full kinematic freedom for toppling or sliding failures, where blocks are considered to be laterally unconfined. In reality, however, the 3-D geometry of lateral and rear release surfaces has been shown to have a strong influence on kinematic freedom and the resultant failure mechanism for sliding and toppling failures in rock slopes (e.g. Hungr and Amman, 2011; Brideau, 2010; Wyllie and Mah, 2004).
Figure 2 - 4: Schematic illustration of planar sliding (a), wedge sliding (b), and block toppling (c) in rock slopes.

Discontinuity orientation is also influential in the development of brittle failure in rock masses with non-persistent joints. For example, Vyazmensky et al. (2010a,b) used the hybrid FEM-DEM code ELFEN to investigate the influence of discontinuity orientation on block cave propagation. Their results suggested that even complex, large-scale failure mechanisms involving brittle fracture of intact rock can be influenced by the orientation of pre-existing discontinuities.

2.2.2.2. Persistence

Persistence describes discontinuity size, and is one of the primary controls on in situ block dimensions in a rock mass. In theoretical models persistence is represented by the diameter of an idealized ellipsoid discontinuity, or the edge length of an idealized square discontinuity (Jennings, 1970). In practice, persistence may be expressed directly as the trace length of discontinuities measured in scanlines or window maps (e.g. Zhang and Einstein, 1998), or may be expressed as a percentage relating the length or surface area of a discontinuity to the length or surface area of the a co-linear or co-planar reference line/plane (e.g. Dershowitz and Einstein, 1988). In 3-D mapping of digital terrain models, persistence is measured as the diameter of best-fit circular discs superimposed on discontinuity planes. True discontinuity shape and persistence are usually impractical to measure in situ, because we cannot observe the interior of a rock mass without completely dismantling it; in other words, because of sampling bias.
Einstein et al. (1983) were among early researches who considered the influence of variable persistence in probabilistic slope stability assessment. They used a statistical slope stability code *SLOPESIM*, which could run many iterations of potential slope configurations based on variable discontinuity persistence, according to probability distributions defined by the user. More recently, Kim et al. (2007) showed with statistical analysis that rock mass strength calculated using the Hoek-Brown criterion could vary by up to 50% with changes in joint set persistence factors.

Statistical treatment of trace length measurements can be used to help predict the distribution of true in situ persistence (e.g. Kulatilake and Wu, 1984a,b; Wu et al., 2011; Wang, 2005; Zhang and Einstein, 1998, 2000). Borehole televiewer logs can also be used to measure trace lengths on cylindrical maps of borehole walls (e.g. Mauldon and Mauldon, 1997). Combining pit mapping observations with emerging techniques in borehole televiewer mapping and remote sensing may help to better understand true discontinuity persistence.

Trace length measurements in outcrops (and boreholes) are subject to sources of bias and uncertainty, some of which are specific to remote sensing methods (Figure 2 - 5). Consequently, the distribution of persistence derived from field investigation may over- or under-estimate true persistence:
Figure 2 - 5: Illustration of sources of bias in measuring discontinuity traces.

- **Truncation** occurs when discontinuities are too small or too short to measure. Typically, the field geologist or engineer selects a cut-off trace length based on practical constraints such as time limitations for field work. Truncation bias may be minimised by choosing a cut-off length that is much shorter than the average discontinuity trace length (Sturzenegger and Stead, 2011; Zhang and Einstein, 1998)

- **Censoring** occurs when discontinuity length extends past the sampling region. In this case, measured discontinuity length under-estimates true persistence, and will skew the persistence distribution towards a lower mean value (Priest, 1993)

- **Orientation Bias** occurs when boreholes, scanlines or windows are sub-parallel to discontinuities. In the case of discontinuities sub-parallel to camera or LiDAR scanner line of sight, the discontinuity will appear as a line, and the discontinuity plane itself is occluded. Shooting from multiple camera stations or LiDAR scanner positions can help to minimise occlusion in remote sensing surveys (Sturzenegger et al., 2007; Lato et al., 2009; Terzaghi, 1965)

- **Length bias** strongly affects scanline mapping, because longer discontinuity traces have increased likelihood of intersecting the scanline. Window mapping is less influenced by length bias than scanline mapping, because the added dimension increases sampling area such that many more short discontinuities will be contained within the sampling window (Sturzenegger et al., 2011; Zhang and Einstein, 2000); Wu et al. (2011) demonstrated that estimates of trace length
can also be influenced by the shape of a window map (circular or rectangular); they found that rectangular windows may tend to result in better estimates of (true) discontinuity trace length than circular maps in some cases; however Mauldon et al. (2001) note that rectangular windows may impose a directional bias to the mapping results.

- **Scale bias** occurs due to limitations due to image resolution in remote sensing-derived models. Higher-resolution imagery allows for smaller discontinuities to be distinguished, whereas lower-resolution imagery may render smaller features indistinguishable; also, discontinuities that appear as extremely persistent features in low-resolution imagery may in fact be identified in high resolution imagery as composite surfaces comprising interaction between less persistent discontinuities and fracture of intact rock (Sturzenegger, 2010; Sturzenegger and Stead, 2009a; Ortega et al., 2006)

- **F-bias**: which occurs because discontinuity traces on outcrops or walls are, ostensibly, chords created by the random intersection of circular discontinuity planes with a sampling window (Priest, 2004)

Larger, more persistent discontinuities such as faults are less frequent than smaller, less persistent joints. Field studies have shown that trace length distributions can conform well to negative exponential or power-law functions (e.g. Ortega and Marrett, 2000; Ortega et al., 2006; Priest, 1993). The ISRM Commission on Standardization of Laboratory and Field Tests (1978) proposed five categories for classification of discontinuity persistence:

<table>
<thead>
<tr>
<th>Persistence</th>
<th>m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very low persistence</td>
<td>&lt; 1 m</td>
</tr>
<tr>
<td>Low persistence</td>
<td>1 – 3 m</td>
</tr>
<tr>
<td>Medium persistence</td>
<td>3 – 10 m</td>
</tr>
<tr>
<td>High persistence</td>
<td>10 – 20 m</td>
</tr>
<tr>
<td>Very high persistence</td>
<td>&gt; 20 m</td>
</tr>
</tbody>
</table>
Palmström (1995) proposed an alternative classification system for discontinuity persistence as part of his Rock Mass index (RMi) characterization system, which categorizes persistence based on discontinuity type:

<table>
<thead>
<tr>
<th>Persistence</th>
<th>Discontinuity Type</th>
<th>Measurement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very short</td>
<td>Bedding/foliation partings</td>
<td>&lt; 0.5 m</td>
</tr>
<tr>
<td>Short/small</td>
<td>Joints</td>
<td>0.1 – 1 m</td>
</tr>
<tr>
<td>Medium</td>
<td>Joints</td>
<td>1 – 10 m</td>
</tr>
<tr>
<td>Long/large</td>
<td>Filled joints, seams, shear zones</td>
<td>10 – 30 m</td>
</tr>
<tr>
<td>Very long/large</td>
<td></td>
<td>&gt; 30 m</td>
</tr>
</tbody>
</table>

Since the publication of ISRM guidelines, open pit slopes have been developed and planned to increasingly larger heights. In large open pits, inter-ramp scale joints and faults features beyond the 20 m upper limit of “very high” persistence may become influential to inter-ramp and overall pit stability. Sturzenegger (2010) suggested a new class for “extremely persistent” discontinuities over 40 m in persistence, but noted that such large-scale features are subject to scale bias; extremely persistent structures that appear to be singular discontinuities at low resolution may be revealed at higher resolution to comprise zones of interaction between different non-persistent discontinuity sets and intact rock fractures. Discontinuity classification methods may thus benefit from considering a lower-bound limit for discontinuity spacing, where highly fractured zones with extremely closely-spaced discontinuities may be effectively treated as one large discontinuity (i.e. approaching the behaviour of a fault core or highly damaged zone, Laws, 2001).

2.2.2.3. Termination

Two possible termination types which may occur at intersections of discontinuities are show in Figure 2 - 6, in which the discontinuities may pass through one another (X-termination), or one discontinuity will terminate against the other (T-termination). T-type terminations can be specifically included in discrete fracture network codes such as
FracMan (Golder Associates, 2001), which use user-input discontinuity data in order to stochastically generate 3-D networks of planar discontinuities.

Figure 2 - 6: Discontinuity termination types in FracMan (Reproduced after Elmo, 2006).

Termination style is an important consideration for establishing the hierarchy of discontinuities in a rock mass. In general, younger, less persistent discontinuities may be more likely to terminate against older, more persistent discontinuities: for example, recently-formed strata-bound tension fractures in sedimentary rock tend to terminate against older bedding foliation; however, this does not always occur, as tension joints may also propagate directly across bedding foliation, or step sideways along bedding before continuing into adjacent strata (Mandl, 2005).

A third termination condition describes discontinuities tips that end in intact rock ($T_{ir}$). Dershowitz and Einstein (1988) described three styles of termination that may occur for discontinuities that terminate in intact rock:

- Termination with curving at the end;
- Termination without curving;
- Termination with bifurcation at the end.

Curving or irregular extension of discontinuity tips is expected in areas where blast-induced damage has occurred (Hagan, 1982); termination in intact rock without curving may thus suggest that the rock mass is relatively undisturbed. Field investigations undertaken in this thesis included wherever possible records of discontinuity termination hierarchy; specific occurrences of termination in intact rock (with or without curved or bifurcated tips); and evidence of extension and dilation of pre-existing discontinuities.
2.2.2.4. Spacing

Discontinuity spacing is a measure of fracture frequency, and is usually measured either with respect to specific discontinuity sets, or presented as a generalized measure of linear frequency observed in borehole logging or scanline mapping.

Set-specific discontinuity spacing measures the perpendicular distance separating parallel or sub-parallel discontinuities of similar orientation. In a 3-D volume of rock mass, set-specific discontinuity spacing values can be incorporated into a single measure of fracture quantity, the volumetric joint count $J_V$ proposed by Palmström (1982; Figure 2 - 7). In practice, care must be taken in noting that discontinuity spacing observed in outcrops reflects apparent spacing, which may be larger or smaller than true spacing, depending on the orientation of the outcrop with respect to that of the discontinuity set (Priest, 1993).

Figure 2 - 7: Relationship between set-specific discontinuity spacing and volumetric joint count $J_V$ (Reproduced after Palmström, 1982).

Linear discontinuity spacing, measured on scanlines or in boreholes, is subject to random variation based on location of the sampling line (Priest, 1993, Figure 2 - 8).
Based on the assumption of random discontinuity occurrences along a scanline, the probability density function of discontinuity spacing can be expressed with a negative exponential function of distance $x$ along a scanline (Priest, 1993):

$$f(x) = \lambda e^{-\lambda x}$$

Integrating the probability density function yields the cumulative probability distribution:

$$F(x) = 1 - e^{-\lambda x}$$

For an exponential distribution, the values of population mean and standard deviation are equivalent. Therefore, mean and standard deviation of discontinuity spacing will tend to converge toward the same value as the sample size of measured discontinuities becomes large (Priest, 1993).

### 2.2.2.5. Roughness and Waviness

Roughness and waviness describe the deviation of a discontinuity surface from an idealized, perfectly flat best-fit plane; roughness usually describes millimetre- to centimetre-scale surface irregularities called asperities, whereas waviness describes larger-scale undulations with amplitude and wavelengths of metres or more.

Roughness is usually measured in the down-dip direction, in order to characterise the benefit of added frictional resistance to sliding or shearing. Experiments with artificial saw-tooth joints by Patton (1966) showed that the effective friction angle of a joint was increased proportional to the inclination of asperities (Figure 2 - 9).
Patton divided components of roughness into first-order (equivalent to large-scale waviness) and second-order (equivalent to small-scale asperity roughness), and showed that for small shear displacements, second-order roughness controls discontinuity strength and deformation. At larger shear displacements, second-order roughness (asperities) may be destroyed, and subsequent displacements are governed by first-order roughness.

Barton (1971), Bandis (1980) and Bandis et al. (1981) used results from laboratory shear tests to demonstrate the scale dependence of discontinuity shear strength. Based on their results they proposed that the shear strength of a discontinuity may be expressed in terms of three components (Figure 2 - 10):

1. Residual friction, or the basic friction angle (\(\phi_r\) or \(\phi_b\)) represents an intrinsic frictional strength component that is independent of scale

2. Roughness contributes scale-dependent strength in two forms:
   a. Internal shear strength of asperities, \(S_a\), represents the cohesive strength of asperities that must be exceeded in order to shear through the roughness and allow displacement
   b. A geometrical component, \(d_n\), relates to the angle of inclination of asperities, and influences the amount of dilation that must occur in order for shear displacement to occur
One popular field method for characterising discontinuity roughness, proposed by Barton and Choubey (1977), involves tracing discontinuity surface profiles with comb profilometers to measure roughness over 100 mm to 300 mm intervals. Profilometer measurements can be related to the suggested characteristic profiles for JRC (Barton and Choubey, 1977), which may in turn be used to characterise discontinuity shear strength using an empirical relationship such as the Barton-Bandis shear strength criterion (Barton and Bandis, 1990). Alternatively, discontinuity profiles may be explicitly digitised for use in laboratory-scale numerical models (Tatone and Grasselli, 2012).

Another field method proposed by Fecker and Rengers (1971) involves measuring the orientation of a discontinuity surface with a geological compass with a series of platens of different size connected to the lid. For a rough surface, the orientation measurements measured using smaller platens will show deviation from the “smoothed” orientation measured on the larger platens. When the full set of orientation data is plotted on a stereonet, the amount of scatter of the poles may be used a proxy indicator of surface roughness: low scatter occurs with smooth surfaces and high scatter with rough surfaces.

Figure 2 - 10: Effect of sample scale on components of joint shear strength
(Reproduced after Bandis et al., 1981)
With the increasing use of terrestrial remote sensing methods, researchers have also investigated the applicability of measuring roughness from 2-D error map contours or 1-D direction profiles extracted from point clouds derived from digital photogrammetry or LiDAR scanning (e.g. Sturzenegger, 2010; Tatone and Grasselli, 2010; Tatone, 2009; Poropat, 2008; Haneberg, 2007; Kemeny et al., 2006). Despite the advantages of remote sensing methods, particularly in the measurement of inaccessible high rock slopes, most researchers and practitioners recommend that remote sensing should always be complemented by field verification (Figure 2 - 11).

### 2.2.2.6. Aperture

Aperture represents the open width of a discontinuity, and relates directly to fracture permeability. Discontinuity aperture may range from cavernous voids (> 1 m) which may occur in karstic limestone, for example, to very tight (<0.1 mm) or welded discontinuities that may occur in massive, confined crystalline rock (Wyllie and Mah, 2004). Hydraulic aperture is one of the primary influences on secondary permeability of a rock mass. The processes of fracture flow of groundwater, crack growth in intact rock, and stress
redistribution throughout a rock mass all interact in a coupled hydro-mechanical process that influences rock mass dilation, effective stress state, and stability of both discontinuity-controlled and brittle failure mechanisms (Lorig, 2009).

In a review of the engineering aspects of sheet joints, Hencher et al. (2011) noted that a progressive degradation of cohesion may occur as discontinuities grow and dilate, transitioning from embryonic micro-fractures, to tightly-closed proto-joints and eventually to fully open exfoliation joints. As the joints grow, the tips extend and the aperture dilates, destroying cohesion imparted by intact rock bridges. Hencher et al. (2011) also noted, however, that wide aperture discontinuities may regain some cohesive strength contributed by soil infilling deposited in fractures by groundwater, or from zones of weathered rock, where the discontinuity surface has degraded to soil-strength material.

2.2.2.7. Alteration, Coating or Infill

Alteration or coating of discontinuity surfaces, or infill by hydrothermal precipitates, can increase or decrease the shear strength of discontinuities relative to a fresh rock surface. Rock mass classification systems including \( \text{RMR}_{89} \), \( Q \), and the quantified GSI chart (Cai et al., 2004) incorporate joint alteration status using a numerical factor called \( J_a \) based on qualitative descriptions of alteration, coating or infill.

Joint alteration factor is incorporated into an overall joint condition factor, \( J_C \), which also includes factors for large-scale joint waviness \( J_W \) and small-scale joint smoothness \( J_S \) according to the relationship:

\[
J_C = \frac{J_W J_S}{J_A}
\]

Coatings or infill of crushed material like fault gauge, or fine-grained weathering products like clay, will reduce discontinuity stiffness and frictional strength. Conversely, hydrothermal mineralization with hard minerals like quartz may effectively seal a discontinuity together, adding cohesive strength, which must be broken before shearing can occur. Even if discontinuities are healed or tightly closed, they may still provide a preferential path of weakness (Figure 2 - 12).
2.2.2.8. Intensity

Fracture intensity is an aggregate measure of the degree of fracturing in a rock mass (Table 2 - 1). Intensity can be used to construct discrete fracture network models (DFN), by using borehole (P_{10}) or window map (P_{21}) measurements of fracture intensity in addition to joint set orientations and estimated persistence.
Table 2 - 1: Measures of fracture quantity; intensity is used in DFN generation
(Based on Golder Associates, 2001; and Elmo, 2006).

<table>
<thead>
<tr>
<th>Dimension of feature</th>
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<tbody>
<tr>
<td></td>
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</table>

<table>
<thead>
<tr>
<th>Dimension of sampling region</th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>P00  Length$^0$</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Number of fractures</td>
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<td></td>
<td></td>
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<tr>
<td>P10  Length$^{-1}$</td>
<td></td>
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<tr>
<td>Number of fractures per unit length of scanline (linear intensity)</td>
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<tr>
<td>P11  Length$^0$</td>
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<tr>
<td>Length of fractures intersects per unit length of scanline</td>
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<td></td>
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<tr>
<td>P20  Length$^{-2}$</td>
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<tr>
<td>Number of trace centres per unit area of sampling (areal density)</td>
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<tr>
<td>P21  Length$^{-1}$</td>
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<tr>
<td>Length of fracture traces per unit area of sampling plane (areal intensity)</td>
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<td></td>
<td></td>
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<tr>
<td>P22  Length$^0$</td>
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<tr>
<td>Area of fractures per unit area of sampling plane</td>
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<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>P30  Length$^{-3}$</td>
<td></td>
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<tr>
<td>Number of fracture centres per unit volume of rock mass (volumetric density)</td>
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<td></td>
<td></td>
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<tr>
<td>P32  Length$^{-1}$</td>
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<tr>
<td>Area of fractures per unit volume of rock mass (volumetric intensity)</td>
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<tr>
<td>P33  Length$^0$</td>
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<td></td>
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<tr>
<td>Volume of fractures per unit volume of rock mass</td>
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<td></td>
</tr>
<tr>
<td>Density</td>
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<td></td>
</tr>
<tr>
<td>Intensity</td>
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<td></td>
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<tr>
<td>Porosity</td>
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</table>

True volumetric fracture intensity, $P_{32}$, represents the total area of fracture surfaces per unit volume of rock mass, and cannot practically be measured in the field.

Four computer programs that have been developed to generate stochastic DFN models for rock mechanics applications include: FracMan (Dershowitz et al., 1998), JointStats (JKMRC, 2000), 3FLO (Billaux et al., 2006; ICSAS, 2006), and RESOBLOK (Merrien-Soukatchoff et al., 2012).
2.2.3. Influence of Blast-Induced Damage

Blasting in open pit slopes is undertaken to break up in situ rock mass and increase ease of excavation. As an additional consequence of blasting, the remaining in situ rock mass is disturbed via the extension and opening of existing discontinuities by explosive gasses, and by the generation of new shockwave-induced cracks. Intact rock bridges may thus be destroyed by blasting, and the rock mass strength and modulus will be degraded for some distance behind the slope, delineated as the “blast damaged zone” (Hoek and Karzulovic, 2000).

Early research into probabilistic step-path simulation at Bougainville copper mine, undertaken by McMahon (1979) led to the development of the STPSIM code, which was later modified by Baczynski and rewritten as STEPSIM4 which was originally used as part of the risk-based pit slope optimisation study at Ok Tedi mine between 1997-1999. In attempting to assess the influence of blasting damage on stability of the final pit wall, Baczynski, Little, and colleagues used a 50% degraded cohesion value within the rock mass in the blast damage zone, which was estimated to extend 50 m behind the final pit wall (Baczynski, 2000; Little, 1999; Little et al., 1999).

Hoek and Karzulovic (2000) published a set of preliminary recommended guidelines for estimating the potential distance $D$ that the blast damage zone may extend behind an open pit slope, based on bench height $H$, confinement of the blast, and the use of controlled blasting procedures (Table 2 - 2).
Table 2 - 2: Preliminary recommended guidelines for assessing thickness of blast damage zone, from Hoek and Karzulovic (2000).

<table>
<thead>
<tr>
<th>Blast size</th>
<th>Confinement</th>
<th>Controlled Blasting Techniques</th>
<th>Distance $D$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Large production</td>
<td>Confined</td>
<td>Little or None</td>
<td>2 to 2.5 $H$</td>
</tr>
<tr>
<td>blast</td>
<td>Free face</td>
<td>None</td>
<td>1 to 1.5 $H$</td>
</tr>
<tr>
<td>Production blast</td>
<td>Confined</td>
<td>Some control: one or more buffer rows</td>
<td>1 to 1.2 $H$</td>
</tr>
<tr>
<td></td>
<td>Free face</td>
<td>Some control: one or more buffer rows</td>
<td>0.5 to 1 $H$</td>
</tr>
<tr>
<td></td>
<td>Free face</td>
<td>Carefully controlled</td>
<td>0.3 to 0.5 $H$</td>
</tr>
</tbody>
</table>

In a study of the mechanisms of blasting damage, Hagan (1982) identified four processes that create overbreak around blasts in rock. In chronological order from the moment of detonation, they are:

1. Crushing and radial fracturing around loaded blast holes
2. Internal spalling and fractures along boundaries of high stiffness contrast
3. Extension of pre-existing discontinuities by gas pressure and strain wave-induced cracking; and
4. Fracturing induced by load relief

Figure 2 - 13 presents a schematized view of the damage mechanisms occurring around a blast hole, and Figure 2 - 14 illustrates the process of load relief fracturing with the physical analogue of a steel plate, representing the blast-induced compressive strain wave, impacting a stack of rubber mats, representing a rock mass.
The crushed zone around a borehole usually extends less than 2 borehole diameters into the rock mass, but can be as high as $7.5d$ in highly porous material. In bedded or foliated strata the crushed zone will usually be much less than $7.5d$ (Hagan, 1982).

Immediately after detonation, gas expansion causes pre-existing fractures to extend. Longer fractures extend first, because stress concentration at crack tips increases with crack length. In well-bedded rock masses with persistent foliation (as exists at Jwaneng), extension of pre-existing discontinuities can be the dominant cause of rock mass damage (Hagan et al., 1978). For sub-horizontal to horizontal bedding planes, the upward force applied by expanding gas causes rotational uplift and cross-bedding tensile fracturing.

The severity of cross-bedding tensile fractures tends to increase towards the top of a bench, and in general, damage is less severe in thicker beds (Hagan et al., 1978). Overbreak also tends to be less severe if pre-existing discontinuities are tight or infilled, because wide aperture discontinuities are more easily infiltrated by explosive gases (Hagan, 1982).
Yu and Vongpaisal (1996) explained that blast damage at a free face is dominated by strain wave-induced internal spalling. When an incident compression strain wave impacts a free face, energy is reflected backwards in tensional strain waves causing internal spalling (Hagan, 1982), also called “scabbing failure” (Yu and Vongpaisal, 1996). Both internal spalling and release-of-load fractures (Figure 2 - 14) are caused by reflected stress waves, and both mechanisms generate tension fractures. In multi-row blasts with large burden and long time delays between row detonation, tension cracks have been recorded to occur as far as 60 m behind the final face (Hagan et al., 1978).

![Diagram of falling steel plate and layers of rubber](image)

*Figure 2 - 14: Load relief-induced extension and separation of layers in a compressible medium (Reproduced after Hagan, 1982).*

Where pre-existing joints are persistent and widely spaced, fragmentation is suppressed, because the strain wave energy will work to extend existing joints preferentially, before nucleating new microcracks and extending existing, smaller discontinuities (Hagan and Morphet, 1986). In intensely fractured rocks, overbreak can be more severe. Mining-induced movements caused by blasting and long-term relaxation can also cause slickensiding on pre-existing fractures, effectively pre-conditioning them for shear failure (Hagan et al., 1978).
Although current numerical codes are not capable of simulating both stress wave-induced fracturing and gas penetration and extension of blast-induced fractures, researchers have conducted controlled experiments to characterise the specific influence of dynamic loading by stress-wave induced by blasting. Recent studies have combined laboratory-scale blasting experiments with numerical modelling in order to better characterise dynamic fracture behaviour and failure mechanisms associated with blasting-induced stress-waves in rock (Dehghan Banadaki and Mohanty, 2012; Dehghan Banadaki, 2010; Mohanty et al., 2005).

In practice, blast-induced damage can be mitigated by incorporating controlled blasting techniques such as buffer, trim, and pre-split blasting (e.g. Read and Stacey, 2009), and by designing blasts according to pre-existing discontinuity properties and rock mass characteristics. Lizotte and Scoble (1994) summarise the findings of Burkel (1979) who investigated the influence of blasting direction relative to geological foliation on fragmentation and overbreak:

- **Blasting with dip** (Figure 2 - 15A) can cause severe backbreak and large movement of the blasted volume away from the face; breakage occurs preferentially along pre-existing foliation; the result is a lower muckpile profile and smoother floor
- **Blasting against dip** (Figure 2 - 15B) tends to cause less backbreak as strata dips into the wall; final wall geometry is controlled by more intact rock fracture perpendicular to bedding planes than when blasting with dip; fragmentation may be poor in the toe of the slope; smaller movement away from the face occurs; the result is a higher muckpile profile and rougher floor
- **Blasting with strike** (Figure 2 - 15C) causes blast energy to propagate in channels along parallel pre-existing discontinuities; stress waves will propagate farther than in an against-strike blast; as with against-dip blasts, final wall geometry is influenced by destruction of out-of-plane intact rock bridges, as new cracks are generated perpendicular to foliation, with potential for severe backbreak
Figure 2.15: Schematic illustration of blast configuration relative to persistent foliation; rows of blast holes are detonated in sequence from 1 to 4 (Modified after Burkel, 1979).
2.3. Slope Stability Assessment with Non-Persistent Discontinuities

Mine designers assess open pit slope stability at three scales: bench-scale, inter-ramp scale, and overall slope scale (Figure 2 - 16). Bench-scale analysis considers stability of individual benches, usually up to 30 m high. Inter-ramp stability analysis considers “bench stacks” which separate haul ramps, and are generally up to 300 m high. Overall slope analyses consider the complete toe-to-crest pit height.

![Diagram of open pit geometry and associated terminology](image)

**Figure 2 - 16: Illustration of open pit geometry and associated terminology (Wyllie and Mah, 2004, by permission).**

Bench-scale stability may be evaluated using simple kinematic assessment or limit equilibrium calculations for simple block failures. As slope height increases, however, the effects of geological heterogeneity and increased stresses elevate the probability of more complex failure mechanisms. Broadly, rock slope failure mechanisms can be divided into three types:

**Type 1:** discontinuity-controlled failure tends to occur where stresses are low relative to rock mass strength, and adverse discontinuities have kinematic freedom to form blocks that may slide or topple;
**Type 2:** brittle failure of intact rock tends to occur where stresses are high relative to rock mass strength. These conditions may occur for extremely high slopes or for highly weathered rock masses with weak intact rock strength;

**Type 3:** combined failure modes including (1) and (2) may occur where pre-existing, adverse discontinuities form a non-continuous potential failure surface. If stresses are sufficiently high, brittle failure of intact rock may occur, facilitating breakout and the development of kinematic freedom.

Type 3 slope failures become more common with increase in stress and slope height (Read and Stacey, 2009). Interaction between pre-existing discontinuities and brittle failure of intact rock bridges may result in step-path type failures (Figure 2 - 17), or combined sliding or toppling failures where failure of intact rock creates release or sliding surfaces (Figure 2 - 18). Improved characterisation of the persistence of adverse discontinuities, and the potential proportion of intact rock bridges, may help to better assess potential for large Type 3 slope failures.

![Figure 2 - 17: Idealized relationship between three discontinuity sets and intact rock bridges, forming a potential failure surface in a slope (Reproduced after Jennings, 1970 and ISRM, 1978).](image)
2.3.1. Incorporating Intact Rock Bridges

Methods for incorporating rock bridges into slope stability analysis can be divided into two categories: (1) models with “in-plane” rock bridges, and (2) models with non-coplanar or “out-of-plane” rock bridges.

2.3.1.1. In-plane rock bridges

In-plane rock bridges are represented by conceptual “patches” of intact rock along a theoretical, fully-persistent discontinuity plane or failure surface. The first method for incorporating the strength benefit of rock bridges on a candidate slope failure surface was proposed by Jennings’ (1970).

In Jennings’ method, apparent shear strength parameters are applied to a conventional limit equilibrium analysis. Strength parameters are weighted according to the proportion
of intact rock and pre-existing discontinuity along a candidate failure surface in 2-D or 3-D (Figure 2 - 19). He defined a coefficient of continuity, \( k_L \), as the ratio of total discontinuity length compared to a theoretical failure path length:

\[
k_L = \frac{\sum \text{jointed length}}{\sum \text{jointed length} + \sum \text{rock bridge length}}
\]

Extending the concept to a 2-D failure surface, with square or circular discontinuities, Jennings described areal continuity as:

\[
k_A \approx \frac{(\sum \text{jointed length})^2}{(\sum \text{jointed length} + \sum \text{rock bridge length})^2} = (k_L)^2
\]

Where fractures are not fully continuous, \( k_L < 1 \) and therefore \( k_A << 1 \). Measures of linear fracture continuity, therefore, are inherently larger than true areal continuity. If the assumed joint geometry is valid, linear fracture continuity represents a larger estimate of discontinuity persistence.

Figure 2 - 19: In-plane rock bridges are idealized as “patches” of intact rock on an idealized persistent discontinuity plane.
More recently, researchers have mapped in-plane rock bridges \textit{a posteriori} on failure surfaces and detachment niches of past rockfalls and wedge failures (Frayssines and Hantz, 2006; Paronuzzi and Serafini, 2009); others have used statistical methods involving discrete fracture network analysis to estimate the potential range of out-of-plane rock bridge configurations (Elmo et al., 2011). Table 2 - 3 presents selected estimates of intact rock bridge content from studies of natural slopes and open pits.

\textbf{Table 2 - 3: Summary of selected case studies quantifying intact rock bridge content.}

<table>
<thead>
<tr>
<th>Location</th>
<th>Method &amp; Measurement</th>
<th>Rock bridge content (%)</th>
<th>Authors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Palliser slide, Canada</td>
<td>Length %</td>
<td>2-3</td>
<td>Sturzenegger &amp; Stead, 2012</td>
</tr>
<tr>
<td>Åknes slide, Norway</td>
<td>Length %</td>
<td>1 to 3</td>
<td>Groneng et al., 2009</td>
</tr>
<tr>
<td>Highway Roadcuts, Idaho USA</td>
<td>Length %</td>
<td>0 to 3</td>
<td>Ristau, 1994</td>
</tr>
<tr>
<td>Gypsum Mine, Arizona USA</td>
<td>Length %</td>
<td>16 to 36</td>
<td>LeBaron, 2011</td>
</tr>
<tr>
<td>Diavik A154*, Canada</td>
<td>DFN; length %</td>
<td>&gt;5</td>
<td>Karami et al., 2007</td>
</tr>
<tr>
<td>Failed roof slab above road, Friuli, Italy</td>
<td>Failure mapping; area %</td>
<td>26</td>
<td>Paronuzzi and Serafini, 2009</td>
</tr>
<tr>
<td>Rockfall, French Sub-alps</td>
<td>Failure mapping; area %</td>
<td>0.2 to 5.0</td>
<td>Frayssines &amp; Hantz, 2006</td>
</tr>
<tr>
<td>Western Alps, France</td>
<td>Failure mapping; length % +numerical simulation</td>
<td>7.3</td>
<td>Lévy et al., 2010</td>
</tr>
</tbody>
</table>

*Stability analyses assigned equivalent rock mass strength parameters to rock bridge intervals, not intact rock strength.

For smaller-scale failures, researchers have mapped rock bridges as zones where brittle failure occurs through intact material. These zones may be distinguished from pre-existing discontinuities by the lack of staining or weathering and by the conchoidal or
irregular surface texture of the fracture surface. Surfaces with no evidence of shearing may also be interpreted to have failed in direct tension loading (Lévy et al., 2010).

Large slope failures, such as the Palliser rock slide investigated by Sturzenegger and Stead (2012) may include large step intervals which resemble brittle fracture surfaces from long-range, but which in fact comprise a combination of multiple sets of low-persistence discontinuities connected by smaller intervals of intact rock fracture.

2.3.1.2. Out-of-plane rock bridges

Out-of-plane rock bridges occur where discontinuities are non-coplanar. Usually out-of-plane rock bridges are traced as the shortest distance of intact rock between discontinuity tips (Figure 2 - 20). This assumed rock bridge geometry may however not always accurately predict the path of crack growth. As the intensity of pre-existing fractures increases, localization effects cause the stress distribution and crack growth behaviour in a rock mass to become more complex, making interpretation of intact rock bridge geometry a non trivial task (Elmo et al., 2011).

---

Figure 2 - 20: Delineation of possible intact rock bridges becomes complex for non-coplanar fracture networks (Modified after Elmo et al., 2009).
Many experimental laboratory studies and numerical modelling studies have been undertaken to investigate the influence of pre-existing discontinuity geometry, material parameters and loading conditions on fracture initiation, growth and coalescence in out-of-plane rock bridges (e.g. Yan, 2008; Gehle and Kutter, 2003; Sagong and Bobet, 2002; Wong et al., 2001; Bobet and Einstein, 1998; Lajtai, 1969a,b,c). However, there is no published set of field methods for characterising out-of-plane rock bridges at the bench, inter-ramp, or overall slope scale in open pits.

To better capture true 3-D rock bridge geometry, field procedures may benefit from mapping whenever possible in two or more orthogonal and offset sampling windows. As with field methods that recommend measuring discontinuity persistence in both along-strike and down-dip directions (e.g. Narendranathan et al., 2012), intact rock bridges should be characterised with consideration to 3-D geometry and the location of rock bridges with respect to potential instabilities. An identical quantity of intact rock bridges could have a markedly different impact on slope stability depending on the location of rock bridges with respect to the unstable volume of rock mass. For example, the influence of a rock bridge occurring as a strong lateral constraint or buttress (Hungr and Amann, 2011) may be different from the influence of rock bridges occurring in the toe of a slope for a non-daylighting wedge (Styles, 2009; Yan, 2008). Both pre-failure slope deformations and dynamics of the final failure mechanism may be influenced by the 3-D geometry and location of the rock bridge.

Trace mapping of out-of-plane rock bridges is subject to potentially random variation in interpreted geometry, depending on the size, location and orientation of the sampling line or window (Figure 2 - 21).
Figure 2-21: Conceptual illustration showing the influence of sampling plane location on observed discontinuity traces and rock bridge geometry.
2.4. Numerical Methods for Simulating Slopes with Non-persistent Discontinuities

Improvements in computing technology since the late 1970s have facilitated the development of increasingly sophisticated slope stability codes. This section provides a brief introduction of the most common methods used for slope stability assessment involving step-path failure geometry, non-persistent discontinuities and intact rock bridges including: probabilistic step-path simulators; continuum methods; discontinuum methods; and advanced numerical methods such as hybrid FEM-DEM and fracture mechanics codes.

Widespread adoption of advanced numerical codes in engineering practice has been limited, due in part to computational requirements, model run-times and the need for specialist training. Another barrier to the widespread adoption of advanced numerical methods, however, is the lack of reliable data concerning discontinuity persistence and intact rock bridge content in rock slopes. Without reliable input data for discontinuity persistence, intact rock bridge content, strength and stiffness parameters for rock mass and discontinuities, the output of advanced numerical models is often unreliable and limited to conceptual “what if” scenarios.

2.4.1. Probabilistic Step-Path Simulation

Jennings’ (1970) modified limit equilibrium approach was followed in the late 1970s and 1980s by the development of probabilistic slope stability codes and step-path simulators. The probabilistic method is an extension of limit equilibrium analysis, based on evaluating many iterations of potential failure paths and combinations of material parameters using multiple random samples by Monte Carlo simulation. Each combination of failure path geometry and rock mass parameters either fails or reaches stable equilibrium; the resultant distribution of outcomes reflects the relative probability of failure (POF).

McMahon (1979) developed the first step-path simulator, STPSIM, as part of the pit slope design study for Bougainville Copper’s Panguna mine in Papua New Guinea (McMahon, 1979; and Read and Lye, 1984). Baczynski (2000) modified and re-wrote the code as STEPSIM4 as part of the 1997-99 final pit wall optimisation study at Ok Tedi.
mine (Little, 1999). More recently the STEPSIM4 code has been applied to an integrated study of slope deformation using microseismic monitoring results, finite element analysis and step-path predictions to better understand the processes of rock bridge failure in slopes (Dight and Baczynski, 2009). The code has also been subject to recent revision and modifications (Baczynski, 2008). Other researchers to apply probabilistic step-path methods include Call and Nicholas (1978), Glynn (1979), and Ristau (1994). Other step-path analysis codes include Bstepp (Miller et al., 2007); and SLOPESIM (Einstein et al., 1983).

Although probabilistic slope codes are able to simulate the effects of uncertain failure paths and variable material properties, they do not explicitly represent the process of brittle failure of intact rock bridges and deformation of the slope. Moreover, results are extremely sensitive to discontinuity persistence and intact rock bridge content, because the shear strength of intact rock can be orders of magnitude stronger than that of discontinuities (Kemeny, 2005).

2.4.2. Continuum Methods: Finite Element and Finite Difference

All continuum methods involve the discretization of the model domain into a mesh of connected elements. Popular continuum methods for modelling rock masses include implicit-solution Finite Element Method (FEM) codes, such as Phase² (Rocscience, 2011a) and explicit-solution Finite Difference Method (FDM) codes, such as FLAC (Fast Lagrangian Analysis of Continua, Itasca 2012). Implicit codes have a solution process that requires convergence to equilibrium, such that large post-failure strain problems cannot be directly simulated. Explicit codes, in contrast, use a time-marching solution of the full equations of motion for each element, according to Newton’s second law, allowing simulation of continuum failure and large post-peak strains (Itasca, 2012).

Conventional continuum codes are well-suited to engineering problems where rock mass behaviour can be approximated by a homogenous continuum. For example, Hoek and Guevara (2009) used the FEM code Phase² in order to help assess support requirements for a deep tunnel in highly stressed, weak rock mass, where the primary engineering challenge arises from squeezing ground. With the addition of a bi-linear rock mass failure criteria, Diederichs (2007) and others showed that FEM codes may also be
useful in predicting the depth of stress-induced brittle fracturing around deep excavations in massive, strong rock (Diederichs, 2007). In modelling slope failure, the Shear Strength Reduction (SSR) method can simulate incipient rock bridge failure and block movement up to the initiation of slope instability (Sturzenegger and Stead, 2012; Franz, 2009; Hammah et al., 2008; Figure 2 - 22).

With modification, finite element codes can be also used to directly model brittle failure. The modified finite element codes RFPA/RPFA³D (Rock Failure Process Analysis) are able to simulate brittle failure in rock by (1) introducing material heterogeneity by using a Weibull or Normal distribution to randomly assign different strength properties to individual finite elements; and (2) automatically reducing the elastic modulus of failed elements to a near-zero value to simulate crack nucleation or propagation. The RFPA codes are also able to simulate the rate of acoustic emission events associated with brittle cracking, by recording the event rate as failed finite element events (Tang and Tang, 2011; Tang et al., 2004; Tang, 1997).
Figure 2-22: Application of the finite element method code Phase\textsuperscript{2} in simulating failure of intact rock bridges from (A) a study of the Cadia Hill Open Pit, NSW Australia by Franz (2009); (B) in an example step-path failure, by Hammah et al. (2008); and (C) in an investigation of the Palliser rock slide near Kananaskis Alberta, by Sturzenegger and Stead (2012).
2.4.3. **Discontinuum Methods**

Discontinuum methods simulate a rock mass as an assembly of discrete blocks or particles that are separated by block contacts or discrete discontinuities. The ability to explicitly simulate discontinuity-controlled deformation, block separation, and large displacements makes discontinuum methods well suited to characterising the response of jointed rock masses where pre-existing discontinuities are expected to be the overriding influence on stability (Barton, 2012).

2.4.3.1. **Discrete Element Method**

The discrete element method is characterised by the ability to simulate displacement and rotation of discrete blocks, and by the automatic detection of new block contacts as calculation progresses (Itasca, 2010a).

The distinct element code *UDEC* (*Universal Distinct Element Code*, Itasca, 2010) represents a sub-set of the discrete element method, proposed by Cundall and Strack (1979). The features that distinguish *UDEC* as a distinct element code are the (1) the inclusion of deformable contacts and (2) the use of explicit time-domain solutions to the equations of motion for each model node, based on Newton’s second law (i.e. *Force = Mass x Acceleration*) (Fournier, 2008). The explicit solution scheme allows for stable calculation of large displacements along discontinuities.

Blocks may be specified as either deformable or rigid. Two common constitutive models applied to deformable blocks include idealized linear elastic behaviour (i.e. infinitely elastic with no failure) and Mohr-Coulomb plasticity; other constitutive models are also available, and the built-in programming language FISH can be used to create customized non-linear constitutive laws. Internal block elements may fail either in tension or in shear, when local stresses exceed the shear strength criteria. Contacts are commonly assigned a joint area or point contact Mohr-Coulomb slip criterion (Itasca, 2010b). As with internal block elements, contacts may fail by either shear slip or tensile opening when local contact stresses exceed the contact shear strength.

Although *UDEC* cannot explicitly simulate the initiation, propagation, and coalescence of new cracks, the built-in Voronoi tessellation function can be used to simulate brittle...
fracture behaviour by pre-generating a network of random polygons which represent potential paths for brittle fracture (Figure 2 - 23). Opening and shearing of Voronoi contacts has been demonstrated capable of reproducing brittle fracture behaviour in laboratory test-scale specimens of intact and jointed rock (Alzo'ubi et al., 2010, 2007; Kazerani and Zhao, 2010; Yan, 2008; Alzo'ubi, 2009; Christianson et al., 2006).

![Image of Voronoi tessellation](image)

**Figure 2 - 23:** Voronoi tessellation can be used to simulate (A) zones of equivalent rock mass for large models; or (B) individual mineral grains or domains of intact rock for small models of intact rock samples; pre-existing discontinuities are shown in red (Modified after Christianson et al., 2006).

If blocks are defined as linear elastic or rigid material, then rock mass strength, deformability and failure mechanisms are controlled by the strength and stiffness of the Voronoi contacts. Five contact parameters are required: normal stiffness, shear stiffness, cohesion, friction angle and tensile strength.
2.4.3.2. Synthetic Rock Mass Method and Bonded Particle Method

The Synthetic Rock Mass (SRM) method is an extension of the distinct element method, where a Discrete Fracture Network (DFN) is inserted into a matrix of bonded particles representing intact rock. By explicitly including intact rock and discontinuities, SRM models can simulate rock mass failure as an emergent behaviour, without the need for including explicit a priori failure surfaces (Read and Stacey, 2009). The Itasca codes PFC/PFC3D use bonded circles or spheres to represent intact rock, and a smooth joint model to represent pre-existing discontinuities, which allows particles to overlap during shear displacement, allowing for perfectly planar discontinuities (Mas Ivars et al., 2008).

SRM formulations based on representation of intact rock by uniform assemblies of bonded spheres or discs may tend to show dominance of tensile contact failure over shear failure (Diederichs, 1999). As a consequence, bonded-disc or sphere models may exhibit unrealistically high ratios of tensile strength to UCS ($\sigma_T / \text{UCS}$), in the order of 25% (Kazerani and Zhao, 2010). In contrast, SRM formulations using angular Voronoi polygons to represent rock produce an inherent particle interlocking effect, which causes shear failure to dominate over tensile failure (Kazerani and Zhao, 2010). However, the recently proposed flat-jointed contact model for PFC (Potyondy, 2012) may help to increase particle interlocking between bonded discs, and thus yield more realistic $\sigma_T / \text{UCS}$ ratios in future work.

Other recent SRM developments include the application of the open-source 3-D bonded particle code YADE Open DEM to modelling intact rock bridge failure in the 1991 Randa rockslide and other rock slopes (Sholtès and Donzé, 2012). Another SRM code using a lattice-spring formulation is Slope Model (Itasca 2010b), which has been developed as part of the CSIRO Large Open Pit project.
2.4.4. Fracture Mechanics Approach and the Boundary Element Method (BEM)

Fracture mechanics codes are based on the early work of Griffith (1921, 1924), who explained that the disparity between the theoretical atomic-scale tensile strength of brittle materials, and the much lower strengths observed in laboratory strength tests, was due to the presence of small pre-existing flaws or microcracks. Stress magnification around the tips of pre-existing Griffith Flaws results in failure at stresses much lower than the theoretical atomic-scale lattice strength.

Griffith explained that the creation of new crack surface area during brittle failure absorbs strain energy that can accumulate by external loading, and that the crack surface area is proportional to the energy absorbed. Further developments in fracture mechanics led to the adoption of the fracture toughness parameters $K_{IC}$, $K_{IIC}$, and $K_{IIIC}$ which describe the critical stress intensity, with units of $\text{stress} \sqrt{\text{crack length}}$, necessary to propagate a crack in Mode I, II, or III failure, as described in Section 2.2 previously.

Gudmundsson et al. (2010) summarised that fracture toughness is in turn related to the material toughness, $G_{TOTAL}$ which describes, in units of $\text{energy} / \text{unit area}$, the amount of strain energy released in mixed-mode fracture propagation:

$$G_{TOTAL} = G_i + G_{ii} + G_{iii} = \frac{(1-v^2)K_{IC}^2}{E} + \frac{(1-v^2)K_{IIC}^2}{E} + \frac{(1+v)K_{IIIC}^2}{E}$$

Where $G_i$, $G_{ii}$, and $G_{iii}$ correspond to material the Mode I, II, and III material toughness, $E$ is Young’s modulus, and $v$ is Poisson’s ratio.

The Boundary Element Method (BEM) is a numerical technique that only requires the boundaries of the problem domain to be discretized; the interior of the model is not meshed as in FEM models. The displacement-discontinuity method (DDM) is a fracture mechanics-based sub-set of the boundary element method. DDM models are able to simulate the propagation of pre-existing discontinuities according to the stress intensity factor approach (i.e. $K_{IC}$, $K_{IIC}$), however they cannot simulate the nucleation of new fractures without the inclusion of pre-existing flaws (Yan, 2008, Whittaker et al., 1992).
Studies using fracture mechanics based codes to investigate rock mass failure in slopes and laboratory-scale simulations have been undertaken by many researchers including: Yan, 2008; Castelli et al., 2007; Shen and Rinne, 2007; Adey and Pusch, 1999; Singh and Sun, 1990, 1989; Scavia, 1995, 1990; Scavia and Castelli, 1996; and Castelli, 2000.

2.4.5. **Hybrid FEM-DEM Methods**

Hybrid FEM-DEM codes can simulate progressive crack growth and the division of a continuum in discrete blocks. Using an energy release rate (*i.e.* \( G \)) criterion, FEM-DEM models can also simulate the nucleation of new cracks.

In the FEM-DEM code *ELFEN* (Rockfield, 2009) a DFN can be inserted into an initial continuum model in order to investigate the influence of pre-existing structure on the development of brittle failure. Researchers have investigated the FEM-DEM-DFN approach to study the response of underground mine pillars (Elmo, 2006); surface subsidence induced by block cave mining (Vyazmensky et al., 2010a,b); and progressive rock slope failure involving brittle failure of intact rock bridges (Elmo et al., 2011, 2009; Yan, 2008; Eberhardt et al., 2004). The solution process can use two potential methods: (1) a continuum-based method, with failure flagging of finite elements to simulate potential fracture growth; or (2) explicit simulation of fracture initiation and propagation with generation of new intra-element cracks. For each time-step in the solution process, the stresses at each mesh element are evaluated with respect to a material failure criterion based on energy release rate. If stresses are sufficient to generate new cracks or to extend pre-existing cracks, then new contacts are created, and new finite element mesh is generated if necessary.

Another FEM-DEM code, *Y2D*, was developed by Munjiza (2004) and Munjiza et al. (2004, 1995). Mahabadi et al. (2010a) developed a graphical user interface, *Y-GUI*, for *Y2D*, and research applying the code to rock mechanics applications is ongoing (Mahabadi et al., 2010b).
3. Site Selection and Methodology

3.1. Introduction

This chapter briefly introduces the three open pit mines selected for field study, and presents information on the survey methods and post-processing techniques used for geotechnical slope characterisation. Additional observations from a pilot investigation at a natural rock slope in Squamish, BC are also presented. Preliminary interpretation of naturally occurring discontinuity-intact rock fracture relationships from the Squamish survey were used to help formulate subsequent strategies for characterising discontinuity persistence and intact rock bridges in open pit slopes.

The three open pit mines were chosen for field investigation in order to develop an improved understanding of discontinuity persistence and intact rock bridges and help increase confidence in slope stability. At each site, ground-based remote sensing surveys were undertaken to construct 3-D terrain models of the pit slopes (Figure 3 - 1). Rock mass conditions are different at each site; consequently, field investigations, post-processing and analysis methods were adapted according to local geology, blasting-induced damage conditions, and practical limitations including site access constraints, lighting conditions, and atmospheric dust.
Figure 3 - 1: Digital terrain models of (a) Jwaneng East Wall; (b) Diavik A418 Pit; and (c) Highland Valley Copper, Valley Pit Upper West Wall.
3.1.1. **Jwaneng Mine, Botswana**

Jwaneng Diamond Mine is in southern Botswana, about 130 km west of the national capital, Gabarone. The mine is owned and operated by Debswana Diamond Company, a joint venture between DeBeers and the government of Botswana. Operations comprise one open pit which intersects four diamond-bearing kimberlite pipes (Figure 3 - 2). The long axis of the pit is oriented Northeast-Southwest and is 2.4 km long; the short axis is 1.8 km wide, striking Northwest-Southeast. The pit depth as of 2011 was 360 m, and the currently underway Cut 8 expansion will enlarge the pit footprint and increase the overall depth to 622 m, extending the mine life to 2024 (Tunono et al., 2011).

![Figure 3 - 2: Location of Jwaneng mine open pit, Botswana (Google Maps).](image)

This thesis focuses on an investigation of the East wall of the pit, which is comprised of foliated meta-sedimentary rocks of the Paleo-Proterozoic age Pretoria Group (Barnett, 2009, by permission). The foliation dips between 10° and 40° toward Northwest, daylighting out of the East wall. Potential instabilities in the East wall may be characterised by susceptibility to planar sliding on persistent foliation, with lateral and rear release surface formed by orthogonal tension joints that may include a combination of pre-existing tectonic structures and blasting-induced damage.
3.1.2. **Diavik Mine, NWT Canada**

Diavik Diamond Mine is in Northwest Territory, Canada, about 300 km northeast of the territory capital, Yellowknife (Figure 3-3). The mine is a joint venture between Harry Winston Diamond Corporation and Diavik Diamond Mines Incorporated, which is a subsidiary of Rio Tinto Group. Operations comprise two open pits and underground sub-level caves. The A154 pit intersects two diamond-bearing kimberlite pipes, and the A418 pit intersects one pipe. The open pits are situated on the formerly submerged lakebed of Lac de Gras, enclosed behind two water-tight rock fill dikes with jet-grouted concrete cores.

![Figure 3-3: Location of Diavik mine A154 and A418 pits (aerial photograph taken prior to excavation of A418 pit), NWT Canada (Google Maps).](image)

The property is in the Slave Structural Province of the Precambrian-age Canadian shield. The country rock at the mine site is a mix of deformed Archean granite and meta-turbidites that have been intruded by Proterozoic diabase dyke swarms and pegmatite (Roscoe and Postle, 2005). The pit walls comprise mostly strong, moderately fractured granite with UCS greater than 100 MPa (Moffitt et al., 2007). This thesis focuses on investigation of the A418 pit, and in particular its Southeast wall, which is situated behind a 900-m long section of the dike. At the time of field investigation in July 2011, the A418 pit was 220 m deep, with individual bench heights of 30 m.
3.1.3. Highland Valley Copper, BC Canada

Highland Valley Copper is in southern British Columbia, Canada, about 200 km northeast of Vancouver (Figure 3 - 4). The mine is majority owned and operated by Teck Resources Limited. Operations currently comprise three open pits: Valley, Lornex, and Highmont.

Figure 3 - 4: Location of Highland Valley Copper mine, BC Canada (Google Maps).

This thesis focuses on an investigation of the Upper West Wall of the Valley Pit, which is comprised of altered and silicified granodiorite of the Guichon Creek batholith (Casselman et al., 1995). The Upper West Wall is intersected by the steeply-dipping Yellow fault, which trends Northeast-Southwest across the Valley pit. At the time of the field investigation in August 2011, the Upper West Wall was 140 m high, with bench heights of 15 m.

3.2. Field Methods

In field studies of natural landslides or rockfalls, the critical slip surface or detachment niche can often be studied explicitly, allowing for a posteriori interpretation of failure
kinematics and direct measurement of pre-existing discontinuities and interpreted brittle fracture through intact rock bridges (e.g. Sturzengger and Stead, 2012; Paronuzzi and Serafini, 2006; Frayssines and Hantz, 2006). In contrast, open pit bench face maps sample potentially non-critical surfaces which may not directly relate to deep-seated failure surfaces (Figure 3 - 5).

**Figure 3 - 5:** Conceptual illustration of a potential failure surface in red; bench faces sample non-critical cross-sections through the slope, and do not reveal the full 3-D structure of a potential failure surface.

Although a single bench face map cannot fully capture 3-D rock mass structure, correlation across multiple benches at different elevations and orientations can help to increase confidence in 3-D discontinuity-rock bridge configurations. Complementing bench face maps with geophysical survey data or borehole logs can further help to characterise the interior of the rock mass, behind the bench face.

Mapping methods for this thesis were based on traditional field techniques for discontinuity survey, with focus on recording pre-existing 2-D discontinuity trace lengths, dip and dip direction, and surface condition. Modifications were incorporated to add information wherever possible on: brittle fracture features, which may indicate previously failed rock bridges; discontinuity termination style; and existing intact rock bridges (Figure 3 - 6).
Figure 3 - 6: (A) Example of discontinuity and rock-bridge geometry parameters recorded in previous studies (Modified after Yan et al., 2007; and Gehle and Kutter, 2003); and (B) potential small-scale (~3 cm) intact rock bridge observed between two patches of a discontinuity in the field.

Remote sensing surveys were used to help improve 3-D characterisation of discontinuities and kinematic release surfaces, and also to help characterise step path geometry and intact rock fracture patterns wherever possible.

3.2.1. Terrestrial Remote Sensing

Remote sensing allows for accurate survey of inaccessible areas on high slopes, and also yields a permanent electronic record of bench conditions, which can be re-analysed
repeatedly without disrupting mine operations (Lee, 2011; Sturzenegger et al., 2011). As an added benefit, data acquisition for remote sensing can be completed much faster than conventional survey methods over equivalent area. However, project planning must also account for time needed for computer-based post-processing, mapping, and analysis of digital terrain models.

Although remote sensing surveys can facilitate mapping of physically inaccessible slopes, digital discontinuity measurements are subject to sources of bias and uncertainty. Some of the most important sources of error and uncertainty relate to censoring bias and truncation, which influence measurement of discontinuity trace lengths; and also occlusion and orientation bias, which are influenced by the orientation of the outcrop or rock face and the line-of-sight direction of the camera or laser scanner (see Chapter 2, Section 2.3.2).

For this study, photogrammetry and LiDAR surveys cover larger areas than the traditional field window maps, up to the full height of overall pit slopes. Both photogrammetry and LiDAR surveys were carried out at varied resolutions, in order to investigate the influence of scale bias. The following sections briefly describe some advantages and limitations of photogrammetry and LiDAR for field characterisation of open pit slopes. Some advantages and limitations of photogrammetry for rock mass characterisation are listed in Table 3 - 1, and selected advantages and limitations of LiDAR survey for rock mass characterisation are listed in Table 3 - 2.
Table 3-1: Selected advantages and limitations of photogrammetry survey for rock mass characterisation (Based on Lee, 2011; Sturzenegger, 2010; and Sturzenegger and Stead, 2009a,b).

<table>
<thead>
<tr>
<th>Advantages</th>
<th>Limitations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Long range (i.e. 2 km) is possible with a telephoto lens of ( f = 400 \text{ mm or more} )</td>
<td>Sensitive to weather &amp; lighting conditions</td>
</tr>
<tr>
<td></td>
<td>• Changes in clouds &amp; sunlight can disrupt image matching process and decrease image accuracy</td>
</tr>
<tr>
<td>Camera equipment is easily portable and cheap relative to some alternate remote sensing systems</td>
<td>Discontinuities easier to map in blocky rock mass with high relief:</td>
</tr>
<tr>
<td></td>
<td>• Limited value in low-GSI, disintegrated rock mass with little relief</td>
</tr>
<tr>
<td>Unmanned Aerial Vehicle (UAV) options are available for aerial survey¹</td>
<td>Subject to orientation, scale bias, occlusion, truncation</td>
</tr>
<tr>
<td></td>
<td>• Limit orientation bias and occlusion by shooting from varied positions and line-of-sight angles with respect to the rock face</td>
</tr>
<tr>
<td></td>
<td>• Limit scale bias and truncation by selecting high resolution and small cut-off-length of discontinuities, if possible</td>
</tr>
<tr>
<td>Creates a permanent electronic record of 3-D bench face</td>
<td>Not a stand-alone substitute for conventional mapping</td>
</tr>
<tr>
<td></td>
<td>• Field verification is important in order to characterise rock hardness, state of weathering or alteration</td>
</tr>
<tr>
<td>Less time in-field compared with conventional survey techniques</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Minimise disruption to mine operations</td>
</tr>
<tr>
<td></td>
<td>• Less person-hours exposed to slope-related hazards</td>
</tr>
<tr>
<td>Allows mapping of high slopes or dangerous areas that are physically inaccessible</td>
<td></td>
</tr>
<tr>
<td>Has potential for characterising outcrop-scale or larger roughness (Haneberg, 2007).</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Advantages</th>
<th>Limitations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Long range (up to 4 km) is possible with recent scanner models (e.g. Riegl, 2012a)</td>
<td>High cost of LiDAR equipment relative to camera for photogrammetry</td>
</tr>
<tr>
<td>3-D data can be derived from one single scan at a single location</td>
<td>LiDAR apparatus and tripod are less portable than camera equipment for photogrammetry</td>
</tr>
<tr>
<td>Mobile options are available:</td>
<td>Subject to orientation bias, occlusion, scale bias, truncation and censoring</td>
</tr>
<tr>
<td>• Vehicle-mounted options (e.g. Riegl 2012b)</td>
<td>• Multiple scanning locations help to limit orientation bias and occlusion (Lato et al. 2009)</td>
</tr>
<tr>
<td>Unmanned Aerial Vehicle (UAV) options are becoming increasingly available (Eisenbeiss, 2011, 2009)</td>
<td>Discontinuities easier to map in blocky rock mass with high relief</td>
</tr>
<tr>
<td>• Minimise disruption to mine operations</td>
<td>• Limited value in low-GSI, disintegrated rock mass with little relief</td>
</tr>
<tr>
<td>• Less person-hours exposed to slope-related hazards</td>
<td>Sensitive to seepage and vegetation</td>
</tr>
<tr>
<td>Less sensitive to sunlight/cloud conditions than photogrammetry</td>
<td>• Water seepage and vegetation reduces reflectivity and can cause “holes” or contrasts in intensity values in the point cloud; However, this may provide a potential advantage as a method for mapping groundwater seepage and vegetation coverage</td>
</tr>
<tr>
<td>• Rain, however, can obstruct beam path and affect rock reflectivity</td>
<td></td>
</tr>
<tr>
<td>Allows mapping of high slopes or dangerous areas that are physically inaccessible</td>
<td>Can characterise discontinuity roughness:</td>
</tr>
<tr>
<td></td>
<td>• In laboratory settings (Tatone and Grasselli, 2010; Grasselli, 2001)</td>
</tr>
<tr>
<td></td>
<td>• In situ (Fekete et al., 2010; Tatone, 2009)</td>
</tr>
</tbody>
</table>
3.2.2. **Window Mapping**

Traditional window maps measure projected discontinuity traces on a rectangular or circular plane. The focus of window mapping is on measuring discontinuity properties including trace length, orientation, terminations, and discontinuity surface conditions. Additional information including lithology, estimated intact rock strength, weathering and groundwater conditions are also recorded. The combined assessment of discontinuities and intact rock properties can be used to assign rock mass quality measurements using systems like the quantified GSI suggested by Cai et al. (2004, 2007) (Figure 3 - 7).

![Quantified GSI](image)

**Figure 3 - 7: Quantified GSI proposed by Cai et al. (2004, 2007, by permission).**
Trace maps can also be used to calculate a “termination index” $T_i$ (ISRM, 1978; Priest, 1993) which compares the proportion of discontinuity tips that terminate in intact rock, to those which terminate against other joints, are censored by the sampling window, or are obscured by vegetation or other obstructions:

$$T_i = \frac{\sum \text{terminations in intact rock}}{\sum \text{all terminations}} = \frac{\sum \text{terminations in intact rock}}{2 \times \text{Number of discontinuities}}$$

Priest (1993) proposed that larger values for $T_i$ indicate a higher proportion of discontinuities terminating in intact rock, and thus a high proportion of intact rock bridges. Conversely, a rock mass comprised of fully-persistent joints and completely formed, discrete blocks would have a termination index of 0.

In Figure 3 - 8, for example, 15 discontinuity tips terminate within intact rock. In total, 16 discontinuities are within the sampling window. The termination index is therefore:

$$T_i = \frac{15}{16 \times 2} \approx 47\%$$

**Figure 3 - 8: Idealized window map with terminations in intact rock.**

Researchers have recently suggested alternative window shapes, in some cases specifically for mapping of remote sensing-derived terrain models. Circular windows have been investigated by researchers including Zhang and Einstein (1998) and Mauldon et al. (2001). Sturzenegger et al. (2011) proposed a circular, non-planar “topographic window” which conforms to the outcrop surface may be useful in limiting orientation bias and under-estimation of the trace intensity, $P_2$, that may occur with planar windows (Figure 3 - 9).
Kulatilake and Wu (1984a,b) and Wu et al. (2011) proposed that rectangular sampling windows can produce more accurate estimates of trace length distributions and fracture density than circular windows. Wu et al. (2011) compared predicted trace length and density values using variably sized circular and rectangular windows on an outcrop in Yingxiu, China. Their results illustrated that in some circumstances circular windows may over or under-estimate true discontinuity trace length and density (measured as number of discontinuity trace centres per unit area), with larger error than equivalently dimensioned rectangular windows. In this study, window maps were configured as approximately rectangular, except where sloping ground resulted in approximately parallelogram windows (Figure 3 - 10).
As a supplement to field mapping, digital trace maps were used to derive areal intensity $P_{21}$ values for pre-existing discontinuities, and also equivalent $R_{21}$ intensity values, calculated as the total length of inferred intact rock bridge traces per square metre of window surface area.

The same aerial intensity procedure was used to calculate a $B_{21}$ intensity value for blast induced damage, calculated as the total length of blast-induced fractures including cross-hole pre-splitting fractures (Figure 3 - 11) per square metre of window surface area. Blast-induced fractures were identified as radial fractures and non-systematic crushing around boreholes, and conchoidal or irregular brittle extensions of pre-existing planar discontinuities.

![Blast-induced horizontal cross-hole fractures](image)

**Figure 3 - 11:** Example of blast-induced fractures associated with pre-splitting in massive granite at Diavik diamond mine.

Field window maps were supplemented with notes and sketches characterising brittle fracture features including step surfaces, niches and arches (Bahat et al., 2005); and also blast-induced spalling or radial fractures (Hagan, 1982). Where they could be clearly distinguished, potential in-plane intact rock bridges were noted in the field with
respect to specific discontinuities or discontinuity sets. Figure 3 - 12 presents a transcribed example of selected field notes taken at a bench mapping location at Diavik.

For this thesis, all window map measurements of GSI should be considered local assessments of rock mass quality, which consider the local extent of blast-induced damage to the rock mass within each window map. Thus, if GSI measurements from window mapping done in this thesis are subsequently used to derive Hoek-Brown shear strength parameters for use in slope stability assessment, the resultant parameters will reflect the influence of blasting in reducing rock mass quality locally; additional application of a blast damage factor $D$ (Hoek, 2012) would effectively be “double-dipping” and may result in underestimation of rock mass strength.

Field data collected for this thesis was transcribed into Excel (Microsoft, 2007), with fields recording discontinuity information; date and time of survey; weather conditions; and location (i.e. bench elevation and approximate location within the pit). Although Excel may not be ideally suited to handling larger volumes of data (i.e. managing life-of-mine geotechnical program data), it is well suited to handling smaller scale investigations, and provides built-in analysis functions capable of carrying out linear regression and statistical analysis of univariate datasets.
3.3. Post-Processing

Post-processing and mapping of photogrammetry surveys was carried out with the 3DM Mine Mapping Suite (Adam Technology, 2010), and LiDAR point clouds were aligned and mapped using the PolyWorks software suite (InnovMetric, 2006). ArcMap (Esri, 2012) was used to create a slope steepness map based on a LiDAR-derived digital terrain model of the Chief rock slope (Section 7.4.2.1).

Digital 2-D trace mapping was carried out using the image editing software Illustrator (Adobe, 2012a) and Photoshop (Adobe, 2012b). The resultant trace maps were analysed using the freeware raster image analysis software ImageJ (Rasband, 2008). Methods for calculating discontinuity trace lengths and inferred rock bridge trace lengths are based on the Digital Rock Mass Rating (DRMR) method proposed by Monte (2004), with extensions to consider:

1. Two methods for deriving fracture intensity $P_{21}$

   **Fracture Intensity Delineation Method 1:** In $P_{21}$ calculation Method 1, all fractures are included in the summation of trace lengths, including irregular blast-induced brittle fractures.

   **Fracture Intensity Delineation Method 2:** In $P_{21}$ calculation Method 2, only discontinuities interpreted as pre-existing before blasting were included in the trace length summation.

2. Two methods for deriving intact rock bridge trace intensity $R_{21}$, with inferred rock bridge geometries based on observations of naturally-occurring brittle fracture features:

   **Rock Bridge Delineation Method 1:** In the first protocol, rock bridges are traced only between pre-existing discontinuities of similar sets, preferentially as parallel in-plane rock bridges, or as perpendicular out-of-plane rock bridges if pre-existing discontinuities are not coplanar.

   **Rock Bridge Delineation Method 2:** In the second protocol, rock bridges are traced between all adjacent discontinuities, including blast-induced fractures,
dissecting the window into an assembly of discrete blocks with continuous perimeters.

3. A parameter for blast damage trace intensity, $B_{21}$, calculated as the sum length of traces of blast-induced fractures per unit surface area, including irregularly-shaped, brittle intact rock fractures, cross-hole pre-splitting fractures, and radial fractures and crushing around boreholes

All photographs are subject to lens distortions which cause error (i.e. the location of a pixel in the image is offset from the real-world location of the corresponding point). Typically, distortion becomes more severe with increasing radial distance away from the centre of the image.

Close-range photographs for 2-D trace maps were taken with a Canon 5D Mark II, with a fixed focal length (prime) lens with $f = 20$ mm. The dimensions of the camera’s image sensor are 36 mm x 24 mm, yielding a diagonal distance of approximately 43 mm. Because the lens focal length is small compared to the diagonal dimension of the image sensor, the lens is considered to be wide-angle. As a consequence, close-range bench face photographs are subject to horizontal foreshortening towards the top of the image and horizontal extension towards the bottom of the image (Figure 3 - 13).

To account for distortion in 2-D trace maps, an approximate correction was applied by adjusting the vertical perspective of each image. Each image was manually adjusted so that borehole half-barrels appear approximately vertical and parallel throughout the entire image. Figure 3 - 14 demonstrates that the perspective correction introduces horizontal length extension of up to 25% at the top of bench face photographs, and horizontal length compression of up to 30% at the bottom of bench face photographs.
Figure 3 - 13: (A) Example of uncorrected bench map with “fish-eye” distortion due to close shooting distance with short focal length lens ($f = 20\text{mm}$) and (B) Modified bench map, corrected for distortion. Frame is about 30 m high.
Figure 3 - 14: Approximate distortion correction using the Lens Correction feature.
3.4. Pilot Study at Stawamus Chief

3.4.1. Geological Background

Stawamus Chief is a Cretaceous-age (Friedman and Armstrong, 1990) granitic monolith near Squamish, BC. Monger and McNicoll (1993) proposed that the Chief is an exposed protuberance of the partially-exhumed, 120 km-long Howe Sound Batholith, which intrudes Early Cretaceous Gambier Group volcanic rocks of the southern Coast Plutonic Complex. The monolith was exposed by orogenic uplift of the Coast Mountains, and was covered by up to 1.3 km of ice at the last glacial maximum approximately 20 ka BP (Turner et al., 2010). Glacial polishing features occur on the exposed peaks of the Chief and on granitic outcrops along Highway 1 near the base of the mountain (Figure 3 - 15).

![Glacially polished outcrop at viewpoint pullout near the Chief](image)

Figure 3 - 15: Glacially polished outcrop at viewpoint pullout near the Chief (Turner et al., 2010, by permission)

3.4.2. Survey Setup

A LiDAR survey of part of the western face of the Chief was undertaken with an Optech ILRIS-3D scanner, from a range of approximately 400 m, based on an automatically calculated, average line-of-site distance from the scanner to the rock face. The point cloud covers a rectangular region of interest (ROI) 150 m high and 300 m wide, with a
ground pixel size ranging from 10 cm to 20 cm depending on the local line-of-sight distance from the rock face to the scanner and the beam angle of incidence (Figure 3 - 16, Figure 3 - 17). Orientation of the survey was achieved by manual rotation of the point cloud according to the trend and plunge of the scanner line-of-sight. Over the ROI, the cliff face is steeply dipping towards northwest; the average dip and dip direction of the face is about 77° / 294°.

A supplementary close-range photogrammetry survey was also undertaken on a nearby glacially-polished outcrop, using a fixed focal length (prime) lens with $f = 20$ mm. The close-range photogrammetry survey was intended to help characterise the geometry of pre-existing discontinuities and recent blasting-induced brittle fractures on a smaller scale than the Chief monolith. Comparison of discontinuities and intact rock fractures between the small-scale outcrop and the large-scale features on the Chief may provide insight into scale effects on discontinuity persistence and intact rock bridge geometry for subsequent guidance in large open pit slope surveys.

### 3.4.3. Preliminary Rock Mass Assessment

The western face of the Chief comprises mostly massive granite with fresh to slightly weathered discontinuity surfaces. Evaluated over the scale of the LiDAR scan, the rock mass quality is excellent; conservative GSI estimates could range from 80 to 90. Near the centre of the surveyed area, a black basaltic dyke intrudes the host granite. The dyke is steeply dipping (~80°), ranging from about 2 m to 4 m thick, and has been dated to about 30 Ma BP (Turner et al., 2000).

Based on evaluation of the LiDAR scan, the rock mass within the dyke is blockier and more intensely fractured than the host granite, with approximately cubic blocks and typical block size ranging from approximately 0.5 - 2 m$^3$. The increased blockiness within the dyke is associated with a change in surface texture that can be readily distinguished on the LiDAR point cloud (Figure 3 - 17). The GSI within the black dyke ranges from about 70 to 80, with blocky to massive block size.

Blank patches or “holes” in the LiDAR point cloud correspond to areas of vegetation or seepage; the association can be confirmed by visual inspection of the site photograph.
(Figure 3 - 16) which shows trees growing along a persistent discontinuity in the upper right of the ROI, and vertical dark grey streaks where groundwater seepage is present.

The discontinuity structure of the Chief’s western face is dominated by tensile fracture features; no obvious macroscopic shear features occur in the region of interest. High to very high persistence (i.e. persistence typically greater than 10 m and frequently greater than 20 m; ISRM, 1978), surface-parallel exfoliation joints are ubiquitous, with associated orthogonal brittle intact rock fractures forming lateral- and rear-release surfaces for slab failures. Based on measurements of step surface heights on the cliff face, exfoliation slabs range from about 10 cm to 5 m thick. Smaller, horizontally-oriented brittle fractures may be associated with gravity-driven failure of exfoliation slabs in direct tensile loading (e.g. Bahat et al., 2005). Conversely, larger brittle fractures may be associated with glacial scour shearing off large slabs during down-valley ice flow in the Pleistocene (Turner et al., 2010).
Figure 3 - 16: Western face of the Stawamus Chief; camera looking east. Dashed rectangle shows region of interest for LiDAR scan.
Figure 3 - 17: LiDAR point cloud of region of interest; blank spots indicate areas of poor reflectance due to vegetation coverage (trees in top right of ROI) and water seepage trails on the face.
3.4.4. Fracture Observations

The following section discusses the morphology of discontinuity surfaces and brittle intact rock fractures at the Chief, and implications of the observed discontinuity geometry and persistence for the role of intact rock bridges. Section 7.6 uses the observations from the Chief to help suggest preliminary methods for discontinuity persistence and rock bridge characterization at the open pit study sites.

3.4.4.1. Exfoliation Jointing

Exfoliation and sheet jointing are often used interchangeably to describe persistent, approximately planar or gently curved tension joints that occur in concentric layers, usually sub-parallel to topography. However, in a recent review of studies into sheet structure and exfoliation, Bahat et al. (2005) distinguished that sheet joints are characterised as sub-horizontal “pre-uplift” discontinuities, and are thus older than modern topography. Conversely, exfoliation joints tend to occur parallel to modern topography, and thus represent more recently formed “post-uplift” discontinuities.

Hencher et al. (2011) explained using case studies of sheet joint slope failures in Hong Kong, Korea, and other countries, the importance of first- and second-order roughness and waviness in stabilising potential sliding blocks. He notes that the formation of sheet joints under Mode I tensile splitting means that sheet joints are unlikely to contain patches of intact rock bridges which provide true cohesion. Rather, the strength benefit of roughness and waviness has been treated in the past by Hoek (2009) and Wyllie and Mah (2004) as an apparent cohesion. Unlike true cohesion provided by intact rock bridges, however, the strength benefit of roughness is derived from friction angle and dilation angle, both of which are sensitive to effective stress, and thus may be reduced by the presence of excess groundwater pore pressures.

Hencher and Knipe (2007) also stress the importance of characterising evolution of sheet joints and other discontinuities with time. The growth in persistence and the reduction of mechanical strength properties through time-dependent mechanical and chemical processes is a critical consideration in engineering evaluation of rock slopes. Furthermore, consideration of the full geological history of a rock mass is necessary to identify potential planes of weakness which may not be initially fractured in situ, but may
act as preferential planes for discontinuity nucleation and extension. For example, Almeida et al. (2006) described three orthogonal planes of weakness that commonly exist in granite (rift, grain, and hardway), that are used to define the orientation of axes for splitting blocks of granite in quarrying.

The persistent surface-parallel joints at the Chief occur parallel to modern topography, and are therefore post-uplift exfoliation joints. In a study of sheet joint structure, Holzhausen (1989) noted that sheet or exfoliation joints are rarely offset in shear by more than a few millimetres, thus they are not preconditioned to shear failure by slickensides or coatings of disintegrated joint wall material. Although exfoliation can occur in any rock type, it is most commonly associated with massive plutonic granite (Bahat et al., 2005; Holzhausen, 1989). Persistence of exfoliation joints tends to be greater in domains where the rock mass is homogenous and where the stress field is uniform over tens to hundreds of metres of more (Bahat et al., 2005).

Exfoliation is a brittle phenomenon driven initially by microscopic Mode I failure (tensile splitting). As a result, many of the same morphological features that occur on laboratory-scale brittle tension cracks (i.e. arrest marks, hackles, plumes, and bifurcation, see Section 3.4.4.2) also occur in association with large-scale exfoliation joints. Some of the brittle features observed on the Chief are discussed in Section 3.4.4.2.

3.4.4.2. Brittle Intact Rock Fractures

Fractographic studies in glass, ceramics, and rocks have identified three distinct zones that occur on brittle fracture surfaces (Figure 3 - 18, Figure 3 - 19). From the initiation point of a crack, the following morphological zones occur:

- **The mirror plane** spreads outward from the initiation point as a flat surface without visible steps or roughness; during crack propagation, crack velocity reaches its maximum value inside this zone (Bahat and Rabinovitch, 2000)
- **The mist zone** occurs outside mirror plane; roughness increases and secondary fractures occur wherever the critical Mode 1 (i.e. tensile) stress intensity $K_{IC}$ occurs around pre-existing micro-flaws. For fractures in rock, the mist zone is usually not visible with the naked eye (Bahat et al., 2005)
Figure 3 - 18: Schematic illustration of mirror, mist, and fringe zones of a brittle crack (Modified after Bahat et al., 2005).

- The fringe zone occurs outside the mist zone; secondary crack surfaces occur in the fringe, aligned parallel to the direction of propagation; the characteristics of the fringe depend on crack velocity: historical experiments in glass have shown that en echelon fringes with regularly-spaced secondary cracks of similar shape are produced by slow crack propagation \((v_c \leq 0.1 \text{ m/s})\), whereas irregular and ragged hackle fringes form at high crack velocity \((v_c \geq 0.1 \text{ m/s})\) (Wiederhorn et al., 1974; Bahat et al., 2005).

Figure 3 - 19: Mirror and hackle fringe in a blast-induced fracture in chalk. (Modified from Bahat and Rabinovitch, 2000).
In addition to the mirror, mist, and hackle zones, bifurcation of the fracture surface may occur as a fracture propagates further beyond the fringe zone, where a crack may split into two or more non-coplanar branches, separated by a step surface parallel to the direction of crack propagation (Figure 3 - 20). Step surfaces formed by bifurcation commonly occur on exfoliation joints.

**Figure 3 - 20**: Bifurcation of sheet joints can produce step surfaces parallel to the direction of crack propagation (Reproduced after Holzhausen, 1989).

Observations from laboratory-scale fractography studies are equally applicable to large rock exposures. Strata-bound tension joints in layered sedimentary rock, for example, frequently have curved, concentric arrest marks and plume markings that indicate the path of joint propagation and help to reconstruct the stress-state of the propagating crack front (Figure 3 - 21).

Plume marks are striae that occur parallel to the direction of crack front propagation, and extend radially outward from the crack initiation point. Arrest marks are concentric step surfaces or undulations that are aligned perpendicular to the direction of crack propagation, indicating meta-stable positions where crack growth ceased or was temporarily slowed.
Figure 3 - 21: Tension joints at Jwaneng mine, with (A) Concentric undulations or partial arrest marks; (B) and (C) show plume markings with mirror plane (M) and partial hackle fringe (H).
Figure 3 - 22 shows a schematic view of common fractography features associated with exfoliation joints, including arrest marks, plume marks, and secondary hackle fringe cracks.

![Schematic view of common fractography features](image)

Figure 3 - 22: Schematic of fracture surface morphology of a tension joint with plume structure and arrest marks, bounded by a hackle fringe (Modified after Bahat et al., 2005).

Some of the brittle fracture features observed at the Chief are described below, and highlighted in Figures 3-29, 3-30, and 3-31.

- **Large-scale arrest marks** are present; 10-50 m long, with associated step heights varying from 0.2 m to 1 m, and step width (i.e. perpendicular distance between arrest marks) between 10 m and 35 m
  - Arrest marks occur in curved-concentric arrangements and also as angular or linear step surfaces. The pattern of arrest marks indicates that exfoliation joints nucleated from multiple initiation points, and subsequently propagated and interacted to form larger, merged composite joints; exfoliation joint interaction is identified by rotation of the crack front (i.e. rotation of arrest marks (Figure 3 - 23, upper left)

- **Partial arches** occur in the centre of the region of interest, where composite exfoliation joints have deviated perpendicular to the cliff face, forming step surfaces up to about 5 m wide. Similar arches have been investigated on the El Capitan monolith and Half Dome in Yosemite National Park, USA (Bahat et al., 2005).
Although they may be present in any rock type, arches always occur in association with exfoliation joints, with arch thickness depending directly on the spacing of exfoliation joints.

- The lower terminations of arches at the Chief taper to curved merges with the sub-vertical cliff face. The upper terminations branch and bifurcate forming an anastomosing interaction zone with sub-vertical exfoliation joints (Figure 3 - 25).

![Figure 3 - 23: Selected partial arches and arrest marks identified on the Chief cliff face.](image)

- **En echelon or hackle fringes** occur where non-coplanar curved fractures are aligned in parallel along the fringe zone of exfoliation joints (Figure 3 - 24)
  - En echelon and hackle fractures can form rough step surfaces that increase surface roughness.
- **Niches** occur where exfoliation joints have deviated perpendicular into the cliff face; with brittle fractures forming recessed embayments or release surfaces for exfoliation slabs (Figure 3 - 24)
Niches may occur as isolated angular shapes, or concentric crescents resulting from brittle fracture initiation at pre-existing arrest marks. Bahat et al. (2005) propose that niches serve as initiation points for arch formation and release of larger exfoliation slabs.

Researchers have proposed that exfoliation jointing with persistence of hundreds of metres or more results from the growth and interaction of many smaller-scale joints propagating from separate initiation points, producing a composite surface (e.g. Holzhausen, 1989; Bahat et al., 2005). The morphology of the Chief cliff face supports this position: a complex configuration of superimposed plumes and arrest marks records the history of nucleation, growth, bifurcation and merging of many distinct exfoliation joints. Similar features have been identified and characterized on other granite monoliths like El Capitan and Half Dome in Yosemite National Park, California, and also in sedimentary strata like the Navajo Sandstone of Zion National Park, Utah (Bahat et al., 2005).
Figure 3 - 24: Brittle fracture features observed at the Chief a) LiDAR point cloud and b) 2-D digital photograph. Brittle features are highlighted including: arrest marks (A); bifurcation of exfoliation joints (B); niches (N); hackle fringes (HF); and partial arches (PA).
Figure 3 - 25: View of step surfaces formed by large partial arches at the Chief. Lower arch terminations are gradational, forming curved intersections with the cliff face; in contrast, upper terminations bifurcate and form an anastomosing zone of interaction with sub-vertical exfoliation joints.
3.4.4.3. Post-processing with GIS

A colour contour map of slope steepness was produced using ArcMap (Esri, 2012) to help illustrate the structural contrasts associated with interacting exfoliation joints and perpendicular step surfaces (Figure 3 - 26). Large-scale branching of composite exfoliation joints produced thick partial arches that are highlighted in the centre of the image. Shape and orientation of brittle fracture features, and changes in cliff orientation are also highlighted by contrasts in colour shading. The plot serves as a proof-of-concept that simple slope steepness maps can be used to quickly produce colour-coded maps from long-range remote sensing data, which help to accentuate major contrasts in slope structure and highlight major geomorphological and fractographic features.

Figure 3 - 26: Slope steepness map of the Chief produced using ArcMap.
3.4.5. Implications for Persistence and Intact Rock Bridges

Based on remote sensing observations, potential slope instability at the Chief is probably influenced most strongly by the persistence and spacing of non-daylighting exfoliation joints inside the monolith. Although past studies have concluded that exfoliation joint spacing tends to increase with depth (Bahat et al., 1999), the precise interior configuration of exfoliation joints at the Chief is uncertain.

Beam buckling has been applied in the past as a mechanical analogue to describe exfoliation slab failure. Bahat et al. (2005) adapted an earlier beam formula from Holzhausen and Johnson (1979) to show that the critical stress which causes unstable exfoliation joint propagation is proportional to the squared distance to a free face (Figure 3 - 27):

\[
\sigma_{\text{critical}} = \frac{\pi^2 E}{3} \times \left(\frac{d}{2c}\right)^2
\]

Figure 3 - 27: Schematic illustration of non-persistent exfoliation joints in buckling analysis applied to a granite monolith.

Where \( E \) is the Young’s Modulus of intact rock, \( 2c \) is the internal fracture length and \( d \) is the distance to the free face.
If joint spacing is assumed to increase linearly with distance \( d \) to the free face, then strength of potential exfoliation slabs will show quadratic growth over the same interval. Consequently, deeper exfoliation slabs, further back from the free face will tend to be more stable than shallow slabs occurring near the free face. Gravity-driven failure should be characterised by progressive ravelling, beginning with small slabs at or near the cliff face. Because the strength of exfoliation slabs is dependent on distance to the free face, estimates of effective “intact rock bridge length” may be directly related to exfoliation joint spacing.

This simplified beam buckling model considers perfectly elastic beams with fixed ends, which may be a reasonable approximation for exfoliation slabs comprised of massive, intact granite. In jointed rock masses, however, where laminated bedding or exfoliation slabs are cross-cut by other joint sets, more complex treatment is needed. The presence of cross-cutting joints reduces the tensile strength parallel to the roof of an underground excavation, or parallel to the wall of a cliff face. Diederichs (1999) introduced a modified application of the Voussoir beam analogue, earlier proposed by Beer and Meeke (1982) to improve estimates of yield threshold for snap-through failure, where buckling occurs through intact rock failure perpendicular to the free face.

Although deep exfoliation joints inside the cliff are unobservable, the height of step surfaces formed by partial arches, niches, arrest marks, and bifurcated exfoliation joints may be a useful proxy measure of spacing. To investigate the distribution of step surface heights, a sample of 101 steps from the LiDAR point cloud was used to produce a histogram of step surface height (Figure 3 - 28). The maximum measured step height was about 5.7 m; the minimum was 0.2 m; and the average was about 1.1 m; the limited resolution of the LiDAR point cloud results in complete truncation of small steps below a height of approximately 0.2 m.
Figure 3 - 28: Step surface heights associated with exfoliation joints at the Chief.

The distribution of step heights appears to fit well to a negative exponential function, where step heights between 0.4 m – 0.6 m are under-sampled due to truncation, and step heights smaller than 0.4 m are mostly truncated. The suitability of negative exponential functions to characterise discontinuity spacing distributions has been demonstrated in the past by Priest and Hudston (1976), and Ortega et al. (2006).

Although the length of internal exfoliation joints is uncertain, the distance between concentric arrest marks can be used to estimate the size of meta-stable exfoliation joints, which may in turn be used to estimate peak strength of a potential exfoliation slab. To estimate internal crack length, the distance between concentric arrest marks in the upper left quadrant of the point cloud were measured. Extremely persistent composite joints were excluded, because they represent fractures that have already merged and may reflect unstable propagation (i.e. mixed-mode loading and complex failure).

The large arrest marks in the upper left quadrant are about 15 m apart, but a smooth planar area up to 35 m wide occurs above the crest of the large arch in the centre of the wall (Figure 3 - 29).
Figure 3 - 29: The maximum distance between arrest marks is approximately 35 m, occurring in the upper left quadrant of the cliff face.

Peak strength was therefore calculated for a candidate exfoliation slab about 1.1 m behind the face, with an initial crack length of $2c = 35$ m and Young’s Modulus of 50 GPa, a typical value for intact granite (e.g. González de Vallejo and Ferrer, 2011):

$$\sigma_{\text{critical}} = \pi^2 \frac{50 \times 10^3 \text{ MPa}}{3} \times \left(\frac{1.1 \text{ m}}{35 \text{ m}}\right)^2$$

$$\sigma_{\text{critical}} = 162 \text{ MPa}$$

The calculated peak strength of 162 MPa is four orders of magnitude greater than gravity-induced vertical stress. The high strength of the slab relative to gravity-induced stress supports the proposal that glacial processes likely played an important role in the large slab failures indicated by the 5 m-thick arches in the centre of the wall. Failure may have been influenced by loading and unloading by overlying ice, thermal freeze/thaw cycles and the associated pore pressures arising from glacial melting, and mechanical weathering by glacial scour.
The simplified beam buckling model considers instantaneous rupture, where in reality the growth of internal exfoliation joints and brittle cracks is a time-dependent process, that may in some cases be characterised by slow, sub-critical crack growth and time-dependent decay of discontinuity shear strength over geologic timescales, due to weathering processes including chemical dissolution of discontinuity surfaces by groundwater, infill of sediment transported by groundwater, dilation of discontinuities and intermittent episodes of shear slip and meta-stability (Hencher et al., 2011; Rinne, 2008; Hencher and Knipe, 2007).

3.4.6. Application to Open Pit Slope Mapping

Measurements of discontinuity persistence and intact rock bridges are both subject to error and uncertainty arising from scale effects, orientation bias, and survey resolution. Intact rock bridge measurements, however, are subject to additional uncertainty arising from rock mass heterogeneity and the influence of stress localization on crack initiation and propagation. Diederichs (2007) noted that brittle failure is always driven by localization effects; cracks nucleate and grow wherever stress concentrations occur around microscopic flaws, geometrical irregularities or boundaries of modulus contrast.

Predictions of intact rock bridge geometry carry an implicit assumption of new brittle fracture geometry that will occur if the rock bridge fails. The accuracy of intact rock bridge measurements depends therefore on the quality of data describing factors that influence brittle failure, including: intact rock properties, pre-existing discontinuities, and loading conditions.

The following sections use observations from the Chief to help formulate preliminary investigation strategies for characterising discontinuity persistence and intact rock bridges in open pit slopes.
3.4.6.1. From Brittle Fracture Observations to Inferred Intact Rock Bridge Geometry

Although experimental studies have provided useful insight into intact rock bridge failure patterns in laboratory-scale specimens (e.g. Gehle and Kutter, 2003; Sagong and Bobet, 2002; Wong et al., 2001; Bobet and Einstein, 1998; Lajtai, 1969a,b,c), observations of large-scale intact rock bridge failures in slopes are uncommon. Thus, in lieu of explicit large-scale measurements of intact rock bridges, records of brittle fracture features from natural slopes like the Chief may help to bracket the potential range of rock bridge geometry and brittle failure behaviour that could develop in an engineered slope. Features like joint bifurcation steps, arrest marks, arches and niches, help to demonstrate potential modes of discontinuity growth and rock bridge failure.

A series of cross sections was extracted from the LiDAR point cloud of the Chief, to help characterise the surface morphology and relative length proportions of surface-parallel exfoliation joints and perpendicular brittle fracture features (Figure 3 - 30). The cross sections are oriented parallel to the down-dip direction of the overall slope based on the usual convention for characterising discontinuity roughness in the down-dip direction. Cross sections thus do not directly relate to the crack-front orientations during phases of exfoliation joint growth and interaction. However, changes in slope roughness along-strike and down dip can still reveal useful information on slope morphology. The total length proportion of perpendicular steps ranges from near zero in cross-section CS3, up to a maximum of about 11% near the peak of the partial arches in CS12.
Figure 3 - 30: Cross-sections across the cliff surface reveal the along-strike development of step surfaces, reflecting tensile release surfaces for exfoliation slab failure
### 3.4.6.2. Influence of Observation Scale

Observation scale and scale effects in mapping can have profound influence on measurements of discontinuity and rock bridge characteristics. As scale increases, the influence of surface roughness on peak strength of discontinuities tends to diminish (Bandis et al., 1981), and surface texture of 3-D terrain models becomes more uniform and smooth. In slopes, large-scale interaction zones between brittle fractures and many low persistence joints may appear at larger observation scales as singular continuous fault planes or step surfaces (Sturzenegger and Stead, 2012).

To investigate the influence of observation scale at the Chief, a supplementary photogrammetry survey was undertaken on a roadside granite outcrop about 400 m from the base of the mountain (Figure 3 - 31). The outcrop is approximately 4 m high, and trends north-south for 40 m along the side of Highway 1. Whereas failure of large exfoliation slabs on the cliff face of the Chief was likely influenced by glacial action, intact rock fractures on the roadside outcrop were caused by recent blasting during the construction of a vehicle pullout and viewing point at the side of Highway 1.

Three main factors suggest that this outcrop and the Chief are both part of the Howe Sound Batholith:

1. Visual inspection suggests a granitic composition of the roadside outcrop, similar to published descriptions of the Chief and the Howe Sound Batholith (Monger and McNicoll, 1993)
2. The roadside outcrop and Chief both show the same characteristic exfoliation jointing and evidence of glacial polish and scour
3. A 2-m wide steeply-dipping basalt dyke intersects the roadside outcrop about 20 m further back from the road (not pictured), and correlates with the black dyke intersecting the Chief
Figure 3-31: Photogrammetry DTM of a roadside outcrop near the Chief, highlighting glacial polish features, exfoliation joints and blast-induced damage.
One very high persistence (i.e. persistence > 20 m; ISRM, 1978) sub-vertical exfoliation joint strikes parallel to the outcrop. The joint surface is curviplanar and bifurcated, with a step width of up to 0.75 m between separate branches of the joint.

At the centre of the outcrop, the joint surface spans the full 4 m height of the outcrop. The most continuous section of the joint surface is about 20 m long along-strike; the total length including separate branches of the joint is about 33 m.

To compare the length proportions of exfoliation joints and intact rock fractures, a series of vertically-oriented cross-sections were constructed (Figure 3 - 32), similar to those measured on the west wall of the Chief.

Figure 3 - 32: Roughness profiles traversing bifurcated exfoliation joint connected by blast-induced brittle fractures.
The cross sections from the roadside outcrop show some similarity to those extracted from the west wall of the Chief:

- Beginning in the south, the sections are continuous curvilinear profiles of the outcrop surface formed by the exfoliation joint
- Toward the north, the development of the bifurcation step surfaces is shown on cross sections CS2 to CS8 (Figure 3 - 32), reaching a maximum step width of about 0.75 m, and then tapering out further to the north

In section CS2, the bifurcation step surface reaches its maximum width, and a smaller step surface also occurs at the base of the outcrop. The length proportion of stepped fracture surfaces represents about 29% of the total vertical profile length at this location, however this figure is skewed by the censoring of the exfoliation joint surface beneath the ground. If the exfoliation joint plane is continuous beneath the ground surface, the true length proportion of stepped fracture surfaces will be lower than 29%.

When step surface width is compared to the maximum along-strike persistence of the exfoliation joint (33 m), the length proportion of stepped surface width drops to about 5%; which is closer to the range commonly cited values for intact rock bridge percentage cited in the literature (i.e. approximately 0 – 10%, Chapter 2, Section 3.2).

Comparison of the cross-sections from the Chief and from the roadside outcrop suggests that the processes of exfoliation, bifurcation and brittle step-surface formation can produce similar stepped roughness patterns over observation scales varying by at least 3 orders of magnitude.

Self-organized criticality (Bak et al., 1987) and fractal scaling have been used to investigate universal, scale-independent characteristics of geological phenomena like landslides and cliff failures (Gomez et al., 2002; Amitrano et al., 2005), and rock discontinuity parameters including trace length, spacing, aperture and roughness (Guerriero et al, 2011; Ortega et al., 2006; Hanson and Schmittbuhl, 2003; Chávez-Guerrero et al., 2010). However, the application of scale-independent methods to characterise rock bridges and persistence may be problematic. Although discontinuity roughness and step-path geometry may seem to scale as a fractal, rock mass strength

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and deformability, and discontinuity shear strength have been shown to be more scale-dependent (e.g. Hoek, 2007; Bandis et al., 1981).

Observations of failed step-surfaces at the Chief can help to guide a suggested upper limit for mapping intact rock bridges at the open pit study sites. By comparing the length of an inferred intact rock bridge to the sum length of pre-existing discontinuities, a “rock bridge ratio” can be calculated, where a value of 0 indicates a fully persistent discontinuity, and a value of 1 represents equal length properties of intact rock bridge and pre-existing discontinuities (Figure 3-33).

Conceptually, a rock bridge ratio of 0 can be assigned a confidence level of 100%, such that the direction of potential failure is expected to follow the continuous pre-existing discontinuity. For larger rock bridge ratio values, the mode of failure and pattern of crack growth becomes increasingly uncertain due to additional rock mass heterogeneity and uncertain stress localization effects. Based on brittle fracture observations at the Chief, a preliminary limit of 0.2 for rock bridge ratio may serve as a realistic cut-off for guiding inferred intact rock bridge tracing at the open pit study sites.

![Figure 3-33: Conceptual illustration of confidence in rock bridge size, based on observed exfoliation slab failure geometries.](image)

Figure 3-33: Conceptual illustration of confidence in rock bridge size, based on observed exfoliation slab failure geometries.
3.4.6.3. Site-Specific Adaptations

The conditions at each mine site are different and site-specific adaptations to field methods, post-processing techniques, and numerical modelling approaches may be needed in order to better capture the influence of discontinuity persistence and intact rock bridges on slope stability. Some of the factors that are likely to influence slope investigation strategies include:

1. Geological variation: rock mass quality, lithology, alteration and weathering
2. Hydrogeology: presence of surface water and fracture flow of groundwater
3. Tectonic history: rock mass structure, damage history (faulting) and folding
4. Blasting conditions: severity of overbreak, quality of wall control

The specific adaptations applied at each mine site are discussed more fully in Chapter 4.

3.4.6.4. Geological Variation

Geological factors including mineralogy and grain size, and heterogeneity of these parameters, have been shown to influence joint development and brittle fracture characteristics at macroscopic scales (Bahat et al., 2005), and also to effect the stress thresholds corresponding to microscopic crack initiation, accumulation and interaction (e.g. Eberhardt, 1998; Diederich, 1999). Moreover, rock type strongly influences regional fracture network characteristics (Ortega and Marrett, 2000). For instance, slopes in bedded (meta)sedimentary rocks are influenced by bedding orientation, whereas slopes in massive plutonic rock may be influenced more strongly by regional-scale fault structures and interaction between non-persistent, non-coplanar discontinuities.

3.4.6.5. Tectonic History

Tectonic history determines the degree of pre-mining rock mass disturbance, the genesis of discontinuities associated with folds and faults, and the state of pre-mining stress. Laws (2001) proposed that large-scale shear zones could be divided into three concentric domains: (1) a strongly foliated zone, (2) a fractured zone, and (3) a heavily fractured, cohesionless zone. More tectonically-disturbed slopes are likely to have reduced rock mass quality, larger fracture intensity and reduced likelihood of large, discrete intact rock bridges.
Tectonic history and paleostress state influence the structure of hydrothermal veins and brittle shears from microscopic to regional scales (Gudmundsson, 2011; Dight and Bogacz, 2009). Crustal plate-scale compression can also cause high “locked-in” horizontal stresses in the pre-mining rock mass. In his study of a Chelmsford granite quarry in Massachusetts, Holzhausen (1989) concluded that horizontal sheet joints forms in areas where high tectonic compressive stresses occur parallel to the ground surface. Although he measured in situ horizontal stress of up to 30 MPa in the quarry floor, laboratory tests of the same granite measured UCS of about 160 MPa.

The disparity between UCS and stress required to produce exfoliation joints relates to the concepts of crack initiation and crack damage stress thresholds (Diederichs, 2007). Researchers who studied the URL Mine-by experimental test tunnel in Pinawa, Manitoba, found that brittle failure occurred in a tunnel excavated in very strong granite, at stresses of about half of the laboratory-measured UCS (Figure 3-34).

![Figure 3-34](image.png)

**Figure 3-34:** Influence of confining stress on brittle failure development in massive hard rock (Diederichs, 2007, by permission).
Acoustic emission monitoring of laboratory UCS tests of intact rock revealed that initiation of new microscopic tension cracks begins to occur at stresses between 30% to 50% of UCS (Diederichs, 1999; Eberhardt, 1998; Martin, 1994). At tunnel boundaries, where confining stresses are negligible, individual tension cracks are free to propagate more easily, forming exfoliation-type spalling fractures that can progressively ravel away the roof or walls. Deep within the rock mass, far away from an excavation, confining stresses restrict the propagation of individual tension cracks. Instead, material failure will be characterised by a critical accumulation of small tension cracks, which may interact to form a macroscopic shear structure.

For slopes in regions of high horizontal tectonic stresses, the subsequent removal of confining stress via pit excavation may promote brittle, stress-induced tensile failure of intact rock bridges. Numerical modelling studies of such slopes may require adaptations in order to consider the potential for stress-induced failure of intact rock bridges.

3.4.6.6. Blasting Conditions

Field investigation methods will require adaptations to consider the effects of blast-induced damage. Large-scale production blasting techniques used in open pit mining inevitably disturb the rock mass for some uncertain distance behind the pit wall (Hoek, 2012; Hoek and Karzulovic, 2000). Blast-induced fractures may mask pre-existing discontinuities, making it difficult to accurately measure persistence and to identify intact rock bridges. However, patterns in blast-induced cracking may also help reveal potential “paths of least resistance” through intact rock bridges that have failed under blast-induced loading, analogous to the brittle fracture features observed on the Chief.

Investigations at Ok Tedi mine in Papua New Guinea showed that blasting in stronger rocks under low in situ stresses tends to reduce cohesion more than friction (Little, 1999). Thus, at low normal stresses, in strong, massive rocks, blasting tends to first destroy cohesion contributed by intact rock bridges. Weak rock masses were found to undergo greater strength loss in response to blasting than strong rocks, and sensitivity calculations with the Hoek-Brown criterion showed that the strength loss arising from blast damage is more severe at higher stress levels. Thus highly stressed sections of rock mass are more susceptible to blast-induced damage. Zones of high stress
concentration, such as the toe region of a large open pit slope, may be more sensitive to potential instabilities arising from poor blasting control.

Based on the findings from Ok Tedi, Little (1999) and Baczynski (2000, 2008) proposed a model for large slope instabilities with complex failure surfaces, where failure may occur through up to 5 regions with different mechanical properties (Figure 3 - 35).

![Figure 3 - 35: Schematic diagram of blast-damaged inter-ramp or overall pit slope with failure occurring through 5 distinct regions (Modified and reproduced after Little, 1999).]

The model can incorporate a combination failure involving:

1. Failure on major structural control (faults)
2. Rock mass rupture
3. Minor structural control (joints or foliation)
4. Intact rock bridge failure
5. Failure through a blast damage zone extended behind the slope.
4. Field Investigation at Jwaneng, Botswana

4.1. Mine Site Overview

Jwaneng is the richest diamond mine in the world by production value (Tunono et al., 2011). This investigation focuses on the East wall of the pit, and was undertaken during September 2011. The pit depth at the time of the field investigation was 360 m, and the Cut 8 expansion will push back the crest and deepen the pit to 622 m (Tunono et al., 2011).

4.2. Geological Setting

The pit at Jwaneng intersects four kimberlite pipes, which have intruded metasedimentary rocks of the Lower Proterozoic Transvaal Supergroup. The generalized local stratigraphy was summarised by SRK (Barnett, 2006) as comprising:

1. Approximately 15-20 m of Kalahari Sand cover, underlain by
2. Up to 40 m of calcrete, underlain by
3. Up to 30 m of thinly bedded metasedimentary units including laminated shales and fissile carbonaceous shales of the Timeball Hill Formation, underlain by
4. Up to 500 m of quartzitic shale of the Rooihoogte Formation.

The quartzitic shale is divided into an upper and a lower unit, separated by a thin (0-4 m) marker unit of conglomerate. Dolerite dykes and sills have also been identified throughout the pit.

The country rock and kimberlite are dissected by a complex network of predominantly steeply-dipping normal faults, which have developed during four distinct phases of tectonic deformation (Tunono et al., 2011). The working geological model for slope stability assessment and design is divided into 11 geotechnical domains based on interpreted fault boundaries (SRK / Barnett, 2009). The investigation in this thesis
focuses on the East wall within Domain 7a, and Domain 6 near the toe of the slope (Figure 4 - 1).

Figure 4 - 1: Geotechnical Domains at Jwaneng, bounded by a complex network of faults, which are shown in red (SRK / Barnett, 2009; reproduced by permission).

Figure 4 - 2 presents an interpreted geological section through the pit, CS-J1, looking Northeast, with the as-built 2008 pit profile and design profile for Cut 8 superimposed (SRK / Barnett, 2009).
Figure 4 - 2: Selected cross-section CS-J1, looking Northeast, extracted from geological model developed by Debswana and SRK (Modified after SRK / Barnett, 2009, reproduced by permission).
4.3. Geotechnical Characterisation

This section summarises the East wall slope geometry at the time of field investigation in September 2011, and reviews past geotechnical characterisation results for the main rock units in the East wall. Although the current investigation focuses on discontinuity persistence and intact rock bridge characterisation, reliable estimates of intact rock and discontinuity shear strength parameters are necessary to help estimate intact rock bridge shear strength and the stability of potential step-path failure surfaces.

### 4.3.1. Slope Geometry

The overall East wall at the time of investigation was 360 m high, however the photogrammetry models of the East wall focus on a section of slope 290 m in height. Figure 4 - 3 presents a selected profile of the East wall, looking northeast, approximately aligned with cross-section CS-J1 shown previously in Section 4.2.

![Figure 4 - 3: Cross-section CS-J1 through Jwaneng East wall, looking Northeast, overall slope angle of 37°.](image-url)
Inter ramp height is 130 m for the lower slope, 50 m for the mid-slope section, and approximately 60 m for the upper slope. Inter-ramp angles are approximately 50° for the lower slope, 45° for the mid-slope section, and 47° in the upper slope. The overall slope angle for the selected profile is approximately 37°. Bench face angles are variable, ranging from approximately 70° where standing walls are maintained, however frequently lower angles occur where benches indicate backbreak to the local dip angle of the metasedimentary foliation (Figure 4 - 4).

Bench-scale instabilities in the East wall are dominated by planar sliding on foliation, which dips approximately northwest at angles between 10° to 40°, daylighting throughout the slope. Figure 4 - 4 presents a panoramic merged photograph of the east wall, highlighting contrasts in lithology, and traces of selected examples of very persistent discontinuities.

Variations in dip angle occur due to undulations associated with minor local folding, in addition to translation and rotation of discrete structural blocks bounded by faults (Barnett / SRK, 2009). Previous investigation by SRK (Barnett, 2009) has shown that locally within Domain 6, foliation dip is expected to vary between approximately 29° and 31°, dipping toward northwest. Previous borehole investigations have shown that in undisturbed rock mass, metasedimentary foliation discontinuities are generally tight and closed, however the more recently developed sub-vertical joints are mostly open (Tunono et al., 2011). Investigations by Barnett (2008) have shown that kimberlite rock mass shows evidence of past failure of the surrounding metasedimentary rock mass, where slope failures (at shallow depth, < 1 km) and stress-induced rock burst events (at depths >> 1 km) have caused the metasedimentary rock mass to fail and cave inward into the kimberlite pipe. The resultant failure deposits are variously present as breccias or isolated massive “floating” blocks of metasedimentary rock within the kimberlite pipe.
Figure 4 - 4: Panoramic view of part of Jwaneng East wall, with foliation dipping out of the slope, traces show major structures, and insets show the extent of the field mapping area and the close range photogrammetry survey (f = 100 mm, distance = 50 m).
4.3.2. **Rock Mass and Discontinuity Properties**

At the time of the field investigation, the East wall intersected mostly Quartzitic Shale of the Rooihoogte Formation, with some laminated and carbonaceous shales of the Timeball Hill Formation occurring in the upper benches of the slope. Previous investigations by SRK developed a set of recommended rock mass strength and deformability properties, based on borehole drilling, field assessment and laboratory testing.

In general, the country rock has been classified as mostly fair to good quality rock mass. The quartzitic shale is distinctly foliated, and the bed thickness is thus an important determinant of block size. The quartzitic shale has been described by others as mostly massive (Tunono et al., 2011), corresponding to block sizes exceeding 1 m edge lengths according to the quantified GSI method suggested by Cai et al. (2004, 2007). Some areas of more thinly-bedded, fissile shale occur in the upper parts of the slope, causing progressive ravelling failure (Tunono et al., 2011).

The kimberlite ore is more porous than the country rock (Tunono et al., 2011): the resultant potential for groundwater flux through the kimberlite may have contributed to its heavily weathered or altered condition. Structure in the kimberlite ranges from blocky/disturbed to very blocky, with GSI ranging from approximately 35 to 45.

Although the kimberlite rock mass is a more disturbed, weaker overall rock mass, the quartzitic shale may be more susceptible to sliding failures controlled by the preferential weakness planes provided by persistent foliation. Figure 4 - 5 highlights a region near the base of the East wall where blasted quartzitic shale has failed along foliation, directly adjacent to a sub-vertical sidewall comprised of blocky/disturbed kimberlite. The absence of foliation in the kimberlite results in a (meta)stable face, whereas the shale has failed by planar sliding induced by blasting.
Figure 4 - 5: Despite low intact rock strength, a vertical sidewall remains standing in kimberlite, adjacent to a series of planar sliding failures in quartzitic shale.
4.4. Field Investigation

4.4.1. Overview

The investigation into the East wall is focused on using results from modified field discontinuity survey, photogrammetry, and 2-D trace mapping to improve our understanding of the role of discontinuity persistence and intact rock bridges in the East wall of the pit.

The dominant structural feature in the East wall is the northwest-dipping foliation, which can be traced almost continuously along the full length of the wall. Preliminary slope stability assessments may thus consider the foliation as fully persistent. However, even with fully persistent foliation, kinematic release surfaces are required in order for discrete blocks to slide (Brideau, 2010). Attention was therefore given to characterising the geometry, persistence, and spatial distribution of lateral and rear release surfaces, which include a combination of

1. Persistent inter-ramp or overall slope-scale faults and lithological contacts
2. Pre-existing tension joints formed during episodes of tectonic deformation
3. Recent brittle fractures arising from blasting damage and progressive slope relaxation.

Emphasis was also given to searching for evidence of coplanar or non-coplanar intact rock bridges between non-persistent foliation discontinuities. Although foliation discontinuities were identified as typically tight and closed in undisturbed rock mass, near-surface foliation discontinuities mapped in the slope are frequently opened as a result of blasting-induced damage.

4.4.2. Modified Discontinuity Survey Procedure

A modified discontinuity survey was carried out for six windows near the toe of the East wall, at local elevation +870 m (Figure 4 - 6). Each of the six windows was also subsequently mapped digitally using 2-D trace mapping techniques.
The field discontinuity survey followed traditional window mapping procedures (e.g. Wyllie and Mah, 2004), with modifications to emphasize:

1. Searching for evidence of intact rock bridges at outcrop scale
2. Identification of different styles of discontinuity termination, including termination in intact rock ($T_{\text{IR}}$)
3. Characterisation of persistence of lateral and rear releases surfaces for planar sliding failures.

The dimensions and orientations of each window are summarised in Table 4 - 1.
<table>
<thead>
<tr>
<th>Window Name</th>
<th>Dip (°)</th>
<th>Dip Direction (°)</th>
<th>Length (m)</th>
<th>Height (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W-01</td>
<td>90</td>
<td>225</td>
<td>6</td>
<td>2</td>
</tr>
<tr>
<td>W-02</td>
<td>70</td>
<td>320</td>
<td>3.3</td>
<td>2.3</td>
</tr>
<tr>
<td>W-03</td>
<td>80</td>
<td>275</td>
<td>5.5</td>
<td>2</td>
</tr>
<tr>
<td>W-04</td>
<td>85</td>
<td>030</td>
<td>5.1</td>
<td>1.8</td>
</tr>
<tr>
<td>W-05</td>
<td>80</td>
<td>325</td>
<td>5.1</td>
<td>1.7</td>
</tr>
<tr>
<td>W-06</td>
<td>80</td>
<td>330</td>
<td>5.7</td>
<td>1.6</td>
</tr>
</tbody>
</table>

Window W-04 was oriented perpendicular to the predominant orientation of the East wall, on a “bull nose” exposure protruding from the slope (Figure 4 - 7). Survey of variably oriented windows is a useful way to limit orientation bias inherent in mapping a single window orientation. Most of the windows are oriented approximately parallel to the slope face (i.e. along-strike), but W-04 provides a valuable view of discontinuity persistence and intact rock bridges perpendicular to the slope (i.e. down-dip).

Figure 4 - 7: Window W-04 “bull nose” oriented perpendicular to overall East wall.
4.4.2.1. Discontinuity Description

In addition to the persistent northwest-dipping foliation, three other discontinuity sets were identified, oriented approximately orthogonal to foliation (Figure 4 - 8). The three sets may act as kinematic release surfaces, and are denoted R1, R2 and R3.

Figure 4 - 8: Pole plot and best-fit great circles for discontinuity sets identified in field window mapping at base of East wall.

The orientation statistics for the mapped discontinuities are summarised in Table 4 - 2.

Table 4 - 2: Summary of discontinuity set orientations identified in field mapping.

<table>
<thead>
<tr>
<th>Set</th>
<th>Mean Dip (°) [Standard Deviation]</th>
<th>Mean Dip Direction (°) [Standard Deviation]</th>
<th>Sample Size, n</th>
<th>Fisher K</th>
</tr>
</thead>
<tbody>
<tr>
<td>R1</td>
<td>61 [6.3]</td>
<td>134 [17.7]</td>
<td>22</td>
<td>24</td>
</tr>
<tr>
<td>R2</td>
<td>77 [7.1]</td>
<td>99 [7.1]</td>
<td>26</td>
<td>68</td>
</tr>
<tr>
<td>R3</td>
<td>75 [8.0]</td>
<td>199 [15.8]</td>
<td>23</td>
<td>22</td>
</tr>
</tbody>
</table>
The orientation data include mean dip and dip direction for each set; standard deviation; sample size; and the Fisher K value, which describes the “tightness” or concentration of the set. Lower K values indicate a higher degree of scatter, and higher K values indicate a more tightly-clustered set with less variation in dip and dip direction.

The stereographic plot in Figure 4 - 8 shows a possible “blind zone” where almost no poles occur, centred in the southeast quadrant of the stereonet. The blind zone corresponds to planes that dip moderately to steeply, with dip directions ranging clockwise from approximately west to northeast. If such discontinuities exist, they are under-sampled or occluded, and may occur as non-daylighting structures in the East wall. Further investigation in different domains of the pit, supplemented with boreholes data from East wall, would help to confirm or refute the existence of joints that are occluded from face mapping.

The results from the current field survey are generally similar to the combined observations from previous discontinuity surveys carried out by Debswana from 2007 to 2010 (Figure 4 - 9). However, some important differences occur between the current and the historic data:

- Pole contours from previous mapping show different orientations for sets R1, R2, and R3
- Additional sets (R3b and R4a&b) are observed in the previous mapping data that do not occur in the current field investigation mapping.

The presence of additional joint sets, and poles populating the blind zone from the current investigation, may be attributed to the broader spatial range of previous mapping data, which were gathered from locations throughout the entire pit, throughout the different structural domains, whereas the current investigation is limited to a section of the East wall.

Notably, the total number of discontinuity measurements from the current field mapping investigation is much smaller than the number of historical measurements, amounting to approximately 5% of the 2400+ measurements from past investigations. However, Fisher K values, which indicate the “tightness” of clustering for major discontinuity sets, are similar, typically ranging from approximately K = 20 to K = 60.
Figure 4 - 9: Pole plot of historical discontinuity data undertaken by Debswana throughout the Jwaneng pit.

Table 4 - 3 summarises the discontinuity set orientation data for the historical mapping.

Table 4 - 3: Summary of discontinuity set data identified in historic investigations

<table>
<thead>
<tr>
<th>Set</th>
<th>Mean Dip (°) [Standard Deviation]</th>
<th>Mean Dip Direction (°) [Standard Deviation]</th>
<th>Sample Size, n</th>
<th>Fisher K</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foliation</td>
<td>34 [14.5]</td>
<td>312 [19.9]</td>
<td>401</td>
<td>26.8</td>
</tr>
<tr>
<td>R2</td>
<td>77 [6.9]</td>
<td>095 [13.5]</td>
<td>528</td>
<td>32.1</td>
</tr>
<tr>
<td>R3a</td>
<td>79 [7.8]</td>
<td>207 [7.4]</td>
<td>265</td>
<td>58.2</td>
</tr>
<tr>
<td>R4a</td>
<td>78 [8.6]</td>
<td>176 [9.2]</td>
<td>426</td>
<td>42.7</td>
</tr>
<tr>
<td>R4b</td>
<td>83 [4.9]</td>
<td>356 [14.9]</td>
<td>216</td>
<td>60.1</td>
</tr>
</tbody>
</table>
Two main differences occur between the discontinuity sets observed in the current investigation, and the historical mapping data:

1) An additional set of release structures, designated set R4 (75°/176°), occurs in the historical data, but was not identified in the current survey. The absence of set R4 from the current survey may result from the limited coverage of the current mapping campaign; data was only collected from a small area at the base of the East wall in Domain 6; whereas the historical data shown in Figure 4 - 9 is drawn from mapping carried out at different locations all throughout the pit, including access to different geotechnical domains, design sectors and wall orientations.

2) Sets R3 and R4 have been mapped as each comprising a conjugate set of steeply dipping joints, with clusters of poles occurring both in the north and south of the stereonet; the variation in dip direction reflects standard stochastic variability in the joint sets. The north-dipping joints (i.e. the poles clustered at the south of the stereonet) are not noted in the current field survey, and fall within the “blind zone”.

The maximum size of the survey windows in the current investigation was 6 m in length and approximately 2 m high; with the limit being constrained by safety considerations and the need for physical access to the rock face for mapping orientation, roughness, and other rock mass characteristics. As a result of the limited observation scale, close-range measurements of persistence in the field discontinuity survey may be biased towards easily distinguished outcrop-scale discontinuities in the range of 0.1 m to 2 m persistence.

The distribution of discontinuity trace lengths from current field window mapping appears to conform well to a negative exponential best-fit function (Figure 4 - 10), however the negative exponential function may be poorly suited to predicting the frequency of high persistence structures (i.e. > 10 m). Furthermore, the influence of observation scale may be an important source of inaccuracy.
Figure 4 - 10: Trace lengths of field mapped discontinuities, with 0.5 m bin intervals.

The peak frequency occurs for trace length < 0.5 m, reflecting sampling bias which may result in favouring of approximately person-size discontinuities during field mapping; over-sampling of smaller discontinuities with trace lengths in the range of 0.5 m to 3 m, and under-sampling of larger features.

Higher persistence discontinuities were included in field window maps wherever present, by carrying out an initial global sketch of the window mapping area from a greater distance (~ 50 – 100 m). Figure 4 - 11 illustrates the limited size of window maps compared with very persistent structures that occur throughout the East wall, and emphasizes the need for an initial survey of major discontinuities, that would be otherwise censored by small windows. Section 4.4 discusses measurements of discontinuity persistence from digital photogrammetry, and compares the results with the field mapping observations.
Surface condition and planarity varies with discontinuity type. Foliation discontinuities are typically planar and rough at close range, but may also be undulating and rough (ISRM, 1978; evaluated over length < 1 m), with some instances of smooth or slickensided foliation partings with coating of clay minerals and chlorite, indicating past shearing and potential infiltration by groundwater. At larger-scale (evaluated over profile lengths > 10 m), foliation is smooth and undulating, with amplitudes of 1–2 m and wavelengths in the order of 10 m.

Tension joints belonging to sets R1 to R4 range from smooth and planar to very rough and stepped; some surfaces have iron oxide staining from groundwater (potentially associated with older, pre-existing tectonic joints), and others are clean and unweathered (potentially associated with recent blast-induced fractures). Field-estimated JRC values for foliation range from about 5 – 10, and field-estimated JRC values for joints belonging to sets R1 to R4 can be grouped in two categories (Figure 4 - 12): smooth surfaces ranging from JRC ≈ 2-6 (occurring in fracture mirror planes), and stepped, rough surfaces in the range of JRC ≈ 10-15 (associated with peripheral hackle fringes, as described in Chapter 3).
The rough or stepped hackle fringe regions of joint surfaces indicate high velocity crack propagation and potential dynamic loading likely associated with recent blasting damage. In contrast, the slickensides occurring on foliation discontinuities may originate from shearing associated with episodes of tectonic deformation (Figure 4 - 13).

Figure 4 - 12: Typical field roughness profiles, with associated dip/dip direction, for foliation discontinuities and lateral and rear release tension joints.

Figure 4 - 13: Slickensided foliation discontinuity, with roughness measured in the direction of slickensiding.
The difference between modern dip direction and slickenside orientation may support the hypothesis that foliation slickensides developed during tectonic episodes of shearing. Although some discontinuities may be preconditioned to shearing parallel to tectonic slickensides, the risk of sliding may be less severe where the dip direction does not align with this preferential weakness direction.

The greater the difference between slickenside trend and the foliation dip direction, the greater will be the increase in discontinuity shear strength contribution from roughness, with a maximum occurring at 90° (Figure 4 - 14).

![Diagram of slickensided surface with roughness](image)

**Figure 4 - 14: Schematic illustration of anisotropic roughness produced by slickensides (After Price and De Freitas, 2009).**

### 4.4.3. 2-D Digital Trace Mapping Procedure

For each field-mapped window, a 2-D digital trace map was produced using Illustrator (Adobe, 2012a). Trace mapping procedures for this study were based on the Digital Rock Mass Rating (DRMR) method proposed by Monte (2004), with extensions to consider:

- Two different methods for intact rock bridge tracing to derive rock bridge intensity
- Two different methods for calculating fracture intensity
- A parameter for intensity of blast-induced damage fracture intensity
After tracing discontinuities, intact rock fractures, and potential intact rock bridges, individual layers corresponding to specific trace sets were extracted and analysed with the freeware raster image processing software ImageJ (Rasband, 2008). The resultant sums of trace lengths were used to calculate areal intensity values, where

\[
\text{areal intensity} = \frac{\text{total trace length}}{\text{window surface area}}
\]

for pre-existing discontinuities, intact rock bridges, and blast-induced damage. Thus:

- Fracture intensity \( P_{21} = \frac{\sum \text{discontinuity trace lengths}}{\text{window surface area}} \)
- Rock bridge intensity \( R_{21} = \frac{\sum \text{intact rock bridge trace lengths}}{\text{window surface area}} \)
- Blast-induced damage intensity \( B_{21} = \frac{\sum \text{blast-induced fracture trace lengths}}{\text{window surface area}} \)

The digital trace maps capture more detail than the initial field survey by applying a smaller cut-off length than the field window maps, of approximately 5 cm. The digital trace maps also include traces of brittle intact rock fractures that are impractical to measure and record individually using conventional field scanline or window methods.

The two trialled methods for intact rock bridge tracing applied the following guidelines, as introduced in Chapter 3, Section 3.3:

1. **Rock Bridge Delineation Method 1:** In the first protocol, rock bridges are traced only between pre-existing discontinuities of similar sets, preferentially as parallel in-plane rock bridges.
   a. For example, if two conjugate joint sets are identified, nominally called J1 and J2, then rock bridges are preferentially traced separately for each set. Rock bridges for set J1 are traced between the tips of non-persistent joints belonging to set J1, and likewise for set J2.

2. **Rock Bridge Delineation Method 2:** In the second protocol, rock bridges are traced between all adjacent discontinuities, including blast-induced fractures, dissecting the window into an assembly of discrete blocks with continuous perimeters.
The two methods for calculating fracture intensity $P_{21}$ were designed to examine the influence of including or excluding irregularly-shaped blast-induced brittle fractures. In $P_{21}$ calculation Method 1, all fractures are included in the summation of trace lengths, including irregular blast-induced brittle fractures.

In $P_{21}$ calculation Method 2, only discontinuities interpreted as pre-existing before blasting were included in the trace length summation. Thus $P_{21}$ (Method 1) can be related to $P_{21}$ (Method 2) and blast damage intensity $B_{21}$ as follows:

$$ P_{21} \text{(Method 1)} = P_{21} \text{(Method 2)} + B_{21} $$

Just as the records of naturally-formed brittle fractography features at the Chief helped to reveal potential modes of intact rock bridge failure in a natural granite slope, the traces of irregular brittle features in blast-damaged slopes may help to characterise the influence of blasting in the extension of pre-existing discontinuities, the formation of new cracks, and the destruction of intact rock bridges.

4.4.3.1. Summary of Intensity Measurements

Table 4 - 4 summarises the rock mass quality estimates and trace intensity measurements for the six digital window maps, and Figures 4-15 to 4-20 present the overall fracture trace maps showing discontinuities and blast-induced damage. Figure 4 - 15 presents a legend of the trace convention used in digital trace mapping.

**Trace Map Legend**

- Metasedimentary foliation / bedding discontinuities
- Conjugate joints, mostly from sets R1 and R2
- Conjugate joints, mostly from sets R3 and R4
- Blast-induced intact rock fractures

**Figure 4 - 15: Trace mapping legend used for Jwaneng digital trace maps.**

The results for all windows, except for W-04, indicate larger $R_{21}$ values result from Method 2, where rock bridges are traced between any nearest adjacent discontinuity tips, irrespective of orientation or belonging to similar discontinuity sets. The result
seems intuitive: with Method 1, rock bridges are preferentially traced between discontinuities of similar sets, and thus the R_{21} value is more likely to reflect in-plane rock bridge content for specific joint sets.

With Method 2, where rock bridges are traced between any pre-existing discontinuities or blasting-induced fractures, the R_{21} value is more likely to reflect out-of-plane rock bridge content.

**Table 4 - 4: Areal intensity of inferred rock bridge traces (R_{21}) and blast-induced fracture intensity (B_{21}) derived from window maps.**

<table>
<thead>
<tr>
<th>Window</th>
<th>Surface Area (m²)</th>
<th>Local GSI (±5)</th>
<th>P_{21} Method 1 (m⁻¹)</th>
<th>P_{21} Method 2 (m⁻¹)</th>
<th>R_{21} Method 1 (m⁻¹)</th>
<th>R_{21} Method 2 (m⁻¹)</th>
<th>B_{21} (m⁻¹)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W-01</td>
<td>5.7</td>
<td>70</td>
<td>9.9</td>
<td>5.3</td>
<td>1.9</td>
<td>2.9</td>
<td>4.6</td>
</tr>
<tr>
<td>W-02</td>
<td>5.1</td>
<td>60</td>
<td>14.9</td>
<td>5.0</td>
<td>3.2</td>
<td>3.9</td>
<td>9.9</td>
</tr>
<tr>
<td>W-03</td>
<td>9.9</td>
<td>65</td>
<td>8.1</td>
<td>4.4</td>
<td>1.6</td>
<td>2.8</td>
<td>3.7</td>
</tr>
<tr>
<td>W-04</td>
<td>7.2</td>
<td>55</td>
<td>15.9</td>
<td>8.1</td>
<td>5.9</td>
<td>5.3</td>
<td>7.8</td>
</tr>
<tr>
<td>W-05</td>
<td>8.6</td>
<td>50</td>
<td>11.0</td>
<td>6.0</td>
<td>3.9</td>
<td>4.5</td>
<td>5.0</td>
</tr>
<tr>
<td>W-06</td>
<td>12.2</td>
<td>60</td>
<td>9.9</td>
<td>2.8</td>
<td>1.3</td>
<td>3.4</td>
<td>7.1</td>
</tr>
</tbody>
</table>

The results demonstrate that the inclusion of irregular blast-induced fractures greatly increases measurements of fracture intensity P_{21}, as the estimates of P_{21} calculated with Method 1, which includes blast damage, are consistently at least double the value of estimates derived using Method 2, which only considers pre-existing discontinuities.
Figure 4 - 16: Window W-01 with discontinuity traces overlain; metre stick for scale.

Figure 4 - 17: Window W-02 with discontinuity traces overlain; metre stick for scale.
Figure 4 - 18: Window W-03 with discontinuity traces overlain; metre stick for scale.

Figure 4 - 19: Window W-04 with discontinuity traces overlain; metre stick for scale.
Figure 4 - 20: Window W-05 with discontinuity traces overlain; metre stick for scale.

Figure 4 - 21: Window W-06 with discontinuity traces overlain; metre stick for scale.
Figure 4-22: Scatter plot of $P_{21}$, $R_{21}$, and $B_{21}$ versus local GSI for each window map.

Figure 4-22 shows a scatter plot of the combined trace intensity measurements for all windows versus local GSI. The results show two trends: (1) the difference between $P_{21}$, $R_{21}$, and $B_{21}$ tends to remain similar across the range of field-estimated local GSI values; and (2) Increases in GSI tend to correlate with increase in intensity for all trace sets, with one apparent exception to the trend, occurring at GSI $\approx 60$. The exception is marked by a discontinuity occurring at GSI $\approx 60$. In general, the windows with GSI $< 60$ tend to have higher overall intensity values than the windows with GSI $> 60$. The result may occur partially because as a rock mass becomes more massive (i.e. higher GSI, there may be fewer discontinuities (and thus fewer rock bridges) visible for trace mapping, thus yielding lower intensity values.

In Section 5, the combined results of field mapping, digital trace mapping and photogrammetry are discussed with reference to discontinuity persistence; block size and shape; and implications for the role of intact rock bridges in the East wall.
4.4.4. **Ground-Based Photogrammetry**

This section discusses results from mapping of two ground-based photogrammetry surveys. The first survey was undertaken from long-range (i.e. shooting distance ≈ 1 km), and covers a large section of the East wall. Photographs for the long-range survey were captured from a series of camera stations along the crest and ramps of the West wall of the pit, using a telephoto lens set at focal length \( f = 100 \text{ mm} \) (Figure 4 - 23).

![Figure 4 - 23: Plan view of camera stations used in long-range photogrammetry survey (\( f = 100 \text{ mm}; \text{distance} = 1 \text{ km} \).](image)

Mapping of the long-range survey was not restricted to specific windows; instead prominent structural features were mapped wherever they were observed within the 3-D terrain model of the East wall. The long-range survey was intended to characterise major structural features and to provide an “overall slope” view of predominant attitudes of foliation and major tectonic structures.

The second survey was undertaken for a sub-section of the lower East wall from close range (shooting distance ≈ 50 m), also using a telephoto lens fixed at \( f = 100 \text{ mm} \), from a series of camera stations positioned approximately at the toe of the East wall, at local elevation +870 m (Figure 4 - 24).
Figure 4 - 24: Plan view of camera stations used in close-range photogrammetry survey, with inset showing image matching points used to construct 3-D terrain models ($f = 100$ mm; distance $\approx 50$ m).

Mapping of the close-range survey provides a higher resolution picture of local slope conditions at the toe of the East wall. The window area mapped in the close-range survey is shown in the inset in Figure 4 - 25.

By using a smaller cut-off limit for the discontinuity survey, a larger inventory of discontinuity measurements is obtained. The close-range survey thus giving a more detailed picture of (1) local step-path geometry; (2) the spatial distribution and orientations of blast-induced and pre-existing release structures; and (3) bedding thickness, block size and block shape within the quartzitic shale unit.
Figure 4.25: Two photogrammetry models of the East wall, with circular discs fitted to selected discontinuity planes; foliation is shown in green and release surfaces are shown in red.
4.4.4.1. Long Range Survey Results

The discontinuity mapping data from the long-range photogrammetry survey are similar to the observations from current and historical field mapping. The most prominent structural feature is the northwest-dipping foliation. Clusters of release surfaces comprising blast-induced fractures and pre-existing joints also occur. Figure 4 - 26 presents a stereographic plot of discontinuities mapped in the long-range photogrammetry survey (f = 100 mm; distance = 1 km).

Three major sets are identified in the long range survey, including the quartzitic shale foliation, and joint sets R2 and R3. Sets R2 and R3 correspond approximately with the same sets identified in the initial field mapping analysis (Section 4.2). Table 4 - 5 summarises the orientation data for each major discontinuity set, including sample size and Fisher dispersion coefficient, $K$. Larger $K$ values indicate more tightly-clustered sets; smaller $K$ values indicate more dispersed sets.

![Figure 4 - 26: Stereographic plot of discontinuity poles from long-range photogrammetry survey (f = 100 mm; distance = 1 km).](image-url)
Table 4 - 5: Summary of major set orientation data from long range ($f = 100$ mm; distance = 1 km) photogrammetry survey.

<table>
<thead>
<tr>
<th>Set</th>
<th>Mean Dip (°) [Standard Deviation]</th>
<th>Mean Dip Direction (°) [Standard Deviation]</th>
<th>Sample Size, n</th>
<th>Fisher K</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foliation</td>
<td>35 [10.7]</td>
<td>313 [10.8]</td>
<td>118</td>
<td>45.1</td>
</tr>
<tr>
<td>R2</td>
<td>84 [4.4]</td>
<td>104 [52.2]*</td>
<td>25</td>
<td>83.5</td>
</tr>
<tr>
<td>R3</td>
<td>77 [6.9]</td>
<td>189 [11.8]</td>
<td>40</td>
<td>37.2</td>
</tr>
</tbody>
</table>

*NOTE: The large standard deviation in dip direction for set R2 is a result of the set including conjugate structures; two groupings occur with opposite dip direction. Instead of standard deviation, the Fisher K value should be used in order to characterise the degree of “tightness of clustering” of the poles within set R2.

Table 4 - 6 summarises the persistence data for the major discontinuity sets, expressed as the diameter of fitted planes. The foliation has the highest average persistence (approximately 28 m), however set R3 contains the most persistent discontinuity overall (177 m; Figure 4 - 27). A histogram of fitted discontinuity plane diameters from the long-range survey is presented in Figure 4 - 28.

Table 4 - 6: Summary of fitted plane diameter data from long-range photogrammetry survey ($f = 100$ mm; distance = 1 km).

<table>
<thead>
<tr>
<th>Set</th>
<th>Persistence / Diameter of Fitted Planes (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum</td>
</tr>
<tr>
<td>Foliation</td>
<td>4.9</td>
</tr>
<tr>
<td>R2</td>
<td>5.4</td>
</tr>
<tr>
<td>R3</td>
<td>2.2</td>
</tr>
</tbody>
</table>
Figure 4 - 27: Overall slope view of East wall from preliminary survey of major structural features mapped in the long range photogrammetry survey (f = 100 mm; distance = 1 km).

Figure 4 - 28: Histogram of fitted discontinuity diameter, derived from long range photogrammetry data; f = 100 mm; distance ≈ 1 km.
The sample population from the long-range survey indicates a maximum discontinuity diameter of 177 m, a minimum of 2.2 m, and an average value of 22.3 m. The overall distribution of discontinuity plane diameters may conform well to a negative exponential function, however the sample population indicates that discontinuities smaller than approximately 15 m are under-sampled in the long-range survey. This result reflects the intentional focus of the long-range survey on mapping the overall slope to identify major structural trends, and thus the intentionally imposed truncation bias. Thus, very high persistence structures are preferably sampled over smaller ones, in order to characterise the predominant rock mass structure. From Chapter 3, Section 5.1: the approximate ground pixel size for the long-range survey may be calculated as follows:

\[
\text{Ground Pixel Size} = \text{Pixel size on face} = \frac{d}{f} \times (\text{pixel size on image sensor})
\]

\[
\text{Ground Pixel Size} = \text{Pixel size on face} = \frac{1000 \text{ m}}{0.1 \text{ m}} \times (6.41 \times 10^{-6})
\]

\[
\text{Ground Pixel Size} \approx 6.4 \text{ cm}
\]

Although the ground pixel size is less than 10 cm, features smaller than 2.2 m are cut-off intentionally in order to increase emphasis on the characterization of major structures. The R3 discontinuity with persistence of 177 m is a major dyke feature (Barnett, 2012, personal communication), and probability originated during one of the four local deformation episodes which produced the modern structural environment. The preliminary results from the long-range survey suggest that tension fractures including major joint sets may tend to occur parallel to major tectonic structures.

The negative exponential function for discontinuity persistence can reasonably estimate the frequency of persistence values up to approximately 60 m in persistence; however extremely persistent features such as the major R3 discontinuity may be under-predicted by the negative exponential function. The results emphasize the need for thorough site investigation in order to identify discrete instances of extremely persistent structures. Structures within the top 1% of persistence values should be identified discretely through field investigation, rather than treated statistically by a best-fit function.
4.4.4.2. Close Range Survey Results

The close range photogrammetry survey was carried with a focal length of \( f = 100 \text{ mm} \) from a shooting distance of approximately 50 m; as a result, the ground pixel size of 3-D terrain models is much higher than that of the long range survey:

\[
\text{Ground Pixel Size} = \frac{\text{Pixel size on face}}{f} \times (\text{pixel size on image sensor})
\]

\[
\text{Ground Pixel Size} = \frac{50 \text{ m}}{0.1 \text{ m}} \times (6.41 \times 10^{-6})
\]

\[
\text{Ground Pixel Size} \approx 3.2 \text{ mm}
\]

The higher resolution imagery allows for a smaller cut-off length for discontinuity survey, and thus a larger inventory of poles was collected (Figure 4 - 29).

Figure 4 - 29: Stereographic plot of discontinuity poles from long-range photogrammetry survey \((f = 100 \text{ mm}; \text{distance } = 50 \text{ m})\).

In addition to thorough spot-mapping of discontinuities throughout the 3-D terrain model, discontinuities were also fitted to step-path segments, measured along cross-sections oriented perpendicular to the long-axis of the pit, with 10 m spacing. Although the orientation of step-path segments accurately reflects local discontinuity orientation, the length of step path segments does not directly reflect true discontinuity persistence,
because the discontinuities will extend back into the slope face. Step-paths on the slope face are the surface expression of sub-surface discontinuity structure, but the true extent of the major discontinuities is hidden inside the rock mass (Figure 4 - 30); this effect is related to f-bias, which states that exposed discontinuity traces are random chords formed by the intersection of ellipsoid discontinuity surfaces with an outcrop. The following section (Section 4.4.3) focuses specifically on the analysis of step-path geometry.

![Figure 4 - 30: Conceptual illustration of a 2-D step path comprising a sliding set and a release set. Step segment length may not reflect true discontinuity persistence.](image)

Pole concentrations in the stereographic plot suggest similar orientation of major discontinuity sets including foliation, R2 and R3. However an additional set, denoted R5, occurs in the southeast quadrant of the stereonet, where other mapping data did not indicate any significant concentration of poles. However, the lower K value of R5 compared to the other sets (K = 10.7 rather than K ≈ 25 to 30) suggest that R5 may be a minor set, or may be localized to the area of the close-range survey.

The region defining set R5 corresponds to the “blind zone” where almost no poles occur in the field mapping data. Although the blind zone may be influenced by truncation bias and limited sampling area, the results clearly emphasize the need for survey from multiple observation scales, exposure locations and viewing directions in order to limit the influence of occlusion, orientation bias, censoring and truncation. Table 4 - 7
summarises the orientation data for each of the major discontinuity sets, including sample size and Fisher dispersion coefficient, $K$.

**Table 4 - 7: Summary of discontinuity set orientation data identified from long range photogrammetry survey ($f = 100$ mm; distance = 1 km).**

<table>
<thead>
<tr>
<th>Set</th>
<th>Mean Dip (°) [Standard Deviation]</th>
<th>Mean Dip Direction (°) [Standard Deviation]</th>
<th>Sample Size, n</th>
<th>Fisher K</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foliation</td>
<td>37 [12.8]</td>
<td>311 [15.3]</td>
<td>725</td>
<td>26.4</td>
</tr>
<tr>
<td>R2</td>
<td>78 [7.3]</td>
<td>104 [13.1]</td>
<td>287</td>
<td>30.8</td>
</tr>
<tr>
<td>R3</td>
<td>78 [7.0]</td>
<td>174 [12.2]</td>
<td>87</td>
<td>34.4</td>
</tr>
<tr>
<td>R5</td>
<td>78 [6.9]</td>
<td>309 [48.0]*</td>
<td>262</td>
<td>10.7</td>
</tr>
</tbody>
</table>

*NOTE: The large standard deviation in dip direction for set R5 is a result of the encompassing poles to steeply-dipping discontinuities with a wide range of dip directions; the set window includes the entire southeast quadrant of the stereographic plot, approximately from azimuth 090° to 185°; the preliminary set window for R5 was selected because the pole concentration contours are approximately uniform, varying from approximately 0.5% to 3.4% density. Further mapping may help to better resolve potential contrasts and sub-sets within the broader category of R5 discontinuities.*

Table 4 - 8 summarises the persistence data for the major discontinuity sets, expressed as the diameter of fitted planes. The average persistence for all sets is skewed towards low persistence features (i.e. 1 to 3 m), because the majority of fitted planes correspond to step-path segments collected along specific cross-sections. Step-segment lengths may be a helpful indicator of the typical continuous planar length of a stepped slope failure surface, but they do not reflect the actual maximum persistence.

That maximum persistence for foliation is approximately 44 m, corresponding to the longest uninterrupted length of a bedding discontinuity. The major R3 discontinuity identified in the long range survey (persistence = 177 m) is truncated by the limited window size of the close-range survey, to a maximum persistence value of 47 m. The maximum persistence of set R2 was measured detail visible in the close-range survey.
resulted in more precise recognition of discontinuity termination, contributing to an increased persistence for the largest R2 discontinuity.

Table 4-8: Summary of fitted plane diameter data from close-range photogrammetry survey ($f = 100$ mm; distance = 50 m).

<table>
<thead>
<tr>
<th>Set</th>
<th>Minimum</th>
<th>Average</th>
<th>Maximum</th>
<th>Standard Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foliation</td>
<td>0.1</td>
<td>2.6</td>
<td>44.3</td>
<td>5.1</td>
</tr>
<tr>
<td>R2</td>
<td>0.2</td>
<td>2.6</td>
<td>47.5</td>
<td>4.5</td>
</tr>
<tr>
<td>R3</td>
<td>0.2</td>
<td>2.4</td>
<td>46.6</td>
<td>5.4</td>
</tr>
<tr>
<td>R5</td>
<td>0.1</td>
<td>1.3</td>
<td>9.7</td>
<td>1.5</td>
</tr>
</tbody>
</table>

A histogram of discontinuity diameters from the survey is presented in Figure 4-31.

Figure 4-31: Persistence distribution based on discontinuity diameter, from close-range photogrammetry data; $f = 100$ mm; distance $\approx 50$ m.
The results support the assertion of Sturzenegger (2010), that multi-scale discontinuity survey is needed to limit censoring of extremely persistent features (such as the 177 m-long R3 discontinuity). Furthermore, high resolution, close-range survey is a useful complement to long-range, lower-resolution preliminary surveys, that may result in more accurate persistence measurement for major structural features (such as the increased maximum persistence observed in set R2, in the close-range survey data).

The close-range sample population has a maximum discontinuity diameter of 47.5 m, a minimum of 0.08 m, and an average value of 2.2 m, which is a full order of magnitude smaller than the average diameter measured in the long-range survey (22.3 m), mostly due to sampling bias, occurring due to the intentional focus of the long-range survey on measurement of major structures that characterise the upper-bounds of discontinuity persistence.

The distribution conforms well to a negative exponential function; the peak frequency value occurs for the smallest persistence range, for discontinuities up to 1 m in diameter. The negative exponential function for discontinuity persistence can reasonably estimate the frequency of persistence values up to approximately 5 m in persistence; however extremely persistent features such as the major R3 and R2 discontinuities with persistence of approximately 50 m or more will likely be under-predicted.

### 4.4.4.3. Step-Path Mapping

In addition to spot mapping of important structural features, the close-range photogrammetry survey (inset in Figure 4-25, Section 4.4) was used to investigate step path geometry, by taking cross-sections at 10 m spacing, parallel to the long-axis of the pit, which trends approximately northeast-southwest. The step path geometry was manually mapped, by fitting discontinuity planes to each segment of step path along the slope surface. The results may help characterise the variation in step path geometry along-strike in the East wall.
Length, dip and dip direction were recorded for each planar segment of the step paths (Figure 4 - 32). The relationships between step path geometric parameters were examined in order to identify spatial trends along the mapped portion of the East wall.

**Figure 4 - 32:** Length, dip, and dip direction were recorded for each planar segment of step paths along cross-sections taken at 10 m spacing.

Figure 4 - 33 shows the overlain cross-sections and planes fitted to each step-path segment. In general, step-paths transition from more shallow overall angles, with larger step segment lengths in the southwest of the terrain model, towards steeper overall angles and much smaller step segment lengths in the northeast portion of the slope.
Figure 4 - 33: Step-path segments were manually mapped along cross-sections taken at 10 m spacing, oriented perpendicular to the long axis of the pit to coincide with the overall dip direction of the East wall.
A density-contoured plot of step segment dip angle versus dip direction (Figure 4 - 34) shows two major clusters, suggesting that a bi-modal distribution could be suitable for characterising discontinuity segments. This conclusion is intuitive, and agrees with observations from previous research into step-path slope failures by Ristau (1994); Baczynski (2000, 2008) and others. In general, step-path slope failure are described as comprising of two major discontinuity sets: a shallow basal set and a steeper release or “step-surface” set, which may include a proportion of intact rock bridge failure.

The step-path geometry at Jwaneng is formed by shallow to moderately-dipping foliation, and moderate to steeply dipping step-surfaces comprising blast-induced brittle fractures, and pre-existing tectonic joints. Northwest-dipping segments vary from sub-horizontal (dip ≈10°) to vertical (dip ≈ 90°), potentially due to micro-folding. Northwest-dipping release structures tend to be more concentrated around moderate to steep dip angles ranging from 60° to 90°.

Figure 4 - 34: Dip angle vs. dip direction for combined step-path segment measurements, showing two clusters; data from close-range photogrammetry survey (f = 100 mm; distance = 50 m).
A density-contoured plot of step segment length versus dip angle shows two possible peaks, reaffirming the suitability of a bi-modal distribution for step path segment geometry (Figure 4 - 35).

The first peak occurs for a dip angle of approximately 35°, with a maximum step segment length of 17 m. This peak corresponds to the set of moderately dipping, low to high persistence foliation planes in the quartzitic shale. Although some rare points indicate high persistence step surfaces (> 10 m; ISRM, 1978), the majority of measurements are clustered below the 5 m value, in the range of low to medium persistence. The second peak occurs for dip angle of 82°, with a maximum step segment length of approximately 11 m. This peak corresponds to the set of moderately to steeply dipping release structures, including blast-induced fractures and tectonic joints that may belong to sets R2-R5 identified in the field mapping and photogrammetry data.

Figure 4 - 35: Step segment length vs. dip angle, with density contours illustrating concentration of poles; data from close-range photogrammetry survey (f = 100 mm; distance = 50 m).
With the persistence limits recommended by the ISRM (1978) guidelines overlain, it becomes clear that the majority of step surface segments are of medium persistence or smaller (Figure 4 - 36). Care should be taken in noting that the plot indicates the 2-D length of exposed step segments, visible on the slope surface, which is expected to be much smaller than true persistence due to f-bias, which means that the true size of the discontinuities extending into the rock mass behind the slope is unknown.

Figure 4 - 36: Step segment length vs. dip, with ISRM persistence limits overlain; data from close range photogrammetry survey (f = 100 mm; distance = 50 m).

There are 1222 total step segment measurements; each measurement includes an orientation and a diameter for the fitted plane. Only 5 measurements exceed the ISRM (1978) limit for high persistence, comprising only 0.4% of the total sample population.

There are 97 measurements with medium persistence (3 m – 10 m), comprising approximately 8% of the sample population. The majority of measurements indicate persistence less than 3 m (1120 points, comprising 92% of the sample population).
results suggest that even when extremely persistent structures are present, such as the foliation of the quartzitic shale, exposed step-path geometry may still tend to be dominated by the interconnectivity of low persistence features.

A plot of step segment length versus dip direction (Figure 4 - 37) shows two similar peaks. One peak occurs at a dip direction of 98° (i.e. southeast-dipping), with a maximum step segment length of 11 m. This peak corresponds to the set of southeast-dipping step-path segments, which dip into the East wall. This set is likely dominated by blast-induced release structures, and may also include tectonic joints relating to set R2.

![Figure 4 - 37: Step segment length vs. dip direction for all step-path measurements, with ISRM suggested persistence limits overlain; data from close-range photogrammetry survey (f = 100 mm; distance = 50 m).](image)

The second peak occurs at dip angle of 332° (i.e. northwest-dipping), with a step segment length of 17 m. This peak corresponds to the set of northwest-dipping step path segments, which are likely dominated by sliding surfaces formed by foliation. Despite the
rare occurrence of step-path segments up to 17 m in length, the majority of measurements indicate medium persistence or less. Figure 4 - 38 presents a stereographic plot of pole concentrations for all 1222 step-path segments, with overlays showing the approximate daylight envelopes for bench, inter-ramp, and overall slopes.

Figure 4 - 38: Pole concentrations for step-path mapping data, with daylight envelopes for bench, inter-ramp and overall pit slopes.

The concentration contours are dominated by the foliation and set R2, which effectively “drown out” other potential discontinuity sets. In order to identify other potential release sets, the foliation and set R2 were extracted from the data set, and the remaining data were re-contoured (Figure 4 - 39).

Figure 4 - 39: Pole concentrations for secondary sets, derived by removing poles belonging to the foliation and set R2.
The orientation data for step-path segments are summarised in Table 4 - 9. These sets do not include major structural features that were spot-mapped throughout the close-range photogrammetry survey. Instead, these discontinuity sets indicate the orientation and size of major step-path segments, as identified along specific cross-sections taken perpendicular to the long-axis of the pit. Notably set A has a maximum pole concentration (corresponding to set R5), of approximately 7.5% to 9.5%, up to approximately three times higher than the maximum pole concentration for R5 identified in the generalized close-range discontinuity survey (0.5% to 3.4%; Section 4.4.2).

Table 4 - 9: Summary of main sets identified in step-path segment mapping.

<table>
<thead>
<tr>
<th>Step Segment Set</th>
<th>Mean Dip (°) [Standard Deviation]</th>
<th>Mean Dip Direction (°) [Standard Deviation]</th>
<th>Mean Length (m) [Standard Deviation]</th>
<th>Sample Size, n</th>
<th>Fisher K</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>87 [5.9]</td>
<td>340 [106.1]*</td>
<td>1.0 [1.2]</td>
<td>159</td>
<td>14.7</td>
</tr>
</tbody>
</table>

*NOTE: The large standard deviation in dip direction for set A is a result of the set including conjugate joints with two distinct groupings with opposite dip direction. Instead of standard deviation, the Fisher K value should be used in order to characterise the degree of “tightness of clustering” of the poles within set A.

Step segments associated with foliation set are slightly steeper dipping (40°) than measured in the other mapping data. Set R2 shows similar orientation to other results, dipping steeply towards east-southeast. The secondary step-path sets, A and B, encompass a wide range of moderately to steeply-dipping release structures, which may include members of sets R1, R3, R4, R5. In this analysis, sets A and B are considered separately from other sets, in order to specifically distinguish them as step surface segments (i.e. components of step-path geometry).
Figure 4 - 40 presents a histogram of step segment lengths for the combined sample of 1222 measurements, expressed as the diameter of the fitted planes. The sample population has a maximum length of 17.1 m, a minimum of 0.08 m, and an average of 1.2 m. The results suggest that the step path geometry on the slope surface is dominated by step segments with low persistence (i.e. 1 - 3 m), although the true persistence of exposed discontinuities is much larger, extending behind the slope, into the undisturbed rock mass.

Figure 4 - 40: Overall histogram of step segment lengths for all mapped cross-sections in Jwaneng East wall.

The distribution conforms well to a negative exponential function; the peak frequency value occurs for the smallest step segment length, for values up to 1 m. The negative exponential function can reasonably estimate the frequency of step segment length values up to approximately 3 m; however, the rare occurrence of large step-path segments (e.g. Figure 4 - 33, southwest end) may be under-predicted. Unless a corrected frequency curve is applied to account for major step structures. Care should be taken, however, in noting that a blanket frequency correction factor of \( + n \) discontinuities may also result in overestimates of the frequency of extremely persistent structures.
The step-path geometry is a product of interaction between natural geologic factors and engineering factors such as blasting and excavation methods. Although natural joints, foliation and micro-fabric may play a role in post-mining slope relaxation and incipient brittle failures, the overriding influence of blasting design may dominate the final slope geometry and performance. More data pertaining to blast design, including borehole geometry, explosive loading specifications and detonation timing parameters would be required to better establish causal links between blast design and the resulting slope geometry and performance.

4.4.5. Block Size and Shape

Block size and shape has an important influence on discontinuity-controlled slope instabilities that are characterised by discontinuum interaction between discrete blocks. For this investigation, field assessment of typical failed block sizes and shapes is based on qualitative descriptions of failed debris and in situ block shape and size was assessed based on discontinuity orientations recorded in window maps.

Kalenchuk et al. (2006) proposed a ternary diagram-based methodology for characterising block shape based on variation between platy, elongated, and cubic geometry. The diagram is constructed with two geometry parameters that can be qualitatively measured for each individual block (Figure 4 - 41).

![Figure 4 - 41: Ternary block shape diagram proposed by Kalenchuk et al. (2006), with suggested contours based on field inspection of typical block shapes observed at Jwaneng.](image)
The $\beta$ parameter describes the elongation of a block, and is a function of the median chord length for a block, including all chords that can be drawn across all block face diagonals, internal diagonals, and external edges. Elongated rod-like blocks will approach $\beta$ values of 10, whereas a perfect cube has a $\beta$ value of 0.82 (Kalenchuk et al., 2006).

The $\alpha$ parameter describes the flatness of a block, and is related to the ratio of block surface area to volume. Kalenchuk et al. (2006) noted that slabs have higher surface area to volume ratios than cubes; a normalized $\alpha$ value of 1.0 is assigned to perfect cubes, and $\alpha$ increases to a value of 10 for typical platy rock block geometry.

Although precise $\alpha$ and $\beta$ values may be calculated individually for given blocks, exhaustive block shape characterization is beyond the scope of this thesis. As an alternative, the idealized example blocks shown in Figure 4-41 can be used as a guide for qualitative assessment of typical block shapes observed in the field.

Figure 4-42 presents typical examples of block sizes and shapes observed during window mapping. Block shape tends to be platy-cubic in quartzitic shale, with upper and lower block surfaces defined by foliation, and side surfaces defined by bedding-orthogonal joints and blast-induced fractures. In situ block size in the quartzitic shale is thus expected to be controlled predominantly by bedding thickness (i.e. the spacing of foliation discontinuities).

Figure 4-43 presents typical block sizes and shapes for a pile of failed blocks in quartzitic shale, with the in situ rock mass in the background. Although the layered structure of the in situ rock mass suggests that blocks may be mostly platy, the pile of debris indicates that a diverse range of block shapes occurs. Comminution of the rock mass by blasting has resulted in debris varying across all extremes of the ternary diagram proposed by Kalenchuk et al. (2006), from platy to cubic and elongate. Most blocks within the pictured debris have maximum edge lengths less than 1 m; thus the average block volume in the pile ranges from approximately 0.05 to 0.15 m$^3$. 
Figure 4 - 42: (A) Approximate upper-bound block sizes based on detached block outlines (local GSI ≈ 65-75; local blast damage factor D ≈ 0.7); and (B) Reduced block size and increased intensity of blasting-induced damage (local GSI ≈ 45 – 55; local blast damage factor D ≈ 1); metre stick in centre of each frame for scale.
Figure 4-43: Example of block size and shape for quartzitic shale in situ (top) and interpreted from debris (bottom).
Figure 4 - 44 presents a suggested block size distribution curve for the quartzitic shale in the East wall, based on bench mapping and qualitative assessment of bench debris. Block size is expected to be controlled by bed thickness. Detailed analysis of the photogrammetry data can help to better characterise the variation in bed thickness throughout the wall. Figure 4 - 45 shows an example region extracted from the close-range photogrammetry survey in order to assess apparent bed thickness. Figure 4 - 46 shows the composite image extracted from the 3-D terrain model, with markings to delineate approximate locations of distinct bedding interfaces.

Figure 4 - 44: Suggested block size distribution curve for quartzitic shale, modified after the suggested method of Kalenchuk et al. (2006).

Figure 4 - 45: Overview of close-range photogrammetry survey with sample region extracted for apparent bedding thickness assessment.
Figure 4 - 46: Sample region from close-range photogrammetry survey ($f = 100$ mm, distance ≈ 50m) for assessing apparent bed thickness, in order to better characterise potential in situ block sizes and potential step-path geometry.
4.5. Discussion

4.5.1. Characterising the Slope with Step-Path Domains

Dividing the East wall into step-path domains based on variations in step surface geometry may help to better understand potential step-path instability mechanisms. In the close-range photogrammetry survey, at least two step-path domains can be distinguished. Step-Path Domain 1 occurs in the northeast of the slope, where step path segments are small, and Step-Path Domain 2 occurs in the southeast of the slope, where steps are much larger. The transition between the two domains is indicated by the presence of a large detachment niche indicating a previous wedge failure (Figure 4 - 47).

In Step-Path Domain 1, blasting damage is estimated as severe (i.e. $D \approx 1$), and the rock mass is very blocky to blocky (i.e. GSI $\approx 45 – 55$), with discontinuity spacing typically less than 0.5 m. Typical block volumes are in the range of 0.1 to 0.5 m$^3$ and do not typically exceed 1 m$^3$. Step surfaces are dominated by low-persistence step-path segments, proportional to block size.

In Step-Path Domain 2, large detachment niches indicate past planar failure of pentahedral wedges, with basal sliding on foliation and lateral release on conjugate tectonic structures and blast-induced fractures. The overall rock mass however appears to be less damaged by blasting than Step-Path Domain 1, with little visible evidence of pervasive blast-induced brittle fracturing (i.e. a local damage factor of $D \approx 0.7$ could be applied if the level of disturbance can be more rigorously assured). Although the rock mass is foliated, the foliation spacing is large (commonly in excess of 1 m), and step surfaces indicate that block volumes often exceed 1 m$^3$, corresponding to local GSI in the range of 55 to 65.
Figure 4: Oblique view of step-path mapping area, with two potential step-path domains identified. Detachment niches indicate large past wedge failures in Step-Path Domain 2. Selected cross-sections are numbered along the base of the slope, from 1 to 19.
Despite the appearance of reduced blasting damage in Step-path Domain 2, care should be taken in noting that blast-induced damage is not restricted to visible blast-induced fractures and dilated joints on the slope face. Blast-induced damage will also exist at depth behind the slope face, where shear strength reduction has occurred along foliation discontinuities, induced by infiltration of explosive gasses and reflection of tensile strain waves. In particular, the evidence of major wedge failures in Step-path Domain 2 suggests that the largest foliation step surfaces have been weakened by blasting sufficiently to allow pentahedral wedge sliding to occur.

Figure 4 - 48: Plan view of step-path segment centres mapped on selected cross-sections, from CS-1 in the lower left, to CS-19 in the upper right.

Figure 4 - 48 presents a plan view of the centres of step-path segments for cross-sections 1 to 19. Each point on the plan view represents the Easting and Northing coordinates of a manually mapped step-path segment. Every cross-section has a different length, and thus any comparison of step path geometry must include a factor to normalize for step-path length.
Figure 4 - 49 presents a conceptual geometric method for characterising step-path steepness. The overall angle or steepness of a step path is dependent on both the dip of the step path segments and the height of steeply-dipping steps. The effective length, $L_e$ of a step-path can be expressed as the toe-to-crest distance in a straight line; the total length, $L_T$ of a step path is expressed as the sum of all step-path segment lengths.

![Step-path Steepness: based on dip & height of steps](image)

Figure 4 - 49: Conceptual geometric criteria for characterizing step-path steepness.

Figure 4 - 50 presents a conceptual geometric method for characterising step-path roughness. Step-path roughness can be expressed based on the number $n$ of steeply-dipping steps, the height of steeply-dipping steps, $H$, and the effective path length, $L_e$. Smooth planar surfaces have no steps; roughness progressively increases as more steps are introduced. The normalized $\left( n \times H \right) / L_e$ parameter can be used to express roughness corrected for path length.
Table 4-10 summarises the step-path geometry for cross-sections 1 to 19, in terms of:

1. Overall toe-to-crest angle $\alpha$ for each cross-section
2. Number of segments mapped along each cross-section, $n$
3. Effective path length $L_e$, representing the straight-line toe-to-crest vector distance of each cross-section
4. Total path length $L_T$, representing the sum length of all step-path segments for each cross-section
5. Two ratio parameters combining items 2, 3, and 4:
   a. $L_e / L_T$ is the ratio of effective path length to total path length. For a smooth plane, the value will be 1. As the path becomes more rough, the total path length will become larger than the effective path length, causing the ratio to decrease.
   b. $L_T / n$ is the ratio of total path length to the number of step-path segments, and thus represents the average step-path segment length. Step-paths comprised of large step surfaces will have larger values for the ratio.

Figure 4-50: Conceptual geometric criteria for characterizing step-path roughness
Cross-sections 1 to 10 are in Step-Path Domain 2; cross-sections 11 and 12 are within the transition zone; and cross-sections 13 to 19 are in Step-Path Domain 1.

In Step-Path Domain 1, overall step path angle varies from a minimum of 61° to a maximum of 87°. The length ratio parameter $L_e / L_T$ varies from 0.53 to 0.62. The ratio of total path length to number of step segments $L_T / n$ varies from 0.55 to 1.02, reflecting step-paths that are typically comprised of very low persistence (i.e. $< 1$ m) segments. In Step-Path Domain 2, overall step path angle varies from a minimum of 65° to a maximum of 87°. Although the variation in overall angle is similar to that of Domain 1, the length ratio parameter $L_e / L_T$ is typically smaller, varying from 0.28 to 0.51, reflecting the transition to a rougher step-path geometry comprised of larger step surfaces. The ratio of total path length to number of step segments $L_T / n$ varies from 1.05 to 4.13; reflecting an increase in the average step segment length, with typically medium persistence step surfaces.

Figure 4 - 51 highlights the major step surfaces in Step-Path Domain 2, which strike oblique to the long-axis of the pit, and thus are not perpendicular to the mapped step-paths. The major step surfaces may relate to a conjugate set of tectonic structures; the conjugate orientation of these major discontinuity sets may thus be an important factor contributing to the “zig-zag” geometry of pentahedral blocks on planar sliding surfaces.
Table 4 - 10: Summary of geometric parameters for selected step path cross-sections.

<table>
<thead>
<tr>
<th>Section Number</th>
<th>Overall Angle, $\alpha$ (°)</th>
<th>Number of Segments, $n$</th>
<th>$L_e$ (m)</th>
<th>$L_T$ (m)</th>
<th>$L_e/L_T$</th>
<th>$L_T/n$</th>
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<td>13</td>
<td>11.3</td>
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<td>0.33</td>
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<td>41.8</td>
<td>67.9</td>
<td>0.62</td>
<td>0.90</td>
</tr>
</tbody>
</table>

Overall Step Path Angle < 70°  Overall Step Path Angle < 80°  Overall Step Path Angle > 80°
Figure 4 - 52 illustrates the contrast in 3-D step path geometries for Step-path Domains 1 and 2, and the transition zone, that result in the computed $L_T/n$ values summarised in Table 4 - 10.

Figure 4 - 52: Selected photographs of typical 3-D step-paths for Step-path Domain 1, 2, and transition zone.
4.5.2. Role of Microfabric and Paleostress

In a review of the origin and shear strength of sheet joints in granite, Hencher et al. (2011) observed that the development of macroscopic joints can be influenced by the small-scale fabric of pre-existing microcracks in intact rock. In reviewing studies of rock slopes along the Tuen Mun Highway in Hong Kong, they note that sheet joints tend to develop parallel to the network of microcracks, which were originally created by cooling stresses. Thus, the paleostress regime of the rock mass can influence the development of modern joints and brittle fractures.

By the same mechanism, the stress history of the deformed metasediments at Jwaneng may influence the development of modern layer-bound tension joints and blast-induced fractures. The influence of the historic stress conditions on the development of modern fractures has been studied for decades; Gerber and Scheidegger (1969) proposed to classify natural weathering processes as either:

1. **Exogenous weathering**: occurring due to external processes including action by water and the atmosphere; or
2. **Endogenous weathering**: occurring due to internal stress conditions within a rock mass.

In the context of open pit mining, blasting and mechanical excavation represent an artificial exogenous process. Although the input of energy is exogenous (i.e. from blasting), the comminution of the rock mass may also be influenced by endogenous factors, such as microcrack networks created by past tectonic deformation.

The occurrence and orientation of high persistence (20 m+) steeply-dipping joints (noted in the previous section and pictured in Figure 4 - 51) can be attributed to the four major historic episodes of tectonic deformation and faulting that created the complex configuration of structural domains on site; in particular, the mapped location of Fault H is near the toe of the East wall in the close-range survey area, dividing geotechnical domains 6 and 7a (Barnett, 2009). Tectonic deformation episodes may also have created a fabric of microcracks aligned with the high persistence joints. Thin-section
study of intact rock samples would be required to confirm the presence of a systematic microcrack network.

If a systematic network of microcracks permeates the intact rock, then the development of layer-bound tension joints and even more recent blast-induced release surfaces may preferentially follow the orientation of the microfabric. The influence of microfabric on propagation of blast-induced fractures may be analogous to the three orthogonal weakness planes exploited during quarrying of granite: rift, grain and hardway (Almeida et al., 2006; Chapter 3, Section 7.4). The relationships between macroscopic blast damage, joint formation and tectonic microfabric may help to explain the geometry of blast-induced release surfaces, which are characterised by “zig-zag” patterns that are repeated from metre-scale (Figure 4 - 53) to bench scale (Figure 4 - 54).

At a larger scale, detachment niches throughout the East wall reveal a trend of failure mechanisms resembling pentahedral or complex, stepped wedges with basal sliding along foliation and release along blast-induced fractures, tectonic joints and faults (Figure 4 - 55). The pattern of release surfaces suggests that wedges may be “nested”, such that smaller wedges close to the slope surface are likely to fail shortly after blasting and excavation. Successive near-surface failures may progressively cause larger wedges to release, promoted by interaction of pre-existing structures with blast-induced fractures within the zone of blast damage. Whereas small wedges may have kinematic release along low-persistence blasting-induced fractures, larger wedges are more likely to abut against high persistence or extremely persistent (i.e. > 30 – 50 m) tectonic joints or faults.
Figure 4-53: Blast-induced damage creates kinematic release for planar sliding block failure on undulating foliation planes.
Figure 4 - 54: Bench-scale “zig-zag” release surface pattern, induced by blasting
Figure 4 - 55: Examples of pentahedral and stepped complex wedge failures in the East wall, involving sliding on foliation and release on conjugate tectonic structures and blast-induced damage.
4.5.3. **Role of Persistence**

The field investigations at Jwaneng were carried out at multiple scales ranging from metre-scale bench face maps to long-range photogrammetry survey of the entire East wall, photographed from approximately 1 km distance. The resultant populations of discontinuity persistence measurements reflect the influences of observation scale, length bias, f-bias, censoring and truncation in face mapping of open pit slopes.

The length scale of discontinuities within a rock mass ranges across several orders of magnitude, from sub-millimetre size microcracks and atomic-scale crack surface interactions (e.g. Diederichs, 1999) to kilometre-scale faults and crustal-scale structural detachment features (e.g. Ortega et al., 2006). Comparison between field mapping and photogrammetry observations from the current investigation can help to identify the controlling length scale and range of persistence values which may most strongly influence potential slope instabilities.

The close-range photogrammetry survey shows that most rear and lateral-release discontinuities are less than 10 m in persistence; although some extremely persistent release structures of tectonic origin are observed (up to 47 m in the close range survey), the average persistence is only approximately 2.2 m. Analysis of step-path geometry (Section 4.4.4.3) shows that the largest step path segments are up to 17 m in length, consisting of exposed foliation planes. The average step path segment, however, is only 1.2 m long, indicating that step-path geometry is comprised mostly of low persistence structures (ISRM, 1978) likely comprising blast-induced fractures and tension joints (e.g. Figure 4 - 53).

The largest persistence measurement, a 177 m-long structure belonging to set R3 was made in the long-range photogrammetry survey. Foliation in the quartzitic shale unit is the overall most persistent structural feature. Although the largest fitted plane to a foliation discontinuity measured 70 m, overall assessment of geological structure throughout the East wall (see panorama, Figure 4 - 4 Section 4.3) suggests that foliation persistence may be in the order of hundreds of metres of more. Distinct terminations and offsets of foliation occur only on major structural features including inter-ramp scale faults and dykes, or geological contacts.
Past investigations concluded that foliation is mostly tight and closed in deep, undisturbed rock mass (Tunono et al., 2011; SRK 2010, 2009), however the current investigation confirms that near-surface foliation has frequently been opened and disturbed by blasting and mining-induced stress perturbations. The transition from open and dilated foliation planes near the slope surface, to tight closed discontinuities at depth, may be gradational or abrupt. In either case, potential for deep-seated, inter-ramp or overall slope failures involving basal sliding on foliation would be reduced by the added strength of tight foliation within the deeper, undisturbed rock mass.

The most evident slope instability hazards in the East wall are characterised by progressive bench-scale block sliding. Ongoing gravity-driven, sub-critical crack growth of blast-induced fractures may lead to periodic formation and linkage of new release surfaces, providing kinematic release for planar sliding failures. The observed range of discontinuity persistence values observed in field mapping, digital photogrammetry mapping, and 3-D step-path characterization, suggests that planar sliding failures tend to involve low persistence structures.

No direct evidence indicates specific instances of incipient inter-ramp or overall slope failure. However, the potential for (1) unseen adverse structures within the deep rock mass; (2) incipient shearing on deep sliding surfaces, or (3) progressive destruction of intact rock bridges leading towards kinematic release would require additional characterization and geomechanical modelling.

4.5.4. Role of Intact Rock Bridges

Based on the structural geology of Jwaneng, the most common mode of slope failure in the East wall is likely to be characterised by translational planar or wedge sliding along foliation discontinuities, with step-path sliding lateral and rear release on blast-induced fractures or pre-existing structures.

The sequence of sediment deposition, burial and genesis has produced continuous bedding contacts; the presence of extensive bridges of intact rock between coplanar, non-persistent foliation discontinuity tips is improbable. However, intervals of tight or “over-closed” (Barton, 2007) foliation may separate discontinuity segments that have been opened by blasting or mining-induced slope relaxation. Inter-ramp or overall slope
stability analysis involving deep-seated failure could consider the cohesive strength benefit of over-closed or tight intervals of foliation as an “effective rock bridge” length, but with lesser shear strength than true intact rock bridges and based instead on the results of laboratory direct shear tests on tightly-closed foliation discontinuities.

Although co-planar intact rock bridges are unlikely to influence planar sliding, non-coplanar intact rock bridging may be an important influence in the formation of undulating or non-planar brittle fractures that cross layer boundaries and may contribute to the formation of step-path type failures. Close-range observations from window mapping and 2-D trace mapping suggest that intact rock bridges between the tips of non-persistent release surfaces may play a role in arresting bench-scale sliding failures on near-surface foliation that has been dilated by blasting.

Figure 4 - 56 shows an example using window W-04 from the field mapping. The high intensity of blast-induced damage ($D = 1$) has resulted in new brittle cracks (red irregular lines), which cross-link the foliation discontinuities, creating numerous potential step-paths for slope failure; one such path is highlighted.

**Figure 4 - 56: Potential sliding surface in W-04, with small intervals of intact rock separating non-persistent release structures.**
Cook and Underwood (2001) and Thomas and Pollard (1993) conducted a numerical experiment to investigate the influence of strata-bound fractures in sedimentary rocks on rock mass deformation. In particular, they investigated the role of localized sliding and opening of cross-bedding fractures. They found that the strength of strata contacts strongly controls the type of resulting fracture intersection (i.e. whether the cross-strata fracture will terminate at the bedding contact, cut across the bedding contact, or “step along” the bedding contact horizontally). Propagation of cross-bedding fractures is thus dependent on the strength of bedding interfaces.

The research of Thomas and Pollard (1993) shows that two cases describe the propagation or termination of cross-bedding fractures:

- **Case 1 - Termination:** termination is favoured at weak bedding contacts; thus, new kinematic release fractures are more likely to terminate on blast-disturbed, dilated foliation discontinuities, restricting the formation of new multi-layer release surfaces.

- **Case 2 – Continued Propagation:** fractures will tend to propagate straight through strong contacts; thus the formation of new joints crossing bedding boundaries may be more favoured at depth, within the undisturbed rock mass, where confining stress is high and the foliation is tightly-closed and strong.

In reality, sedimentary contacts often fall between Case 1 and Case 2, resulting in local opening on the contact and production of new fractures. Figure 4 - 57 presents a conceptual diagram where sedimentary bedding interface strength results in a mixture of propagation and termination of cross-bedding fractures. At shallow depth, confining stresses are low and fracture termination is more likely, with lower layer-parallel tensile stress imparted by the weight of overburden. However, blast-induced damage is more severe at shallow depths near the slope surface, increasing the number and extent of weak interfaces and high-angle cross-bedding fractures that may provide kinematic release. In contrast, thicker sedimentary layers and greater burial depths result in larger layer-parallel tensile stresses, which may promote continued propagation of fractures and stepping of fractures across bedding.
Preliminary observations from this investigation suggest that intact rock bridges may occur in the near-surface rock mass as minor intervals (i.e. < 5% of projected 2-D step-path lengths) of intact rock between the tips of blast-induced intact rock fractures and other potential release surfaces. Intact rock bridges may act to resist bench-scale sliding failures on dilated foliation, however their role in inter-ramp or overall slope failures requires additional information on the persistence of joints and faults at depth within the undisturbed rock mass. This information will require further development of remote sensing techniques to incorporate geophysical methods and borehole investigation data.
5. Field Investigation at Diavik, NWT Canada

5.1. Mine Site Overview

Diavik diamond mine is located on Lac de Gras in Northwest Territory, Canada, 300 km northeast of the territory capital, Yellowknife. The two open pits, A154 and A418, are partially surrounded by a waterproof dyke that allows mining to proceed below the former lakebed. Production from the A154 pit began in 2003, and the A418 began production in 2008. Trial underground operations began in 2009 beneath the A154 pit, and mining will transition to fully underground operations by the end of 2012.

This investigation focuses on the A418 pit and in particular the A418 Southeast wall. At the time of the field investigation the depth of the pit was approximately 190 m. The ultimate pit design proposes a final depth of 240 m before transitioning to sub-level caving operations (Infomine, 2010).

5.2. Geological Setting

Diavik is located in the Slave Province of the Precambrian age Canadian Shield, and the property contains four diamond-bearing kimberlite pipes that have been dated to approximately 55 Ma (Roscoe and Postle, 2005). The A154 pit contains two pipes: A154 North and A154 South; the A418 pit contains only one pipe, the eponymous A418 pipe. A fourth kimberlite pipe, A21, occurs to the south of the A418 pit, but at the time of the field investigation during July 2011, had not yet been developed for production.

The host rock is mostly Archean age, comprising deformed and metamorphosed greywacke-mudstone metaturbidites that have been extensively intruded by granite and granodiorite. The granitic rocks are intruded locally by pegmatite dykes, and also by extensive diabase dyke swarms (Roscoe and Postle, 2005). Previous investigations have described the A154 slopes as comprising a predominantly good quality, strong, moderately fractured granite rock mass with UCS typically greater than 100 MPa (Moffitt
et al., 2007). Diabase dykes tends to be more blocky and jointed than the host granite and metasediments; consequently, the network of diabase dykes may act as important conduits for groundwater flow (Golder Associates 2010, 2007; Roscoe and Postle, 2005).

Regional structure is dominated by two faults that intersect the pits, both striking approximately Northeast-Southwest. The four kimberlite pipes on the property are also aligned in a Northeast-Southwest direction, indicating that regional-scale deformation may have influenced the emplacement of the kimberlite intrusions (Stubley, 1998). The A154 pit is cut by the Dewey’s Fault, which is steeply-dipping, and intersects both the A154N and A154S kimberlite pipes.

The A418 pit is cut by the steeply-dipping A418 fault, which intersects the Southwest and Northeast walls of the pit. In the Southwest wall, the A418 fault is associated with a 10 to 20 m thick zone of oxidized and highly weathered rock, and an increase in fracture intensity. Several extremely persistent (i.e. 100 m+) multi-bench joints occur sub-parallel to the fault. In the Northeast wall, the A418 fault zone is less thick (<10 m), and is sub-vertical oriented.

The A418 pit is divided into three geological domains with different metasediment content (Figure 5 - 1A). Domain 1 occurs in the Northeast, and is interpreted to contain less than 10% metasediment; Domain 2 occurs in the Southeast and the Northwest, and is interpreted to contain between 10% and 50% metasediment. Domain 3 occurs in the Southwest, and contains less than 10% metasediment. There are three Design Sectors (Figure 5 - 1B) based on the interpreted geology and also on pit geometry including face orientation, ramp configuration and proximity to the waterproof dyke and Lac du Gras (Golder Associates, 2010). This field investigation uses mapping observations made throughout the pit to help characterise the variation in discontinuity persistence and possible intact rock bridge content, with specific reference to potential instability mechanisms within the Southeast wall, in Design Sector 2.
Figure 5-1: (A) Geological domains around A418 pit; and (B) Design Sectors in the A418 pit, with two interpreted design cross-sections through the Southeast Wall (Golder Associates, 2010, by permission).
5.3. Geotechnical Characterisation

This section summarises the slope geometry of the A418 Southeast wall, and also presents relevant geotechnical parameters for the rock mass and discontinuities, derived from previous investigations. Reliable estimates of intact rock strength and deformability, rock mass characteristics and discontinuity shear strength and stiffness are necessary to help estimate the intact rock bridge shear strength and to assess potential complex mechanisms of slope instability.

5.3.1. Slope Geometry and Major Structures

The overall slope of the A418 pit at the time of investigation was approximately 190 m high, with 30 m high benches. Due to safety considerations and access constraints, the photogrammetry models cover a section of slope 150 m high that does not fully extend to the bottom of the pit. Figure 5 - 2 presents an as-built cross-section, A418-CS1, through the Southeast wall.

![Figure 5 - 2: Selected cross-section A418-CS1 through Diavik A418 Southeast wall, looking Northeast (Air photo from Rio Tinto, 2009).](image)

Section A418-CS1 was extracted from a 3-D terrain model derived from an $f = 100$ mm photogrammetry survey. The section is located along approximately the same azimuth
as design Section 150 from the previous investigations (Figure 5 - 1). The details of the photogrammetry survey procedures, including camera station positioning, lens focal length, and shooting distance, are further discussed in Section 4.4.

The cross-section shows that bench face angles are steep to sub-vertical (80-90°). The main haul ramp descends in a clockwise spiral, from the crest of the pit in the north, across the Southeast wall towards the base of the pit. At the location of the cross-section, the inter-ramp angle is approximately 51° in the lower slope below the haul ramp, and 45° in the upper slope, above the haul ramp. The overall slope angle is approximately 40°.

Figure 5 - 3 presents two annotated panoramic images of the West wall (top image) and East wall (bottom image) of the pit, showing major structural features and selected instances of observed bench-scale failure mechanisms:

- Failure of poor quality, oxidised and highly weathered rock mass occurs in the A418 fault zone
- Groundwater seepage is frequently observed near undulating, sub-horizontal discontinuities, and the rock mass surrounding the diabase dykes
- Several bench-scale wedge failures occur towards the north end of the West wall, and in the lower section of the Southeast wall
  - Partial wedges occur throughout the pit, formed by moderately to steeply-dipping conjugate joints.
- Brittle blasting-induced cracks form steps between pre-existing discontinuities, however, backbreak is typically localized and not severe
  - Wall control is typically excellent; benches are mostly free of debris and sub-vertical bench faces are maintained.
- The groundwater regime is controlled by fracture flow, and shows some evidence of compartmentalization. Seepage zones (visible as dark sections of wet wall rock) frequently terminate at abrupt boundaries, where flow is apparently restricted by steeply dipping joints or fault-associated fractures.
- Extensive planar exposures of major, steeply-dipping discontinuities daylight in bench faces in the Southeast wall, with dimensions in the order of 100 m x 30 m.
Figure 5 - 3: Annotated panorama of West wall (top image) and East wall (bottom image) of A418 pit, with major structures highlighted. Bench height is 30 m for scale.
5.3.2. Rock Mass and Discontinuity Properties

The rock mass in the A418 Southeast wall can be grouped into two major geotechnical units: granite and metasediment. Both rock mass units are generally strong (UCS > 100 MPa) and moderately fractured with permeability controlled by sparsely-distributed, widely spaced discontinuities of enhanced permeability (Chorley et al., 2009). Despite the importance of the groundwater inflow from Lac de Gras, coupled hydro-mechanical analysis of discontinuity-controlled instability involving excess pore pressure is beyond the scope of this thesis. However, estimates of rock mass and discontinuity mechanical properties are necessary to better understand the strength benefits of intact rock bridges in potential slope failures.

Elmo et al. (2011) carried out a series of numerical analyses of the A154 pit Northwest wall using both the continuum finite element code Phase\textsuperscript{2}, and the hybrid FEM-DEM code ELFEN. Table 5 - 1 summarises their published rock mass properties and Mohr-Coulomb failure criteria for granite, kimberlite, and major joints.

Table 5 - 1: Summary of Mohr-Coulomb strength parameters for joint shear strength, based on field assessment and laboratory shear tests on rock joints (Elmo et al., 2011).

<table>
<thead>
<tr>
<th>Geological Unit</th>
<th>( E_{RM} ) (GPa)</th>
<th>UCS (MPa)</th>
<th>Cohesion, ( c ) (MPa)</th>
<th>Friction Angle, ( \phi ) (°)</th>
<th>Tensile Strength, ( \sigma_T ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Granite</td>
<td>25.3</td>
<td>130</td>
<td>3.56</td>
<td>53.2</td>
<td>1.18</td>
</tr>
<tr>
<td>Kimberlite</td>
<td>6.0</td>
<td>25</td>
<td>2.58</td>
<td>44.0</td>
<td>-</td>
</tr>
<tr>
<td>Joints</td>
<td>-</td>
<td>-</td>
<td>0.025</td>
<td>42.0</td>
<td>-</td>
</tr>
</tbody>
</table>

Due to computational requirements and design considerations, the FEM-DEM models only explicitly included joints with high persistence or larger (i.e. > 10 m; ISRM, 1978). Continuum material parameters for the granite rock mass were based on a GSI of 75, corresponding to blocky to massive structure.
Karami et al. (2007) also investigated the potential for step-path failure in the Northwest wall of the A154 pit, using the discrete element code \textit{UDEC}. Table 5 - 2 presents a summary of their published Mohr-Coulomb failure criteria for granite rock mass and major discontinuities including a shallow-dipping basal sliding set, and a steeply-dipping step surface set.

\begin{table}
\centering
\caption{Summary of Mohr-Coulomb strength parameters for joint shear strength, based on field assessment and laboratory shear tests on rock joints (Karami et al., 2007)}
\begin{tabular}{l|cc|cc}
\hline
Geological Unit & Base Properties (Residual) & & Scaled Properties (5\% rock bridge) & \\
 & Cohesion, c (MPa) & Friction Angle, $\phi$ (°) & Cohesion, c (MPa) & Friction Angle, $\phi$ (°) \\
\hline
Granite & 2.3 & 60 & - & - \\
Joints & & & & \\
\hspace{0.5cm} Shallow Dip & 0.025 & 35 & 0.140 & 37 \\
\hspace{0.5cm} Steep Dip & 0.025 & 35 & 0.025 & 35 \\
\hline
\end{tabular}
\end{table}

Base properties for joints conservatively use residual strength values. Scaled discontinuity cohesion and friction angle values were derived using the apparent strength method of Jennings (1970) in order to account for the influence of intact rock bridges along the failure surface. Based on the results of Moffitt et al. (2007), intact rock bridges were considered only along the basal sliding discontinuities; the steeply-dipping joint set is expected to be continuous over the scale of the global failure surface, with no rock bridges.

The scaled properties are based on an intact rock bridge content of 5\%, which was derived through an iterative DFN study of potential stepped failure paths in the Northwest wall (Karami et al., 2007). The DFN study used field mapping data to generate multiple stochastic realizations of the 3-D fracture network. The cumulative results from all the DFN realizations suggested with 95\% confidence that intact rock bridge content should exceed 5\% on the shallow-dipping basal sliding discontinuity set.
The current investigation thus aimed to gather additional observational evidence for potential intact rock bridge geometry and also overall rock bridge quantity (as a length or area percentage) associated with specific discontinuities or with respect to potential instability mechanisms such as bench-scale wedge failure.

5.3.3. **Local Variation in Rock Mass Quality**

Major structures including the diabase dykes and the A418 fault were observed in this field study to be associated with local decreases in rock mass quality, increases in fracture intensity and potential presence of groundwater seepage. Figure 5 - 4A illustrates how the diabase dyke unit tends to be more fractured and blocky than the host rock, with typically cubic in situ blocks, with edge lengths less than 1 m. The diabase dykes also show evidence of groundwater seepage and weathering. Figure 5 - 4B shows a blasted exposure of diabase dyke and massive granite in the southeast of the RL + 270 m bench, after a potentially unstable bench-scale wedge was removed by secondary blasting. Local inflows of groundwater from two point sources in the diabase dyke continued at rates of approximately 10 L/min over the remaining duration of the field investigation (at least 3 days).

The observation of seepage from outcropping diabase supports the suggestions of Roscoe and Postle (2005) and Chorley et al. (2009) that groundwater flow is governed by fracture flow, and that the flow regime is probably dominated by sparse, widely spaced major structures with enhanced permeability.

Faults intersecting both pits are also important zones of enhanced permeability (Chorley et al., 2009). Figure 5 - 5 illustrates the degradation of the rock mass in the core of A418 fault. Damage is mostly localized within the fault core, which has a local thickness of approximately 3 m to 5 m. The surrounding rock mass is massive, and although some blasting-induced brittle fractures are present, there is little evidence of tectonic damage.
Figure 5 - 4: Diabase dyke in south wall of the pit, RL + 390 m bench (A); and southeast wall, RL + 270 m bench (B).
According to Hoek (2012), a $D$ factor of 1 corresponds to severe damage associated with heavy production blasting. Actual blast damage conditions in the A418 pit are variable. Results from localized bench-scale window mapping are discussed in Section 4.3, where local GSI values are assigned based on rock mass quality and severity of blasting damage at each mapping location. Special attention was given during mapping to search for evidence of blast-induced damage in the form of recent intact rock fractures, as well as extension and dilation of pre-existing discontinuities.
5.4. Field Investigation

5.4.1. Overview

The field investigation of the A418 pit incorporates results from modified discontinuity survey, 2-D digital trace mapping, and ground-based photogrammetry. Observations gathered from locations throughout the entire A418 pit are interpreted with reference to the role of discontinuity geometry, persistence and intact rock bridges on potential slope instability mechanisms. Particular attention was given to searching for evidence of persistent adverse structures within the southeast wall, including: (1) northwest-dipping joints or continuous sub-horizontal “rafts” of metasediment that could act as basal sliding surfaces; and (2) systematic joint sets or major structures that may provide lateral or rear release for inter-ramp or overall slope-scale instabilities.

Section 5.5 presents a discussion of the combined field results with focus on the roles of discontinuity persistence, fracture connectivity, intact rock bridges and blasting-induced damage in inter-ramp and overall slope scale stability.

5.4.2. Modified Discontinuity Survey

This section briefly reviews findings from previous field mapping and borehole drilling undertaken by Golder Associates (2007) and others, and then presents mapping results and relevant observations from the current field investigation for comparison. Previous investigation data include an inventory of over 1500 discontinuity orientation measurements, which have been analysed by others to identify dominant joint sets.

Figure 5 - 6 presents a stereographic plot of the major joint sets in the southeast wall, identified in previous investigations. Each pole in the stereographic plot represents the mean orientation of a major set. In order to provide a simplified representation of overall trends in discontinuity structure, the joint sets have been broadly divided into four major joint set groups. The orientation data for each major joint set group are summarised in Table 5 - 3.
Figure 5 - 6: Mean orientation of major joint sets in Domain 2, identified by Golder (2007, 2010) from historic borehole drilling and field mapping data.

Set Group 1 contains sub-vertical joints trending northeast-southwest, parallel to the strike of regional faults (Roscoe and Postle, 2005). Set Group 2 contains shallow sub-horizontal discontinuities, which probably correspond to the undulating discontinuities identified in the pit wall panorama photographs (Figure 5 - 3). Set Groups 3 and 4 may correspond to moderately dipping conjugate joint sets, oriented approximately perpendicular to the northeast-southwest trend of regional faulting.

Table 5 - 3: Summary of orientation data for major discontinuity set groupings from previous investigations on the A418 pit by Golder Associates (2007, 2010, by permission).

<table>
<thead>
<tr>
<th>Discontinuity Set Group</th>
<th>Mean Dip (°)</th>
<th>Mean Dip Direction (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Set Group 1</td>
<td>89</td>
<td>307</td>
</tr>
<tr>
<td>Set Group 2</td>
<td>6</td>
<td>318</td>
</tr>
<tr>
<td>Set Group 3</td>
<td>41</td>
<td>024</td>
</tr>
<tr>
<td>Set Group 4</td>
<td>57</td>
<td>240</td>
</tr>
</tbody>
</table>
Field mapping locations for the current investigation were selected based on safety considerations and operational constraints. Locations were spread throughout the entire pit, with the aim of gathering a representative sample of discontinuity data for all geotechnical domains (Figure 5 - 7).

![Plan view of A418 pit with selected field mapping areas and investigation dates highlighted.](image)

Figure 5 - 7: Plan view of A418 pit with selected field mapping areas and investigation dates highlighted.

Although conventional discontinuity orientation data were also collected in the current investigation, emphasis was given to characterising additional characteristics such as roughness, aperture, and potential intact rock bridge content. Wherever possible, data was also recorded on the geometry and prevalence of non-systematic, brittle blast-induced fractures, intact rock bridges, and evidence of extension and dilation of major joints. Figure 5 - 8 presents a pole plot of major discontinuities mapped throughout the pit during the current field investigation. Mean orientations for the preliminary field-mapped discontinuity sets are summarised in Table 5 - 4.
Figure 5-8: Poles to discontinuities identified in current field mapping investigation, sorted according to local bench elevation and corner of the pit.

Due to the limited sample size \((n = 65)\), the preliminary field sets cannot be considered statistically representative of the fracture network throughout the A418 pit. However, the data provide a useful simplified overview of major discontinuity sets that may be more thoroughly characterised with more detailed mapping of photogrammetry models.

Table 5-4: Summary of orientation data for preliminary discontinuity sets identified in field mapping.

<table>
<thead>
<tr>
<th>Discontinuity Set Group</th>
<th>Mean Dip (°)</th>
<th>Mean Dip Direction (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Field Set 1</td>
<td>83</td>
<td>140</td>
</tr>
<tr>
<td>Field Set 2</td>
<td>63</td>
<td>329</td>
</tr>
<tr>
<td>Field Set 3</td>
<td>6</td>
<td>232</td>
</tr>
</tbody>
</table>

A northwest-dipping discontinuity set, denoted Field Set 2, is identified in field mapping data, but does not precisely overlap with the major joint sets from previous investigations. However, Field Set 2 may relate to the major grouping SG-1 from
previous investigations, which includes steeply-dipping to sub-vertical joints striking northeast-southwest. The poles belonging to Field Set 2 are not localized to the southeast wall.

The maximum persistence measured in close-range bench face mapping was approximately 40 m. In this instance the 40 m figure represents an under-estimate of the persistence of a multi-bench scale joint, where both ends of the joint were censored, as the joint extends above and below the mapped bench face. The average recorded persistence was approximately 16 m and the minimum persistence was 1 m; smaller discontinuities were not recorded, as they were judged to be less reflective of systematic joint sets and thus less significant to potential structurally-controlled instability mechanisms than the high and very high persistence (i.e. > 20 m) structures that occur throughout the pit.

The typical GSI range for field mapped rock masses in granodiorite and metasediment range from approximately 65 to 95, indicating good quality blocky to massive rock mass. In highly-stressed, deep underground excavations, this range of GSI has been proposed as a practical limit for the occurrence of brittle spalling failure (Diederichs, 2007). At Diavik, the combination of good quality rock and very high persistence discontinuities suggests that intact rock bridges may play an important role in stabilising structurally-controlled slope failures.

Figure 5 - 9 presents a selection of small scale roughness profiles measured on field-mapped discontinuity planes using a 10 cm-long profilometer. Most of the mapped discontinuities are planar or undulating and rough to very rough, at the centimetre scale. Field estimates of JRC range from approximately 5 to 15. Although the majority of the observed joint profiles are planar, some discontinuities are stepped or undulating at the centimetre scale, with small sections of intact rock fracture comprising up to approximately 5% of the exposed discontinuity plane surface area. The occurrence of very rough undulations and stepped discontinuity surfaces indicates that some interlocking effects may occur at the centimetre-scale within the undisturbed rock mass, contributing effective cohesive strength to the rock mass. At small scales, the strength benefit of intact rock bridges may be inseparable from the asperity strength component in rough, tightly-closed discontinuities.
Figure 5 - 9: Selected roughness profiles for discontinuities and intact rock fractures in the A418 pit.
Bench-scale conjugate wedge-forming joints are typically planar, and are frequently tight or closed for some of their trace length, with partial opening induced by blasting. Intervals of possible intact rock bridges are commonly observed between the tips of open coplanar joint segments (Figure 5 - 10).

Figure 5 - 10: Example of conjugate joints partially opened by blasting; both discontinuity traces are interrupted by intervals of apparently intact rock bridges (RL + 270 m bench; west wall).
Detachment niches from bench-scale wedges reveal that blasting-induced damage can interact with pre-existing discontinuities to provide kinematic release. Figure 5 - 11 shows an example from the RL + 360 m bench, where blast-induced brittle cracks step between pre-existing joints, and contribute to lateral release of a wedge.

Figure 5 - 11: Typical bench-scale wedge failures formed by conjugate joints; in some cases kinematic release is provided by brittle fractures induced by blasting (RL + 360 m Bench; Southwest wall).
Controlled blasting (pre-splitting) has produced excellent wall conditions throughout much of the A418 pit, indicated by sub-vertical bench face angles, preserved borehole half-barrels, and bench faces comprised of clean tensile fracture surfaces through intact rock. Figure 5 - 12 shows an example of bench face conditions resulting from effective controlled blasting techniques, in the west wall of the pit, on the RL + 330 m bench. Traces of planar, non-persistent northwest-dipping joints are visible throughout the face, separated by intervals of tight, apparently intact rock. Irregularly-shaped, non-systematic brittle cracks induced by blasting are also visible, with blast damage generally increasing towards the top of the bench face. Blast-induced tension fractures are typically very low persistence (< 1 m), however they often occur in clusters, forming step-paths that link between pre-existing discontinuities.

In describing the formation of tension joints in rock, Hencher (2012) noted that joints tend to grow at orientations defined by a pre-existing fabric of micro-cracks, or proto-joint network, originating from cooling stresses or tectonic deformation. Over geological time, stress changes and weathering processes can cause microfractures to merge and extend, eventually producing persistent mechanical discontinuities. However, even if joint traces appear to be fully persistent on surface exposures, a joint may still retain tensile strength from intact rock bridges inside the rock mass. The potential for intact rock bridges occurring inside a rock mass with developed joint traces was incorporated into a conceptual step-path classification method by Yan (2008).

Hencher (2012) noted that columnar basalts are an instructive example of tension joints with intact rock bridges. Overhanging basalt columns at Svartifoss waterfall in Iceland illustrate the influence of intact rock bridges: although the traces of columnar joints appear to be continuous, overhanging blocks remain in place because the bounding joints have not fully developed into persistent mechanical discontinuities (Figure 5 - 13; Photograph by Ciangottini, 2012; National Geographic Traveler Photo Contest 2012).

Preliminary observations from the Diavik A418 pit show that although many near-surface joints have been extended and dilated by blasting, traces frequently terminate within apparently intact rock. Where good blasting control is maintained, bench-scale wedge-forming joints may not be fully developed into persistent mechanical discontinuities, and thus may often retain a tensile strength component from intact rock bridges.
Figure 5 - 12: Partially-developed northwest-dipping joints exposed in West wall, RL + 330 m bench.
Figure 5 - 13: Columnar basalts are an instructive example of tension joints with intact rock bridges. Although the joint traces appear continuous, overhanging blocks remain in place because the joints retain intact rock bridges (Photograph by Ciangottini, 2012; National Geographic Traveler Photo Contest 2012).
5.4.3. 2-D Digital Trace Mapping

Digital trace maps were created for seven bench face windows, using the same procedures that were applied at Jwaneng. Window maps at Jwaneng were restricted to a maximum size of approximately 6 m long and 2 m high, corresponding to the practical size limit that allowed field mapping with direct physical access to the bench face. Due to the more close fracture spacing at Jwaneng than at Diavik, the smaller field windows were judged appropriate for gathering a field sample of discontinuity measurements.

The windows in the Diavik A418 pit cover larger areas including full bench heights of up to approximately 30 m. Although field discontinuity measurements were taken along the base of the walls, the majority of the bench faces are physically inaccessible. Due to the massive, sparsely fractured rock mass at Diavik, larger digital windows were needed to capture a useful representative sample of discontinuity measurements. The trace maps are presented in Figures 5-15 to 5-21 along with photographs of the bench face at each location. As with the trace mapping survey at Jwaneng:

- Fracture intensity $P_{21} = \frac{\sum \text{discontinuity trace lengths}}{\text{window surface area}}$
- Rock bridge intensity $R_{21} = \frac{\sum \text{intact rock bridge trace lengths}}{\text{window surface area}}$
- Blast-induced damage intensity $B_{21} = \frac{\sum \text{blast-induced fracture trace lengths}}{\text{window surface area}}$

$P_{21}$ calculation Method 2 only considers pre-existing discontinuities, before blasting. Thus $P_{21}$ (Method 1) can be related to $P_{21}$ (Method 2) and blast damage intensity $B_{21}$ as previously described:

$$P_{21} \text{ (Method 1)} = P_{21} \text{ (Method 2)} + B_{21}$$

Table 5-5 summarises the statistics for each window, and Figure 5-14 provides a legend for interpreting trace sets.
Table 5 - 5: Summary of trace mapping results from Diavik.

<table>
<thead>
<tr>
<th>Window</th>
<th>Surface Area (m²)</th>
<th>Local GSI (±5)</th>
<th>( P_{21} ) Method 1 (m⁻¹)</th>
<th>( P_{21} ) Method 2 (m⁻¹)</th>
<th>( R_{21} ) Method 1 (m⁻¹)</th>
<th>( R_{21} ) Method 2 (m⁻¹)</th>
<th>( B_{21} ) (m⁻¹)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>205</td>
<td>85</td>
<td>2.1</td>
<td>0.6</td>
<td>0.9</td>
<td>0.6</td>
<td>1.5</td>
</tr>
<tr>
<td>B</td>
<td>289</td>
<td>80</td>
<td>1.1</td>
<td>0.2</td>
<td>0.1</td>
<td>0.4</td>
<td>0.9</td>
</tr>
<tr>
<td>C</td>
<td>163</td>
<td>75</td>
<td>1.4</td>
<td>0.2</td>
<td>0.3</td>
<td>0.2</td>
<td>1.2</td>
</tr>
<tr>
<td>D</td>
<td>334</td>
<td>75</td>
<td>1.2</td>
<td>0.3</td>
<td>0.3</td>
<td>0.9</td>
<td>0.9</td>
</tr>
<tr>
<td>E</td>
<td>355</td>
<td>70</td>
<td>1.7</td>
<td>0.4</td>
<td>0.4</td>
<td>0.6</td>
<td>1.3</td>
</tr>
<tr>
<td>F</td>
<td>117</td>
<td>90</td>
<td>0.2</td>
<td>0.1</td>
<td>-</td>
<td>0.2</td>
<td>0.1</td>
</tr>
<tr>
<td>G</td>
<td>293</td>
<td>75</td>
<td>0.7</td>
<td>0.2</td>
<td>0.1</td>
<td>0.2</td>
<td>0.5</td>
</tr>
</tbody>
</table>

**Trace Map Legend**

- Sub-horizontal "cross-hole" blast-induced fractures associated with pre-splitting
- Irregular blast-induced brittle fractures, isolated or extended from pre-existing discontinuities
- Undulating, shallow to moderately dipping discontinuities, may be associated with groundwater seepage or pegmatite dykes
- Conjugate joint set 1, may be associated with wedge-forming discontinuity sets identified in photogrammetry
- Conjugate joint set 2, may be associated with wedge-forming discontinuity sets identified in photogrammetry
- Inferred intact rock bridge traces derived using Method 2, dissecting the window into an assembly of discrete blocks with continuous perimeters

Figure 5 - 14: Legend for interpretation of discontinuity and rock bridges traces.
Figure 5 - 15: Diavik Window A and accompanying trace map.
Figure 5 - 16: Diavik Window B and accompanying trace map.
Figure 5 - 17: Diavik Window C and accompanying trace map.
Figure 5 - 18: Diavik Window D and accompanying trace map.
Figure 5-19: Diavik Window E and accompanying trace map
Figure 5 - 20: Diavik Window F and accompanying trace map.
Figure 5 - 21: Diavik Window G and accompanying trace map.
Discontinuity spacing, block size and block shape are variable, but most windows show in situ block sizes usually exceeding 30 cm and frequently exceeding 1 m, thus corresponding to blocky to massive rock mass according to the quantified GSI scale suggested by Cai et al. (2004). Spacing of high persistence or larger, moderately dipping to steeply-dipping conjugate joints ranges from approximately 2 m to over 10 m. Spacing of non-systematic, brittle blast induced cracks is variable, but in general the intensity of blast-induced fractures tends to increase (and the spacing tends to decrease) towards the top of bench faces, supporting the observations of Hagan et al. (1978). Blast-induced fractures also frequently form step-paths between pre-existing discontinuities. Section 5.1 further discusses block size and shape based on the combined results of general field mapping, digital trace mapping, and digital photogrammetry.

The relationship between trace intensity values and rock mass quality (GSI) is uncertain (Figure 5 - 22). Higher GSI values seem to be associated with a decrease in fracture intensity ($P_{21}$), which is intuitively expected. However, more massive rock mass may also associate with a decrease in rock bridge trace intensity ($R_{21}$). As a rock mass becomes more massive, fewer pre-existing discontinuities exist, and thus fewer intact rock bridge traces can be identified. The result occurs because intact rock bridge traces can only be characterised with reference to the pre-existing discontinuities that are present.

![Figure 5 - 22: Scatter plot of areal intensity versus local GSI for each window map.](image)
The most massive rock mass is observed in Window F (Figure 5 - 20), where all borehole half-barrels are preserved, blasting damage is locally negligible (i.e. the face locally resembles controlled excavation resulting from excellent controlled blasting or manual excavation as suggested by Hoek (2012), corresponding to a local D value of 0) and GSI may range from 85 to 95+. Only one major discontinuity crosses the entire window: a shallow to moderately dipping undulating discontinuity with amplitude of approximately 2 m and wavelength of 10 m. The rock mass is massive and almost entirely unfractured, except for the one major discontinuity and some minor blast-induced damage. Nevertheless, the $R_{21}$ value is low ($0.2 \text{ m}^{-1}$) because intact rock bridge traces may only be defined with reference to the one major pre-existing discontinuity.

Windows where no major discontinuities can be distinguished are thus subject to scale effects: major discontinuities are not identified, or are under-sampled, because they are less likely to intersect small windows. If no discontinuities or intact rock bridges can be identified at close range, then it may be necessary to carry out a survey from greater distance, covering a larger surface area. By successively “zooming out”, major structures will eventually occur inside the survey region, and the controlling length scale for discontinuity persistence can be identified.

Preliminary trace mapping results suggest that $R_{21}$ values derived from surface traces may not always reveal the actual in-plane rock bridge content along specific discontinuity surfaces inside the rock mass. The example of overhanging blocks in columnar basalt (Figure 5 - 13) presented by Hencher (2012), illustrates that fully developed discontinuity traces may still retain tensile strength from intact rock bridges. Despite the limitations in identifying in-plane rock bridges, the $R_{21}$ procedure is useful for preliminary identification of out-of-plane rock bridges (i.e. rock bridges that separate non-coplanar discontinuity tips). Additional trace mapping carried out on perpendicular windows may also be helpful in assessing the relationship between $R_{21}$ values in the “along-strike” direction (parallel to the slope strike) and $R_{21}$ in the “down-dip” direction (parallel to the slope dip direction).
5.4.4. **Ground-Based Photogrammetry**

This section summarises the predominant discontinuity set orientations and persistence measurements from three photogrammetry surveys taken in the A418 pit. Camera stations were positioned along the crest of the pit and selected benches, subject to safety considerations and access constraints. The average distance to the face was approximately 500 m, corresponding to the approximate mean diameter of the pit. Each model was registered in local mine coordinates by manually flagging of at least six geodetic monitoring prisms with known coordinates as control points (Figure 5 - 23). Further discussion of discontinuity persistence is presented in Section 5.2.

![Examples of pre-existing geodetic monitoring prisms used to register photogrammetry models.](image)

**Figure 5 - 23: Examples of pre-existing geodetic monitoring prisms used to register photogrammetry models.**

5.4.4.1. **Photomodel using \( f = 100 \text{ mm} \) Survey**

The \( f = 100 \text{ mm} \) survey covers the entire pit circumference; however, it does not fully extend to the bottom of the pit. Photographs were taken with a telephoto lens from a series of 13 camera stations positioned around the pit crest and on the RL + 360 m bench. Figure 5 - 24 presents a selection of the camera stations used to generate 3-D terrain models, and the common points auto-matched between corresponding photographs. The large number of camera stations provides redundancy in identifying common points between matching images, however not all available photographs were utilized in generating the final 3-D terrain models for mapping.
Discontinuity mapping data from the $f = 100$ mm survey include 282 measurements made throughout all sectors of the pit (Figure 5 - 25). The cut-off length for measured discontinuities is 1.6 m; smaller joints are censored. The purpose of the initial survey is not to provide an exhaustive inventory of all discontinuities, but rather to provide a representative sample of major discontinuity sets and structural trends in different sectors of the pit. The results from the $f = 100$ mm survey are subsequently compared with observations from higher resolution surveys taken with focal lengths of $f = 200$ mm and $f = 300$ mm.
A zone of reduced density in the mapping data occurs in the north wall, where coverage of the lower benches below RL + 300 m is limited, and where the upper benches are characterised by non-systematic blast-induced brittle fractures. Subsequent mapping with the $f = 200$ mm survey attempted to gather additional data for this region. Each terrain model from the $f = 100$ mm survey covers a 3-D window area up to approximately 150 m x 180 m, however the exact dimensions of each window are variable, depending on the local line-of-sight distance to the rock face, and on the amount of overlap between matching images (Figure 5 - 26).
Mapping was not restricted to rectangular windows comprised of single 3-D terrain model (DTM) files, but was carried out for multi-bench sections of the pit, by overlaying eight or more DTMs simultaneously. Figure 5 - 27 illustrates the typical combined mapping area resulting from simultaneous overlay of eight DTM files.

The windows illustrate the typical persistence of discontinuities identified in mapping the \( f = 100 \) mm survey. The most persistent features include multi-bench segments of the A418 fault and diabase dykes, and also traces of sub-horizontal, undulating discontinuities with trace length up to approximately 150 m. Other discontinuities included in the survey predominantly include major, distinct planar features that can be reliably categorized as tension joints, in general varying from medium to very high persistence or greater (i.e. approximately 3 m or greater; ISRM, 1978).

Although the theoretical mean ground pixel size in \( f = 100 \) mm terrain models is approximately 3.2 cm, small, irregular brittle blast-induced fractures are easier to distinguish reliably in the higher resolution \( f = 200 \) mm and \( f = 300 \) mm surveys, and in the close-range (i.e. distance < 30 m) trace maps photographed with \( f = 20 \) mm. The influence of survey resolution on characterisation of discontinuity persistence is further addressed in Section 5.2.2.
Figure 5 - 27: Example of mapping windows from f = 100 mm survey, each comprised of 8 individual 3-D terrain models overlain (Bench heights are 30 m for scale).
Figure 5 - 28 presents a stereographic plot of all mapped discontinuities from the $f = 100$ mm survey ($n = 282$), organized according to the location within the pit where they occur (a) and the ISRM recommended persistence category, measured by diameter of the fitted planes (b).

Based on pole density contours, three major discontinuity sets are proposed, denoted J1, J2 and J3. Sets J1 and J2 are separately identified according to pole concentration contours, however, they may correspond to the same sets of steeply-dipping, northeast-southwest striking sets SG-1 (from previous investigations), FS-1 and FS-2 (from current
field mapping), which strike parallel to regional faults. Although some sub-horizontal discontinuities were measured in the \( f = 100 \) mm survey, the major groupings of sub-horizontal joints (SG-2 from previous investigations, and FS-3 from the current field mapping data) are not present. Random joints, not identified as belonging to specific sets, occur throughout the stereonet and may be indicative of localized sets that are not present throughout the entire pit. Thus, local set concentrations may be suppressed by the major discontinuity sets in the combined mapping data. The higher-resolution \( f = 200 \) mm and \( f = 300 \) mm surveys help to provide more measurements of less-persistent, localized discontinuity sets that do not occur as significant pole concentrations in the preliminary \( f = 100 \) mm data. The results from the \( f = 200 \) mm and \( f = 300 \) mm surveys are discussed in sections 4.4.2 and 4.4.3.

In the preliminary \( f = 100 \) mm data, set J1 is more concentrated than any other set, containing poles to 106 discontinuities. By removing J1 from the plot, an additional minor set, J4, becomes visible (Figure 5 - 29).

![Figure 5 - 29](image)

**Figure 5 - 29: Removal of set J1 increases the apparent density of secondary sets, indicating a possible set J4.**

Although set J1 contains 106 measurements, only 18 poles occur in the southeast wall. In contrast, 55 poles from J1 occur in the southwest corner of the pit. The results agree with field observations that northwest-dipping joints are more common near the A418 fault. The orientations data for the major discontinuity sets are listed in Table 5 - 6.
Table 5 - 6: Summary of discontinuity set orientation data identified in \( f = 100 \) mm photogrammetry survey.

<table>
<thead>
<tr>
<th>Set</th>
<th>Mean Dip (°) [Std. Dev.]</th>
<th>Mean Dip Direction (°) [Std. Dev.]</th>
<th>Sample Size, ( n )</th>
<th>Fisher K</th>
</tr>
</thead>
<tbody>
<tr>
<td>J1</td>
<td>69 [7.0]</td>
<td>334 [9.3]</td>
<td>107</td>
<td>52.4</td>
</tr>
<tr>
<td>J2</td>
<td>77 [8.0]</td>
<td>138 [9.8]</td>
<td>38</td>
<td>42.8</td>
</tr>
<tr>
<td>J3</td>
<td>88 [3.2]</td>
<td>036 [75.6]*</td>
<td>14</td>
<td>229.8</td>
</tr>
<tr>
<td>J4</td>
<td>68 [9.1]</td>
<td>012 [9.3]</td>
<td>27</td>
<td>42.2</td>
</tr>
</tbody>
</table>

*NOTE: The large standard deviation in dip direction for set J3 is a result of the set including steeply-dipping joints with opposite dip direction; the Fisher K value should be used in order to characterise the degree of “tightness of clustering” of the poles within set J3.

Large K values indicate tightly-clustered sets, whereas small K values indicate dispersed sets. The maximum, minimum and average diameter of fitted discontinuity planes for each set from the \( f = 100 \) mm survey are listed in Table 5 - 7.

Table 5 - 7: Summary of discontinuity set persistence data identified in \( f = 100 \) mm photogrammetry survey.

<table>
<thead>
<tr>
<th>Set</th>
<th>Persistence / Diameter of Fitted Planes (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum</td>
</tr>
<tr>
<td>J1</td>
<td>1.6</td>
</tr>
<tr>
<td>J2</td>
<td>5.9</td>
</tr>
<tr>
<td>J3</td>
<td>4.9</td>
</tr>
<tr>
<td>J4</td>
<td>4.1</td>
</tr>
</tbody>
</table>
The most persistent discontinuity set is J2, which strikes approximately parallel to the A418 fault. The most persistent individual feature in set J2 is approximately 111 m long, occurring in the west wall of the pit (Figure 5 - 31). In the southeast wall, the maximum recorded persistence of a J2 discontinuity is 28 m. The spatial variability of discontinuity persistence throughout the four regions of the pit is discussed further in Section 5.21. The highest maximum persistence values of 89 m and 111 m occur for J1 and J2, respectively, which are aligned parallel to the strike of regional faulting. These measurements likely correspond to the northeast-southwest striking sets SG-1 (from previous investigations), FS-1 and FS-2 (from current field mapping), which strike parallel to regional faults.

Figure 5 - 30 presents a series of density-contoured plots of discontinuity dip versus dip direction for four sectors of the pit. Each sector represents a 90° interval of azimuth, measured from the centre of the pit, thus representing one quarter of the pit circumference.

![Figures 5 - 30](image_url)  
*Figure 5 - 30: Density-contoured plots of dip versus dip direction for four sectors of the A418 pit, from f = 100 mm mapping data.*
Figure 5 - 31: Multi-bench joint with persistence > 100 m showing dilation up to approximately 200 mm in West wall; large structures may also act as groundwater flow boundaries.
5.4.4.2. Photomodel using $f = 200$ mm Survey

The $f = 200$ mm survey covers the majority of the pit circumference; however an occluded zone with no data occurs in the southwest corner. Photographs were taken with a telephoto lens from four camera stations on the RL + 390 m bench. Figure 5 - 32 presents a plan view of the camera stations used to create the 3-D terrain models, showing the common-points auto-matched in corresponding photographs, and the associated distance:base values for the paired stations.

![Figure 5 - 32: Plan view of common points and a selection of camera stations from the $f = 200$ mm survey, highlighting an occluded zone where insufficient image overlap results in a section of no data.](image_url)

The positions of camera stations in the $f = 200$ mm survey results in an occluded section with no data, occurring in the southwest section of the pit. In the occluded zone, there is insufficient image overlap, and thus automatic recognition of common points was not possible. As a result, no discontinuity mapping data is available between approximately azimuths 200° to 250°.

Discontinuity mapping data from the $f = 200$ mm survey included 368 measurements made throughout the pit, with the exception of the southwest corner. The cut-off length used in discontinuity mapping is 0.8 m; smaller structures are truncated. Figure 5 - 33 presents a plan view of the centre points to all the mapped discontinuities.
Figure 5 - 33: Plan view of mapped discontinuity centres, \( f = 200 \text{ mm survey} \).

Each terrain model from the \( f = 200 \text{ mm survey} \) covers a 3-D window area up to approximately 50 m x 50 m, however the exact dimensions of each window are variable, depending on the local line-of-sight distance to the rock face, and on the amount of overlap between matching images. Figure 5 - 34 illustrates the typical dimensions of a single 3-D window produced by a pair of matching images.

Mapping was not restricted to rectangular windows comprised of single 3-D terrain model (DTM) files, but was carried out for multi-bench sections of the pit, by overlaying eight or more DTMs simultaneously to form a 3-D window up to approximately 200 m square. Figure 5 - 35 shows the combined mapping area resulting from simultaneous overlay of 12 individual DTM files.
Figure 5 - 34: Example of a single 3-D window produced from a pair of matching images, stored in a 3-D terrain model file (DTM file) with .JPG texture overlays from the \( f = 200 \) mm survey; north wall, RL + 300 m bench.
Figure 5 - 35: Example of mapping windows from $f = 100$ mm survey, each comprised of 12 individual overlain 3-D terrain models.

Figure 5 - 36 presents a stereographic plot of all mapped discontinuities in the $f = 200$ mm survey ($n = 368$), organized according to the location within the pit where they occur (a) and according to the ISRM recommended persistence category, measured by diameter of the fitted planes (b). Based on pole density contours, four major discontinuity sets are proposed: J1, J2, J3, and J5.
Figure 5 - 36: Major discontinuity sets from the \( f = 200 \) mm survey.

The \( f = 200 \) mm survey uses a shorter cut-off length (0.8 m; half the length of 1.6 m cut-off used in the \( f = 100 \) mm survey) and thus includes more data than the \( f = 100 \) mm model. Four major joint sets are proposed: delineated J1, J2, J3 and J5. Although some of the joint sets are similar to those identified in the \( f = 100 \) mm survey, discontinuity sets from each survey have been treated separately in order to avoid potential confusion and overlap of measurements.

- Set J1 includes two concentrations of moderately to steeply northwest-dipping poles occur, delineated J1a and J1b. The clusters of poles correspond to set J1 identified in the \( f = 100 \) mm survey, and are also similar to the sets of steeply-
dipping, northeast-southwest striking sets SG-1 (from previous investigations), FS-1 and FS-2 (from current field mapping), parallel to regional faults

- Set J2 dips **steeply to the southeast**, also approximately parallel to the A418 fault; this set correspond to set J2 identified in the $f = 100$ mm survey, and may also relate to sets SG-1 (from previous investigations), FS-1 and FS-2 (from current field mapping)

- Set J3 is **sub-vertical**, striking northwest-southeast, and corresponds to the same set J3 identified in the $f = 100$ mm survey; there is no analogous set equivalent to J3 from either the previous investigation data or the current field mapping results

- The secondary set J4 from the $f = 100$ mm survey (steeply dipping toward north) is not observed

- Set J5 dips shallowly to **moderately** toward north, and corresponds to the sub-horizontal set group SG-2 identified from previous investigations, and the preliminary field set FS-3 from the current investigation

The orientation data and sample statistics for the major sets are listed in Table 5 - 8.

**Table 5 - 8: Summary of discontinuity set orientation data identified in $f = 200$ mm photogrammetry survey.**

<table>
<thead>
<tr>
<th>Set</th>
<th>Mean Dip (°) [Std. Dev.]</th>
<th>Mean Dip Direction (°) [Std. Dev.]</th>
<th>Sample Size</th>
<th>Fisher K</th>
</tr>
</thead>
<tbody>
<tr>
<td>J1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>J1a</td>
<td>74 [8.4]</td>
<td>357 [168.4]*</td>
<td>61</td>
<td>55.4</td>
</tr>
<tr>
<td>J1b</td>
<td>72 [6.9]</td>
<td>330 [6.3]</td>
<td>49</td>
<td>80.0</td>
</tr>
<tr>
<td>J3</td>
<td>87 [2.6]</td>
<td>034 [81.1]*</td>
<td>16</td>
<td>204.7</td>
</tr>
<tr>
<td>J5</td>
<td>26 [6.1]</td>
<td>357 [161.6]*</td>
<td>36</td>
<td>39.4</td>
</tr>
</tbody>
</table>
NOTE: The large standard deviation in dip direction for sets J1a and J5 results from the sets containing approximately north-dipping joints which includes dip directions of approximately 340°-360° on one side of North (000°), and dip direction of approximately 000-010° on the other side of North. The large standard deviation of dip direction in set J3 is a result of the set including steeply-dipping joints with opposite dip direction; the Fisher K value should be used in order to characterise the degree of “tightness of clustering” of the poles within set J3.

The major discontinuity sets J1 to J5 represent only approximately 53% of the total number of poles mapped in the $f = 200$ mm survey. This result suggests that other localized or minor discontinuity sets may exist, but local concentrations may be suppressed by the greater concentration of the dominant joint sets. To better characterise localized or minor joint sets, the major discontinuity sets were removed and the plots were re-contoured (Figure 5 - 37).

![Figure 5 - 37: Secondary discontinuity sets from the $f = 200$ mm survey.](image-url)
Based on the resulting concentrations of poles (n = 175), five secondary discontinuity sets are proposed, denoted SS-1 to SS-5 (i.e. Secondary Set 1 to Secondary Set 5). Table 5 - 9 summarises the orientation data for the secondary sets. The secondary set windows were deliberately over-sized beyond the dimensions of the minimum pole concentration contours, in order to better capture the dispersion of poles in minor sets.

Table 5 - 9: Summary of discontinuity set orientation data identified in f = 200 mm photogrammetry survey.

<table>
<thead>
<tr>
<th>Set</th>
<th>Mean Dip (°) [Std. Dev.]</th>
<th>Mean Dip Direction (°) [Std. Dev.]</th>
<th>Sample Size</th>
<th>Fisher K</th>
</tr>
</thead>
<tbody>
<tr>
<td>SS-1</td>
<td>88 [5.1]</td>
<td>287 [93.8]*</td>
<td>53</td>
<td>13.1</td>
</tr>
<tr>
<td>SS-2</td>
<td>57 [10.6]</td>
<td>013 [150.9]*</td>
<td>35</td>
<td>22.2</td>
</tr>
<tr>
<td>SS-3</td>
<td>5 [9.9]</td>
<td>086 [97.3]*</td>
<td>29</td>
<td>12.3</td>
</tr>
<tr>
<td>SS-5</td>
<td>59 [5.0]</td>
<td>262 [28.1]</td>
<td>16</td>
<td>10.6</td>
</tr>
</tbody>
</table>

*NOTE: The large standard deviation in dip direction for sets SS-1 occurs because the set contains steeply dipping structures with poles on opposite sides of the stereonet, with opposite dip directions; the large standard deviation in dip direction for SS-2 occurs because the set contains approximately north-dipping poles which vary in dip direction from West-northwest (~330° or more, to East-northeast (~030° or less); the large standard deviation in SS-3 occurs because the set contains a scatter of sub-horizontal joints, with dip direction that varies from 0° to 360°. For all cases with such high standard deviation the Fisher K value should be used in order to characterise the degree of “tightness of clustering” of the poles within the sets.

SS-1 is a sub-vertical set that strikes approximately 015°-195°, similar to the northeast-strike associated with regional faulting and kimberlite emplacement, and may represent the same set group SG-1 identified in previous mapping. The set occurs throughout all four sections of the pit. SS-2 is a moderate to steeply north-dipping set that also occurs throughout all for section of the pit. SS-3 comprises a broad scattering of sub-horizontal structures with dip direction varying from 0° to 360°. SS-4 is a steep southeast-dipping
set that is most frequently observed in the northeast section of the pit. SS-5 is a steep west-dipping set that is most frequently observed in the southeast section of the pit, and may represent the same group of discontinuities identified in SG-4 from previous investigations, which may include conjugate joints that strike oblique to the trend of regional faulting.

The maximum 3-D window size used for mapping in the $f = 200$ mm survey was up to approximately 200 m side length, resulting from simultaneous overlay of approximately 12 DTM files. This maximum mapping area is smaller than the large overall-slope scale mapping windows used in the $f = 100$ mm survey. As a result of the decreased size of mapping windows, the maximum measured persistence from the $f = 200$ mm survey is slightly smaller than observed in the $f = 100$ mm survey.

The highest persistence features was 45.8 m; the minimum persistence was 0.8 m (corresponding with the cut-off length of 0.8 m); and the average recorded persistence was approximately 9.6 m. The maximum, minimum and average diameter of fitted discontinuity planes for each set from the $f = 200$ mm survey are listed in Table 5 - 10.

Where major discontinuities were censored by the limited size of 3-D windows, additional DTMs were overlain wherever possible in order to extend the mapping area, thus limiting the influence of censoring bias, which tend to cause under-estimation of discontinuity persistence (Figure 5 - 38).

![Figure 5 - 38: Censoring in 3-D mapping windows is minimized by using larger mapping areas, achieved by overlaying additional DTMs.](image)
Despite efforts to measure the true maximum extent of discontinuity trace and plane exposures, the resulting measurements still may not reflect true discontinuity persistence inside the rock mass, because the interior rock mass of the slope cannot be observed (i.e. f-bias). The influence of survey resolution on persistence measurements is further discussed in Section 5.5.2.2.

Table 5 - 10: Summary of discontinuity set persistence data identified in \( f = 200 \) mm photogrammetry survey.

<table>
<thead>
<tr>
<th>Set</th>
<th>Persistence / Diameter of Fitted Planes (m)</th>
<th>Minimum</th>
<th>Average</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Major Sets</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>J1a</td>
<td></td>
<td>1.2</td>
<td>9.6</td>
<td>45.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>J1b</td>
<td></td>
<td>1.3</td>
<td>11.2</td>
<td>44.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>J2</td>
<td></td>
<td>1.1</td>
<td>11.4</td>
<td>32.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>J3 a/b</td>
<td></td>
<td>2.6</td>
<td>12.8</td>
<td>21.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>J4</td>
<td></td>
<td>0.8</td>
<td>6.1</td>
<td>29.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS-1</td>
<td></td>
<td>1.3</td>
<td>8.8</td>
<td>40.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS-2</td>
<td></td>
<td>1.3</td>
<td>8.2</td>
<td>30.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS-3</td>
<td></td>
<td>1.5</td>
<td>9.2</td>
<td>36.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS-4</td>
<td></td>
<td>1.9</td>
<td>10.0</td>
<td>27.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS-5</td>
<td></td>
<td>4.7</td>
<td>11.1</td>
<td>25.3</td>
</tr>
</tbody>
</table>
5.4.4.3. Photomodel using $f = 300$ mm Survey

The $f = 300$ mm survey was focused exclusively on the southeast wall. Photographs were taken with a telephoto lens from camera stations positioned on the RL + 300 m and RL + 330 m benches. Figure 5 - 39 presents a plan view of the camera stations used to generate 3-D terrain models, the common points auto-matched between corresponding photographs, and the maximum and minimum distance:base ratio values associated with the different camera station pairings.

![Figure 5 - 39: Plan view of common points and camera stations for the $f = 300$ mm photogrammetry survey of the southeast wall](image)

Figure 5 - 40 shows a perspective view of the common points automatically identified in the southeast wall, with annotation indicating bench crest elevations. Blank areas with no common points indicate occlusion, occurring predominantly along the floor of the main haul ramp, and along most bench floors, where camera line-of-sight is obscured by the bench crests. The higher resolution and lower discontinuity cut-off length allowed for a larger inventory of discontinuity measurements to be collected than both the $f = 100$ mm and $f = 200$ mm surveys.
Figure 5 - 40: Perspective view of common points and camera stations for the $f = 300$ mm photogrammetry survey of the southeast wall.

The mapping data for the $f = 300$ mm survey include 532 measurements (Figure 5 - 41). The cut-off length for discontinuities is 0.3 m; smaller joints are truncated.

Figure 5 - 41: Plan view of mapped discontinuity centres, $f = 300$ mm survey.

To take advantage of the higher resolution of the $f = 300$ mm survey, mapping efforts thus focused on additional characterisation of bench-scale or smaller discontinuities, from medium persistence (3 to 10 m) down to the cut-off limit of 0.3 m. Low persistence features are frequently associated with blasting-induced damage, whereas high
persistence or larger structures more likely result from geological processes. Each terrain model from the $f = 300$ mm survey covers a 3-D window area up to approximately 50 m x 30 m, however the exact dimensions of each window are variable, depending on the local line-of-sight distance to the rock face, and on the amount of overlap between matching images.

Although mapping focused on gathering data on lower-persistence features that may have been indistinguishable in the $f = 100$ mm and $f = 200$ mm surveys, mapping was not restricted to discrete windows comprised of a single DTM file. Wherever major discontinuities extended beyond the limits of a single DTM, additional DTMs were overlain in order to allow mapping the adjacent areas. Figure 5-42 illustrates the typical dimensions of a single 3-D window produced by a pair of matching images, and also the combined mapping area provided by simultaneous overlay of up to 18 DTM files.

![Figure 5-42: Example of 3-D windows produced from the $f = 300$ mm survey.](image-url)
Due to the computer memory requirements for displaying the high resolution $f = 300$ mm DTM files, the maximum mapping area simultaneously analysed was limited to three-bench stacks of approximately 100 m to 150 m side length.

Figure 5 - 43 presents a pole plot of all discontinuities mapped in the $f = 300$ mm model. Based on the concentrations of poles, five major discontinuity sets are proposed. Although some of the joint sets are similar to those from the $f = 100$ mm and $f = 200$ mm surveys, discontinuity sets from each survey have been considered separately in order to avoid potential confusion and overlap of measurement. The discontinuity sets for each survey should be considered separately.

![Pole plot of discontinuities for $f = 300$ mm survey with major discontinuity sets overlain.](image)

- J1 is a steep northwest-dipping joint set, similar to set J1 observed in the two other photogrammetry surveys, and corresponds to sets SG-1 (from previous investigations) and FS-2 (from the current field mapping), which are oriented parallel to regional faults.
• J2 occurs is a **sub-vertical** set that strikes northeast-southwest, approximately parallel to the strike of the A418 fault

• J3 is a **sub-vertical west-dipping** set that is not observed in the other photogrammetry survey observations

• J4 is a shallow to **moderately southwest-dipping set** of predominantly low-persistence discontinuities that may be associated with preserved sedimentary structure within metasediments; the set likely corresponds to the same southwest-dipping set group SG-4 identified in previous investigations

• J5 is a **sub-horizontal** set comprised of mostly low to medium persistence discontinuities; many of these measurements may be associated with blast-induced cross-hole fractures identified between preserved half-barrels of production blast holes; the set also relates to the groupings of sub-horizontal discontinuities in sets SG-2 (from previous investigations), and FS-3 (from current field mapping)

• Although the previous investigation data indicated a northeast-dipping (41°/024°) group of sets denoted SG-3, no corresponding set occurs in the $f = 300$ mm data

The orientation data for the major joint sets are summarised in Table 5-11.
Table 5 - 11: Summary of major discontinuity set orientations identified in the \( f = 300 \) mm photogrammetry survey.

<table>
<thead>
<tr>
<th>Set</th>
<th>Mean Dip (°) [Std. Dev.]</th>
<th>Mean Dip Direction (°) [Std. Dev.]</th>
<th>Sample Size</th>
<th>Fisher K</th>
</tr>
</thead>
<tbody>
<tr>
<td>J1</td>
<td>68 [6.4]</td>
<td>337 [7.4]</td>
<td>52</td>
<td>75.2</td>
</tr>
<tr>
<td>J2</td>
<td>90 [2.5]</td>
<td>323 [91.4]*</td>
<td>21</td>
<td>192.5</td>
</tr>
<tr>
<td>J3</td>
<td>81 [4.7]</td>
<td>280 [9.1]</td>
<td>24</td>
<td>64.0</td>
</tr>
<tr>
<td>J5</td>
<td>5 [7.6]</td>
<td>080 [93.8]*</td>
<td>179</td>
<td>22.1</td>
</tr>
</tbody>
</table>

*NOTE: The large standard deviation in dip direction for set J2 occurs because the set contains steeply dipping structures with poles on opposite sides of the stereonet, with opposite dip directions; the large standard deviation in dip direction for set J5 occurs because the set contains a scatter of sub-horizontal joints, with dip direction that varies from 0 to 360°. For all cases with such high standard deviation the Fisher K value should be used in order to characterise the degree of “tightness of clustering” of the poles within the sets.

Figure 5 - 44 presents a density-contoured plot of discontinuity dip versus dip direction from the \( f = 100 \) mm survey, and Figure 5 - 45 presents the plots from the \( f = 300 \) mm survey. The results illustrate the strong influences of

1. **Survey resolution**: increased sample size and smaller cut-off length of the \( f = 300 \) mm survey has resulted in a larger dataset

2. **Mapping location**: the \( f = 100 \) mm survey covers the entire pit circumference in all design sectors; whereas the \( f = 300 \) mm survey focuses only on the southeast wall in Design Sector 2

The plots from the \( f = 100 \) mm survey show almost no poles with dip less than 40°.
The $f = 300$ mm dataset indicates two major clusters with shallow to moderate dip (i.e. 0 to approximately 60°), that are not present in the $f = 100$ mm data. Although the $f = 100$ mm survey covers a larger mapping area than the $f = 300$ mm survey, the two clusters of shallow to moderately-dipping discontinuities are excluded, due to truncation and other sampling bias: that is, the $f = 100$ mm survey focussed on major structures of predominantly high persistence or greater, whereas the $f = 300$ mm survey focussed on smaller structures at the bench scale and below. The influence of survey resolution is further discussed in Section 5.2.2.
Figure 5 - 45: Density contour plot of discontinuity dip versus dip direction for mapping data from $f = 300$ mm surveys.

The major joint sets J1 to J5 contain 341 poles of the total 532 measurements made using the $f = 300$ mm dataset – amounting to 64% of the sample population. The remaining 191 poles account for 36% of the survey data, and may belong to localized or minor discontinuity sets, where the local pole concentrations may be suppressed by the overall pole concentrations of the major sets.

To investigate the potential for localized or minor sets, poles from the major discontinuity sets were removed and the remaining data ($n = 189$) were re-contoured. Figure 5 - 46 presents a pole plot of the remaining discontinuities, organized according to the ISRM recommended persistence category, measured using the diameter of fitted discontinuity planes.
Figure 5 - 46: Minor discontinuity sets from the $f = 300$ mm survey.

The secondary pole plot shows a large degree of randomness and scatter; over 12 small localized clusters occur with Fisher concentration of 2% or more. In order to simplify the engineering geology interpretation of secondary discontinuity sets, the set windows were deliberately over-sized beyond the dimensions of the minimum pole concentration contours for single clusters. Four minor sets are proposed, denoted MS-1 to MS-4 (i.e. Minor Set 1 to Minor Set 4). Table 5 - 12 summarises the set orientation data.

Table 5 - 12: Summary of minor discontinuity set orientations identified in the $f = 300$ mm photogrammetry survey.

<table>
<thead>
<tr>
<th>Set</th>
<th>Mean Dip (°) [Std. Dev.]</th>
<th>Mean Dip Direction (°) [Std. Dev.]</th>
<th>Sample Size</th>
<th>Fisher K</th>
</tr>
</thead>
<tbody>
<tr>
<td>MS-1</td>
<td>72 [10.2]</td>
<td>087 [15.2]</td>
<td>34</td>
<td>21.2</td>
</tr>
<tr>
<td>MS-3</td>
<td>76 [8.6]</td>
<td>187 [16.5]</td>
<td>27</td>
<td>20.4</td>
</tr>
<tr>
<td>MS-4</td>
<td>57 [11.7]</td>
<td>019 [138.6]*</td>
<td>39</td>
<td>15.5</td>
</tr>
</tbody>
</table>
*NOTE: The large standard deviation in dip direction for set MS-4 occurs because the set contains approximately north-dipping poles which vary in dip direction from West-northwest (i.e. ~330° or more, to East-northeast (~030° or less. For all cases with high standard deviation in dip direction, the Fisher K value should be used in order to characterise the degree of “tightness of clustering” of the poles within the sets.

MS-1 and MS-2 are steeply-dipping, approximately north-south striking sets that may reflect conjugate joints that strike slightly oblique to the northeast-southwest strike of regional faults. MS-2 in particular is similar to set SG-1 identified in previous data, which occurs parallel to the A418 Fault and Dewey’s Fault. MS-3 is a steep south-dipping set that is not present in either the previous investigation data or the current field mapping. MS-4 is a moderately northeast-dipping set that may relate to the local presence in the southeast wall of the set SG-3 identified in previous mapping data.

In the $f = 300$ mm survey, set J5 is characterised by low persistence, shallowly-dipping or sub-horizontal fractures that are frequently observed crossing between borehole half barrels, and thus likely includes blasting-induced brittle fractures (Figure 5 - 47). The extremely persistent discontinuities (100 m+) recorded in the $f = 100$ mm survey are located in other sectors of the pit, and thus are not present in the $f = 300$ mm model of the southeast wall.

Figure 5 - 47: Low persistence (<1 m) sub-horizontal blast-induced fractures identified in the $f = 300$ mm model.
The maximum, minimum and average diameter of fitted discontinuity planes for each set from the $f = 300$ mm survey are listed in Table 5 - 13.

**Table 5 - 13: Summary of discontinuity set persistence data identified in $f = 300$ mm photogrammetry survey.**

<table>
<thead>
<tr>
<th>Set</th>
<th>Persistence / Diameter of Fitted Planes (m)</th>
<th>Minimum</th>
<th>Average</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Major Sets</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>J1</td>
<td></td>
<td>1.1</td>
<td>8.4</td>
<td>26.4</td>
</tr>
<tr>
<td>J2</td>
<td></td>
<td>1.2</td>
<td>8.0</td>
<td>24.8</td>
</tr>
<tr>
<td>J3</td>
<td></td>
<td>1.1</td>
<td>5.7</td>
<td>24.8</td>
</tr>
<tr>
<td>J4</td>
<td></td>
<td>0.9</td>
<td>3.9</td>
<td>19.3</td>
</tr>
<tr>
<td>J5</td>
<td></td>
<td>0.3</td>
<td>4.3</td>
<td>30.3</td>
</tr>
<tr>
<td>Secondary Sets</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MS-1</td>
<td></td>
<td>1.0</td>
<td>5.1</td>
<td>18.1</td>
</tr>
<tr>
<td>MS-2</td>
<td></td>
<td>1.4</td>
<td>10.8</td>
<td>38.2</td>
</tr>
<tr>
<td>MS-3</td>
<td></td>
<td>1.0</td>
<td>7.1</td>
<td>42.6</td>
</tr>
<tr>
<td>MS-4</td>
<td></td>
<td>0.6</td>
<td>4.8</td>
<td>14.6</td>
</tr>
</tbody>
</table>
5.5. Discussion

5.5.1. Block Size, Shape, and Discontinuity Spacing

Discontinuity spacing and block geometry have important effects on discontinuity-controlled slope instabilities, influencing the kinematics of bench-scale wedge failures and the volume of rockfalls. In larger inter-ramp or overall slope-scale failures, block size may also influence the internal mechanics of a large volume of unstable rock mass; research by Kalenchuk et al. (2008) has demonstrated with numerical simulation that block geometry can be an important control on the progressive development of failure in underground sub-level caves.

Figure 5 - 48 presents suggested contours of typical and outlier block shapes expected in the A418 pit, overlain on the ternary diagram proposed by Kalenchuk et al. (2006), based on field mapping observations and interpretation of the 3-D digital terrain models constructed from the photogrammetry surveys.

Figure 5 - 48: Suggested density contour-based qualitative field assessment of block shape, superimposed on the ternary diagram proposed by Kalenchuk et al. (2006).
Based on the field observations and mapping results, the rock mass in the A418 pit is blocky to massive according to the quantified GSI recommended by Cai et al. (2004). The major joint sets are very widely to extremely widely spaced (i.e. 0.6 m to > 6 m; ISRM, 1978), and bench-scale wedge failures tend to be pyramidal, cubic-elongated or irregular cubic blocks. Detachment niches from wedge failures, such as those previously discussed in Section 4.2, indicate that blocks formed by pre-existing discontinuities tend to range from approximately 1 m$^3$ to 10 m$^3$; however debris that is comminuted by blasting contains smaller blocks with volume much less than 1 m$^3$. Despite localized occurrence of backbreak and wedge failures up to ~ 10 m$^3$, rockfall debris accumulation is not significant along the bench faces, and vertical bench face angles are maintained throughout most of the A418 pit.

### 5.5.2. Length Scale for Discontinuity Persistence

Histograms of discontinuity diameters extracted from the photogrammetry surveys help to characterise the dominant length scale of discontinuities. All mapping results tend to suggest a common trend approximately fitting a negative exponential probability density function for persistence. However, the combined influences of censoring, truncation, length bias, f-bias, scale effects, and survey resolution cause subtle differences between the results from the three photogrammetry surveys. Mapping location with respect to geotechnical domains and pit design sectors is another important consideration: not all photogrammetry surveys cover the same area. The $f = 100$ mm covers the entire pit circumference in all design sectors; the $f = 200$ mm covers the entire pit circumference with the exception of an occluded zone in the southwest corner, comprising part of Design Sector 3 (in geotechnical domain 3); the $f = 300$ mm survey focuses exclusively on the southeast wall of the pit in Design Sector 2 (in geotechnical domain 2).

Histogram results are presented in terms of absolute frequency, reflecting the number of features measured in the photogrammetry surveys. Consequently, the $f = 100$ mm survey has the smallest dataset and the smallest frequency values, with increasing frequency for the $f = 200$ mm and $f = 300$ mm surveys, which have larger datasets. Alternatively, histogram results could be presented in terms of percentage of the sample population, however the distribution will show identical shape, with the only change occurring in the labelling of the vertical axis, because the relative proportions of
discontinuity frequencies remains identical. Figure 5 - 49 shows a histogram of all discontinuity diameters for the $f = 100$ mm dataset, with bin intervals of 1 m.

![Histogram of all fitted discontinuity diameters for $f = 100$ mm photogrammetry survey.](image)

**Figure 5 - 49:** Histogram of all fitted discontinuity diameters for $f = 100$ mm photogrammetry survey.

A peak in frequency occurs for discontinuities between 8 m and 9 m in persistence, and smaller discontinuities are under-sampled. A calculated negative exponential best-fit curve is overlain, with a correction of +1 to all frequency values in order to avoid convergence of the curve to 0 frequency at large persistence values. Without the correction, the negative exponential function converges to 0 and thus may under-predict the frequency of important major structures.

Of the 282 total measurements, 194 (88%) are within the ISRM (1978) suggested persistence limits, up to a maximum of 20 m. The remaining 88 discontinuities (31%) are larger than 20 m. The results support the case of Sturzenegger (2010) who suggested an additional category may be need for extremely persistent structures.

The $f = 100$ mm survey is the only survey that captures the true maximum trace length of the most persistent discontinuities intersecting the pit, which includes a steeply east-dipping joint that occurs in the west wall with persistence greater than 100 m; the A418 fault which crosses the entire pit; and also the diabase dykes, up to 10 m in the north.
and south walls, that can be correlated across the pit. Figure 5 - 50 presents a histogram of discontinuity diameters for the major joint sets from the $f = 100$ mm survey.

**Figure 5 - 50: Histogram of fitted discontinuity diameters for major joint sets identified in the $f = 100$ mm photogrammetry survey.**

The plot illustrates the importance of outlier structures with extremely high persistence. Despite the peak in frequency that occurs for medium persistence structures (3 -10 m, ISRM, 1978), all sets contain outlier structures with extremely high persistence, greater than 35 m and up to approximately 110m. The results emphasize the conclusion that corrected frequency curves are necessary to avoid under-prediction of major structures.

Figure 5 - 51 presents a histogram of discontinuity diameters for the $f = 200$ mm dataset, with bin intervals of 1 m. A corrected negative exponential best-fit function is overlain.
Figure 5 - 51: Histogram of all fitted discontinuity diameters for $f = 200$ mm photogrammetry survey.

A peak in frequency occurs for discontinuities between 4 m and 5 m in persistence, and smaller discontinuities are under-sampled. The finer resolution of the survey, compared with the initial $f = 100$ mm model, shifts the distribution towards a smaller peak value of persistence. As the focal length is doubled from the $f = 100$ mm model, the most frequent discontinuity persistence value is approximately halved. The curvature of the suggested negative exponential fit is steeper than that of the $f = 100$ mm model and the transition to asymptotic growth at smaller persistence values is more abrupt.

Figure 5 - 52 presents a histogram of discontinuity diameters for the major joint sets from the $f = 200$ mm survey. All sets contain outlier structures with very high persistence.
Figure 5 - 52: Histogram of fitted discontinuity diameters for major joint sets identified in the $f = 200$ mm photogrammetry survey.

The results reinforce the importance of accounting for very high or extremely high persistence outlier structures, beyond the 20 m upper limit suggested by the ISRM (1978) guidelines.

Figure 5 - 53 shows a histogram of discontinuity diameters for the $f = 300$ mm dataset, with a calculated negative exponential fit overlain. The negative exponential function applies a correction factor of +1 to all frequency values in order to avoid convergence of the curve to 0 frequency at large persistence values. Without the correction, the negative exponential function converges to 0 and thus may under-predict the frequency of important major structures.

A peak in frequency occurs for discontinuities between 3 m and 4 m in persistence; smaller discontinuities may be under-sampled. Very low persistence, shallowly-dipping or sub-horizontal blasting-induced fractures are better resolved in the $f = 300$ mm model than in other surveys, adding to the frequency of low-persistence measurements.
Figure 5 - 53: Histogram of fitted discontinuity diameters for \( f = 300 \) mm photogrammetry survey.

Figure 5 - 54 presents a histogram of discontinuity diameters for the major joint sets from the \( f = 300 \) mm survey. All sets contain structures with high to very high persistence.

Figure 5 - 54: Histogram of fitted discontinuity diameters for major joint sets identified in the \( f = 300 \) mm photogrammetry survey
The 3-D window mapping area provided by all three surveys is sufficient to yield a population of discontinuity measurements that supports the utility of a negative exponential best-fit function.

5.5.2.1. Spatial Variation in Discontinuity Persistence

Spatial variation in discontinuity persistence throughout the pit can be investigated by deriving a vector for each mapped discontinuity, with an origin at the centre of the pit and a termination at the discontinuity centre (Figure 5 - 55). The bearing of the vector relative to North, denoted as $\phi$, describes the location of the discontinuity centre.

Figure 5 - 55: Plan view of A418 pit; the bearing of the location vector, $\phi$, describes the location of the discontinuity centres occur, and persistence parameter $P$ describes the diameter of the discontinuity.

Figure 5 - 56 shows a plot of discontinuity persistence versus bearing of the location vector for the $f = 100$ mm mapping data. The grouping of points do not follow a consistent trend, however several observations can be made:
• Most discontinuities (88%) have persistence of 30 m or less, for all locations throughout the pit.
• The highest persistence discontinuities are outliers of the dataset, belonging to joint sets J1 and J2, which strike approximately parallel to the strike of the A418 fault; these structures occur between bearings of approximately 220° to 320°, in the west wall of the pit.
• The highest persistence observed in the southeast wall, between bearings of 90° to 180°, is approximately 62 m, however the majority of features mapped in the southeast wall have persistence of 20 m or less.

**Figure 5 - 56**: Discontinuity persistence vs. bearing of location vector, $f = 100$ mm survey; ISRM (1978) upper limit for very high persistence (20 m) is highlighted.

Figure 5 - 57 shows a plot of discontinuity persistence versus bearing of the location vector for the $f = 200$ mm mapping data. The grouping of points shows a wide scatter, however several general observations can be made:
Most discontinuities (86%) are clustered below persistence of 17.5 m; the remaining 14% of high and very high persistence features appear randomly distributed.

No data is available between bearings of approximately 200° to 250° due to the occluded zone where insufficient image overlap prevented the construction of digital terrain models.

The maximum measured persistence is 46 m (less than the 125 m measured in the $f = 100$ mm survey) due to the influence of truncation of large discontinuities and the computational restrictions on model size and resolution.

Figure 5-57: Discontinuity persistence vs. bearing of location vector, $f = 200$ mm survey; ISRM (1978) upper limit for very high persistence (20 m) is highlighted.

Figure 5-58 shows a plot of discontinuity persistence versus bearing of the location vector for the $f = 300$ mm mapping data. Discontinuity data is only available between bearings of approximately 110° to 230°, because the $f = 300$ mm survey focused only on the southeast wall. As with the previous surveys, results show a wide scatter, however several generalized observations can be made:
Most discontinuities (93%) are clustered below persistence values of 15 m; the remaining 7% of high and very high persistence features appear randomly distributed about azimuth bearings of 120° to 220° with the exception of:

- A cluster of high to very high persistence discontinuities (15 m to 31 m) occurs between bearings of 180° to 220°, in the southwest corner of the pit near the A418 fault.

The maximum measured persistence is 43 m (less than both the $f = 200$ mm and $f = 100$ mm survey results) due to the influence of truncation of large discontinuities and the computational restrictions on model size and resolution.

![Figure 5-58: Discontinuity persistence vs. bearing of location vector, $f = 300$ mm survey; ISRM (1978) upper limit for very high persistence (20 m) is highlighted.]

### 5.5.2.2. Influence of Survey Resolution

Computational requirements and available software partially control the size and resolution of 3-D terrain models that can be simultaneously displayed. In the $f = 100$ mm survey, resolution is sufficiently low such that the entire A418 pit wall can be simultaneously displayed and mapped. However, the increased resolution of the $f = 200$ mm and $300$ mm surveys restricts the total area that can be simultaneously displayed.
The ground pixel size can be calculated for each survey according to the by the equation:

$$\text{Ground Pixel Size} = \frac{\text{distance}}{\text{focal length}} \times \text{pixel size on CCD image sensor}$$

For the camera model used to carry out the photogrammetry survey (Canon EOS 5D Mark II) the pixel size on the image sensor is 6.41 μm. For a camera distance of 500 m (corresponding to the average line-of-site the distance from the pit crest to the opposite wall), the ground pixel sizes are:

- $f = 100$ mm
  $$\text{Ground Pixel Size} = \frac{500\text{m}}{0.1\text{m}} \times (6.41 \times 10^{-6}) = 3.2 \text{ cm}$$

- $f = 200$ mm
  $$\text{Ground Pixel Size} = \frac{500\text{m}}{0.2\text{m}} \times (6.41 \times 10^{-6}) = 1.6 \text{ cm}$$

- $f = 300$ mm
  $$\text{Ground Pixel Size} = \frac{500\text{m}}{0.2\text{m}} \times (6.41 \times 10^{-6}) = 1.1 \text{ cm}$$

The distributions of discontinuity persistence reflect the varying resolution of each survey. Higher resolution is associated with an increase in the recognition and mapping of smaller discontinuities, and with truncation of larger discontinuities due to limited window size. Table 5 - 14 summarises the overall minimum, average, and maximum persistence values for each survey. To limit the influence of censoring bias, slope stability analyses should always consider the maximum expected persistence values, including explicit representation of at least the top 10% most persistent structures from the $f = 100$ mm survey. Statistical methods need to convert persistence frequency observations from mapping to probability values in order to derive statistical distributions that can be applied to probabilistic analysis.
Table 5 - 14: Summary of mapped discontinuity diameters of from photogrammetry survey.

<table>
<thead>
<tr>
<th>Survey Focal Length (mm)</th>
<th>Minimum Persistence (m)</th>
<th>Average Persistence (m)</th>
<th>Maximum Persistence (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>1.6</td>
<td>17.4</td>
<td>125.3</td>
</tr>
<tr>
<td>200</td>
<td>0.8</td>
<td>9.5</td>
<td>45.8</td>
</tr>
<tr>
<td>300</td>
<td>0.3</td>
<td>5.7</td>
<td>42.6</td>
</tr>
</tbody>
</table>

Figure 5 - 59 shows a section of bench face intersected by a segment of diabase dyke. The transition from $f = 100$ mm to $f = 200$ mm and $300$ mm is accompanied by an increase in resolution that allows progressively smaller features to be resolved, including finer resolution of small brittle blast-induced cracks that may not be distinguishable in the lower resolution surveys. Initial assessment at low resolution provides useful insight into large-scale major structures, but refined analysis at high resolution helps to quantitatively assess rock mass parameters including block size, shape and discontinuity spacing at bench scale or smaller.
Figure 5 - 59: Comparison of model detail for increasing focal lengths, highlighting influence of resolution of discontinuity cut-off lengths (After the method for comparing photogrammetry resolution suggested by Lee, 2011).
5.5.3. Relating Persistence, Intact Rock Bridges and Fracture Connectivity

Fracture connectivity is defined by the average number of fractures intersections in a rock mass. Discontinuity networks with greater connectivity have higher potential to form linked pathways for fluid flow, or continuous surfaces for potential slope failures. Ozkaya and Mattner (2003) applied the concept of percolation threshold to 2-D conceptual analysis of fractured hydrocarbon reservoirs. In stochastic 2-D DFN models, they found that the average number of fracture intersections, denoted $\lambda$, describes three stages of fracture connectivity (Figure 5 - 60).

- **Isolated Fractures** occur when $\lambda < 1$
- **Clusters** of connected fractures occur when $1 < \lambda < 2$
- **Fully-connected** fracture networks extend across the entire model domain when $\lambda > 2$

![Figure 5 - 60: Three cases of fracture connectivity (Ozkaya and Mattner, 2003, reproduced by permission).](image)

In addition, they found that the relative frequency of connected fractures required for percolation is inversely correlated with persistence. The conclusion is intuitive: for a rock mass with more persistent discontinuities, fewer intersections are needed to form a connected fracture network that traverses the model domain, because each discontinuity can form a larger segment of the connected path (Figure 5 - 61).
Figure 5-61: Conceptual 2-D fracture networks above percolation threshold ($\lambda > 2$) showing A) low persistence discontinuities with many fracture intersections, and B) high persistence discontinuities with fewer fracture intersections.

The wall rock in the A418 pit is mostly blocky to massive with frequent occurrence of high persistence (10 - 20 m) to very high persistence (> 20 m) discontinuities, and some outlier structures with much larger persistence ranging from 20 m to over 100.

Peak frequency for fitted discontinuity diameters occurs for values as high as 8 m to 9 m for the $f = 100$ mm photogrammetry survey, and as low as 3 m to 4 m for the $f = 300$ mm survey. Despite the predominant quantity of medium persistence joints, the demonstrated potential for extremely persistent discontinuities of 100 m or more indicates the potential, albeit low, based on frequency, that inter-ramp or overall-slope scale failure surfaces could occur along a failure surface comprised of a small number of extremely persistent adverse structures.

The findings of this field investigation could form the basis for additional DFN-based studies where statistical analysis of DFN iterations may help to better understand the connectivity ($\lambda$ values) of the fracture network around the A418 pit, and the likelihood of inter-ramp or overall failure surfaces comprising extremely persistent structures.
5.5.4. Surface Morphology of Major Discontinuities

Discontinuities are rarely perfectly planar. Although some of the most commonly used DFN models simplify discontinuities as planar polygons or circles (Staub et al., 2002), true morphology of discontinuities is often rough, irregular and branching. Microscopic studies have repeatedly demonstrated that the microfabric of crystalline rock is frequently comprised of multiple overprinted generations of irregular, non-planar discontinuities including inter- and intra-grain scale cracks (e.g. Catlos et al., 2011; Tentler and Amcoff, 2010; Onishi and Shimizu, 2005; Fonseka et al., 1985).

Similarly, studies of regional-scale structural geology and fault tectonics have shown striking parallels between the development of kilometre-scale brittle faults and laboratory-scale fracture in unconfined or triaxial compression tests (Figure 5 - 62).

![Figure 5 - 62: A) Coyote Creek Fault in Southern California (After Segall and Pollard, 1980; and Sharp and Clark, 1972, reproduced by permission); and B) simulated UCS test with tensile wing cracks growing from inclined cracks separated by an intact rock bridge.](image)

Faults may branch and bifurcate, such that discontinuous fault segments observed on surface may converge at depth into a single structure. Intervening zones between discontinuous fault segments are, effectively, regional-scale rock bridges (Figure 5 - 63, modified after Segall and Pollard, 1980).
At Diavik, joint roughness profiles indicate that brittle step surfaces occur on discontinuities ranging from the centimetre scale (Figure 5 - 9, Section 5.4.2) up to steps in the order of 1 m on very high persistence discontinuity surfaces exposed in bench faces (Figure 5 - 64).

The influence of intact rock bridges at Diavik may be divided into two categories:

1) **Cohesive rock bridge effect**: conventional bridging of intact rock between the tips of non-persistent discontinuities; and bridging of *rock mass* between the tips of major structures separated by distances of 10s of metres of more.

2) **Frictional rock bridge effect**: discontinuity roughness profiles indicate the presence of the step surfaces from the millimetre scale to approximately 1 m in height. If discontinuities bifurcate during propagation, forming step surfaces, then the interlocking effect provided by the step surfaces will add to the peak frictional strength of the rock mass.

The relative importance of the two rock bridge types is uncertain; further investigation by numerical modelling is warranted.
Figure 5 - 64: Step surfaces up to approximately 1 m in size on a very high persistence, curved joint intersecting the bench face in the west wall at RL + 300 m; person for scale.
5.5.5. **Influence of Blasting-Induced Damage**

The severity of blasting damage in the A418 pit is variable, and shown by the range of different $B_{21}$ values observed in trace mapping. Although vertical bench faces and excellent wall quality are common, minor blast damage is frequently observed in window maps, and can generally be divided into two groups:

**Type 1:** Dilation and extension of pre-existing discontinuities by overpressure generated from explosive gases; an example is shown Figure 5 - 65 from the southwest corner of the pit wall, on the RL + 300 - 330 m bench face. Type 1 damage is the primary mechanism by which intact rock bridges (and thus cohesive strength) is destroyed along pre-existing joint planes.

- Little (1999), Little et al. (1999), Baczynski (2000) accounted for the influence of blasting on joint cohesion at Ok Tedi mine by degrading the cohesion on blast-damaged geological structures to zero, and undertaking probabilistic step-path slope stability analysis with the STEPSIM4 code.

**Type 2:** Brittle tension fractures caused by blast-induced tensile strain waves that may form steps between pre-existing discontinuities, or may be dispersed throughout the rock mass. Two further sub-types are proposed:

- **Type 2a:** systematic sub-horizontal fractures between borehole half-barrels, with spacing that tends to decrease towards the top of bench faces, as described by Hagan et al. (1978) (Figure 5 - 66A)
- **Type 2b:** non-systematic brittle “internal-spalling” or “release of load” fractures that may occur anywhere in the bench face, and may be isolated from pre-existing discontinuities, and are induced by strain wave propagation and reflection from free air interfaces (Hagan, 1982; Yu and Vongpaisal, 1996).
  - Type 2b fractures may exploit pre-existing micro-fabric (i.e. the proto-joint network of partially coalesced micro-cracks, described by Hencher (2012) that precedes the formation of fully continuous mechanical joint planes (Figure 5 - 66B).
Figure 5 - 65: Dilation and propagation of a steeply-dipping joint by blasting damage; recent blast-induced brittle crack extends from the tip of the pre-existing joint.
Figure 5 - 66: Proposed examples of Type 2a and Type 2b blast-induced damage.
The severity of Type 2 damage is recorded in window maps with the blast damage intensity parameter $B_{21}$, which describes the length of blast-induced fractures per unit surface area of bench face. In contrast, the severity of Type 1 damage is reflected in the rock bridge trace intensity $R_{21}$ values, which record the total length of intact rock bridges traced between pre-existing discontinuity tips. As the severity of Type 1 blasting damage increases, pre-existing discontinuities are extended and thus the length of intact rock bridges between discontinuity tips is decreased, causing the rock bridge trace intensity $R_{21}$ to decrease.

Type 1 damage is most easily distinguished in association with joints of high persistence (i.e. $\geq 10$ m) or greater, where new, curving brittle fractures can be readily identified extending from the tips of the pre-existing joint (Figure 5 - 65). The damage mechanism of discontinuity extension by explosive gas infiltration favours longer joints over less persistent structures, because stress concentration at crack tips increases with crack length (Hagan et al., 1978). Strain wave energy, causing Type 2 damage, will also work to extend existing joints preferentially, before nucleating new microcracks and extending existing, smaller discontinuities (Hagan and Morphet, 1986).

Blasting has caused the extension and dilation of pre-existing near-surface discontinuities throughout the A418 pit, with a general trend that shows increases in dilation induced by blasting proportional with increases in persistence. The role of blast-induced damage in potential slope instability will differ with scale.

Bench-scale failures and inter-ramp scale failures occur only within the blast damage zone, which may be as thick as 50 m behind the face where blasting techniques is not optimised for geological conditions (Little, 1999). However, based on the guidelines of Hoek and Karzulovic (2000), the blast damage zone may be as shallow as 5 to 10 m behind the face in areas where (1) controlled blasting techniques were used; (2) the blast was detonated towards a free face; and (3) excellent final wall control is maintained, with minimal overbreak, a clean vertical bench face and preserved borehole half barrels. In inter-ramp or overall slope scale instability, the influence of blasting damage may be confined to the near-surface parts of the critical surface, at the toe and crest of the slope.
6. Field Investigation at Highland Valley, BC Canada

6.1. Mine Site Overview

Highland Valley is the largest base metal mine in Canada, producing predominantly copper and minor amounts of molybdenum. The property is located approximately 50 km southwest of Kamloops, BC. The owner-operator, Teck Resources Limited, holds 97.5% share of the mine, which includes three active open pits: Valley, Lornex and Highmont. The Valley pit is currently the only pit producing ore, but the mine plan includes expansion projects for all three pits that will extend the mine life to 2026 (Teck, 2011).

This investigation focuses on the Upper West Wall push-back of the Valley pit, where modified discontinuity survey and ground-based LiDAR were used to help characterize slope geometry, discontinuity orientation, persistence and potential intact rock bridge content.

6.2. Geological Setting

The Highland Valley porphyry district hosts five major porphyry copper-molybdenum deposits: Valley, Lornex, Bethlehem, Highmont and JA. All five deposits occur within the Bethsaida granodiorite, a unit which covers a 15 km² area in the central region of the larger (1000 km²) Upper Triassic age Guichon Creek batholith (Figure 6 - 1). The Bethsaida granodiorite was intruded during the earliest major phase of the batholith (Casselman et al., 1995).
Figure 6 - 1: Simplified regional geological map of the Guichon Creek Batholith around Highland Valley, highlighting major granodiorite phases, faults and the footprints of the open pits at Highland Valley (Modified and reproduced after McMillan, 1978).
Two major fault systems intersect near the Valley deposit: the Highland Valley fault, which varies between approximately West-trending and Northwest-trending; and the Lornex Fault, which trends North-South. Discontinuities in the local rock mass fabric, from faults down to the scale of joints and quartz veinlets, tend to parallel these two regional trends in orientation (Casselman et al, 1995).

The fracture network produced by regional faulting played an important role in the development of the ore deposits. Casselman et al. (1995) described how all five major ore deposits developed through multiple phases of sulphide mineralization by hydrothermal fluids. They noted that almost all sulphide mineralization occurs in fractures, veins, faults or breccias, with sulphide-rich ore zones distributed around sulphide-poor, siliceous core zones. Because mineralization was governed by fracture-flow of hydrothermal fluids, higher grade ore zones are frequently associated with more intensely fractured rock mass. The geomechanical implications of this emplacement model suggest that higher-grade ore zones occur in poor quality rock mass with more intense fracturing, higher degree of alteration, and lower shear strength than non-mineralized rock.

The Valley deposit (and the Valley pit rock mass in general) is characterised by pervasive silicic alteration, which is expressed in two main features:

(1) A strongly-developed quartz stockwork occurs throughout the Valley deposit, called the “silicic re-entrant”. The stockwork zone plunges to the Northwest, with quartz veins typically ranging from 1 cm to 2 cm in width, with some veins up to 25 cm wide.

(2) Zones of massive, pervasive silicification occur where quartz has “flooded” the plutonic granodiorite.

The secondary quartz features listed above consist of silica that was leached from pre-existing, rock forming minerals during episodes of hydrothermal alteration. Two geotechnically significant alteration types occur in association with the secondary quartz: argillic alteration and phyllic alteration.
Argillic alteration zones are characterised by the replacement of plagioclase feldspar by clay minerals including kaolinite and montmorillonite; sericite is also common (Casselman et al., 1995). Osatenko and Jones (1976) showed using X-ray analysis that the primary product of argillic alteration in the Valley deposit is kaolinite clay. They also showed that the degree of argillic alteration is directly related to fracture intensity. Areas of more intense fracturing show more complete degradation of plagioclase and higher concentrations of kaolinite and sericite.

Phyllic alteration zones are characterised by the fracture-controlled replacement of pre-existing feldspars with micaceous minerals including muscovite and sericite. Casselman et al. (1995) noted that phyllic alteration in the Valley pit is strongly associated with copper-mineralized quartz veins.

The relationship between degree of alteration with intensity of fracturing has important implications for potential slope instabilities. Phyllic and argillic alteration minerals possess low frictional strength, and may play an important role in preconditioning altered discontinuities to shear failure. Day et al. (2012) demonstrated through case studies of (1) inter-bedded limestone and shale, and (2) a porphyry copper deposit that variability of secondary “intrablock” structures including veins, stockworks and microfractures can have a significant impact on overall rock mass quality. Past studies into the Lornex pit by Tosney et al. (2004) and Newcomen et al. (2003) have reported frictional angles for discontinuities in Bethsaida granodiorite as low as 12°, with negligible cohesion, based on laboratory direct shear test results.

In the Valley pit Upper West Wall, areas of intense alteration can also be expected to have lower global rock mass strength, as the inter- and intra-grain scale tensile strength is reduced in zones of kaolinite and sericite relative to unweathered granodiorite. Conversely, barren quartz-filled discontinuities with no associated argillic or phyllic alteration products may be strengthened in comparison with fresh rock, as an apparent cohesion is imparted by the quartz cement.
6.3. Geotechnical Characterisation

This section summarises the slope geometry of the Upper West Wall at the time of the field investigation, and also presents some relevant geotechnical parameters from previous investigations. Past investigations into the Upper West Wall have been undertaken by Piteau Associates (Rose and So, 2010), who also undertook a review of previous work by Golder Associates (2006).

Previous studies into the stability of the Lornex pit, to the south of the Valley pit, were undertaken by researchers including Newcomen et al. (2011; 2003), Tosney et al. (2004) and Tosney (2001).

6.3.1. Slope Geometry

The Upper West Wall is a push-back of the Valley pit west wall that forms a curved amphitheatre-shaped slope. The slope is excavated in 15 m-high benches, with typical bench width of approximately 15 m, and no haul ramps. The maximum overall height of the slope at the time of the field investigation was approximately 120 m, and the planned final height is 240 m.

Figure 6 - 2A presents a plan view of the Upper West Wall slope design, showing the two geotechnical domains, North and South, which are divided by the steeply northwest-dipping Yellow Fault (~80°/300°). The boundaries of four design sectors are also overlain, as recommended by Piteau Associates (Rose and So, 2010). Figure 6 - 2B shows a plan view of the LiDAR point cloud from the current investigation, as well as the traces of two cross-sections that were extracted to examine the as-built slope profile.

The lithology of the Upper West Wall is approximately uniform, consisting entirely of granodiorite; thus, previous investigations have differentiated geotechnical units based on changes in intensity of fracturing, weathering and alteration, with less emphasis on contrasts in lithology. Figure 6 - 3 shows cross-section UW1 from a 3-D block model of interpreted RQD, constructed by Piteau Associates (Rose and So, 2010).
Figure 6-2: (A) Design plan of Valley Pit Upper West Wall, divided into North Domain and South Domain by the Yellow Fault, with Design Sectors DS-1 to DS-4 (Modified after Rose and So/Piteau Associates 2010, by permission); and (B) LiDAR point cloud of Upper West Wall scanned in August 2011.
Figure 6 - 3: Interpreted cross-section UW1 through Valley Pit Upper West Wall, based on pre-mining, undisturbed RQD interpreted from a block model constructed from previous borehole drilling data (Modified after Rose and So / Piteau Associates 2010, by permission).
The block model of RQD was developed by Piteau Associates (2009) using the mine planning software Surpac (Gemcom Software, 2012), with input data from borehole logs and previous mapping (Piteau Associates / Rose and So, 2010). The resulting estimate of RQD shown in cross-section UW1 suggests that fracture intensity may be elevated near the toe of the Upper West Wall. The current investigation emphasizes comparison of the severity of the fracturing throughout the slope face in order to better understand the spatial variation of fracture intensity, persistence and potential intact rock bridge content.

Figure 6 - 4 presents a panoramic view of the Upper West Wall, spanning approximately azimuth 200° to 300°. The main trace of the Yellow Fault and an associated multi-bench scale fault segment are highlighted, and major accumulations of bench-scale debris are outlined. Some major discontinuities are also traced, however in all cases the persistent joint traces have been overprinted by more recent fractures that may include blast-induced damage and tectonic damage arising from fault reactivation. The height of debris generally increases from South towards North, with the maximum height of debris occurring in Design Sector 3.

Cross-sections UW2 and UW3 extracted from the LiDAR survey help to illustrate the effects of progressive bench-scale failures (Figure 6 - 5 and Figure 6 - 6). Cross-section UW2 was extracted from Design Sector 3 and cross-section UW3 was extracted near the boundary of Design Sectors 2 and 3, approximately located along the Yellow Fault. Both profiles show that progressive small-scale failures have caused accumulation of debris in the mid-slope benches, increasing the volume of loose debris that may act as a source for rockfall, and flattening the overall slope angle.

Where benches are not covered with debris, bench face angles are up to approximately 72°. In both profiles the overall slope angle is approximately 36°.
Figure 6-4: Panoramic view of Valley pit Upper West Wall, looking West, with major structures highlighted; the mid-slope benches in the centre of the image are completely covered with debris from progressive bench-scale ravelling. A rockfall catch berm has been constructed on the RL + 1400 m level to protect the working area at RL + 1385 m.
Figure 6 - 5: Cross-section UW2, View North 36.2° overall angle

Bench faces indistinguishable; covered with debris

Figure 6 - 6: Cross-section UW3, View Northwest 36.3° overall angle.

Bench faces angle up to ~ 72°
6.3.2. Rock Mass and Discontinuity Properties

This section reviews discontinuity and rock mass parameters from previous studies of Highland Valley. Although comprehensive characterization of rock mechanical properties is beyond the scope of this thesis, estimates of rock mass and discontinuity strength and stiffness parameters are needed to better understand the role of intact rock bridges in providing support to discontinuity-controlled slope instabilities. Table 6 - 1 summarises the rock mass and discontinuity properties applied to a UDEC study of deep-seated toppling in the Lornex pit by Tosney et al. (2004).

Table 6 - 1: Summary of rock mass and discontinuity parameters applied to a UDEC study of deep-seated toppling in the Lornex pit by Tosney et al. (2004).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Granodiorite Rock Mass</th>
<th>In-dipping, toppling joint set A</th>
<th>Conjugate secondary joint set B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/m³)</td>
<td>2700</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bulk Modulus (GPa)</td>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear Modulus (GPa)</td>
<td>7.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Friction Angle, φ (°)</td>
<td>25</td>
<td>12</td>
<td>27</td>
</tr>
<tr>
<td>Cohesion, c (MPa)</td>
<td>1</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Tensile Strength, σₜ (MPa)</td>
<td>0.1</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Normal Stiffness (GPa/m)</td>
<td>-</td>
<td></td>
<td>4</td>
</tr>
<tr>
<td>Shear Stiffness (GPa/m)</td>
<td>1</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Tosney (2004) found that discrete element models of deep-seated toppling may be controlled by the existence of continuous, widely spaced major discontinuities (i.e. faults) with low frictional strength.

Table 6 - 2 summarises the findings of Newcomen et al. (2003) who reported on the strategies developed at Highland Valley to manage ongoing slope displacements in the Lornex pit. Their results show that the Bethsaida granodiorite (BGD) rock mass is more fractured and weaker near the Lornex fault, and discontinuities show increasing evidence of shearing with proximity to the fault. They also reported the results of laboratory direct shear tests on discontinuity samples from upper and lower BGD, which led to suggested shear strength parameters of $\varphi = 19^\circ$ and $c = 0$ for all joints in those units.

### Table 6 - 2: Summary of selected rock mass classification parameters of Bethsaida Granodiorite and Lornex Fault Zone, from previous study of the Lornex Pit (Newcomen et al., 2003).

<table>
<thead>
<tr>
<th>Unit</th>
<th>$RMR_{76}$</th>
<th>UCS (MPa)</th>
<th>Joint Condition Factor, $J_c$ (Bieniawski, 1976)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper BGD</td>
<td>70</td>
<td>57</td>
<td>19 Slightly rough surfaces; separation &lt;1 mm; hard joint wall contact</td>
</tr>
<tr>
<td>Lower BGD</td>
<td>52</td>
<td>42</td>
<td>13 Slightly rough surfaces; separation &lt;1 mm; soft joint wall contact</td>
</tr>
<tr>
<td>Lornex Fault Zone</td>
<td>28</td>
<td>15</td>
<td>6 Slickensided surfaces OR Gouge &lt;5 mm thick; joints open 1-5 mm; continuous joints</td>
</tr>
</tbody>
</table>

Alteration and mineralization at Highland Valley are associated with fracture flow of hydrothermal fluids. Thus the degree of alteration, and the rock mass strength, may be correlated with measurements of rock mass fracturing such as Rock Quality Designation (RQD) or fracture intensity measurements (i.e. $P_{10}$, $P_{21}$ or $P_{32}$). Measurements of rock
mass fracturing may also be used to help estimate potential intact rock bridge content, and the cohesive strength benefit of intact rock bridges to the rock mass.

Diederichs (1999) compared the support equivalency of cross-sectional rock bridges (i.e. in-plane rock bridges) with the support pressure provided by common rock support elements, reported by Stillborg (1994). He noted that a 1.2% cross-sectional rock bridge content can provide 130 kPa of support pressure, equivalent to a 2x2 m pattern of Double Strand Cablebolts. If field evidence supports the existence of intact rock bridges in the Upper West Wall, then the rock mass cohesive strength may be augmented (Figure 6 - 7) by an effective support pressure in the range of 100 to 300 kPa.

![Figure 6 - 7: Equivalent support pressure of cross-sectional rock bridges loaded in direct tension, from the analysis by Diederichs (1999), and the support pattern data of Stillborg (1999).](image)

The range of published rock bridge contents measured \( a \ posteriori \) on slope failure surfaces (Chapter 2) is in the order of 1 to 10%, suggesting potential strength benefit of over 1 MPa for 10% rock bridges on a failure surface. Thus, in the current field investigation, detachment niches and sliding surfaces of bench-scale wedges are
compared with multi-bench scale trends in slope geometry and rock mass quality, in an attempt to suggest a rationale for quantified estimation of intact rock bridge content.

6.4. Field Investigation

6.4.1. Overview

The field investigation into the Upper West Wall incorporates results from modified discontinuity survey and ground-based LiDAR scans. Additional spot-mapping observations are presented on the characteristics of selected bench-scale failures.

Field observations and post-processing results from the LiDAR survey are interpreted across multiple scales in an attempt to better understand the range of length-scales of discontinuity persistence that are most influential to stability of the Upper West Wall. Potential 3-D configurations of intact rock bridges are discussed further in Section 6.5.2, with specific reference to mapping results and the local structural geology.

6.4.2. Modified Discontinuity Survey

The modified discontinuity survey is based on limited scanline data supplemented with close-range (i.e. outcrop-scale) spot mapping observations. Fracture intensity and discontinuity persistence are interpreted in the context of local geological history, with emphasis on the chronology of fracture network development leading to modern rock mass conditions.

6.4.2.1. Field Rock Mass Classification

Despite the localized occurrence of massive rock mass, the majority of the Upper West Wall is comprised of a blocky to very blocky rock mass with approximately cubic shaped blocks of edge length 1 m or less (Figure 6 - 8). Even where spacing of discontinuity traces is very wide, or where large planar discontinuity surfaces daylight, with edge lengths exceeding 10 m, debris piles indicate the comminuted rock mass tend to fracture into blocks not usually exceeding 3 m edge lengths, with the majority of block volumes less than 1 m$^3$. Discontinuity surface conditions are typically fair to poor, with frequent veneers or coating of argillic and phyllic alteration products.
The modern network of discontinuities reflects a complex, episodic history of tectonic activity and associated fracturing. The stages of fracture development can be broadly considered using a simplified timeline comprising 5 episodes of damage to the rock mass, beginning with the formation of the Bethsaida Granodiorite:

**D1 - Cooling Joints:** the oldest joints are probably associated with cooling-induced stresses, occurring after emplacement of the Bethsaida phase of the Guichon Creek Batholith, approximately 210 Ma BP (Casselman et al., 1995).

**D2 – Stress Change During Uplift and Exhumation:** the Guichon Creek Batholith was rapidly uplifted after cooling, and was exposed at surface during the Early Jurassic (Casselman et al., 1995); the relief of confinement may be associated with the development of topography-parallel brittle tensile fractures (i.e. sheet joints or exfoliation; Leith, 2012; Hencher et al., 2011; Bahat et al., 2005).

**D3 - Preliminary Regional Faulting:** The Lornex and Highland Valley Faults developed prior to mineralization, producing pervasive fault-parallel fractures ranging from the size of quartz veinlets up to large-scale extensional fractures and dykes. Casselman et al. (1995) propose that displacements on the Lornex Fault have occurred in periodic episodes over a long interval spanning the Mesozoic to the Tertiary.

**D4 - Mineralization and Fault Re-Activation:** Episodes of hydrothermal alteration and fluid flow deposited infill within pre-existing tectonic fractures, formed by quartz leached from the country rock. Episodes of hydrothermal alteration were also likely associated with hydraulic fracture growth caused by infiltrating fluid pressures, as well by chemical and thermal alteration. The end products of alteration include the characteristic phyllic and argillic minerals observed throughout the pit. Periodic re-activation of large faults may also be associated with the development of neotectonic structures. Casselman et al. (1995) described the discovery in past borehole drilling investigations of minor, young faults in the Valley deposit that offset, and therefore post-date the mineralized stockwork zone.

**D5 – Quaternary Damage:** Geologically recent damage can be expected to have accumulated during repeated episodes of loading and unloading by glaciers during
the Quaternary. Extensive glaciolacustrine and glaciofluvial deposits, and paleochannels from subglacial drainage, occur on the East side of the Valley pit, indicating that Quaternary glaciation has significantly impacted the local geomorphology (Fortin et al., 2011). Leith (2012) studied stress development in alpine valleys, and found that glacial ice loading can impact sub-glacial development of brittle fractures. However, specific assessment of the degree of glacial-induced brittle damage is beyond the scope of this investigation.

**D6 - Recent Mining-Induced Damage:** The excavation of the Valley open pit has caused further disturbance to the rock mass and created new damage in two main forms:

a. **Blasting-induced Damage:** Immediate damage occurs during production blasting of the slopes. Production blasting is associated with the preferential extension of pre-existing discontinuities via expansion of explosive gases, and also the creation of new fractures via tensile strain-wave reflections from free-surface interfaces, release-of-load type fracturing and other dynamic damage mechanisms (Dehghan Banadaki and Mohanty, 2012; Mohanty et al., 2005; Hagan et al., 1978).

b. **Time-Dependent Slope Relaxation:** Pit excavation reduces confining stresses in the slope and may result in slow displacements during mine operation; past studies in landslide have proposed a primary creep phase (relaxation) that is followed by a secondary phase of linear time-dependent deformation (Mercer, 2006). Although some displacement may be accommodated via shear on pre-existing structures, creep may also be associated with the creation of new cracks and slow sub-critical growth of existing cracks (Damjanac and Fairhurst, 2010; Rinne, 2008).
Figure 6 - 8: Selected examples of locally massive conditions, based on block size interpreted from discontinuity trace spacing, dimensions of planar discontinuity exposures, and detachment niches of past wedge failures. Although the in situ rock mass may suggest locally massive conditions, debris piles indicate comminution of the rock mass to predominantly blocks < 1 m³.
The existing fracture network reflects an intensely fractured rock mass where the older, more persistent discontinuities (i.e. originating from episodes D1 and D2) are overprinted and frequently completely obscured in bench face exposures by intense concentrations of low-persistence fractures that originated in episodes D3 to D6. Figure 6 - 9 illustrates the effect of over-printed rock mass damage, where a northeast-dipping joint set denoted J1 is interpreted to have originated during episodes D2 to D3, and is overprinted by intense fracturing originating from more recent fault re-activation, glacial loading-induced damage, and blasting-induced damage during episodes D4 to D6. The predominant discontinuity sets are further discussed in the following section.

Figure 6 - 9: Older joints associated with cooling and uplift are overprinted by more recent fracturing associated with fault re-activation and blasting (Estimated GSI ≈ 40-50; D ≈ 1).

Past tectonic episodes are associated with large-scale shear displacements (i.e. > 1 km) occurring on the regional-scale Highland Valley and Lornex faults (Casselman et al., 1995). Major deformation episodes may also be associated with the formation of clusters of deformation bands, such that large faults including the Yellow Fault may have
accumulated displacements via repeated localized slip events occurring along finite patches of the fault (e.g. Shipton and Cowie, 2003).

The concentration of shear strains around major faults is reflected in the spatial distribution of fracture intensity in the Upper West Wall. Proximal to the Yellow Fault, the rock mass grades into disintegrated and sheared material (Figure 6 - 10). A combination of mechanical damage by shearing, and chemical weathering by groundwater flowing through the fault has degraded the fault core.

**Figure 6 - 10:** Close-up view of Yellow fault on RL +1385 m bench, showing transition from extremely fractured and altered rock (GSI ≈ 20-30) to completely disintegrated in the fault core.

**6.4.2.2. Predominant Discontinuity Sets**

Preliminary bench-face mapping (n = 30) carried out on the RL + 1400 m bench was used to identify the predominant discontinuity sets that contribute to progressive bench-scale failures. A cut-off length for discontinuity persistence of 0.5 m was applied in order to capture the most dominant joint sets; smaller joints and brittle fracture features were not measured directly. Thus the sample population includes predominantly medium-persistence discontinuities (3 – 10 m trace lengths; IRSM, 1978) with an average persistence of about 6.9 m. The largest measured persistence was estimated in the field
as greater than 60 m, for a segment of the Yellow Fault; the pole is shown with a gold star symbol in Figure 6 - 11.

Figure 6 - 11: Pole plot of discontinuity sets recorded during field mapping, sorted according to ISRM (1978) persistence categories.

The preliminary results suggest that most low to medium persistence joints can be grouped into three major discontinuity sets, denoted J1, J2 and J3. Table 6 - 3 presents a summary of the orientations, sample size and Fisher K value for each discontinuity set. Larger Fisher K values indicate a more tightly-clustered discontinuity set.

Table 6 - 3: Summary of discontinuity set orientations identified in field mapping.

<table>
<thead>
<tr>
<th>Set</th>
<th>Mean Dip (°)</th>
<th>Mean Dip Direction (°)</th>
<th>Sample Size</th>
<th>Fisher K</th>
</tr>
</thead>
<tbody>
<tr>
<td>J1</td>
<td>44</td>
<td>59</td>
<td>7</td>
<td>39.7</td>
</tr>
<tr>
<td>J2</td>
<td>80-84</td>
<td>104/284</td>
<td>10</td>
<td>61.9</td>
</tr>
<tr>
<td>J3</td>
<td>58</td>
<td>212</td>
<td>11</td>
<td>11.8</td>
</tr>
</tbody>
</table>

Due to the limited sample population, standard deviations are not reported, and the reported mean values should be considered only as preliminary indicators of
predominant structural trends. Table 6 - 4 summarises the preliminary persistence (trace length) data for the major sets.

Table 6 - 4: Summary of preliminary persistence data for major discontinuity sets identified in field mapping.

<table>
<thead>
<tr>
<th>Set</th>
<th>Persistence (m)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum</td>
<td>Average</td>
<td>Maximum</td>
</tr>
<tr>
<td>J1</td>
<td>1.5</td>
<td>5.1</td>
<td>10</td>
</tr>
<tr>
<td>J2</td>
<td>0.5</td>
<td>2.8</td>
<td>7.5</td>
</tr>
<tr>
<td>J3</td>
<td>2</td>
<td>6.8</td>
<td>10</td>
</tr>
</tbody>
</table>

Figure 6 - 12 illustrates the mean (toe to crest) slope orientation and daylight envelopes for three major sections of the Upper West Wall. The south section comprises Design Sectors 1 and 2; the west section corresponds directly to Design Sector 3, and the north section comprises Design Sector 4. The local orientation of the slope is a key determinant of kinematic stability for wedge, planar sliding and toppling failures occurring on major joint sets.

Figure 6 - 12: Mean slope dip/dip direction and daylight envelopes for three major sections of the Upper West Wall.
Set J1 dips to the Northeast, and may act as a basal surface for planar sliding and bench scale failures, particularly in Design Sector 3. The distribution of J1 discontinuities may play an important role in the increased severity of backbreak and incidence of bench-scale sliding and wedge failures frequently observed throughout Design Sector 3. The minimum field persistence measurement for J1 is 1.5 m; the maximum is 10 m; and the average is 5 m.

Joints from set J2 may act as release surfaces for bench-scale planar sliding failures. This joint set has a sub-vertical dip with dip direction varying between 104° and 184°. The broad northeast-southwest strike of set J2 is sub-parallel to the strike of the Yellow Fault. The preliminary results support the assertion of Casselman et al. (1995) that outcrop-scale structures in rock mass fabric may occur parallel to the orientations of large faults. The maximum persistence of set J2 assessed in the field is approximately 7.5 m, the minimum is 0.5 m, and the average approximately 5 m.

Set J3 dips moderately to steeply towards the Southwest, and may act as rear release surfaces for sliding failures, or contribute to block toppling failure, particularly in Design Sector 3. The minimum persistence observed for set J3 is 2 m; the maximum is 10 m; and the average 6.8 m.

Figure 6 - 13 presents a selection of roughness profiles collected during field mapping, for a limited sample of discontinuities in sets J1, J2 and J3. Discontinuity surfaces are observed to be commonly planar and rough or undulating and rough. Some stepped discontinuity surfaces are observed in sets J1 and J3 with brittle secondary crack features including hackle fringes and secondary en echelon cracks occurring in association with the major discontinuity sets. Past experimental studies in glass and rock-like materials have shown that such secondary brittle fracture features typically indicate mixed-mode loading conditions (i.e. combination tensile and shear failure; Bahat et al., 2005; Cooke and Pollard, 1996; Guin and Wiederhorn, 2003). The presence of secondary cracks and step surfaces may contribute to interlocking effects within the rock mass at the centimetre-scale, imparting an effective cohesive strength analogous to bridging of intact rock between discontinuity tips.
Figure 6 - 13: Summary of selected roughness profiles collected during field mapping, with dip and dip direction and estimated JRC (After the method suggested by Barton and Choubey, 1977).
6.4.3. **Ground-Based LiDAR Survey**

The Upper West Wall was scanned at two different resolutions (Figure 6 - 14) in order to investigate the influence of ground point resolution on detection cut-off limits for discontinuities and potential for identification of discrete intact rock bridges.

![Figure 6 - 14: Perspective view of overlapping LiDAR scans, showing initial lower-resolution point cloud comprising 8 separate overlapping scans, and higher-resolution point cloud comprised of 3 scans of Design Sector 2.](image)

The first survey was carried out using a spot spacing (i.e. resolution) of 15; spot spacing is a dimensionless resolution setting that can be modified in the scanner controller software. A spot spacing of 15 results in a ground-point spacing of 5 to 20 cm, depending on the local line-of-site site distance to the scanner and the beam angle of incidence. The final low resolution point cloud of the Upper West Wall is comprised of 8
overlapping scans that were captured separately and merged. Each of the 8 scans took approximately 20 minutes to complete; the overall merged point cloud contains approximately 9.5 million points.

The second survey was carried out using a spot spacing of 7 (approximately double the resolution of the first survey). The increased spot spacing corresponds to a ground point spacing varying from approximately 1 cm to 10 cm. As a result of the increased resolution, each scan took approximately 1 hour to capture. Consequently, only three overlapping, high resolution scans were collected, covering the south slopes of the Upper West Wall, in the South geotechnical domain, in Design Sector 2. The final merged point cloud contains approximately 22.3 million points.

Figure 6 - 15 presents a panoramic perspective image of the two LiDAR scans. The images illustrate the influence of occlusion and perspective bias. The most prominent occlusion effect is the absence of bench floors: because the scanner is positioned at the base of the wall, all bench floors are obstructed. However, occlusion of bench floors is not a concern for structural mapping.

Major discontinuity traces are not visible in the panoramic image of the overall Upper West Wall. The rock mass is characterised by intense fracturing resulting from the multiple episodes of damage evolution, D1 to D5. If very high persistence or larger joints are present (i.e. 20 m +), they are overprinted by many low persistence, closely-spaced discontinuities with a high degree of variability in orientation. Discontinuity survey from the LiDAR point cloud thus required close-range mapping of bench faces, from viewing ranges in the order of 10 to 50 m from the face.
Figure 6 - 15: Comparison of point clouds from lower resolution survey (top) and higher resolution survey (bottom); blank white areas indicate occlusion, where the incident laser beam line-of-sight does not reach the slope surface. Both models show that high persistence structures exceeding 10 m are difficult to distinguish from long range, due to the overprinting by recent damage from episodes D4 and D5; and due to major accumulations of debris from progressive bench failures. The rock mass is highly fractured, containing numerous low-persistence, closely spaced discontinuities with a high degree of variability in orientation.
6.4.3.1. Predominant Discontinuity Sets

In total 952 planes were manually fitted to discontinuity surfaces (Figure 6 - 16). In the first stage of mapping, 308 discontinuities were manually mapped in the lower-resolution point cloud, throughout the North and South geotechnical domains. The boundary between the domains is formed by the Yellow Fault. These discontinuities were then imported and overlain on the higher-resolution point cloud, in order to avoid duplicate measurements. In the second stage of mapping, 644 discontinuities were mapped in the high resolution survey, in the South geotechnical domain, in Design Sector 2.

Figure 6 - 16: Plan view of discontinuity centres mapped in two LiDAR surveys; areas of poor mapping coverage are associated with bench faces obscured by debris.
Figure 6 - 17 presents the overall pole plot for the combined discontinuity data from both surveys, with poles colour-coded according to ISRM (1978) persistence.

![Figure 6 - 17: Pole plot and best-fit great circles for discontinuity sets identified in LiDAR surveys (combined high-resolution and low-resolution surveys).](image)

A broad three-part cluster of poles occurs in the North half of the stereonet, corresponding to a series of overlapping discontinuity sets (J3a, J3b, and J4). Set J4 is not present in the field mapping results, and set J2 appears less prominent than in the field mapping results. However, comparison with the field mapping data should not be considered definitive, due to the very limited sample size (n = 30) of preliminary field mapping data.

Table 6 - 5 summarises the orientations, sample size and Fisher K values for each interpreted discontinuity set. Despite the overall increase in sample size relative to the very limited field mapping survey data, a significant amount of variation and dispersion is still observed, as Fisher K values vary from approximately 24 to 170. Large Fisher K values indicate tightly-clustered sets, whereas small K values indicate dispersed sets.

Hoek and Bray (1977) emphasized the cautionary conclusions of Stauffer (1966) that not all pole concentrations must necessarily be meaningful, despite the natural tendency to interpret preferred structure rather than to dismiss measurements as random. They conclude that contoured pole diagrams are a necessary tool for slope stability.
assessment; however, pole concentrations alone are not sufficient to characterize rock mass structure. Additional data and “intelligent field observations” are needed to validate mapping results.

The discontinuity data from the combined LiDAR survey includes non-systematic discontinuities associated with faulting and damage from episodes D3 to D5, and also composite discontinuities comprising non-systematic brittle fractures induced by blasting or mining-induced changes in stress (i.e. episode D6).

Table 6 - 5: Summary of discontinuity set orientation data identified in LiDAR mapping, distance ≈ 500 m.

<table>
<thead>
<tr>
<th>Set</th>
<th>Dip (°)</th>
<th>Dip Direction (°)</th>
<th>Sample Size</th>
<th>Fisher K</th>
</tr>
</thead>
<tbody>
<tr>
<td>J1</td>
<td>45</td>
<td>050</td>
<td>163</td>
<td>34.7</td>
</tr>
<tr>
<td>J2</td>
<td>88</td>
<td>280</td>
<td>18</td>
<td>170.4</td>
</tr>
<tr>
<td>J3a</td>
<td>44</td>
<td>197</td>
<td>324</td>
<td>29.0</td>
</tr>
<tr>
<td>J3b</td>
<td>79</td>
<td>215</td>
<td>89</td>
<td>75.1</td>
</tr>
<tr>
<td>J3c</td>
<td>83</td>
<td>040</td>
<td>25</td>
<td>79.9</td>
</tr>
<tr>
<td>J4</td>
<td>53</td>
<td>137</td>
<td>177</td>
<td>23.5</td>
</tr>
</tbody>
</table>

¹NOTE: J3b and J3c are members of the same set, with opposite dip direction; they reflect natural variability of steeply-dipping joints about the same northwest-southeast strike.

Table 6 - 6 summarises the maximum, minimum, and average persistence for each major discontinuity set, measured as the diameter of fitted discontinuity planes. Set J1, the northeast-dipping sliding set, has the largest maximum and average measured persistence; this observation is consistent field observations that suggest the most persistent joint features are associated with planar sliding failures on northeast-dipping joints in Design Sector 3. The maximum persistence values should be used as minimum persistence values in stability assessments, because orientation and sampling bias
contribute to reducing persistence measurements in the LiDAR survey, such that they do not reflect the true maximum extent of the discontinuities.

Table 6 - 6: Summary of persistence, measured as fitted discontinuity plane diameter, for combined LiDAR mapping results.

<table>
<thead>
<tr>
<th>Set</th>
<th>J1</th>
<th>J2</th>
<th>J3a</th>
<th>J3b</th>
<th>J3c</th>
<th>J4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum</td>
<td>27.8</td>
<td>5.6</td>
<td>12.6</td>
<td>9.2</td>
<td>18.0</td>
<td>18.2</td>
</tr>
<tr>
<td>Minimum</td>
<td>0.6</td>
<td>0.8</td>
<td>0.2</td>
<td>0.4</td>
<td>0.8</td>
<td>0.4</td>
</tr>
<tr>
<td>Average</td>
<td>6.6</td>
<td>3.0</td>
<td>1.4</td>
<td>2.5</td>
<td>4.2</td>
<td>1.9</td>
</tr>
</tbody>
</table>

Figure 6 - 18 presents a histogram of the fitted discontinuity diameters for the major sets. The results show that the majority of features are smaller than the 3 m suggested limit for medium persistence, and thus fall into the low persistence category (ISRM, 1978). Sets J1, J3, and J4 all contain outlier structures with high persistence. Set J1 is the only set containing very high persistence structures.

Figure 6 - 18: Histogram of fitted discontinuity diameters for major sets from combined LiDAR mapping data, with ISRM (1978) persistence limits.
Figure 6 - 19 presents a pole plot of discontinuities from the low resolution LiDAR survey (spot spacing = 15; n = 308), which covered the entire Upper West Wall. The high resolution dataset (spot spacing = 7, n = 644) is excluded.

Figure 6 - 19: Low-resolution LiDAR survey, pole plot of fitted discontinuities, plotted according to ISRM (1978) persistence.

Figure 6 - 20 presents the lower-resolution LiDAR survey results plotted according to the geotechnical domain where discontinuity measurements were taken. The boundary between the North and South domains is formed by the Yellow Fault.

Figure 6 - 20: Low-resolution LiDAR survey, pole plot of fitted discontinuities, plotted according to geotechnical domain.
When the discontinuity poles are plotted according to geotechnical domain, several trends become apparent:

- J1 includes discontinuities from both the North Domain, located north of the Yellow Fault, and the South Domain, located south of the Yellow Fault; J1 discontinuities in the North Domain have a dip direction closer to East (~070°), whereas J1 discontinuities in the South Domain have a dip direction closer to Northeast (~050°)
- J2 includes discontinuities from both geotechnical domains, however the majority of poles are located in the North Domain
- J3a includes poles located inside both geotechnical domains, however J3b/c includes almost exclusively poles from the North Domain
- J4 is almost entirely comprised of discontinuities in the North Domain
- J5 is almost entirely comprised of discontinuities in the South Domain

In the lower resolution survey the southeast-dipping set J4 is less prominent than it appears in the combined dataset. This difference likely occurs due to truncation bias, which may result because the lower resolution survey cannot resolve discontinuities below a cut-off of length of approximately 0.4 m; whereas the higher resolution survey applies a cut-off length of approximately 0.2 m; the histogram of fitted discontinuity diameters in Figure 6 - 18 indicates that J4 is dominated by low persistence structures.

The most prominent discontinuity set in the low resolution survey is J1, the northeast-dipping planar sliding set. An additional set, J5, occurs that is not distinguishable in the combined LiDAR dataset. The absence of set J5 from the high resolution survey and from the overall dataset, is due in part to orientation bias. Set J5 dips steeply towards northwest, and is thus likely to be under-sampled in the high resolution survey, which is confined to the South Domain (Design Sector 2), where the overall slope dip/dip direction is approximately 40°/354°. The high quantity of non-J5 discontinuities from the high resolution survey may suppress the concentration of poles that occurs in the low resolution survey, which covers the entire Upper West Wall.

The results emphasize the importance of considering orientation bias and truncation. Set J5 is an important feature of local rock mass fabric that occurs parallel to a large
segment of the Yellow Fault. Overall, the LiDAR discontinuity mapping data support the assertion that minor structural features including joints often tend to occur parallel to major structures. The orientation, persistence, and statistical data for set J5 are summarised in Table 6 - 7.

Table 6 - 7: Summary of orientation, persistence and sample statistics for set J5.

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Dip (°)</td>
<td>72</td>
</tr>
<tr>
<td>Dip Direction (°)</td>
<td>325</td>
</tr>
<tr>
<td>Sample Size</td>
<td>19</td>
</tr>
<tr>
<td>Fisher' K</td>
<td>46.7</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Persistence (m)</td>
<td></td>
</tr>
<tr>
<td>Maximum</td>
<td>5.3</td>
</tr>
<tr>
<td>Minimum</td>
<td>0.5</td>
</tr>
<tr>
<td>Average</td>
<td>4.6</td>
</tr>
</tbody>
</table>

Figure 6 - 21 shows a pole plot of the discontinuities from the high resolution dataset.

Figure 6 - 21: High resolution LiDAR survey, pole plot of fitted discontinuities, plotted according to ISRM (1978) persistence.
The pole concentrations indicate that J1 is less prominent in the high resolution dataset than in either the combined or low resolution results. The strongest grouping of poles occurs for the south-dipping discontinuities in J3 and J4. Sets J2 and J5 are not present in the high resolution pole plot. J1 is the most probable set to be directly associated with planar sliding failures, particularly in Design Sectors 3 and 4. Specific evidence of bench scale wedge-type failure mechanisms is discussed in Section 5.4.1.

Figure 6 - 22 compares a section of bench face using two imaging methods:

(1) A simplified schematic trace map showing major joint sets, superimposed on a 2-D digital photograph

(2) The corresponding area mapped in the high resolution point cloud from the LiDAR survey, with best-fit circular discs indicating the major joint sets.

The LiDAR image demonstrates the potential for low persistence discontinuities to be mapped in the high resolution point cloud. The point cloud is artificially lit according to the intensity of the reflected beam, which varies according to several parameters including rock colour, presence of water, orientation of the surface, mineralogy and weathering. Discontinuity traces are easily recognizable as dark streaks on the brightly bench face. The reflectivity-based lighting provides useful information on contrasts in structure or rock mass condition, which may be used to identify areas of seepage, zones of altered or weathered rock, or continuous seams of soil-like infill along altered discontinuities.
Figure 6 - 22: Comparison of 2-D photograph with high resolution LiDAR point cloud; the high resolution and relief shading in the point cloud allows for identification of low-persistence discontinuities that are difficult to distinguish in the photograph.

Figure 6 - 23 presents the contoured pole plots for the LiDAR mapping data, with a plan view of the Upper West Wall, showing three major sections of the slope with different overall orientations, and daylight envelopes constructed from the overall slope angles.
Figure 6 - 23: Comparison of overall slope orientations and daylight envelopes for overall slopes, considering three major sections of the Upper West Wall, with discontinuity pole plots derived from LiDAR mapping.
6.4.3.2. **Interpreting Trends in Persistence from LiDAR**

The combined data from both LiDAR surveys appear to conform well to the negative exponential distribution model for discontinuity persistence. Extremely persistent features exceeding 50 m are present but rare; the rock mass is characterised by intense fracturing with low-persistence discontinuities. The two most persistent features are 100 m + segments of the Yellow Fault (Figure 6 - 24). No purely extensional fractures (i.e. typical joints, or dykes) are observed exceeding approximately 20 m in persistence.

![Extensive 30 m bridge of fractured rock mass separating segments of the Yellow Fault.](image)

**Figure 6 - 24: Extensive 30 m bridge of fractured rock mass separating segments of the Yellow Fault.**

The geological timeline of fracture development (D1 to D6) can be used to derive a conceptual hierarchy of discontinuity persistence. Persistent tension fractures like exfoliation joints (driven by external tensile stresses) and dykes (driven by internal fluid overpressure) may originate early in the history of an igneous pluton such as the Guichon Creek Batholith, during cooling and uplift (e.g. Hencher et al., 2011).

Later tectonic activity may be associated with the tilting and deformation of the rock mass, causing pre-existing fractures to propagate and potentially coalesce into regionally persistent shear structures, such as faults including the Lornex and Valley faults, with kilometre-scale persistence (Gudmundsson, 2011).
Subsequent hydrothermal intrusions caused hydraulic fracturing by internal fluid overpressure, creating a stockwork fracture network. Fault re-activation episodes may be associated with localized zones of intensified secondary shear damage and also the formation of minor new secondary faults such as the Yellow Fault.

The most recently-formed discontinuities are caused by mining (D6), and they are also the least persistent structures. The damage timeline indicates that fracture intensity will increase with time, and that more recent processes are associated with the formation of less-persistent discontinuities. Although high-persistence (i.e. > 20 m) tension joints may have originated during early stages of the pluton (i.e. during cooling, uplift, and exhumation), potential evidence of these features has been mostly overprinted by subsequent tectonic damage, hydrothermal alteration and mining-induced damage.

Figure 6-25 presents a histogram of fitted discontinuity diameters for the low resolution LiDAR survey. A peak occurs for discontinuities between 1 m and 2 m persistence, and discontinuities smaller than 2 m are under-sampled due to (1) limited resolution and truncation; and (2) length bias, such that discontinuities larger than 2 m are more easily distinguished, and thus more likely to be preferentially noticed during mapping.

Figure 6 - 25: Low resolution LiDAR survey (spot spacing = 15; resolution = 5 cm to 20 cm), histogram of fitted discontinuity diameters (1 m bin interval).
Figure 6 - 26 presents a histogram of fitted discontinuity diameters for the high resolution LiDAR survey. The exponential increase in frequency for smaller discontinuities is marked and abrupt. The peak frequency occurs for discontinuities smaller than 1 m, and under-sampling of smaller discontinuities does not seem to occur.

![Histogram of fitted discontinuity diameters](image)

**Figure 6 - 26: High resolution LiDAR survey (spot spacing = 7; resolution = 1 cm to 10 cm), histogram of fitted discontinuity diameters (1 m bin interval).**

Re-plotting the high resolution dataset with 0.2 m bin intervals shows the true peak in frequency occurs for a value of 0.8 m, with smaller discontinuities being under-sampled (Figure 6 - 27). Both the high resolution and low resolution datasets support field observations that indicate the rock mass is dominated by very low to low persistence (< 1 m to 3 m; ISRM, 1978) discontinuities probably originating from geologically recent fault re-activation episodes and modern mining-induced rock mass damage.
Figure 6 - 27: High resolution LiDAR survey, histogram of fitted discontinuity diameters (0.1 m bin interval).

Figure 6 - 28 presents a histogram of discontinuity diameters for the combined LiDAR results, including all discontinuity measurements from both surveys. A suggested negative exponential best-fit function is overlain, and the 10 m threshold for “high persistence” (ISRM, 1978) is also shown.

A total of 901 discontinuity measurements, comprising 95% of the sample population, have fitted plane diameter (proxy for persistence) of 10 m or less, indicating that the majority of measurements range from very low to medium persistence. 51 measurements, making up 5% of the sample population, exceed the 10 m threshold for “high” persistence.
Figure 6-28: Histogram of discontinuity diameters for combined results of LiDAR surveys (1 m bin interval), with 10 m boundary indicating lower bound for ISRM (1978) limit for high persistence.

The suggested negative exponential function produces a good approximation of frequency of smaller discontinuities, up to approximately 10 m in persistence, corresponding to the upper bound limit for medium persistence suggested by the ISRM (1978) guidelines. Discontinuities with persistence greater than 10 m, however, will be under-predicted by the negative exponential function. Thus although the negative exponential function is able to capture the natural abundance of smaller discontinuities, the rare occurrence of extremely high persistence outlier features is difficult to predict.

To analyse the sample population of non-systematic discontinuities, the major joint sets and very high persistence composite features (i.e. complex bench-scale wedge failure surfaces, further discussed in Section 5.4.1) were removed. The remaining data are a set of 116 poles, representing approximately 12% of the total sample population. A pole plot of the random discontinuities is presented in Figure 6-29. No clear secondary sets are apparent, pole density concentrations do not exceed approximately 5%.
Figure 6 - 29: Density-contoured pole plot of interpreted non-systematic discontinuities, sorted according to ISRM (1978) persistence category; poles to major joint sets have been removed.

Figure 6 - 30 presents a histogram of fitted diameters for the interpreted random discontinuities. Although the majority of measurements suggest very low to low persistence, outlier features exist with very high persistence, suggesting that not all very high persistence structures can be easily predicted as part of systematic sets.

Figure 6 - 30: Histogram of fitted discontinuity diameters for non-systematic (random) discontinuities not belonging to sets J1 to J5.
6.5. Discussion

This section discusses the implications of the combined field investigation results for the role of discontinuity persistence and intact rock bridges in the performance of the Upper West Wall. Mechanisms of bench-scale instability are discussed with direct reference to failure surfaces observed throughout the wall. The influence of blasting-induced damage is also considered.

6.5.1. Implications of Discontinuity Persistence, Block Geometry and Spacing

The geological history of the Bethsaida granodiorite suggests that highly or extremely persistent (20 m+; 50 m+) tension fractures may have developed during the stages of cooling, uplift and exhumation. However, evidence for such features is not well preserved in surface exposures of the modern rock mass. The largest structural feature in the slope is the Yellow Fault; the two fault segments mapped in the LiDAR survey have persistence of approximately 93 m and 101 m, with the bottom of the larger segment being censored beneath the floor of the RL + 1385 m bench, which was the base of the Upper West Wall at the time of field investigation. The overall orientation of the Yellow Fault appears to be approximately sub-vertical, trending Northeast-Southwest (average dip is 88° with opposing dip directions varying between 132° and 312°).

If joints with high persistence exceeding 20 m daylight within the Upper West Wall, they may have been either:

1. Covered with debris from progressive bench-scale failures; or
2. Overprinted by more recent fracturing induced by:
   a. Geologically recent (i.e. Tertiary age) fault re-activation and shearing
   b. Geologically recent (i.e. Tertiary age) hydraulic fracturing induced by hydrothermal fluid intrusion
   c. Modern blasting-induced damage associated with mining

The condition of the bench faces reflects the history of tectonic fracturing and alteration, with frequent occurrence of sheared, poor rock mass conditions and silicic- and phyllic-altered discontinuities (typically increasing with proximity to the Yellow Fault). The most
significant structural controls for bench-scale failures are low- to medium-persistence, northeast-dipping joints. The limited field mapping data identified an average persistence of approximately 5 m for pre-existing discontinuities, excluding recent, irregularly-shaped blast-induced fractures. The range of block shapes and sizes tends towards mostly cubic to platey-cubic blocks with edge lengths not usually exceeding 3 m (Figure 6 - 31).

![Figure 6 - 31: Suggested range of typical block shapes based on field observations of the Upper West Wall, superimposed on the ternary diagram proposed by Kalenchuk et al. (2006).](image)

The typical block sizes indicate that irregularly oriented, low-persistence discontinuities resulting from blast-induced damage, and stockwork fractures from previous tectonic activity, are typically closely to moderately spaced (i.e. 60 mm to 600 mm; ISRM, 1978). Major planar discontinuities with medium persistence or greater, including joint sets J1 to J5, tend to be more widely spaced, ranging from moderate spacing (200 mm to 600 mm; ISRM, 1978) to extremely wide spacing (> 6 m; ISRM, 1978).

Discontinuities identified in the low-resolution LiDAR survey showed a peak in frequency for fitted diameter values between 1 m and 2 m. The higher-resolution survey resulted in a peak value of approximately 0.8 m. Overall, the LiDAR suggest that rock mass structure is dominated by low- to medium-persistence discontinuities, originating from episodes of tectonic deformation, hydrothermal fluid intrusion, and blasting-induced...
damage. Histograms of fitted discontinuity diameters from the LiDAR survey suggest that the distribution of persistence may conform well to a negative exponential distribution, up to a maximum persistence of approximately 10 m. Structures with higher persistence, comprising approximately the top 5% to 10% of the population of discontinuities, should be identified manually and discretely included in stability analyses or directly inserted into DFN models for discrete element or hybrid FEM-DEM slope stability assessment, rather than treated with statistical methods.

6.5.2. **Role of Intact Rock Bridges**

The dominance of low-to medium-persistence discontinuities, suggests that large (i.e. > 1 m) discrete bridges of intact rock are unlikely. The rock mass fabric is characterised by a pervasive stockwork and discontinuity sets that occur parallel to the regional Lornex and Valley fault orientations.

LeBaron (2011) suggested that the relationship between blast fragmentation and intact rock bridge content could only be clearly related when rock bridges are evaluated as a ratio of pre-existing joint lengths. The results of the current investigation reach a similar conclusion: rock bridges can only be meaningfully interpreted with reference to key parameters describing pre-existing discontinuities, most notably persistence, spacing, intensity and orientation. As a corollary of the typical low persistence values seen throughout the Upper West Wall, and the close to moderate discontinuity spacing, intact rock bridge content may be also be characterised by small length dimensions, not exceeding typical block edge lengths of 0 to 1 m.

Conceptually, an intensely fractured rock mass dominated by low-persistence, closely-spaced discontinuities may contain a dispersed volume of very small intact rock bridges, in order of centimetres in length. Field identification and precise mapping of such small, individual intact rock bridges would be impractical and time-consuming. For very blocky or disintegrated rock masses, a cumulative treatment of overall intact rock bridge content may be more feasible, as long as the spatial distribution of rock bridges is approximately random. In such a highly fractured rock mass, a through-going failure surface comprised of short discontinuity segments with low persistence but very close spacing may contain
a high overall rock bridge content, however the individual rock bridges will be small and dispersed throughout the rock mass.

Figure 6 - 32 shows a conceptual diagram for the dependency of rock bridge content along a 2-D failure surface comprised of a steep joint set and a shallow joint set, with four idealized domains highlighted. The figure demonstrates that potential rock bridge content cannot be considered in isolation; discontinuity spacing, trace length, and ideally fracture intensity must also be assessed.

The axes are labelled with a logarithmic scale for trace length and a linear scale for spacing, with suggested values provided in order to provide context only. The domain boundaries should not be considered as explicit quantitative assessments of limiting values for trace length and spacing, and are providing a conceptual guide only for further investigation. Based on the results of the field mapping and remote sensing, the Upper West Wall may be well-suited to rock bridge representation methods in Domain 3 (Figure 6 - 33).
Figure 6 - 33A shows a schematic trace map for a typical rough, blast-damaged bench face in Design Sector 3 at elevation RL + 1385 m. Major discontinuities are traced but no discrete intact rock bridge traces are shown. The underlying photograph clearly shows that more discontinuities are excluded than are traced; the rock mass is dominated by closely-spaced very low persistence brittle fractures. In agreement with the negative exponential best-fit distribution of persistence, smaller discontinuities occur with greatly increased frequency relative to more persistent structures.

Rather than direct measurement of intact rock bridges, a conceptual model for the rock mass could be created by specifying a characteristic block size, and a factor describing the “completeness” of a typical block, as percentage of perimeter length. Figure 6 - 33B shows a conceptual model for rock bridges in the fractured granodiorite, using an overlay of randomly-generated, discontinuous Voronoi polygons where the perimeter of each polygon ranges between approximately 60% and 90% complete.

No rock bridges greater than approximately 40 cm interrupt the polygon perimeters in the Voronoi assembly. However, a conceptual “failure path” transecting the window, traced along the perimeter of adjoining polygons, may contain a cumulative length of intact rock bridges of 1 m or more, depending on the geometry of the critical path.

Direct identification of discrete intact rock bridges in LiDAR point cloud mapping is difficult, however extensive zones of rock mass bridges are easily recognized between very persistent to extremely persistent (i.e. >> 20 m) structures, such as the segments of the Yellow Fault (see Figure 6 - 24). A highly fractured rock mass such as the Upper West Wall may be well suited to a volumetric-based approach to rock bridge characterisation, where the cumulative influence of a large quantity of very small intact rock bridges, distributed throughout the rock mass, is considered. Future work based on statistical shortest-path analysis of multiple Voronoi realizations or DFNs may help to better bracket probable upper-bound and lower-bound values for cumulative intact rock bridge content for bench, inter-ramp, or overall slope-scale instability mechanisms.
Figure 6 - 33: Blast-induced damage overprints pre-existing structures in an intensely fractured rock mass with field-estimated GSI ≈ 30-40; (A) Discontinuity/block edge length traces; and (B) Random, discontinuous Voronoi polygons as a conceptual model for simulating dispersed intact rock bridges.
6.5.3. Influence of Volumetric Rock Bridges

In modified limit equilibrium slope assessments, the effects of intact rock bridges are usually only considered with respect to added strength along a candidate failure surface. In reality, however, 3-D rock bridges exist throughout a rock mass. As an extension of the out-of-plane rock bridge concept, a volumetric rock bridge measure, $V_{RB}$, may help to better characterise the internal shear strength and deformability of a potential slide volume. Conceptually, a volumetric rock bridge measure could incorporate two factors: (1) in situ block size; and (2) “completeness” of blocks, or a persistence factor related to the size of discontinuities compared with the sample volume (e.g. Elmo et al., 2008; Kim et al., 2006).

**Figure 2 - 24:** Conceptual illustration of volumetric rock bridge content, related to block size and “completeness” of block perimeters.
Figure 2 - 24 conceptually illustrates the difference in $V_{RB}$ for a series of cases where block volume ranges from the order of 1 cm$^3$ to 1 m$^3$, and the “completeness” of blocks ranges from 100% (fully-formed blocks, no intact rock bridges) to less than 10% (i.e. only isolated fractures occur; no fully-formed blocks, 90% intact rock bridges). Methods for simulating volumetric rock bridge content in numerical models are investigated in Chapter 7.

6.5.4. Influence of Blasting-Induced Damage

The occurrence of progressive bench failure in Design Sector 3 may be associated with interaction between blasting damage and pre-existing, northeast-dipping joints from J1. Slope steepness maps produced in ArcGIS reveal anastomosing “zig-zag” geometry of steep section of bench face, and help to show the variability in overbreak and bench width, which was shown to vary typically from approximately 11 m to 15 m. Bench roughness maps show that more highly damaged sections of bench face show evidence of interaction between blast induced damage and pre-existing joint sets J1 to J5.

Tectonic damage, hydraulic fracturing and associated alteration have fractured the rock mass to the extent that it is difficult to distinguish pre-existing fractures from mining-induced damage. Tectonic damage appears to increase with proximity to the Yellow Fault; this relationship is supported by field observations that the rock mass grades from very blocky, to disintegrated, soil-like material in the core of the fault (Figure 6 - 10, Section 6.4).
7. Numerical Modelling of Large Open Pit Slopes with Non-Persistent Discontinuities and Rock Bridges

7.1. Introduction

This chapter applies observations from the field investigations to a series of conceptual numerical models of large open pit slopes with non-persistent discontinuities. The models are constructed to simulate large slopes comprised of strong rock, where failure mechanisms are predominantly discontinuity-controlled. In order to model progressive failure, discontinuities are non-persistent, so that destruction of intact rock bridges is required for kinematic release and initiation of catastrophic failure (e.g. Eberhardt et al., 2004).

First, the continuum approach is investigated with the 2-D finite element code Phase² (Rocscience, 2011a). Discrete, non-persistent discontinuities are inserted into the finite element mesh, forming a discontinuous, bi-planar overall slope failure surface with bridges of rock mass occurring throughout the failure path. Rock mass strength properties are defined using a bi-linear failure envelope in order to investigate the influence of confining stress on post-peak rock mass behaviour (Diederichs, 2007).

Next, the distinct element code UDEC (Itasca, 2010a), and the lattice-spring code Slope Model (Itasca, 2010b) are used to investigate the influence of dispersed rock bridges on internal shearing of an unstable slide volume. Both the UDEC and the Slope Model simulations are constructed with a pre-existing, bi-planar failure surface, so that the overall failure mechanism is predetermined. Instead of characterising the influence of intact rock bridges along the sliding surface, the models investigate the influence of volumetric (or areal, in 2-D models) rock bridges on internal shearing and dilation of the slide mass.
7.2. Review of Two Approaches to Intact Rock Bridge Representation in Slope Stability Modelling

7.2.1. Rock Bridges on a Failure Surface

The first approach to incorporating rock bridges into slope stability analysis originated with Jennings’ (1970) method where intact rock bridges occur as idealized patches or length intervals of intact rock along a candidate failure surface. This method considers the influence of rock bridges solely on the shear strength of the failure surface; and is therefore referred to by the author as the failure surface-specific (FSS) method.

By only considering the influence of intact rock bridges on the shear strength of the failure plane, equations for factor of safety can be derived by adapting a force balance solution from the simple case of a block sliding on an inclined plane. Figure 7 - 1 illustrates three cases of sliding with adapted force balance solutions, expressed in terms of:

- Weight of the sliding block, W
- Inclination of the failure plane, \( \alpha \)
- Joint continuity coefficient, \( k \)
- Tensile strength of intact rock, \( T_o \)
- Inclination of non-coplanar discontinuities, \( \beta \).

In Figure 7 - 1B, the continuity coefficient \( k \) describes the ratio of jointed length to intact rock length (or area) along a failure surface. Apparent shear strength parameters are calculated by summation of the cohesion and friction components contributed by intact rock and jointed portions of the failure surface.

In Figure 7 - 1C a step-path type failure occurs through parallel, non-persistent, non-coplanar discontinuities. The solution considers shear strength along an equivalent failure surface (dashed line), with a cohesive strength component depending on the inclination of the discontinuity segments, and an additional (directionally resolved) strength component contributed by tensile strength of intact rock (\( T_o \)) which must be exceeded in order to fracture the intact rock bridges.
Figure 7-1: Three conceptual limit equilibrium solutions for sliding slope failure in dry conditions (Reproduced from Eberhardt et al., 2004; cited from Jennings, 1970, and Jaeger, 1971).

7.2.2. Rock Bridges in a Volume of Rock Mass

The FSS method described in the previous section may be well-suited to non-dilating slope failures including: rigid-body failures involving translation or rotation of discrete blocks of massive rock mass; or large circular slump failures through a weak, highly weathered rock mass. However, some types of large slope failures may involve complex failure surface geometry, where internal shearing and dilation of the slide mass along internal discontinuities is required to produce kinematic freedom (Corkum and Martin, 2004; Martin and Kaiser, 1984).

Signs of incipient or progressive slope failure, such as toe bulging or crest tension cracks, suggest that internal deformation and dilation of the unstable slide mass occur during the time leading up to failure (e.g. Narendranathan et al., 2012; Eberhardt et al., 2004). Thus, for complex large open pit slope failures, the configuration of discontinuities
and intact rock bridges within the slide mass may have an important influence on stability.

The Prandtl Prism mechanism, which is commonly applied to analysis of foundation bearing capacity failure in soils (Prandtl, 1923) was first adapted for slope stability by Mencl (1966), in order to explain the curved failure surface of the 1963 Vaiont slide. He proposed that where failure surface geometry is curved or irregular, the stiffness of the slide mass influences the development of kinematic freedom and the transfer of driving stress from an upper active block, through a Prandtl transition wedge and into the a lower passive block (Figure 7 - 2).

![Figure 7 - 2: Prandtl wedge applied to bi-linear slope failure; the Prandtl Prism functions as a transition zone, where the direction of driving stresses from the active upper block are transferred to the lower passive block (Modified after Kvapil and Clews, 1979).]
The driving force of the active wedge and the resisting force of the passive wedge are related by the radii \( r_a \) and \( r_b \) which bound the Prandtl wedge (Kvapil and Clews, 1979):

\[
\frac{P_A}{P_B} = \frac{r_B}{r_A}
\]

The radii \( r_a \) and \( r_b \) are related by the prism apex angle, \( \psi \):

\[
r_B = r_A \cdot e^{\psi \tan \phi}
\]

Kvapil and Klews (1979) further discussed the expected sequence leading to development of the Prandtl Wedge and eventual failure, including three stages:

1. Initial displacements in the upper, active wedge, accompanied by the development of tension cracks near the slope crest
2. Larger displacements in the upper, active wedge, and the development of the Prandtl transition zone though the formation of radial and log-spiral secondary shear surfaces; development of the transition zone is associated with bulging of the slope near the prism apex.
3. Initiation of accelerated deformations and intensive shearing of the slide volume, potentially leading to catastrophic failure.

Sarma (1979) proposed a limit equilibrium approach accounting for internal shearing by incorporating inclined slices; Martin and Kaiser (1984) reported that the Sarma method provided good results in back analysis of a rock slope with ongoing deformations, and confirmed the importance of internal shear surfaces in dilating slope failures. Stead (1984) further investigated limit equilibrium assessment of Prandtl Prism geometry by relating the prism apex angle \( \psi \) to the ratio of Prism radii (i.e. \( r_A / r_B \) in Figure 7 - 2) and friction angle of the internal shear surfaces. He found that \( r_A / r_B \) varies from a value of 1 for low friction angles (10°), up to 2.2 for a friction angle of 45°.

Eberhardt et al. (2004) carried out numerical modelling of the 1991 Randa rock slide with continuum, discontinuum, and hybrid FEM-DEM codes and found that a Prandtl zone of yielded rock mass developed at the base of the sliding surface, and propagated upward towards the upper active wedge through progressive tensile damage. Fisher and Eberhardt (2007) used distinct element modelling to investigate bi-planar failure and toe
breakout of cataclinal dip slopes with non-daylighting bedding; they found that the Prandtl wedge mechanism could accurately describe the development of kinematic freedom and toe breakout, as internal shear surfaces form as step-paths along thinly-bedded foliation, closely-spaced cross joints, and rupture through rock mass.

As the Prandtl wedge develops, the formation of radial and log-spiral shear surfaces within the transition zone is needed to facilitate volume change and allow down-slope transport of the active and passive failure blocks. If dilation of the Prandtl wedge cannot occur, then failure is arrested. Conceptually, a fractured rock mass containing persistent joints and fully-formed blocks could be expected to accommodate shearing more easily than a massive rock mass with incomplete blocks (Figure 7-3).

Sections 7.6 and 7.7 investigate the Prandtl mechanism using UDEC and Slope Model, with particular emphasis on the role of intact rock bridges and the development of brittle damage inside the slide mass.

![Figure 7-3: Non-systematic fracturing and rock bridges within the transition zone of a potential slide mass.](image_url)
7.3. Influence of Model Resolution

Model resolution controls the scale of geological structures and failure processes that can be simulated, and represents a fundamental choice in model setup. Selection of resolution frequently represents a trade-off between increased precision given by finer mesh sizes, and concomitant increases in computational expense. For simulation of brittle failure in intact rock, mesh size needs to be sufficiently fine to capture localized stress concentrations that are fundamental to initiation, propagation and coalescence of brittle cracks (Figure 7-4).

Figure 7-4: Three simulated UCS tests in UDEC with identical parameters except for mesh size \( m \); largest mesh size is unable to simulate localized wing crack initiation and propagation.

At the scale of a large open pit slope, models resolution is limited by computational requirements, and thus models cannot explicitly simulate the initiation, propagation and coalescence of millimetre-scale brittle cracks in intact rock. Instead, pit-scale models may consider brittle failure of equivalent rock mass bridges, with scaled shear strength parameters. Stead et al. (2007a) illustrated the relationships between slope model scale, mesh (or particle) size, scale of fracture simulation, and number of model elements for 2-
D analysis. They suggest that a transition towards complex brittle fracture modelling in large, 3-D open pit slope models would benefit from increased utilisation by modellers and modelling codes of parallel processing and 64bit high RAM. Simultaneously, they caution that “bigger models does not mean better models” and that characterisation methods for gathering input data, and theoretical research underpinning our understanding of the mechanisms of brittle fracture in rock slopes, must keep pace with advances in computing.

7.3.1. **Finite Element Mesh in Phase**

The *Phase* models use a graded triangular mesh, with coarse zone edge lengths of up to approximately 200 m at the extreme boundaries of the model, transitioning to finer edge lengths down to approximately 1 m in the region of interest surrounding the slope, the pre-existing discontinuity surfaces and the rock mass bridges (Figure 7 - 5).

![Figure 7 - 5: Graded triangular mesh implemented in Phase, with increased resolution around rock mass bridges and failure surface discontinuity segments.](image-url)
7.3.2. **Mesh and Voronoi Polygon Size in UDEC**

The *UDEC* models use Voronoi tessellation to create an assembly of random polygons, where the polygon contacts can be assigned strong rock mass properties or weak discontinuity properties in order to simulate complex failure paths involving brittle rock mass failure and discontinuity-controlled shearing or opening. Voronoi tessellation has been shown to be capable of reproducing brittle failure behaviour in intact rock samples (Yan, 2008; Kazerani and Zhao, 2010) and has also shown potential capacity for simulating large-scale failure of rock slopes (Alzo’ubi, 2009; Alzo’ubi et al., 2007, 2010; Coggan et al., 2007; Schön, 2011; Strouth and Eberhardt, 2009).

An important feature of Voronoi tessellation in replicating brittle failure mechanisms is the ability to simulate tensile cracking. Figure 7 - 6 illustrates how interlocking Voronoi polygons can allow the development of tensile stresses within an overall compressive stress regime, by transmitting vertical compressive forces from overlying block contacts.

![Interlocking Voronoi polygons](image)

**Figure 7 - 6**: Interlocking Voronoi polygons allow for the development of local contact tensile stresses in an overall compressive stress regime (Reproduced from Poytondy and Cundall, 2004; and Alzo’ubi, 2009).
For this investigation, *UDEC* models assume an idealized linear elastic constitutive law for internal finite element zones (i.e. the inside of Voronoi blocks), such that rock mass failure cannot occur through intra-block shear or tensile failure. Thus rock mass failure is forced to occur by breakage of block contacts. Bulk rock mass behaviour is therefore controlled more by Voronoi polygon size than by intra-block mesh size.

The slope modelled in *UDEC* is 500 m high, and to limit memory requirements and run-times, Voronoi polygon sizes are assigned a maximum of 10 m edge lengths. For simplicity, the edge length for intra-block finite element zones is also limited to 10 m.

### 7.3.3. Lattice Resolution in Slope Model

Slope Model uses a 3-D lattice-spring configuration where point masses, or nodes, are connected by springs that are assigned nonlinear force-displacement laws (Cundall and Damjanac, 2009). Node spacing is the single parameter used to control model resolution, specified as $R$. Large $R$ values result in coarser resolution and faster run-times (Itasca, 2010b), however no structures or processes smaller than the length scale of $R$ can be simulated. The recommended minimum $R$ value should be selected such that a minimum of five nodes occur in the smallest region of interest. For example, if the smallest region of interest is a 1 m-long rock bridge, then resolution should be set to at least 20 cm in order to better capture the localized stress-strain behaviour within the region of interest (Itasca 2010b).

Due to computational requirements for creating a very large slope model (i.e. $H \approx 1$ km), the lattice resolutions for the current investigation are limited to 5 m for most models, with three selected cases run at a higher resolution ($R = 2.5$ m) to investigate the influence of varying $R$.

### 7.4. Influence of Boundary Conditions

Boundary conditions control the interaction between far-field stresses and near-field, mining-induced stress-strain responses in the rock mass including release of confinement and post-excavation slope instabilities. In this study all models consider hydrostatic stress conditions, based on the findings of Hoek et al. (2009b), who
concluded that preliminary numerical analyses suggest the traditional approach of applying hydrostatic stress conditions is appropriate for open pit slopes, and that the influence of high horizontal tectonic stresses is not significant in most cases.

Model extents were based on Read and Stacey’s (2009) recommended dimensions for far-field extents of open pit slope models, in order to minimize boundary effects (Figure 7 - 7). The suggested extents are usually sufficient to minimise boundary conditions for relatively shallow slopes, such as open pits, where overall slope angle is typically in the range of 30° to 60°. However, for very steep slope angles (i.e. > 60° to vertical) as frequently occur in natural high mountain rock slopes, model extents may require extension to much greater dimensions in order to minimise boundary effects and accurately simulate the influence of regional-scale topography on local stress concentrations in the modelled slope (e.g. Gerber and Scheidegger, 1969).

![Figure 7 - 7: Recommended far field model boundaries for open pit slope modelling (After Read and Stacey, 2009)](image)

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7.5. Continuum Approach with \textit{Phase}²

\textit{Phase}² (Rocscience, 2011a) is a finite element code that uses an implicit solution scheme, requiring that the solution process for all models converge to equilibrium. Although separation of discrete blocks and true crack propagation cannot be simulated, discontinuities can be included as interface elements, which can reveal useful information on shear stress and strain distributions in a rock mass, up to the point of failure. This investigation uses the shear strength reduction (SSR) method to investigate the characteristics of the slope at the point of failure initiation. The SSR method has previously been investigated for rock slope stability assessment by others such as Fournier (2008). Through an iterative process, the shear strength parameters of the rock mass and discontinuities are decreased until the solution process no longer converges, indicating failure.

7.5.1. Discontinuities in \textit{Phase}²

Discontinuities can be incorporated as interface elements embedded in the finite element continuum, however the amount of slip that may occur on discontinuities is limited compared with distinct element codes. The tips of discontinuity elements may be specified as either open or closed. Closed discontinuity tips are represented by a single node, and are unable to accommodate any displacement, whereas open tips are represented by two element nodes, which allows for some displacement to occur. The program manual recommends the end of a joint should be set as open if it terminates at another joint, or at a free surface (i.e. the slope face). For this investigation, all discontinuities were specified as open-ended, to allow for displacements to occur within rock mass bridges during progressive slope failure.

Regardless of discontinuity tips being open or closed, the model retains continuum structure: large discontinuity-controlled strains with shearing and separation of discrete blocks cannot be simulated explicitly (Hammah et al., 2008).

7.5.2. Model Setup

The model simulates a 1000 m-high slope with an overall angle of 45° (Figure 7 - 8). The benches range from approximately 75 m to 100 m high, representing multi-bench stacks
(i.e. 3 x 30 m bench stacks) that are commonly implemented in design of large open pit slopes (Read and Stacey, 2009).

Figure 7 - 8: Model setup for conceptual study of confinement dependent rock bridge failure.

Two discontinuity sets are included:

- **Discontinuity Set 1** forms a stepped upper sliding surface for potential bi-planar overall slope failure; there are six segments (170 m in length) in Set 1, separated by bridges of rock mass from 25 m to 60 m in length.

- **Discontinuity Set 2** comprises one extremely persistent structure which forms a basal sliding surface, with 10 m-long bridges of rock mass occurring at both tips of the discontinuity.

The lithology is uniform throughout the model, with Mohr-Coulomb shear strength properties (friction angle $\phi$ and cohesion $c$) based on a strong crystalline rock mass (Lac
du Bonnet granite) described by Diederichs (2007), Martin (1994); Eberhardt (1998); and Kaiser et al. (2000).

The sides of the model are fixed such that no horizontal displacement may occur and the bottom of the model is fixed so that no vertical displacement may occur. Based on the slope geometry and the pre-inserted discontinuities, displacements are expected to be accommodated by the global bi-planar failure surface.

At low confining stresses (i.e. $\sigma_3 < 20$ MPa), as occur in near-surface excavations or around the boundaries of underground excavations, the rock mass is characterised by high cohesive strength and low mobilised frictional strength. That is, pre-peak rock mass strength is dominated by influence of high intact rock strength, because the rock mass is massive and sparsely jointed ($\text{GSI} \approx 85$; strength envelope delineated by the “Damage” parameters in Figure 7 - 9, Diederichs, 2007). In order for shear strength to be mobilised, intact rock must first be fractured.

![Figure 7 - 9: Principal stress plots based on shear strength parameters for Lac du Bonnet granite given by Diederichs (2007), by permission.](image-url)
At high confining stresses (~ $\sigma_3 \geq 60$ MPa), as occur in deeply buried rock mass, the rock mass strength transitions to friction-dominated shear strength parameters, approximating a Hoek-Brown strength envelope for a rock mass with GSI $\approx 70$ (delineated by the “Spall limit” in Figure 7 - 9, Diederichs, 2007).

A blast-damaged zone extends for a distance $T$ ranging from 50 m to 75 m behind the slope face, corresponding to the suggested guidelines recommended by Hoek (2012) and Hoek and Karzulovic (2000), of $T \approx 2$ to $2.5H$ (where $H =$ typical bench height of 30 m) for large production blasts with little or no use of controlled blasting techniques, and accounting for the influence of confinement in causing increased blast damage. The thickness of the blast damage zone thus reflects a “worst case” scenario of very poor blasting practices (Chapter 2, Section 2.2.3).

Table 7 - 1 summarises the rock mass and discontinuity parameters applied in the $Phase^2$ slope model, and Section 5.3 describes in greater detail the rationale for derivation of peak and residual rock mass strength and stiffness parameters.

Care should be taken in noting that although rock mass residual cohesion values are listed as zero, a small, positive non-zero residual cohesion was applied (in the order of $10^{-4}$ MPa) after the recommendations of Diederichs (2007), in order to avoid unrealistic localization, which may occur in finely-meshed models when failure induces the transition from high cohesion peak strength parameters to cohesionless residual shear strength parameters.
Table 7-1: Summary of rock mass and discontinuity strength and deformability parameters for use in Phase$^2$ slope model.

<table>
<thead>
<tr>
<th></th>
<th>Rock Mass</th>
<th>Discontinuities</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Undisturbed</td>
<td>Blasted</td>
<td>Set 1</td>
<td>Set 2</td>
</tr>
<tr>
<td><strong>Cohesion, c (MPa)</strong></td>
<td>35</td>
<td>26</td>
<td>0.5</td>
<td>0.25</td>
</tr>
<tr>
<td>Peak</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Residual</td>
<td>0$^1$</td>
<td>0$^1$</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td><strong>Friction Angle, (\Phi) (°)</strong></td>
<td>22</td>
<td>18</td>
<td>36</td>
<td>30</td>
</tr>
<tr>
<td>Peak</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Residual</td>
<td>50</td>
<td>42</td>
<td>33</td>
<td>27</td>
</tr>
<tr>
<td><strong>Tensile Strength, (\sigma_T) (MPa)</strong></td>
<td>7.5</td>
<td>4.5</td>
<td>0.1</td>
<td>0.05</td>
</tr>
<tr>
<td>Peak</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Residual</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td><strong>Rock Mass Deformation Modulus, (E_{RM}) (GPa)</strong></td>
<td>30</td>
<td>9.9</td>
<td>26</td>
<td></td>
</tr>
<tr>
<td><strong>Poisson’s Ratio, (\nu)</strong></td>
<td>0.26</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Normal Stiffness, (K_n) (MPa/m)</td>
<td>-</td>
<td>10 000</td>
<td>5000</td>
<td></td>
</tr>
<tr>
<td>Shear Stiffness, (K_s) (MPa/m)</td>
<td>-</td>
<td>1000</td>
<td>500</td>
<td></td>
</tr>
</tbody>
</table>

$^1$NOTE: In practice a small, positive non-zero residual cohesion value was applied to ensure numerical model agreement with predicted behaviour and prevent unrealistic localization effects, after the recommendations of Diederichs (2007)
7.5.3. *Derivation of Material Parameters to Simulate Confinement-Dependent Strength*

The peak and residual shear strength parameters for the rock mass define a bi-linear failure criterion, after the method proposed by Diederichs (2007), to investigate the influence of confinement on rock mass failure in large slopes. The bi-linear envelope is based on the S-curve “Damage Initiation and Spalling Limit” developed to explain the occurrence of spalling failure in strong, highly stressed rock mass at the Underground Research Laboratory (URL) at Pinawa, Manitoba (Martin, 1994; Eberhardt, 1998; Diederichs, 1999; and Kaiser et al., 2000). The extensively studied mine-by test tunnel project at the URL showed that brittle stress-induced spalling occurred at stress levels of approximately 50% of laboratory-measured UCS values. The S-curve captures the influence of confinement in highly stressed, strong rock masses by incorporating a reduction in strength for strong, highly stressed, unconfined rock masses.

In unconfined conditions, as occur around tunnel walls, tensile cracks are free to propagate and cause excavation-parallel spalling failures. At high confining stresses, the propagation of tensile cracks is restricted, and the failure mechanism may transition towards a process of damage accumulation, potentially leading eventually to crack interaction and macroscopic shear failure (Diederichs, 2007). In terms of Mohr-Coulomb strength criteria, rock strength at low confining stress is dominated by the cohesion component, whereas rock strength at high confining stress is dominated by the friction component.

In a large rock slope with a height of 1 km or more, the S-shaped failure criterion introduced by (Diederichs 2007; and Kaiser et al., 2000) may be helpful in characterising the range of failure mechanisms that could occur at different locations inside the slope. Even if the potential for high horizontal tectonic stresses is ignored, the vertical stress within the interior of a pit slope approaching 1000 m high may be sufficient to exceed crack initiation thresholds, which range from 30% to 50% of laboratory UCS values for strong crystalline rock (Martin, 1994; Eberhardt et al., 1998; Diederichs, 2007). Figure 7-10 presents a conceptual diagram illustrating the potential for the S-curve failure envelope to capture confinement-dependent failure processes in three different sections of a large slope.
If vertical stress is taken to increase with the typical global stress gradient of 0.027 MPa/m, then a 1000 m-high slope would induce vertical stresses of 27 MPa under peak overburden. Stress concentrations occurring at the toe of the slope may be elevated even further, potentially promoting stress-induced brittle fracture or spalling-type failures.

For this investigation, parameters for the undisturbed rock mass were taken directly from the linear Mohr-Coulomb approximation of Lac du Bonnet granite reported by Diederichs (2007). The parameters describe very strong, massive, sparsely jointed rock mass with GSI ≈ 85 and UCS_{intact rock} = 230 MPa (Diederichs, 2007). According to the quantified GSI suggested by Cai et al. (2004), a GSI of 85 indicates blocky to massive rock mass with typical in situ block edge lengths exceeding approximately 0.7 m, and “very good” or “good” discontinuity surface conditions (Joint condition factor J_c > 2; Cai et al., 2004).

Shear strength parameters for the undisturbed rock mass are thus dominated by the strength of intact rock, with a low friction angle (c = 35 MPa, φ = 22° and σ_t = 7.5 MPa), and the residual strength parameters are friction-dominant, with negligible cohesion and
no tensile strength (c = 0.1 kPa, φ = 50° and σt = 0; the small non-zero cohesion is used to ensure numerical stability and prevent unrealistic localization, after the recommendations of Diederichs, 2007). Deformability parameters Young’s Modulus E and Poisson’s Ratio ν were selected to reflect typical deformability of good quality crystalline rock masses in the range of GSI > 70 (e.g. Hoek and Diederichs, 2006; Gercek, 2007).

Shear strength parameters for the blast damaged zone were derived by scaling the base parameters for the undisturbed rock mass, in order to represent the influence of blast-induced damage in degrading the rock mass. The blast damage factor D (Hoek, 2012) was used indirectly, to derive scaling parameters for the Mohr-Coulomb shear strength parameters and for the rock mass deformation modulus.

The scaling factors were derived using the code RocLab (Rocscience, 2007) to examine the influence of a blast damage factor of D = 1 (Hoek, 2012) on rock mass strength and deformability for a typical good quality rock mass. The steps for scaling the parameters are summarised as follows:

1. Input typical rock mass parameters for a strong, good quality rock mass:
   - UCS = 150 MPa
   - GSI = 75
   - Hoek-Brown parameter m = 25
   - Intact Rock Modulus E = 25 GPa
2. Use a blast damage factor of D = 0 and record resultant best-fit Mohr-Coulomb shear strength parameters, rock mass tensile strength and rock mass deformation modulus
3. Change blast damage factor to D = 1
4. Scaling factors are recorded as the percent change in output for best-fit Mohr-Coulomb shear strength parameters, rock mass tensile strength and rock mass deformation modulus

When applying the blast damage factor D, care should be taken in noting that the influence of D on rock mass strength and deformation modulus varies according to GSI. In Step 1, a hypothetical GSI value of 75 was used to derive the scaling parameters,
because the reduction in rock mass strength and stiffness is actually greater for GSI = 75 than for larger GSI values (Figure 7 - 11). The scaling parameters thus reflect a conservative estimate of the influence of blasting damage in a good quality rock mass.

Figure 7 - 11: Influence of blast damage factor $D$ on the global rock mass strength and rock mass deformation modulus in a good quality rock mass (GSI = 75); modulus calculation using the Hoek-Diederichs (2006) modulus method (Modified after Hoek, 2012).
It must also be noted that the Hoek-Brown failure criterion, and the blast damage factor $D$, are empirically-derived quantities based on published data from rock mechanics projects throughout the world. Parameters used in this investigation thus do not represent the precise mechanical properties for any real sites. However, the undisturbed rock mass and blasted rock mass parameters have been derived in order to construct a rational “what if” investigation into the influence of blasting and confinement-dependent strength for a large open pit slope excavated in good quality rock mass.

Peak strength parameters for both undisturbed and blasted rock are thus characterised by a dominant strength component from cohesive and tensile strength, and low frictional strength. In contrast, residual parameters are characterised by zero tensile strength, zero cohesive strength, and high frictional strength. The result is not a true S-shaped curve, but two intersecting linear functions, which approximate the transition from cohesion-dominated strength at low confining stresses, to frictional-dominated strength at high confining stresses.

When the peak strength is exceeded at low-confinement, strain softening behaviour occurs as the material strength transitions to the purely frictional residual parameters. When peak strength is exceeded a high confinement, strain hardening occurs as the material transitions to the high friction angle residual parameters (Figure 7 - 12).
Figure 7 - 12: Bi-linear approach for characterising confinement-dependent rock mass behaviour in underground excavations. Upper graph shows conceptual principal stress plot after Diederichs (1999, 2003, 2007), and Kaiser et al. (2000); lower plot shows linear Mohr-Coulomb approximation applied to the slope in Phase 2.
7.5.4. Preliminary Results and Discussion

The shear strength reduction function was applied to both rock mass and discontinuity properties. Failure occurred (i.e. non-convergence of the solution process) at a critical stress reduction factor (SRF) of 2 with a maximum displacement of 1.3 m occurring at the toe of the slope. Figure 7 - 13 shows a contour plot of minimum principal stress ($\sigma_3$) at failure, with overlain points indicating elements failed in tension and in shear. Insets show the development of tensile and shear failure in three different regions: (A) at the slope crest; (B) in the interior of the slide mass, under maximum overburden; and (C) at the toe of the slope.

The results show incipient slope failure occurring at three scales:

1. At the toe of the slope, extensive tensile and shear failure of finite elements suggests that stress-induced fracturing may contribute to bench-scale instabilities around zones of stress concentration (i.e. at the toe) in large slopes.

2. Evidence of inter-ramp scale failure occurs in the lower three benches of the slope, where tensile and shear damage propagates upward from the toe of the slope, through the blast-damage zone (Figure 7 - 13C). The results suggest that the thickness $T$ of the blast-damage zone can be an important influence on the depth of inter-ramp scale slope failures, if failure initiates within the blast damage zone.

3. Bi-planar, overall slope failure initiates along a stepped rear sliding surface comprising six parallel, non-coplanar discontinuities from set 1, with sliding on the shallow basal surface formed by the single large structure belonging to discontinuity set 2, with toe breakout occurring at the base of the slope through tensile and shear rock mass failure.
Figure 7 - 13: Highlighted regions of intact rock bridge failure for bi-linear, “s-curve” rock mass with blast damage zone; contours indicate minor principal stress $\sigma_3$. 

Critical SRF = 2.1 

Low confinement: direct tensile failure dominates, “wing crack” type failure of rock mass bridges 

Incipient multi-bench scale failure initiates in blast damage zone 

Toe breakout: rock mass bridge ruptures 

Increased confinement: damage accumulation, macroscopic shear failure increases 

$\sigma_3$
The overall failure mechanism is bi-planar, with a stepped rear sliding surface. A contour plot of total displacement reveals that the largest displacements occur at the toe of the slope (Figure 7 - 14). This result is in contrast with the active/passive biplanar mechanism described by Kvapil and Clews (1979), which suggested the largest displacements will occur in the upper, active wedge.

The results illustrate the dependence of the failure mechanism on the relative shear strengths of the upper and lower sliding surface. Discontinuity set 2 (the lower sliding surface) is weaker than discontinuity set 1, which forms the upper surface. As a result, the relative excess of driving forces versus resisting forces is larger on the lower surface than on the upper surface, resulting in maximum displacements occurring at the toe. Displacement vectors illustrate the curved transition from steep downwards displacements in the upper portion of the slide mass, to shallow sliding displacements along the basal surface.

Figure 7 - 14: Total displacement contours show that the maximum displacement occurs at the toe of the slope; displacement vectors reveal the stepped upper failure surface geometry and the transition from steep downward displacements in upper block to shallow sub-horizontal displacements in the lower block.
At the crest of the pit, confining stress is negligible; the driving force of the unstable rock mass is sufficient to induce direct tension failure in the 2.7 m bridge of rock mass that separates the uppermost segment of Discontinuity Set 1 (Figure 7 - 15).

Because confining stress is low near the ground surface, the transition from peak to residual strength parameters creates a strain softening effect. The transition from peak to residual strength parameters changes the rock mass from a state of high cohesive strength with low friction, to a zero-cohesion state with only frictional strength. Confining stresses ($\sigma_n$) are necessary to mobilize frictional shear strength ($\tau$), according to the Mohr-Coulomb relationship with friction angle ($\varphi$):

$$\tau = \sigma_n \tan \varphi$$

However, the mean stress state is tensile (i.e. $\sigma_n$ is negative), and thus no frictional shear strength is mobilized in the failed elements. The residual parameters thus result in a drop to near-zero mobilized shear strength.

![Figure 7 - 15: Mean stress at the top of the slope is tensile, causing tension cracking to develop in the rock mass bridge separating the uppermost segment of Discontinuity Set 1.](image)
In the middle of the slope, confining stresses are higher and failure of rock mass bridges shows evidence of both curved tension failures extending from pre-existing discontinuity tips, and secondary shear failure extending straight through extensive bridges of rock mass (Figure 7 - 16).

In the centre of the rock mass bridge, the principal stress values indicate approximately 10 MPa of confining stresses:

\[
\begin{align*}
\sigma_1 &= 28 \text{ MPa} \\
\sigma_2 &= 12 \text{ MPa} \\
\sigma_3 &= 10 \text{ MPa}
\end{align*}
\]

Figure 7 - 16: Differential stress for a rock mass bridge in the middle of the upper failure surface; tension failures curve away from the pre-existing discontinuities, and secondary shear failure directly connects the two discontinuity segments.

Despite increased confinement relative to the crest of the slope, the confining stress is insufficient to induce strain hardening when the rock mass transitions to friction-
dominated residual strength. The change from peak to residual strength parameters causes an overall drop in mobilised shear strength, although some frictional shear strength is mobilised within the failed rock mass bridge due to confinement.

Confining stress is highest in the deep interior of the slope, at the hinge point of the overall failure surface, where the shallow basal sliding surface is in closest proximity to the stepped upper sliding surface. Extensive shear failure occurs along the length of the shallow basal sliding surface, where confining stresses are up to approximately 16 MPa (Figure 7 - 17).

![Figure 7 - 17: Differential stresses (σ₁ – σ₃) contours around a rock mass bridge at the hinge in the overall failure surface, where the shallow basal surface meets the steep upper surface; shear failure accumulates along the length of the basal failure surface where confining stress is high.](image)

The occurrence of elements failed in shear along the basal surface supports the description of Kvapil and Klews (1979) and Stead (1984) that extensive secondary shearing and crushing must occur in the transition zone of a large bi-planar Prandtl-type failure before failure is kinematically possible.
7.6. Discontinuum Approach with *UDEC*

The Universal Distinct Element Code (*UDEC*, Itasca, 2010a) uses an explicit solution scheme and Newton’s second law to analyse forces and displacements for discrete blocks separated by discontinuities. The explicit solution scheme allows for simulation of large displacements including full separation and rotation of blocks, with automatic detection of new contacts as blocks are displaced.

The ability of the explicit solution process to step through large intervals of numerical time, past the initiation of slope failure, makes *UDEC* well suited to investigating post-failure internal deformations in a jointed rock mass. This investigation focuses on using *UDEC* to investigate progressive rock mass failure and deformation within the slide mass of a large bi-planar slope instability, with emphasis on characterising the influence of rock bridges within a Prandtl transition zone.

### 7.6.1. Model Setup

The modelled slope is 500 m high with an overall angle of 33° (Figure 7 - 18). Each modelled bench corresponds to a multi-bench stack, as are commonly implemented in large open pit slopes (e.g. Read and Stacey, 2009); bench stack heights range from 33 m to 64 m and bench stack angles vary from 55° to 77°.

A pre-existing, bi-planar global failure surface is inserted with an upper surface dipping at 49° and a curved lower surface dipping at 10°. The interior of the slide volume is discretized into random Voronoi polygons with edge lengths of 10 m. History monitoring points are inserted at the crest of each bench stack in order to record slope movements. Five different cases of the model are assessed, each with the same basic slope geometry, but different quantity of *rock mass bridges* within the slide volume.
Figure 7 - 18: Base slope geometry with three excavation stages, a pre-existing bi-planar failure surface; the slide mass is discretized into random Voronoi polygons with 10 m edge lengths.
The sides of the model are fixed such that no horizontal displacement may occur and the bottom of the model is fixed so that no vertical displacement may occur. Based on the slope geometry and the pre-inserted failure surface, displacements are expected to be confined to the assembly of Voronoi polygons (the overall slide volume is predetermined).

Although no intact rock bridges are explicitly included along the global failure surface, rock mass bridges are included within the slide mass by assigning rock mass strength parameters to Voronoi contacts. For the base case, the entire slide mass is comprised of strong Voronoi contacts with rock mass properties, effectively representing an areal rock mass bridge content of 100%.

Subsequent cases investigate the influence of decreasing rock mass bridge content, by selectively assigning weak frictional joint properties to certain Voronoi contacts, according to dip angle, after the approach of Schön (2011), who used Voronoi tessellation to simulate rock mass failure in the Barrier rock slope, Southwest BC.

Schön (2011) assigned elasto-plastic failure criteria to intra-block materials, allowing rock mass failure to occur through a combination of internal block failure, as well as tensile and shear failure of Voronoi contacts. For this investigation, however, intra-block zone properties are assigned idealized linear elastic parameters, so that failure is forced to occur through breakage of Voronoi contacts, which represent equivalent brittle fracture paths through the rock mass. Due to the large size of the Voronoi polygons (10 m edge lengths), the Voronoi contacts do not represent discrete fracture paths through intact rock, but instead represent contorted, complex failure paths through the rock mass.

The mechanism of overall slope failure is determined by the pre-existing sliding surfaces, but the quantity of rock mass bridges in the slide volume is expected to influence the development of progressive failure.

Table 7 - 2 summarises the angular ranges used to add weak frictional joints into the Voronoi polygon assembly for the five model cases.
Table 7 - 2: Angular range used to assign frictional joint properties to selected Voronoi contacts.

<table>
<thead>
<tr>
<th>Model Case</th>
<th>Rock Mass Bridge Content (%)</th>
<th>Joint Set 1 / Rear Release Angular Range (°)</th>
<th>Joint Set 2 / Basal Sliding Angular Range (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case A</td>
<td>100</td>
<td>Nil</td>
<td>Nil</td>
</tr>
<tr>
<td>Case B</td>
<td>94</td>
<td>80 to 85</td>
<td>-20 to -15</td>
</tr>
<tr>
<td>Case C</td>
<td>81</td>
<td>75 to 90</td>
<td>-25 to -10</td>
</tr>
<tr>
<td>Case D</td>
<td>54</td>
<td>70 to 95</td>
<td>-30 to -5</td>
</tr>
<tr>
<td>Case E</td>
<td>37</td>
<td>60 to 105</td>
<td>-40 to 5</td>
</tr>
</tbody>
</table>

The base case (A) contains no weak frictional joints in the slide mass; each successive case is simulating reduced rock mass bridge content by increasing the angular range of dip angles used to define weak frictional joints. Figure 7 - 19 shows the decrease in rock mass bridges as the angular range for weak frictional joints is increased from Case A to E.

The rock mass bridge content is calculated as the sum length of strong Voronoi contacts with rock mass properties, divided by the total contact length of all Voronoi polygons:

$$Rock\ Mass\ Bridge\ \% = \frac{\sum L_{Voronoi}^{Strong\ Rock\ Mass}}{\sum L_{Voronoi}^{Strong\ Rock\ Mass} + \sum L_{Voronoi}^{Weak\ Frictional\ Joints}}$$

Thus in the base case 100% rock mass bridges exist within the slide mass, because the sum length of Voronoi polygons (1.34 x 10^4 m) is entirely comprised of strong Voronoi contacts, and the sum length of Voronoi contacts with weak frictional joint properties is zero.
Figure 7 - 19: Volumetric rock mass bridges content decreases as the angular range of joint sets 1 and 2 is increased from Case A to Case D, light coloured Voronoi contacts have strong rock mass properties, and dark coloured Voronoi contacts have weak frictional joint properties.
7.6.1.1. Overall Slide Surface Parameters

The overall failure surface is comprised of two parts: a steep upper surface underlying the driving active block, and a shallow basal surface underlying the passive block. Mohr-Coulomb strength parameters for the sliding surfaces were specifically selected to precondition the slope for bi-planar sliding.

The strength of the upper sliding surface reflects a frictional interface associated with typical rock joints, and intended to simulate the influence of a ubiquitous persistent joint set with a minor cohesive and tensile strength component, allowing for a limited amount of intact rock bridges (i.e. <1% length).

The lower surface represents an extremely weak contact characteristic of a potential contact in weak argillaceous sedimentary rocks, or a low-angle listric fault with a minor cohesion component (5 kPa) arising from fault gouge or infill material, and zero tensile strength. The strength parameters for the upper and lower sliding surfaces are summarised in Table 7 - 3.

Table 7 - 3: Summary of bi-planar surface properties.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Upper Sliding Surface (Dip = 49°)</th>
<th>Basal Sliding Surface / Fault (Dip = 10°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction Angle, φ (°)</td>
<td>36</td>
<td>15</td>
</tr>
<tr>
<td>Cohesion, c (MPa)</td>
<td>0.025</td>
<td>0.005</td>
</tr>
<tr>
<td>Tensile Strength, σ_T (MPa)</td>
<td>0.005</td>
<td>0</td>
</tr>
<tr>
<td>Normal Stiffness, k_n (GPa/m)</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Shear Stiffness, k_s (GPa/m)</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>
7.6.1.2. Rock Mass and Minor Discontinuity Parameters

Internal block zones are modelled as ideal linear elastic material with Young’s Modulus of $E = 25$ GPa and Poisson’s Ratio of $v = 0.26$, so that failure of the slide mass is forced to occur through breakage of Voronoi contacts. A unit weight of $27$ kN/m$^3$ is assumed.

Figure 7 - 19 shows how the slide mass comprises an assembly of Voronoi contacts with three material types: strong rock mass, and two minor joint sets representing a steeply-dipping rear release set, and a shallowly-dipping sliding set.

Defining properties for the two minor joint sets is straightforward. Both joint sets simulate frictional tension joints with medium to high persistence (i.e. 3 m to 20 m length) and no cohesive or tensile strength. The joint normal stiffness ($j_{kn} = 10$ GPa/m) and joint shear stiffness ($j_{ks} = 1$ GPa/m) of the two minor joint sets was selected according to the characteristic values suggested by Itasca (2010a) for joints in strong crystalline rock, and previously applied in a UDEC Voronoi-based study of complex toppling by Alzo’ubi (2009).

In contrast to conventional discontinuities, selection of strength and stiffness properties for the Voronoi rock mass contacts is scale-dependent. In small models, millimetre-scale contacts can represent individual mineral grains, and may be assigned very high stiffness (Kazerani and Zhao, 2010). However, as the size of the contacts increases, stiffness must be decreased in order to accurately simulate rock mass deformation. Thus stiffness parameters for the Voronoi contacts were selected according the guidelines suggested by Itasca (2004), Christianson et al. (2006) and Alzo’ubi (2009), where normal stiffness is related to rock mass parameters bulk modulus $K$, shear modulus $G$, and the minimum perpendicular width ($\Delta Z_{min}$) of an adjoining zone for the Voronoi contacts:

$$k_n = \text{max} \left( \frac{K + \frac{3}{2}G}{\Delta Z_{min}} \right)$$  \hspace{1cm} \text{Eq. 7.1 (Alzo’ubi, 2009; and Itasca, 2004)}

To ensure that failure is driven by deformation and breakage of Voronoi contacts, the stiffness properties for Voronoi rock mass are based on a slightly softer material than the linear elastic intra-block zone properties. Stiffness for strong Voronoi contacts (i.e. rock mass) is thus based on a Young’s modulus of $E = 20$ GPa, and Poisson’s Ratio of 0.26.
Based on Equation 7.1, Voronoi contact normal stiffness is calculated as follows:

\[ k_n \approx \frac{13.9 \text{ GPa} + \left( \frac{4}{3} \times 7.9 \text{ GPa} \right)}{10 \text{ m}} \]

\[ k_n \approx 2.5 \text{ GPa/m} \]

The relationship between joint normal stiffness, joint shear stiffness and Poisson’s ratio, suggested by Christianson et al. (2006), was used to estimate shear stiffness for Voronoi rock mass contacts:

\[ \frac{k_n}{k_s} \approx 10\nu = 10 \times 0.26 \]

Thus:

\[ k_s \approx \frac{2.5}{2.6} \approx 1 \text{ GPa/m} \]

The bulk strength of a rock mass represented by a Voronoi assembly is dependent on the cohesion and friction angle of individual small-scale block contacts. Kazerani and Zhao (2010) found that the bulk material friction angle depends on both small-scale contact friction angle and small-scale contact cohesion, whereas bulk material cohesion depends only on contact cohesion. They also found that Voronoi contact tensile strength significantly controls bulk material tensile strength, and also contributes to the bulk material UCS.

Gao (2013) found that the interlocking effect caused by the angular shape of Voronoi polygons has a strong influence on bulk material friction angle; simulated UCS tests showed that a friction angle of approximately 11° applied to millimetre-scale Voronoi contacts resulted in a bulk material friction angle of over 40°.

Table 7 - 4 summarises the strength and stiffness parameters assumed for the Voronoi rock mass contacts and the two minor joint sets.
Table 7 - 4: Summary of discontinuity properties for UDEC Voronoi model.

<table>
<thead>
<tr>
<th></th>
<th>Voronoi Rock Mass Contacts</th>
<th>Steep Rear Release Joints</th>
<th>Shallow Sliding Joints</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction Angle, $\varphi$ ($^\circ$)</td>
<td>11</td>
<td>36</td>
<td>29</td>
</tr>
<tr>
<td>Cohesion, $c$ (MPa)</td>
<td>6</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Tensile Strength, $\sigma_T$ (MPa)</td>
<td>3.5</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Normal stiffness, $j_{kn}$ (Pa/m)</td>
<td>$2.5 \times 10^9$</td>
<td>$1 \times 10^{10}$</td>
<td>$1 \times 10^{10}$</td>
</tr>
<tr>
<td>Shear stiffness, $j_{ks}$ (Pa/m)</td>
<td>$1 \times 10^9$</td>
<td>$1 \times 10^9$</td>
<td>$1 \times 10^9$</td>
</tr>
</tbody>
</table>

7.6.2. Preliminary Results and Discussion

7.6.2.1. Summary

Each model case is based on the same calculation sequence of pre-mining activity, where the full slope is constructed in three major excavation stages. After the slope is excavated, each case investigates the differences in post-mining behaviour of the slope with changes in rock mass bridge content inside the slide mass. Despite each model case having the same overall failure surface geometry and strength parameters, post-mining displacements and rock mass failure are different in each case.

Displacements were monitored from the bench crest history points, and internal contact failure inside slide mass was recorded using a function developed by Gao (2013) in the Itasca programming language FISH (Itasca, 2010c). The function records the length and number of contacts that fail, sorted into three categories:

**Type 1:** Direct contact tension failure

**Type 2:** Pure contact shear failure, with no opening of the contact occurring at any time throughout the cycling of the model; the contact remains closed throughout the cycling, and is failed in shear.
**Type 3:** Shear failures that subsequently undergo dilation/opening. The initial contact failure is still characterised by a shear failure mechanism.

Type 3 failure still indicates shear failure, it is not a separate failure mechanism from Type 2, however the subsequent opening of the failed contacts may lend insight into the extent of post-failure dilation (opening) of failed block contacts.

Table 7-5 summarises the maximum displacements and total length of new cracks generated during post-mining deformations. Case A, B, and C seem to reach equilibrium, based on the cessation of surface displacements and internal cracking. Case D and E, however, undergo extensive internal shear failure, and appear to proceed towards overall slope failure.

The results indicate that shear failure of Voronoi contacts dominates over tensile failure. For Cases A, B, and C there is a negligible quantity of tensile fracturing of the rock mass. Although the absolute quantity of tensile fracturing increases in Case D and E, the cumulative length of contacts failed in shear is up to an order of magnitude larger than the length of contacts failed in tension.

The development of contact failure versus numerical time is further addressed with contact failure vs numerical time plots in Section 6.2.3.
Table 7 - 5: Summary of UDEC model displacements and total lengths of broken contacts during progressive failure.

<table>
<thead>
<tr>
<th>Case</th>
<th>Maximum Displacement (cm)</th>
<th>Length of Internal Fractures (m)</th>
<th>Shear Failure</th>
<th>Tensile Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Total</td>
<td>Shear then Opening [%]</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(Type 2 + Type 3)</td>
<td>(Type 3)</td>
</tr>
<tr>
<td>Case A</td>
<td>8.9</td>
<td>35.3</td>
<td>18.8 [53%]</td>
<td>5.1</td>
</tr>
<tr>
<td>Case B</td>
<td>9.0</td>
<td>56.6</td>
<td>37.0 [65%]</td>
<td>5.1</td>
</tr>
<tr>
<td>Case C</td>
<td>9.6</td>
<td>86.1</td>
<td>75.0 [87%]</td>
<td>0</td>
</tr>
<tr>
<td>Case D*</td>
<td>67.0</td>
<td>859.7</td>
<td>822.6 [96%]</td>
<td>115.2</td>
</tr>
<tr>
<td>Case E*</td>
<td>80.0</td>
<td>1455.9</td>
<td>1415.0 [97%]</td>
<td>98.0</td>
</tr>
</tbody>
</table>

*NOTE 1: Case A, B, and C are interpreted to have reached equilibrium, based on cessation of displacements and generation of new cracks (broken contacts). Case D and Case E appear to result in overall slope failure.

In addition to highlighting the role of internal shearing, the results show that post-failure dilation of the slide volume is an important factor in the development of kinematic freedom. As the slope becomes more unstable from Case A to Case E, the proportion of contact shear failures that undergo subsequent dilation (opening) increases from 53% for Case A, to 97% for Case E.

Figure 7 - 20 presents a plot showing the proportion of contact shears undergoing subsequent dilation, as a function of rock mass bridge content.
Figure 7 - 20: As rock mass bridge content is reduced, a greater proportion of contacts that initially fail in shear are subject to subsequent dilation (opening), Voronoi size = 10 m.

The proportion of shear failures that subsequently open approaches 100% as rock mass bridge content is reduced to values less than approximately 50%. The results appear to conform to a bi-linear trend, with the intersection point occurring at a rock mass bridge content of 80%, where approximately 87% of all contact shear failures subsequently undergo opening. Care should be taken in noting that rock mass behaviour may be dependent on Voronoi polygon size. Future works should investigate the sensitivity of overall slope scale numerical models to changes in Voronoi polygon size.

Figure 7 - 21 presents a plot of maximum displacement versus rock mass bridge content. The results indicate two major point groupings: Case A, B, and C have rock bridge content in the range of 80% to 100%, and remain stable, with approximately 10 cm of total displacement. Case D and E have reduced rock bridge content to 54% and 37%, and appear to be unstable, with maximum displacements of 67 cm and 80 cm. A potential transition zone is highlighted, between rock mass bridge content of approximately 55% and 80%, where a “critical rock mass bridge content” corresponding to an overall safety factor of 1 may exist. Importantly, the precise rock mass bridge
content corresponding to a safety factor of 1 may be dependent on model factors including Voronoi polygon size.

Figure 7 - 21: Maximum displacement versus rock mass bridge content, showing a potential transition zone between stability for Cases A, B, and C, and instability for Cases D and E.

Further investigations across a wider range of rock mass bridge configurations and Voronoi polygon sizes would help to better understand the role of rock mass bridges in (1) arresting the development of the Prandtl transition zone, and overall deformation throughout the unstable volume, before slope failure, and (2) influencing post-failure dilation and breakup of the failed volume.

7.6.2.2. Slope Movement History

Crest movement records are interpreted from a history point at the intersection of the upper sliding surface with the pit crest; mid-slope displacements are interpreted based on averaged total displacement magnitude across four bench crest monitoring points around the mid-slope; toe displacements were evaluated from a history point at the overall toe the slope (Figure 7 - 22).
Displacement records for the pit crest are shown in Figure 7 - 23; averaged displacement across four mid-slope benches is shown in Figure 7 - 24; and Figure 7 - 25 shows the displacements recorded from the toe of the slope. The movement records show the following trends:

- Case A, B, and C all trend towards stability, with the highest magnitudes of total displacement occurring at the crest of the pit, followed by the mid-slope benches, and the lowest magnitude of total displacement occurring at the toe of the slope:

$$\Delta_{\text{CREST}} > \Delta_{\text{MID-SLOPE}} > \Delta_{\text{TOE}}$$

- Case A, B, and C all show similar magnitude of total displacement for a given location on the slope
- Case D and Case E show a marked departure from stability, with magnitude of total displacement approximately 400% larger than Case A, B, and C, and no indication of trending towards stability (i.e. displacements throughout the slope continually increase throughout the modelled numerical time span)
Case D and Case E show the highest magnitude of total displacement occurring at the crest of the slope, with similar displacement occurring at the toe of the slope; in contrast to the other models, the mid-slope benches show the lowest magnitude of total displacement:

\[ \Delta_{\text{CREST}} \approx \Delta_{\text{TOE}} > \Delta_{\text{MID-SLOPE}} \]

The magnitude of total displacement for Case D and Case E diverges with increases in numerical time: Case E has the lowest quantity of rock mass bridges (37%), and shows approximately 20% greater magnitude of total displacement than Case D (54% rock mass bridges), across all monitoring locations across the slope, at the maximum calculated numerical time.

All displacement histories show an initial drop in total displacement that occurs after the initial calculation of equilibrium, during the three-stage excavation of the slope, while the rock mass and contacts are assigned strong elastic properties in order to limit pre-excavation displacements. The transition to increasing total displacements begins at the time-step after the last stage of slope excavation, when the true rock mass properties, described in Section 6.1, are applied to the model.
Figure 7 - 24: Total displacement across mid-slope benches vs numerical time.

Figure 7 - 25: Total displacement at pit toe vs. numerical time.
Figure 7 - 26 shows the contrast between displacements of the slide mass in Case A, which trends toward stability, and Case E, which trends towards failure.

Figure 7 - 26: Displacement vectors for Case A at equilibrium, with maximum displacement of 8 cm at the pit crest, and Case E during failure, with maximum displacement of 0.8 m at the toe.
The small displacements in Case A are insufficient to cause extensive failure of rock mass bridges, and thus no Prandtl transition zone develops. Instead, the slide mass shows uniform, steeply-dipping contours of total displacement, which grade from the lowest displacements at the toe of the slope ($\approx 2$ cm), to the highest displacements at the crest of the slope ($\approx 8$ cm).

In Case E, the reduced content of rock mass bridges results in a weakened slide mass where more internal dilation can occur. As larger displacements progress, the slide mass appears to become partitioned into an active upper block, a passive lower block, and a transition zone with curved shear surfaces.

Figure 7 - 27 presents contour plots of total displacement for Cases A, B, and C. The contours confirm that displacements of up to approximately 8 cm develop within the slide mass in the stable cases, with the contours oriented approximately sub-vertical, showing no indication of radial shear surface development. Larger displacements develop within the pre-defined slide mass than in the surrounding country rock, indicated by the mismatch of total displacement contours at the boundary between the country rock and the slide mass. Despite the presence of mining-induced displacements, the slopes trend towards stability.

Figure 7 - 28 presents contour plots of total displacement for Case D and E, which illustrate the structural changes within the slide mass associated with the transition from stability to instability. Whereas Cases A, B, and C, indicate uniform sub-vertical partitioning of displacement contours, Cases D and E indicate a transition towards instability and displacement contours reveal spiral shear surfaces occurring in a Prandtl-like transition zone between the active upper block and the passive lower block. The displacements in the surrounding country rock are similar to those observed for the stable cases, not exceeding approximately 10 cm. However, displacements of up to 72 cm occur within the Prandtl transition zone, indicating the destruction of rock mass bridges to form complex stepped shear surfaces which link between pre-existing joints.
Figure 7 - 27: Displacement magnitude contours with 2 cm increments, illustrating division of the slide mass into separate blocks.
The results suggest that the transition from stability to instability occurs at an intermediate rock mass bridge content between Case C (81% rock mass bridges) and Case D (54% rock mass bridges). In Case C, the rock mass bridge content within the slide mass is sufficient to create an overall strength and stiffness of the slide mass that is high enough to arrest the development of a Prandtl transition zone under gravitational
loading, and thus denies kinematic freedom and prevents overall slope failure from occurring.

In Case D, the increased content of weak frictional joints within the slide mass causes internal shear surfaces to nucleate and propagate, forming concentric, ubiquitous or closely-spaced shear surfaces that allows larger internal deformations to occur, eventually leading towards the development of the Prandtl transition zone and overall slope failure.

7.6.2.3. Cracking / Rock Mass Dilation History

By monitoring the total length of internal fracturing of rock mass bridges within the slide mass, a damage intensity parameter for the slide mass delineated $D_{21}$ can be calculated, expressing the total length of internal fractures created during failure, divided by the total 2-dimensional surface area of the slide mass in cross-section (Figure 7 - 29):

$$D_{21} = \frac{\sum \text{Length}_{\text{broken/failed}}}{\text{Cross - Sectional Area of Slide}}$$

Figure 7 - 30 presents a graph of $D_{21}$ for the five slope model cases. The results suggest a bi-linear best-fit. Cases A, B, and C are fit by a shallow function with $D_{21}$ values in the order of $1 \times 10^{-3}$ m$^{-1}$. Cases D and E are fit by a steeper function, which varies from 0.01 to 0.02 m$^{-1}$. A potential transition zone is highlighted between the two groupings, indicating that a theoretical “Safety Factor = 1” point exists between a rock mass bridge content of approximately 54% and 82%. Because the best-fit line for Cases D and E is steeper than for Cases A, B, and C, the damage intensity for unstable cases is more sensitive to reductions in rock mass bridge content than for stable cases.

Plots showing the failure state of broken contacts, and the associated $D_{21}$ values, illustrate the extent of damage and the prevalence of dilation (open contacts) within the slide mass. Figures 8-31 to 8-35 show the instantaneous state of failed contacts for Case A to Case E, including both contacts failed in tension and in shear. The results show a trend of increasing intensity in damage (i.e. failed Voronoi contacts) from Case A to Case D, as the content of rock mass bridges is decreased. As the damage intensity increases from Case A to E, the newly-failed contacts indicate extensive shear failure within the transition zone between the active upper block and passive lower block.
Figure 7 - 29: Areal intensity for damage in the slide mass, $D_{21}$ may be expressed as the sum length of internal fractures divided by the surface area of the slide mass.

$$\Sigma \text{Internal crack lengths} = 3a$$

$$D_{21} = \frac{3a}{87432 \text{ m}^2}$$

Figure 7 - 30: Damage parameter $D_{21}$ versus rock mass bridge content for each model case.
Figure 7 - 31: Case A, instantaneous discontinuity status at equilibrium (Cycle 2.2 x 10^6; numerical time 8.9 x 10^2 s); tension crack beginning to develop at crest of the slope; maximum displacement = 8.9 cm

\[ D_{21} = 4.6 \times 10^{-4} \text{ m}^{-1} \]

Figure 7 - 32: Case B, instantaneous discontinuity status at equilibrium (Cycle 2.2 x 10^6; numerical time 8.9 x 10^2 s); minor tension and shear damage beginning to develop throughout the slide mass; maximum displacement = 9.0 cm

\[ D_{21} = 7.1 \times 10^{-4} \text{ m}^{-1} \]
Figure 7 - 33: Case C, instantaneous discontinuity status at equilibrium (Cycle 2.7 x 10^6; numerical time 1.1 x 10^3 s); tension cracking developing at crest, shear damage developing in mid-slope transition zone; maximum displacement = 9.6 cm

$D_{21} = 9.8 \times 10^{-4} \text{ m}^{-1}$

Figure 7 - 34: Case D, instantaneous discontinuity status (Cycle 4.6 x 10^6; numerical time 1.9 x 10^3 s); extensive shear damage and open contacts in mid-slope transition zone; maximum displacement = 67.6 cm

$D_{21} = 1.1 \times 10^{-2} \text{ m}^{-1}$
In addition to simulating internal tensile and shear damage within the slide mass, the Voronoi tessellation also results in bench-scale failures in Case E, where single 10-m polygons have failed in planar sliding (Figure 7 - 35). The coarse resolution of the Voronoi polygons is insufficient to precisely characterise instabilities smaller than 10 m, but the two bench failures suggest that the Voronoi approach may be adapted to characterise multiple scales of slope instability simultaneously.

Figure 7 - 35: Case E, instantaneous discontinuity status (Cycle 4.6 × 10^6; numerical time 1.9 × 10^3 s); extensive shear and tensile damage in mid-slope transition zone; maximum displacement = 34.7 m occurring for bench-scale block failures in lower slope; maximum displacement discounting local instability is approximately 80 cm.

Data recorded by the FISH function developed by Gao (2013) shows that contact shear failure tends to dominate over contact tensile failure during slope failure. In Cases D and E, which trend toward instability, the total number of contacts failed in tension number 50 and 42, respectively, whereas the total number of contacts failed in shear number approximately 330 for Case D and 560 for Case E.

Figure 7 - 36 shows the contacts failed in shear versus numerical time, indicating some initial shear failure in Case A, B, and C, trending towards stability at numerical time of approximately 100 s, after which no additional contacts fail in shear. In contrast, Case D
and Case E show an increasing number of contacts failed in shear throughout the simulation time. Small step-wise increases in contact shear failure occur, where 10 to 20 new contacts (each with edge length in the order of 10 m) fail almost instantaneously. The step-wise increases in contact shear failure associate closely in numerical time with similar step-wise increases in tensile contact failure (Figure 7 - 37).

![Figure 7 - 36: Contacts failed in shear vs. numerical time.](image)

Figure 7 - 36: Contacts failed in shear vs. numerical time.

Figure 7 - 37 shows contacts failed in tension versus numerical time, indicating that negligible tensile damage occurs in Case A, B, and C, which trend toward stability. Case D and E, which trend towards instability, show near-instantaneous step-wise increases in tensile failure in association with the increased shear failure recorded in Figure 7 - 36.

The individual histories of contact failure for each case are presented in Figures 8-38 to 8-42, including all three failure types (direct tensile failure, pure shear with no opening, and shear with subsequent opening of the contact at a later numerical time step). The contact failure histories all begin recording approximately at numerical time of t = 50 s, when the slope excavation is completed and the true rock mass and contact properties are applied to the contacts and zone mesh, according to the parameters discussed in Section 6.1.
Figure 7 - 37: Contacts failed in tension vs. numerical time

Figure 7 - 38: Case A, number of failed contacts vs numerical time.
Figure 7 - 39: Case B, number of failed contacts vs numerical time.

Figure 7 - 40: Case C, number of failed contacts vs numerical time.
Figure 7 - 41: Case D, number of failed contacts vs numerical time.

Figure 7 - 42: Case E, number of failed contacts vs numerical time.
7.6.2.4. Preliminary Conclusions

The results indicate that the transition from stability to instability of the modelled slope occurs at an intermediate quantity of rock mass bridge content between Case C (81% rock mass bridges) and Case D (54% rock mass bridges). With all other parameters held constant, slope displacements and internal contact failure inside the slide mass are influenced by the quantity of rock mass bridges represented by strong Voronoi contacts.

In all model cases, shear failure dominates over tensile failure, and the majority of contacts that fail in shear also undergo subsequent opening, suggesting that initial shear failure may be a precursor to rock mass dilation. In Case A, B, and C the slide mass is sufficiently strong as to prevent the full development of a Prandtl transition zone, and thus the slopes trend towards stability. Case D and E, however, show evidence of Prandtl wedge development at small overall slope displacements (< 1 m), including extensive damage inside the slide mass and curved concentric shear surfaces. The development of failed contacts in Case D and E suggests three broad stages of internal cracking inside the slide mass: (I) instantaneous initial crack development, followed by (II) non-linear primary cracking, with step-wise increases in cracking, followed by (III) steady-state or constant-rate behaviour after overall failure is initiated.

A damage intensity factor $D_2$ analogous to the areal discontinuity intensity factor $P_{21}$ is proposed, expressing the total length of new fractures inside the slope, divided by the 2-D cross-sectional surface area of the slide mass, with units of [m$^{-1}$]. The five model cases show $D_2$ values ranging over two orders of magnitude, from $4.6 \times 10^{-4}$ m$^{-1}$ for stable conditions in Case A (100% rock mass bridges), to $1.8 \times 10^{-2}$ m$^{-1}$ for unstable conditions in Case E (37% rock mass bridges).

7.7. Lattice-Spring Approach with Slope Model

*Slope Model* is a new code developed by Itasca (2010b) as part of the CSIRO Large Open Pit Project. The code applies the Synthetic Rock Mass method (Mas Ivars et al., 2011), where intact rock (or equivalent rock mass) is represented by a 3-D assembly of point masses connected by nonlinear springs, and discontinuities are represented by the smooth joint model, which allows slip and separation according to the predefined joint
orientation, rather than the local orientation of the contacts (Cundall and Damjanac, 2009). Brittle fracture is simulated via the breakage of springs, allowing for explicit simulation of intact rock bridge failure. Intact rock bridges may be modelled using either of two methods: (1) random patches of intact rock may be simulated along a discontinuity surface, according to a persistence factor which describes the areal continuity of the plane; or (2) non-coplanar intact rock bridges may be included explicitly as “the space in-between” pre-existing discontinuities, by importing a discrete fracture network where the size, location and orientation of each discontinuity is explicitly defined.

7.7.1. Model Setup

The 500 m-high slope investigated in UDEC was imported into Slope Model and extruded 50 m in the out-of-plane direction, in order to simulate a pseudo-2-dimensional simplified geometry (Figure 7 - 43). Slope displacements were recorded at history points positioned at the crest of each bench and at the toe of the slope. The cumulative number of fractured lattice-springs (cracks) was also automatically recorded for each case.

![Figure 7 - 43: Base geometry used in Slope Model, based on the same 2-D cross-section from the UDEC analysis, extruded 50 m out-of-plane.](image)
Rock mass and discontinuity strength parameters were selected to approximate the parameters applied in the *UDEC* models. Table 7 - 6 summarises the properties assumed for the lattice-springs, including Mohr-Coulomb friction and cohesion, as well as rock mass UCS, tensile strength, Young’s Modulus and Poisson’s ratio.

**Table 7 - 6: Summary of simulated rock mass properties.**

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Rock Mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction Angle, $\phi$ (°)</td>
<td>41</td>
</tr>
<tr>
<td>Cohesion, $c$ (MPa)</td>
<td>6</td>
</tr>
<tr>
<td>Rock Mass Compressive Strength, $UCS_{RM}$ (MPa)</td>
<td>35</td>
</tr>
<tr>
<td>Tensile Strength, $\sigma_T$ (MPa)</td>
<td>3.5</td>
</tr>
<tr>
<td>Young’s Modulus, $E_{RM}$ (GPa)</td>
<td>25</td>
</tr>
<tr>
<td>Poisson’s Ratio, $\nu$</td>
<td>0.26</td>
</tr>
<tr>
<td>Units Weight (kN/m$^3$)</td>
<td>27</td>
</tr>
</tbody>
</table>

Table 7 - 7 summarises the strength properties of the failure surface, which were selected to pre-condition the slope to bi-planar sliding failure. Friction angle, cohesion and tensile strength are identical to the *UDEC* simulation, but discontinuity stiffness values are, by default, defined by the rock mass matrix (i.e. lattice-springs) that they intersect. Separate joint stiffness parameters currently have no consequence for joint mechanical behaviour (Itasca, 2010b).
Table 7 - 7: Summary of bi-planar surface properties.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Upper Sliding Surface</th>
<th>Basal Sliding Surface / Weak Fault</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction Angle, φ (°)</td>
<td>36</td>
<td>15</td>
</tr>
<tr>
<td>Cohesion, c (kPa)</td>
<td>25</td>
<td>5</td>
</tr>
<tr>
<td>Tensile Strength, σ_T (MPa)</td>
<td>0.005</td>
<td>0</td>
</tr>
</tbody>
</table>

A preliminary base case model (Case 1a) was simulated with an intact slide volume and a lattice resolution of $R = 5$ m. The base case was then re-run using an increased resolution $R = 2.5$ m (Case 1b). Since no pre-existing discontinuities exist within the slide volume, rock mass dilation is forced to occur entirely by brittle failure of lattice springs.

To investigate the influence of pre-existing discontinuities within the slide mass, two additional models, denoted Case 2a and 2b, were run with two pre-existing discontinuity sets inserted inside the expected area of the Prandtl transition zone. As with the UDEC models, the pre-existing joints inside the slide volume are separated into two groups:

1. Steeply-dipping joints that may act as rear release surface or steep step-path segments, with friction angle of $\varphi = 36^\circ$

2. Shallow to moderately-dipping joints that may act as a basal sliding surface for step paths, with friction angle of $\varphi = 29^\circ$

Case 2a and 2b have identical model geometry, but different model resolution. Case 2a was run using a lattice resolution of $R = 5$ m. Case 2b was run using an increased resolution of $R = 2.5$ m in order to investigate potential resolution-dependency of modelled slope behaviour.
7.7.2. Preliminary Results

7.7.2.1. Base Model, Case 1a, R = 5 m

The base model (Case 1a) shows evidence of both global slope instability and bench-scale failure. Displacements on the surface of the slope accelerate throughout the simulation, and brittle failure occurs extensively throughout the slide mass. Kinematic freedom is developed through brittle failure in three important areas:

(1) A tension crack develops at the crest of the slope, as predicted by the theoretical model proposed by Kvapil and Klews (1979)

(2) Inclined shear surfaces develop in the upper active block (α = 33° to 74°)

(3) Extensive fracturing occurs in the transition zone, with curved shearing surfaces propagating upward from the hinge of the overall failure surface.

Figure 7 - 44A illustrates the extent of fracturing within the slide mass as kinematic freedom develops through the formation of a transition zone between the upper active block and the lower passive block. Annotations indicate the angles of inclination of internal shear surfaces.

A plot of displacement vectors is given in Figure 7 - 44B, showing that the largest magnitude of displacement occurs in the lower block, towards the toe of the slope. The occurrence of the highest displacement within the lower block is probably due to the relative excess of driving versus resisting forces on the weak lower surface (representing a fault, for instance), compared with a greater proportion of resisting forces occurring on the stronger upper surface.

The contours of total displacement reveal spiral shear surfaces developing in the transition zone, centred near the hinge of the overall failure surface, similar to the idealized log-spiral surfaces predicted for the Prandtl wedge mechanism. The spiral shear surfaces occur confined between the major curved shear surface, CD, in the lower block, and the major inclined surface in the upper block with overall inclination of 32°, AB.
Figure 7 - 44: Case 1a at numerical time $t = 3.5$ s - brittle rock mass failure, indicated by failed lattice springs, allows kinematic freedom to develop (A); displacement vectors and failed springs plotted together reveal the development of spiral shear surfaces in the transition zone (B).
7.7.2.2. Base Model, Case 1b, R = 2.5 m

To examine the influence of the lattice resolution $R$, Case 1b was rerun using the same base model geometry with $R = 2.5$ m, double the resolution of the first model. Figure 7 - 45 illustrates the development of kinematic freedom through the development of internal shear damage. Some important differences occur between Case 1a and Case 1b:

- No crest tension crack develops at the crest of the pit in Case 1b
- An incipient shear surface occurs near the top of the upper active block, appearing to propagate simultaneously upward from the global failure surface, and downward from the slope surface
- The overall inclination of shear surface that forms the interface between the upper active block and the transition zone, AB, is steeper in Case 1b (38°) than in Case 1a (32°)
- Prandtl wedge development in Case 1b is apparent at a later numerical time ($t \approx 19$ s) than in Case 1a ($t \approx 3.5$ s)

In order to investigate potential resolution-dependency of model behaviour, slope displacements and fracturing of lattice-springs for Case 1a and 1b were compared as a function of numerical time.

Figure 7 - 46 presents plots comparing the total displacement at the slope toe versus numerical time, and total number of broken lattice-springs (cracks) versus numerical time, for both cases. Arrows indicate the numerical times of $t = 3.5$ s and $t = 19$ s where the Prandtl wedge plots for Case 1a and 1b respectively were generated.
Figure 7 - 45: Case 1b at numerical time $t = 19$ s - brittle rock mass failure, indicated by failed lattice springs, allows kinematic freedom to develop (A); displacement vectors reveal the division of the slide mass into active & passive blocks, and the development of spiral shear surfaces in the transition zone (B).
Figure 7 - 46: Comparison of base case with $R = 5$ m and $R = 2.5$ m showing displacement vs. time at the slope toe, and new fractures vs. time.
The displacement versus time plot indicates similar results for both cases, suggesting that the coarser $R = 5$ m resolution is sufficient to produce the same overall slope deformation behaviour that could be modelled with a higher resolution. The displacement trends may be well suited to a bi-linear best-fit. Deformations begin with the slow development of approximately 1 m of displacement at the toe, over the course of approximately 20 seconds of numerical time.

After approximately $20 \text{ s}$ numerical time, the displacements accelerate and approach an accelerated displacement rate of approximately $0.8 \text{ m/s}$, suggesting the onset of overall slope failure.

Although the displacement histories are similar for both cases, the trends in number of fractured lattice springs versus time are very different. In the low resolution case ($R = 5$ m) the rate of fracturing appears to be constant throughout the failure, developing a total of approximately 20 000 broken lattice springs over the course of 50 s of numerical time. Any changes in trend indicating accelerated cracking or instantaneous fracturing with step-wise increases are indistinct.

In contrast, the high resolution results ($R = 2.5$ m) indicate two major step-wise increases in fracture count, occurring after the onset of catastrophic failure, at numerical times of approximately $t \approx 32 \text{ s}$ and $t \approx 43 \text{ s}$. Because the step-wise increases in fracturing occur after the transition to accelerated displacements and overall slope failure, the steps may correspond to “major break-up events” of the failed rock mass, as a preliminary step in the comminution of the failure volume during down-slope transport.

The results suggest that for the base case slope geometry and model parameters, a resolution of $R = 5$ m is sufficient to reproduce the same overall surface slope deformations that would be computed with a finer resolution. However, geometry and timing of internal fracturing within the slide mass is different for each case, indicating that finer resolution models may be necessary in order to carry out detailed analysis of brittle fracturing mechanisms occurring inside the slide mass during failure.
7.7.2.3. Jointed Transition Zone, Case 2a, $R = 5 \text{ m}$

Cases 2a and 2b use identical slope geometry, and overall failure surface geometry as Case 1a/b, with the addition of two cohesionless joint sets inside the expected transition zone (Figure 7 - 47). Case 2a uses resolution of $R = 5 \text{ m}$ and Case 2b uses a resolution of $R = 2.5 \text{ m}$. The shallow joint set has a mean dip of $15^\circ$ and includes nine non-coplanar joints with mean persistence of $30 \text{ m}$ (standard deviation = $5 \text{ m}$, with a Gaussian distribution assumed) and uniform spacing of $50 \text{ m}$. The steep joint set has a mean dip of $85^\circ$ and includes nine non-coplanar joints with equivalent spacing and persistence parameters as the shallow set.

![Diagram of jointed transition zone]

**Shallow Joint Set:**
- Mean Dip = $15^\circ$
- Standard Deviation = $2.5^\circ$
- $c = 0$; $\sigma_t = 0$
- $\varphi = 36^\circ$
- Mean Persistence: 30 m
- Standard Deviation: 5 m
- Spacing: 50 m (s.d. = 0)

**Steep Joint Set:**
- Mean Dip = $85^\circ$
- Standard Deviation = $2.5^\circ$
- $c = 0$; $\sigma_t = 0$
- $\varphi = 29^\circ$
- Mean Persistence: 30 m
- Standard Deviation: 5 m
- Spacing: 50 m (s.d. = 0)

**Figure 7 - 47:** Cases 2a and 2b incorporate two joints sets, with a total of 18 non-coplanar joint segments inserted into the expected transition zone.

Figure 7 - 48 illustrates the final computed state for Case 2a, at numerical time $t = 74s$. The slide mass has failed, and is divided into an upper active block and passive lower block along an approximately vertical interface that forms along the steep (sub-vertical joint set), with at least two major rock mass bridge failures connecting the non-persistent discontinuity tips. A crest tension crack inclined at approximately $70^\circ$ allows release of the upper block, however no transition zone is apparent; the interface between active and passive blocks is abrupt, shown by the sharp contrast in displacement contours.
Figure 7 - 48: Case 2a at numerical time $t = 74$ s - brittle failure of rock mass bridges has allowed kinematic freedom to develop, with a vertical interface between upper and lower blocks (A); displacement contours show the division of the slide mass into active & passive blocks, and no transition zone is apparent (B).
7.7.2.4. Jointed Transition Zone, Case 2b, $R = 2.5$ m

Case 2b uses the same model configuration as Case 2a, however the model resolution is increased to $R = 2.5$ m in order to investigate potential resolution-dependency of fracturing inside the expected transition zone. Figure 7 - 49 illustrates the state of fractured-lattice springs and displacement contours for Case 2b at numerical time $t = 4$ s, before kinematic freedom is fully developed.

Approximately 0.6 m of total displacement has occurred at the toe of the slope. Broken lattice springs indicate the formation of a sub-vertical ($\sim 85^\circ$) interface between the active upper block and passive lower block; however, the development of brittle rock mass failure is different from Case 2a:

- In Case 2a ($R = 5$ m), an inclined tension crack ($\sim 70^\circ$) occurs at the crest of the slope, however no crest tension crack occurs in Case 2b ($R = 2.5$ m) at numerical time $t = 4$ s
- In Case 2a the active/passive interface is vertical, comprising a composite surface formed by the steep frictional joint set, and at least two major rock mass bridge ruptures; in contrast the active/passive interface in Case 2b is steeply inclined ($\sim 85^\circ$), with two additional incipient major shear surfaces, curving sub-vertically upward from the shallow frictional joint set, into the active block
- The displacement contours for Case 2b do not yet indicate full division into active/passive blocks along a sub-vertical displacement contrast interface; instead, the contour intervals in the lower passive block are curved and sub-vertical; contours in the upper active block are curved and sub-horizontal. The jointed transition zone shows transitional displacement contours that follow the orientation of the two major joint sets.

The results indicate that the joints inside the transition zone influence the development of internal shearing and displacements inside the slide mass. Additionally, the two joint sets inside the slide mass appear to provide more initial kinematic release than Case 1 (the unjointed, intact slide mass), thus allowing shear failure along discontinuities to develop with less occurrence of brittle rock mass failure.
Figure 7 - 49: Case 2b at numerical time $t = 4$ s - brittle failure of rock mass bridges is beginning to cause kinematic freedom to develop, with a branching, sub-vertical interface between upper and lower blocks (A); the orientation of displacement contours in the transition zone shows influence from the pre-existing joints.

To investigate the final failed state of the slope, an additional plot was generated at numerical time $t = 29$ s after kinematic freedom is fully developed and catastrophic failure has occurred (Figure 7 - 50).
The displacement plot indicates that approximately 13 m of total displacement has occurred at the toe of the slope. Several changes in slope displacements and brittle fracturing of the slide mass occur between numerical time $t = 4$ s and $t = 29$ s:

- At $t = 29$ s, the displacement contours indicate a sharp division of the slide mass into a fully-developed upper active block, and a passive lower block, separated by a sub-vertical interface indicated by contrasting displacement contours.
- At $t = 4$ s, both the upper active block and passive lower block are predominantly intact, with little rock mass fracturing indicated; however, at $t = 29$ s, broken lattice-springs in the passive lower block suggests major non-systematic brittle rock mass fracture, possibly associated with the early stages of rock mass fragmentation during down-slope transport of the failed slide volume.
- No tension crack is observed to develop at the crest of the slope in Case 2b, neither at time $t = 4$ s nor later at $t = 29$ s.

The contrasts in brittle fracturing between Case 2a and 2b suggest potential resolution-dependency of model behaviour. Figure 7 - 51 presents plots of fracturing versus numerical time (i.e. broken lattice springs) and total displacements versus numerical time at the slope toe. The broken springs versus numerical time plot is annotated with arrows indicating three major step-wise fracturing events that occur at numerical times of $t = 25$ s; $t = 35$ s; and $t = 45$ s. The events are separated by intervals of approximately 10 s (numerical time), and they correlate with temporary deceleration events in the toe-displacement rate versus time plot, followed by renewed acceleration.

The toe displacement versus numerical time plot suggests a bi-linear best fit, with an initial slow displacement phase ($v \approx 0.1$ m per numerical time step) as kinematic freedom develops through brittle failure of rock mass bridges along the transition interface between active and passive blocks. Displacements are initially similar for both Case 2a and 2b. However, at numerical time $t \approx 7$ s the low resolution model (i.e. Case 2a, $R = 5$ m) accelerates to a faster displacement rate than the high resolution model. At $t \approx 15$ s the mean displacement rate for the linear best-fit function accelerates to $v \approx 0.5$ m per numerical time step, suggesting the onset of failure.
Later, at numerical time $t \approx 32 \, \text{s}$, the high resolution model (Case 2b, $R = 2.5 \, \text{m}$) accelerates and displaces faster than the lower resolution model (Case 2a, $R = 5 \, \text{m}$). The numerical time step where Case 2b accelerates to a greater rate than Case 2a occurs after the first major step-wise fracturing event ($t = 25 \, \text{s}$) but before the second major step-wise fracturing event ($t = 35 \, \text{s}$).

Figure 7 - 50: Case 2b at numerical time $t = 29 \, \text{s}$ - brittle failure of rock mass bridges has allowed kinematic freedom to develop, with a branching, sub-vertical interface between upper and lower blocks (A); the orientation of displacement contours shows a sub-vertical division between the upper active block and passive lower block.
Figure 7 - 51: Comparison of Case 2a and 2b showing displacement vs. time at the slope toe, and new fractures vs. time.

The results suggest that the coarser mesh size of $R = 5 \text{ m}$ may be sufficient to reproduce the same overall pattern of slope displacement up to the onset of overall slope failure. However, fine mesh size (high model resolution) is an important prerequisite for more realistic modelling of post-failure deformations and ongoing brittle failure mechanisms occurring inside the slide mass during failure.
7.7.3. Summary of Slope Model Observations

The results of the base case model (Case 1) with an intact slide volume show that internal shearing of the slide mass must occur in order for kinematic failure to develop for bi-planar, overall slope scale failure. If no pre-existing fractures are present, dilation occurs through brittle fractures forming (1) a Prandl Prism-like transition zone at the hinge of the overall failure surface; (2) a tension crack at the crest of the slope; and (3) inclined shear surfaces within the upper, active block.

During the early stages of overall slope failure, when total displacements at the toe of the slope are less than approximately 10 m, displacement vectors reveal the formation of log spiral shear surfaces within the transition zone. At larger displacements, the spiral shear surfaces are less apparent, and the displacement contours suggest a sub-vertical interface dividing the active upper block and passive lower block.

Cases 2a and 2b show that if the rock mass contains at least two pre-existing discontinuity sets (a shallow to moderately dipping set, and a steeply-dipping set), then the slide volume has greater kinematic freedom, and may undergo internal shearing and dilation without intense, pervasive brittle fracturing of the transition zone. Brittle failure forming the active/passive interface tends to initiate at the hinge of the overall failure surface, stepping along pre-existing discontinuities wherever possible, forming a steeply-inclined active/passive interface.

Plots of the toe displacement versus numerical time suggest that both Case 1a/b and Case 2a/b are well suited to a bi-linear best fit, with a slow initial displacement phase as kinematic freedom develops through brittle fracturing, and an accelerated second phase indicating the onset of failure. For both cases, the displacement versus numerical time trend is resolution-independent before the onset slope failure. However, the results diverge once displacement accelerates towards the failure phase. Plots of fracture formation versus numerical time derived from the high resolution (R = 2.5 m) models suggest that both Case 1b and Case 2b undergo step-wise fracturing, with instantaneous major fracturing events occurring after the onset of slope failure.
8. Conclusions

This chapter presents a synthesis of field observation from investigations at the three open pit mine study sites, and makes some preliminary recommendations for field methods and post-processing techniques to help improve characterisation of discontinuity persistence and intact rock bridges in open pit slopes. Recommendations are also provided for areas of future study that may help to improve methodologies for characterising the influence of discontinuity persistence and intact rock bridges in large open pit slopes.

8.1. Preliminary Suggested Methodology

This section presents some preliminary suggested guidelines for geotechnical assessment of open pit slopes, with specific focus on improving characterization of discontinuity persistence, rock mass bridges and intact rock bridges. The recommendations are intended for improving understanding of existing slopes, not for preliminary (pre-feasibility) design.

In open pit slope stability assessment, methods for characterising persistence and intact rock bridges should be applied only after standard investigation measures have been undertaken to identify (1) slope scale and geometry; (2) geological factors including lithology, intact rock properties, rock mass structure, hydrogeology, weathering and alteration; and (3) the most likely expected slope instability mechanisms based on the geological or geotechnical site model, and previous experience of slope performance at the specific site. Figure 8 - 1 presents a simplified, conceptualized flow chart for addressing the roles of discontinuity persistence and rock bridges.
Modified Discontinuity Persistence and Rock Bridge Mapping for Open Pit Slopes

Part 1: Standard Reconnaissance

- Slope Size
- Geology
  - Rock type, mineralogy
  - Fabric, grain-scale structure
  - Intact rock strength, stiffness
  - Rock mass structure:
    1. Massive, blocky, or foliated
    2. Major discontinuity sets
  - Hydrogeology
  - Weathering, alteration

Mechanisms
- Basic kinematics:
  - Sliding
  - Toppling
  - Wedges
  - Step-paths

Part 2: Characterizing Persistence

- Major Structures
  - Discretely map all discontinuities that cross the entire slope (Persistence ≥ Slope Size)
  - Identify most likely influence on slope failure mechanism:
    - Example: listric fault may act as a weak basal sliding surface
    - Example: bedding may form a fully-persistent sliding surface

- Secondary Structures
  - Identify top 10-20% largest discontinuities and classify into sets where appropriate
  - Measure persistence of major sets and estimate in situ block size/shape
  - Carry out kinematic assessment based on secondary structures
  - Do secondary structures create kinematic release for larger failures delineated by major structures?

- Minor Structures
  - Is there systematic structure to the least persistent discontinuities?
  - Orientation of minor structures may define most likely directions for rock mass failure

Part 3: Characterizing Rock Bridges

- Major Rock Mass Bridges
  - If major structures delineate obvious large-scale (i.e., ≥ slope height) failure mechanisms, is there evidence for fractured rock mass bridges providing lateral/rear constraint or toe buttressing?
  - If YES: Measure dimensions of rock mass bridge, carry out limit equilibrium analysis with equivalent rock mass strength parameters

- Intact Rock Bridges
  - In-plane/coplanar rock bridges
    - Is there evidence of microfabric defining a protojoint network, where joints have not all fully developed into persistent mechanical discontinuities?
  - Out-of-plane/non-coplanar rock bridges
    - Given the persistence, spacing, and intensity of secondary and minor joint sets: what is the likely range of sizes for out-of-plane intact rock bridges between adjacent discontinuity tips
    - If YES: Proto-joint shear strength retains true cohesion and tensile strength (Hencher et al. 2011).
      Laboratory strength testing parallel and perpendicular to microfabric will help to assess c and σ;
      Suggestion: Monte-Carlo analysis with DFN may help constrain 3-D intact rock bridge size and geometry; DFN-SRM numerical models may help derive equivalent continuum strength

Figure 8 - 1: Simplified flow-chart with suggested discontinuity persistence and rock bridge characterization approaches for open pit slopes.
A three-tier approach is recommended for discontinuity persistence characterization, addressing first major structures, then secondary structures, and finally minor structures. The major structures category includes all discontinuities with persistence greater than the slope dimensions. All major structures should be mapped and characterised individually and discretely included in stability assessment, as they are the most likely structures to control failure surface geometry and kinematics.

Secondary structures comprise the top 10% to 20% most persistent discontinuities; these should be individually measured and grouped into discontinuity sets where appropriate according to traditional geotechnical field methods (e.g. Wyllie and Mah, 2004). Secondary structures are likely to be an important influence on in situ block size and shape, and also may provide kinematic release (lateral or rear release) for larger failures bounded by major structures. In very high slopes (i.e. H > 500 m), the persistence of secondary structures is likely to be a major influence on bulk rock mass strength and stiffness.

Minor structures comprise the 80% to 90% majority of discontinuities within the rock mass likely to be below the cut-off length in a discontinuity survey, and thus may not be explicitly included in the field records (Dight and Baczynski, 2009). Although it is impractical to discretely measure each minor discontinuity, a sample population of at least n > 100 (Stauffer, 1966) should be gathered in order to characterise key parameters including orientation, persistence, spacing and intensity, because minor rock mass structure may be a controlling intra-block factor (Day et al., 2012) that defines the internal shear strength and deformability of unstable volumes of rock mass.

In selecting practical cut-off lengths for discontinuity persistence to include in field mapping surveys, the recommendations of Dight and Baczynski (2009) can serve as helpful guidelines:

- Moderately jointed rock masses with spacing averaging 0.5 m to 1 m could apply similar cut-off lengths for discontinuity persistence, in the range of 0.5 m to 1 m
- Massive rock masses with few fractures can use a smaller cut-off length, as short as 0.1 m if no other major features are present
• Intensely fractured rock masses with spacing in the range of 1 cm to 10 cm should apply a larger cut-off limit for discontinuity persistence, in the range of 2 m to 3 m, with the highly fractured ground in-between being regarded as rock mass.

Additional DFN studies may also be helpful in characterising the influence of the selected cut-off length on in situ block size distribution, mean trace length and intensity, and intact rock bridge content (Sturzenegger et al., 2011).

The guidelines presented here should not serve as absolute limits for persistence cut-off lengths. Instead, persistence cut-off lengths should be selected on a case-by-case basis for individual site investigations, such that (1) the influence of major discontinuities on the kinematics of slope instability is best understood, and (2) the influence of minor structures in creating rock mass strength anisotropy and defining potential failure pathways through the rock mass is captured.

In characterising intact rock bridges, an important distinction must be made between intact rock bridges and rock mass bridges. Major rock mass bridges may act as important buttresses against failure, such as with toe-breakout of inter-ramp or larger non-daylighting wedge failures (Havaej et al., 2012; Styles, 2009; Yan, 2008). In stability analysis, rock mass bridges represent fractures zones of rock mass, and thus must use equivalent rock mass properties which account for the weakening influence of discontinuities that are too small to explicitly simulate. In contrast, intact rock bridges represent unfractured intervals of rock which are limited in their size by the persistence, spacing, and intensity of pre-existing discontinuities. Absolute size limits for intact rock bridges will be smaller within a heavily jointed rock mass than within a massive rock mass.

Even a rock mass that appears massive at meso-scale may contain a fabric of microcracks that defines principal axes defining preferential fracture propagation directions (Fuji et al., 2007). In some cases major secondary joints may be part of a proto-joint network with tight aperture, indicating that joints have not fully developed into persistent mechanical discontinuities. When joint aperture is tight and traces of parallel or coplanar joints occur with terminations in intact rock, then the joint set shear strength retains true cohesive and tensile strength from intact rock bridges (Hencher et al., 2011).
However, as previously noted, co-planar intact rock bridges are not expected to occur along depositional or erosional discontinuity structures such as sedimentary bedding or unconformities.

In some cases it may be difficult to distinguish the influences of intact rock bridges and discontinuity surface roughness on shear strength. Field observations from the natural exfoliation joint surfaces at the Chief, and from natural and blast-induced fracture surfaces at all three mine sites, show that including fractography-based observations can help to improve characterisation of discontinuity roughness and frictional strength. Furthermore, fractographic characterisation may also help to identify the typical loading conditions (i.e. Mode I, Mode II, Mode III or mixed-mode) and mechanisms of brittle crack growth that can lead to kinematic release and slope failure. The results of this investigation support the suggestion of Hencher and Knipe (2007) that fractography features including hackle marks, bifurcations, arrest marks or other brittle fractures are important structures that should be recorded as part of all discontinuity surveys.

Finally, wherever forensic examination of previous slope failures is possible, a detailed survey of failure surface morphology should be undertaken in order to interpret the failure mechanism (Wolter et al., 2011). Key steps in characterising the previous failure event should include:

1. Measuring the persistence and orientation of major planar discontinuity components along the failure surface

2. Searching for evidence of shearing (i.e. slickensides or crushing of asperities) and quantifying the extent of shear displacement where possible

3. Searching for evidence of brittle fracture of intact rock bridges, and measuring the geometry and surface area comprising intact rock fracture, which may be recognized as patches fresh, unweathered rock without staining, mineralized veneer or coating (e.g. Frayssines and Hantz, 2006), or merely by the absence of frictional features, indicating brittle failure in direct tension (Lévy et al., 2010).

4. Assessing the bulk mechanical properties of the failed rock mass volume: block shapes, block size distribution, and runout distances may help to interpret
the extent of volumetric intact rock bridging within the failed rock mass, and thus better understand the pre-peak bulk shear strength and dilational behaviour of the unstable volume leading up to the failure event.

8.2. Summary of Field Investigation Sites

The Chief, Jwaneng, Diavik, and Highland Valley are situated in different geological provinces with widely varying lithology, tectonic history, and rock mass structure. The dominant mechanisms of slope instability are different at each site, as are the geological and hydrogeological factors that most strongly influence potential instability:

1. The Chief is a massive granite monolith that has been uplifted, exhumed and scoured by glacial action. The primary mechanism of slope instability on the Grand Wall is characterised by failure of tabular slabs along persistent exfoliation joints oriented parallel to the sub-vertical slope face, with release occurring on brittle fracture step surfaces. Recent post-glacial slab failures are mostly small, comprising tabular slab failures in the order of 10 m in length. Major arch structures with width up to 6 m are recessed into the wall, suggesting that glacial scour may have contributed to larger failures in the past, in the order of 10 000 m$^3$ to 100 000 m$^3$.

2. The East Wall at Jwaneng mine is dominantly influenced by layering in the metasedimentary rocks that dip out of the wall, creating potential for planar sliding failure; ongoing slope instabilities are commonly characterised by pentahedral wedges with basal sliding on foliation and release on a combination of pre-existing joints and blast-induced damage.

3. The A418 pit at Diavik is influenced by strong intact rock properties and the common occurrence of high or extremely persistent discontinuities associated with kilometre-scale faults that intersect the pit. Non-persistent bench and inter-ramp scale wedges occur (i.e. 10 m – 100 m+), and the constant head boundary behind the pit crest, Lac de Gras, recharges a complex, compartmentalized fracture-flow groundwater regime.

4. The Upper West Wall of the Valley pit at Highland Valley mine is comprised of highly fractured and altered granodiorite; observed slope instability mechanisms.
are characterised by progressive bench-scale block failures including complex wedges, planar sliding and block toppling failures, with kinematic release provided by blast-induced damage and linkage between very low to low persistence discontinuities (i.e. <1 m to 3 m), with some intact rock failure.

- Sliding commonly initiates on sheared discontinuities that have been influenced by past faulting episodes, and are commonly coated with phyllic or argillic alteration products from hydrothermal fluid infiltration.

The major differences in tectonic history, intact rock and discontinuity properties and hydrogeological conditions suggest that site-specific adaptations are required in characterising the role of discontinuity persistence and intact rock bridges at each site. Table 8 - 1 presents a summary of key rock mass characteristics at each mine including:

- Lithology
- Rock mass structure
- Tectonic history
- Alteration
- Fracture network characteristics
- Observed or expected slope failure mechanisms.
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<td><strong>The Chief</strong></td>
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<tr>
<td>Granite</td>
<td>Massive, with exfoliation joints</td>
<td>Creteaceous intrusion, part of Howe Sound Batholith; major exhumation by glacial erosion during Tertiary; buried by 1.3 km height of ice at last glacial maximum (Turner et al., 2010). Rock mass is not highly damaged, GSI &gt; 75; D = 0</td>
<td>Exfoliation joints aligned parallel to the cliff face, with spacing apparently increasing from &lt; 0.2 m near surface, to &gt; 6 m, indicated by step surfaces and major arch structures; kinematic release of slabs from brittle fractures orthogonal to exfoliation, Jn = 2 corresponding to one major joint set, after Barton et al. (1974).</td>
<td>Seepage paths visible along persistent exfoliation joints, indicated by dark stained patches on the cliff face and extremely persistent (&gt;100m) fracture traces with trees rooted into the joints.</td>
<td>Gravity-driven rockfall of tabular slabs formed by exfoliation joints, with release along a lateral or upper confining surface of intact rock.</td>
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<td>Sub-vertical basalt dyke~ 5 m wide transects slope</td>
<td>Field assessment indicates GSI &gt; 75</td>
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<tr>
<td><strong>Jwaneng: East Wall</strong></td>
<td>Layered with distinct bedding, dipping toward NW, out of east wall</td>
<td>Four episodes of deformation have locally tilted bedding (Tunono et al., 2011); field assessment indicates local GSI = 40 to 60 assessed for bench-scale windows</td>
<td>Persistent moderately dipping, gently undulating bedding; spacing is variable; observed step-path geometries indicate spacing can vary from 10 cm to approximately 6 m. Sparse steeply-dipping inter-ramp size (&gt;100m) faults and fault-associated joints; faults are spaced wider than 100 m; Local Jn = 9-12 corresponding to three joint sets +/- random joints, after Barton et al. (1974).</td>
<td>Deep groundwater table with fracture-controlled flow; compartmentalized aquifers disconnected from regional groundwater regime by steeply-dipping faults (Tunono et al., 2011)</td>
<td>No seepage observed in the slope.</td>
</tr>
<tr>
<td>Metasedimentary sandstones and shale sequence</td>
<td>Blocky to massive, fair to good quality rock mass (Tunono et al., 2011);</td>
<td>Structural domains are bounded by major faults; Kinematic release for progressive bench scale sliding failures is provided by blast-induced fractures; suggested disturbance factor D = 1</td>
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<tr>
<td>Kimberlite pipes (ore)</td>
<td>Layered with distinct bedding, dipping toward NW, out of east wall</td>
<td>Four episodes of deformation have locally tilted bedding (Tunono et al., 2011); field assessment indicates local GSI = 40 to 60 assessed for bench-scale windows</td>
<td>Persistent moderately dipping, gently undulating bedding; spacing is variable; observed step-path geometries indicate spacing can vary from 10 cm to approximately 6 m. Sparse steeply-dipping inter-ramp size (&gt;100m) faults and fault-associated joints; faults are spaced wider than 100 m; Local Jn = 9-12 corresponding to three joint sets +/- random joints, after Barton et al. (1974).</td>
<td>Deep groundwater table with fracture-controlled flow; compartmentalized aquifers disconnected from regional groundwater regime by steeply-dipping faults (Tunono et al., 2011)</td>
<td>No seepage observed in the slope.</td>
</tr>
<tr>
<td><strong>Diavik: A418 pit</strong></td>
<td>Blocky to massive rock mass, no distinct foliation, Local GSI = 70 to 90 assessed for bench-scale windows</td>
<td>Strong, moderately fractured, good to excellent quality rock mass (Moffitt et al., 2007)</td>
<td>Many high persistence joints (10 – 20 m), few inter-ramp scale joints (&gt; 100 m), including sub horizontal, undulating joints with groundwater flow; steeply dipping joints parallel to A418 fault. DFN studies have been undertaken by Moffitt et al. (2007) and Elmo et al. (2011).</td>
<td>Heterogeneous, fracture-controlled flow from constant head source (Lac de Gras) produces a compartmentalized pore pressure distribution controlled largely by sparse, widely spaced major discontinuities with enhanced permeability such as faults and blocky diabase dykes. Excess pore pressures up to + 40 m above collar height occur, but may be located near completely dry boreholes (Chorley et al., 2009)</td>
<td>Observed bench-scale wedge instabilities and planar sliding with release on blast-induced brittle fractures; potential for multi-bench scale wedges on extremely persistent (100 m+) joints, A418 fault, diabase dyke contacts</td>
</tr>
<tr>
<td>Granite and metasedimentary turbidites</td>
<td>Multiple episodes of deformation and intrusion by new granite, from Archean age onward (Rosoe and Postle, 2005)</td>
<td>Many high persistence joints (10 – 20 m), few inter-ramp scale joints (&gt; 100 m), including sub horizontal, undulating joints with groundwater flow; steeply dipping joints parallel to A418 fault. DFN studies have been undertaken by Moffitt et al. (2007) and Elmo et al. (2011).</td>
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<tr>
<td>Diabase and pegmatite dykes</td>
<td>Steeply-dipping faults intersect both open pits (A154 and A418) along NE-SW axis</td>
<td>Many high persistence joints (10 – 20 m), few inter-ramp scale joints (&gt; 100 m), including sub horizontal, undulating joints with groundwater flow; steeply dipping joints parallel to A418 fault. DFN studies have been undertaken by Moffitt et al. (2007) and Elmo et al. (2011).</td>
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<tr>
<td>Kimberlite pipes (ore)</td>
<td>Bench faces mostly vertical, minor backbreak, variable D = 0.5 to 1</td>
<td>Many high persistence joints (10 – 20 m), few inter-ramp scale joints (&gt; 100 m), including sub horizontal, undulating joints with groundwater flow; steeply dipping joints parallel to A418 fault. DFN studies have been undertaken by Moffitt et al. (2007) and Elmo et al. (2011).</td>
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<tr>
<td><strong>A418 pit</strong></td>
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<tr>
<td>Highland Valley:</td>
<td>Granodiorite with quartz stockwork</td>
<td>Blocky to very blocky, fair to good quality rock mass with RQD from 40% to 80% (Piteau Associates / Rose and So, 2010)</td>
<td>Multiple episodes of regional faulting produced a stockwork that was mineralized by repeated hydrothermal fluid infiltration from Mesozoic to Tertiary; mineralized discontinuities are associated with low frictional strength phyllic and argillic alteration products including kaolinite, sericite (Casselman et al., 1995).</td>
<td>Hydrothermally altered joints are altered with kaolinite, chlorite, sericite (i.e. $J_a = 4$ to 8, after Barton et al., 1974); $J_a = 4$ → softening or low friction clay mineral coatings; also chlorite, talc, gypsum, graphite, and small quantities of swelling clays.</td>
<td>Minor seepage visible near the Yellow Fault: wet wall rocks and other high-persistence joints in Design Sector 2</td>
</tr>
<tr>
<td>Valley Pit Upper West Wall</td>
<td>Published RMR values for nearby Lomex Pit vary from approximately 30 to 70, with lesser quality occurring near major fault zones such as the Lomex fault and Valley Fault (Newcomen et al., 2003; Tosney et al., 2004).</td>
<td>Locally blocky/disturbed to disintegrated proximal to Yellow Fault. Evidence of heavy production blasting, suggested $D = 1$.</td>
<td>Probable influence of cyclic loading by glaciers during Quaternary may contribute to brittle fracture development (e.g. Leith, 2012).</td>
<td>$J_a = 8$ → Medium or low over-consolidation, softening, clay mineral fillings (continuous, but &lt; 5 mm thickness).</td>
<td>Minor flow (&lt; 1 L/min) observed during current field investigation, from horizontal drains at foot of slope in Design Sector 3, on RL = 1385 m</td>
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<td>Fracture network characteristics are complex, overprinted fracture network. Modern rock mass is heavily fractured (i.e. $J_a = 15$ corresponding to four or more joint sets, random, highly jointed, “sugar-cube” structure, after Barton et al., 1974); blocks are typically irregular polygons, tending towards cubic shape after Kalenchuk et al. (2006), 10 cm to &lt; 1 m edge lengths.</td>
<td>Medium or low over-consolidation, softening, clay mineral fillings (continuous, but &lt; 5 mm thickness).</td>
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<td></td>
<td>Evidence of heavy production blasting, suggested $D = 1$.</td>
<td>Older, more persistent features associated with uplift and tectonic deformation are overprinted by more recent damage from fault re-activation, hydraulic fracturing by hydrothermal fluid intrusion, and blast induced damage.</td>
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Bench-scale block toppling, planar sliding and wedge sliding are common; discrete failure event volumes are small ($10 – 100 m^3$) comprised of individual blocks in the order of $1 – 10 m^3$; preliminary combinations analysis with Swedge suggests idealized critical wedge volumes of approximately $65 m^3$ in dry conditions, with potential for larger wedge volumes with assumed fully saturated fractures. Progressive backbreak / ongoing failure has resulted in debris accumulation in the order of $100 000 m^3$ across the benches of the Upper West Wall.

Largest accumulation of debris occurs in Design Sector 3, associating with increased risk of rockfall due to loss of bench width for catchment, and increased volume of potential rockfall source (i.e. loose failed rock).
8.3. Summary of Trace Mapping for Areal Intensity

Window maps at Diavik and Jwaneng were constructed at two different scales: the windows at Jwaneng are limited to 6 m long and 2 m high, whereas the windows at Diavik are larger, up to approximately 20 m square (i.e. $L = 20$ m, $H = 20$ m). Comparing the measured trace intensities of intact rock bridges, discontinuities, and blast-induced damage from the two sites reveals two major influences on trace mapping:

1. **External** influences (sampling bias) reflect uncertainty and error arising from factors that are not related to intrinsic properties of the rock mass; the most dominant external errors in trace mapping relate to the influences of:
   a. Observation scale
   b. Sampling bias by the mapping practitioner in preferentially identifying larger discontinuities (length bias)
   c. Orientation bias relating to under-sampling of window-parallel discontinuities
   d. Truncation of discontinuities below an artificially-imposed cut-off length
   e. F-bias which proposes that trace length may not correlate with the maximum diameter of an ellipsoidal discontinuity

2. **Natural** influences (rock mass quality) relate to actual inherent rock mass characteristics including rock mass quality (i.e. block size and shape, weathering and discontinuity surface conditions), fracture intensity, and degree of blasting-induced damage. Although blasting damage is induced by mining, it alters the actual extant state of the rock mass quality.

A summary of the field window mapping results is presented in Table 8 - 2.
Table 8-2: Summary of window mapping results including window size, rock mass quality, and areal intensity of pre-existing discontinuities, inferred rock bridges ($R_{21}$) and blast-induced damage ($B_{21}$).

<table>
<thead>
<tr>
<th>Window.</th>
<th>Surface Area ($m^2$)</th>
<th>Approximate GSI Range</th>
<th>(Pre-existing) Fracture Intensity $P_{21}$ ($m^{-1}$)</th>
<th>Method 1: including blast-induced fractures</th>
<th>Method 2: excluding blast-induced fractures</th>
<th>Intact Rock Bridge Trace Intensity $R_{21}$ ($m^{-1}$)</th>
<th>Method 1: bridges traced only between similar discontinuity sets</th>
<th>Method 2: bridges traced between any nearest fractures to dissect window into complete blocks</th>
<th>Blast-induced Fracture Intensity $B_{21}$ ($m^{-1}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jwaneng</td>
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<tr>
<td>01</td>
<td>5.7</td>
<td>65 – 75</td>
<td>9.9</td>
<td>5.3</td>
<td>1.9</td>
<td>2.9</td>
<td>4.6</td>
<td>9.9</td>
<td>4.6</td>
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<tr>
<td>02</td>
<td>5.1</td>
<td>55 – 65</td>
<td>14.9</td>
<td>5.0</td>
<td>3.2</td>
<td>3.9</td>
<td>9.9</td>
<td>3.9</td>
<td>9.9</td>
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<tr>
<td>03</td>
<td>9.9</td>
<td>60 – 70</td>
<td>8.1</td>
<td>4.4</td>
<td>2.8</td>
<td>2.8</td>
<td>3.7</td>
<td>2.8</td>
<td>3.7</td>
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<td>04</td>
<td>7.2</td>
<td>50 – 60</td>
<td>15.9</td>
<td>8.1</td>
<td>5.9</td>
<td>5.3</td>
<td>7.8</td>
<td>5.3</td>
<td>7.8</td>
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<tr>
<td>05</td>
<td>8.6</td>
<td>45 – 55</td>
<td>11.0</td>
<td>6.0</td>
<td>3.9</td>
<td>4.5</td>
<td>5.0</td>
<td>4.5</td>
<td>5.0</td>
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<td>06</td>
<td>12.2</td>
<td>55 – 65</td>
<td>9.9</td>
<td>2.8</td>
<td>1.3</td>
<td>3.4</td>
<td>7.1</td>
<td>3.4</td>
<td>7.1</td>
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<td>Diavik</td>
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<tr>
<td>A</td>
<td>205</td>
<td>80 – 90</td>
<td>2.1</td>
<td>0.6</td>
<td>0.9</td>
<td>0.6</td>
<td>1.5</td>
<td>0.6</td>
<td>1.5</td>
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<tr>
<td>B</td>
<td>289</td>
<td>75 – 85</td>
<td>1.1</td>
<td>0.2</td>
<td>0.1</td>
<td>0.4</td>
<td>0.9</td>
<td>0.4</td>
<td>0.9</td>
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<tr>
<td>C</td>
<td>163</td>
<td>70 – 80</td>
<td>1.4</td>
<td>0.2</td>
<td>0.3</td>
<td>0.2</td>
<td>1.2</td>
<td>0.2</td>
<td>1.2</td>
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<tr>
<td>D</td>
<td>334</td>
<td>70 – 80</td>
<td>1.2</td>
<td>0.3</td>
<td>0.3</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
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<tr>
<td>E</td>
<td>355</td>
<td>65 – 75</td>
<td>1.7</td>
<td>0.4</td>
<td>0.4</td>
<td>0.6</td>
<td>1.3</td>
<td>0.6</td>
<td>1.3</td>
</tr>
<tr>
<td>F</td>
<td>117</td>
<td>85 – 95</td>
<td>0.2</td>
<td>0.1</td>
<td>-</td>
<td>0.2</td>
<td>0.1</td>
<td>0.2</td>
<td>0.1</td>
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<tr>
<td>G</td>
<td>293</td>
<td>70 – 80</td>
<td>0.7</td>
<td>0.2</td>
<td>0.1</td>
<td>0.2</td>
<td>0.5</td>
<td>0.2</td>
<td>0.5</td>
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</table>
Although the areal intensity of pre-existing discontinuities ($P_{21}$), blast-induced traces ($B_{21}$) and rock bridge traces ($R_{21}$) all tend to decrease for higher GSI values, there is a wide scatter in the results. When the data from Diavik and Jwaneng are plotted together, it is difficult to determine any strong trend (Figure 8-2). The largest scattering is observed for the $B_{21}$ results, suggesting that intensity of blast-induced damage is probably highly dependent on mining-related factors (i.e. use of controlled blasting techniques such as pre-splitting) in addition to inherent properties of the rock mass.

![Figure 8-2: Areal Intensity of rock bridge traces ($R_{21}$); blast-induced damage ($B_{21}$); and pre-existing discontinuities ($P_{21}$) vs rock mass quality for combined results from Diavik and Jwaneng.](image)

Upper-bound and lower-bound limits are plotted in order to illustrate the range of intensity results that may occur for a given GSI value. In general, the variation in intensity measurements becomes larger at small GSI values. However, the results cannot prove a causal relationship. The high-GSI data points (i.e. GSI $\geq 70$) occur at Diavik, and the lower-GSI data points (GSI $\leq 70$) occur at Jwaneng. Major differences in intensity measurements between the two sites are thus likely dependent on site-specific factors including geological history, window map size (Jwaneng was mapped with small windows; Diavik was mapped with larger windows), and also local engineering practices.
such as blasting methods. The results indicate that measurements from each mine should be compared separately, in order to better address site-specific conditions.

Figure 8 - 3 shows the data from Diavik and Jwaneng plotted separately. Intensity measurements at Diavik range from 0 to 2 m$^{-1}$, and intensity measurements from Jwaneng range from 0 to 16 m$^{-1}$. For both sites the largest intensities relate fracture intensity $P_{21}$ measured with Method 1, which includes blasting-induced fractures.

Figure 8 - 3: Summarised trace mapping intensity measurements from digital window maps for Diavik and Jwaneng.
Figure 8 - 4 presents the mean $P_{21}$ intensity values for Jwaneng and Diavik, measured as the average of Method 1 and Method 2 for each window. The $P_{21}$ values from Diavik are all clustered below values of 2 m$^{-1}$ whereas the $P_{21}$ measurements from Jwaneng range from 6 m$^{-1}$ to 12 m$^{-1}$.

Fracture intensity $P_{21}$ was not measured in the Valley pit Upper West Wall, due to the field assessment of the rock mass as highly damaged and fractured ($J_n = 15$ corresponding to four or more joint sets, random, heavily jointed, “sugar-cube” structure, after Barton et al.1974). However, a suggested $P_{21}$ domain box is included for the Upper West Wall at Highland Valley, indicating an expected increase in $P_{21}$ associated with lower rock mass quality in the range of GSI = 20 to 50. The plot also includes qualitative descriptors of expected parameter uncertainty and variability of $P_{21}$ for each site, where parameter uncertainty describes the inherent uncertainty in determining accurate values that reflect true $P_{21}$ in situ. Diavik has the highest GSI, the tightest clustering of results, and the lowest expected standard deviation and parameter uncertainty; Jwaneng has intermediate GSI values, and a larger expected standard deviation and parameter uncertainty; Highland Valley includes the lowest-GSI rock mass, and the highest expected standard deviation and parameter.
Highly damaged rock mass in the range of GSI ≈ 20 is likely to apply proximal to the Yellow Fault, and in highly blast-damaged areas where extensive bench-scale failures have caused major debris accumulations. Precise limits for $P_{21}$ values at Highland Valley would depend on the cut-off limit of fractures included in the calculation. Inclusion of centimetre-scale brittle fractures that occur throughout the rock mass would result in exponentially increased $P_{21}$ values.

Figure 8 - 5 shows a plot of average fracture intensity ($P_{21}$ for pre-existing discontinuities) versus average rock bridge trace intensity ($R_{21}$). The averaged values represent the mean intensity values from the two different methods for calculating $P_{21}$ and $R_{21}$ respectively, reported in Table 8 - 2.

**Figure 8 - 5: Average of $P_{21}$ values denoting intensity of pre-existing discontinuities versus corresponding $R_{21}$ values denoting intensity of intact rock bridge traces.**

The $R_{21}$ and $P_{21}$ values from Diavik are clustered around values in the range of 0 to 1 m$^{-1}$ whereas the average intensity values from Jwaneng are spread across a greater range: $P_{21}$ values range from 6 m$^{-1}$ to 12 m$^{-1}$ and $R_{21}$ values range from 2 m$^{-1}$ to 5.5 m$^{-1}$. The lower $R_{21}$ value at Diavik may be partially explained because the rock mass at Diavik is more massive than that of Jwaneng, and thus rock bridges at Diavik were preferentially
traced between major joints with medium persistence (3 m to 10 m; ISRM, 1978) or
greater, and smaller features were less likely to be included.

In contrast, the rock mass at Jwaneng is more damaged than Diavik, and rock bridges
were thus traced between more very low to low persistence discontinuities (≤3 m; ISRM,
1978). There is a larger absolute number of rock bridge traces for each window map at
Jwaneng dataset; however the typical dimensions of an individual rock bridge are larger
for Diavik trace maps.

When combined, the results suggest a tentative trend of increasing rock bridge intensity
$R_{21}$ with increase in fracture intensity $P_{21}$. However, the error in the suggested best-fit
trend line becomes large for higher intensity values, as indicated by the poor fit with the
data from Jwaneng. Further trace mapping investigations into a wider range of rock
masses with varying lithology and degree of fracturing is needed to better understand
potential relationships between fracture intensity $P_{21}$ and intact rock bridge intensity $R_{21}$.
Additional study of the influence of intermediate window sizes within the same rock mass
would also help to better resolve the dependency of trace mapping results on external
factors such as window dimensions (Sturzenegger et al., 2011).

Figure 8 - 6 presents a plot of mean $P_{21}$, mean $R_{21}$, and blast-damage intensity $B_{21}$
versus window surface area. The results indicate a strong influence of window size on
trace intensity measurements. The results from Diavik and Jwaneng data are separated
into two distinct groups:

- At Jwaneng windows were limited in size to approximately 6 m x 2 m (i.e. surface
area in the order of 10 m$^2$) with photographs taken from a distance of
approximately 10 m. The digital trace maps applied a low cut-off length of
approximately 5 cm; the resulting intensity measurements range from 2 to 12 m$^{-1}$
- At Diavik larger windows were mapped, up to approximately 20 m x 20 m
(maximum surface area = 355 m$^2$), with photographs taken from a distance of
approximately 20 m to 30 m. Due to the more massive rock mass at Diavik, and
the dominance of medium persistence or larger discontinuities (i.e. ≥ 3 m) the
Diavik trace maps used a larger cut-off limit of 0.5 m; an order of magnitude
larger than the cut-off limit for Jwaneng. All intensity values from Diavik are less
than 2 m$^{-1}$. Additionally, rock mass at Diavik was generally higher quality (GSI $\geq$ 70, less blast-induced damage) than the rock mass at Jwaneng (GSI $\leq$ 70).

The results suggest that both external and natural factors are important considerations that influence areal intensity measurements derived from trace mapping. Sturzenegger et al. (2011) investigated sources of bias and uncertainty in estimating mean discontinuity trace length, intensity and in situ block size using field mapping, remote sensing techniques, and DFN simulation based on a granite rock outcrop near Vancouver, BC. Their comparison of mapping results showed that intensity measurements, mean trace length, and block size estimates may be influenced by survey resolution and cut-off length for mapping. They recommend that remote sensing surveys should be optimized to incorporate higher resolution 3-D terrain models or point clouds in order to allow mapping of a sufficiently large sample population of discontinuity measurements. With the use of small remote sensing windows (in the order of 10 m x 10 m, as in the current investigations), intensity and trace length measurements will be sensitive to the cut-off length used in mapping.

**Figure 8 - 6: Areal Intensity of rock bridge traces and blast-induced damage vs surface area of mapping windows.**
8.4. Summary of Remote Sensing For Characterising Persistence and Intact Rock Bridges

This investigation has shown that photogrammetry and LiDAR surveys provide useful 3-D visualization of variations in discontinuity orientation and persistence, and also detailed geometry of 3-D release structures on bench-scale failures. Where conventional scanline and window maps are limited by physically accessibility constraints, remote sensing allows for detailed mapping of important structural regions wherever they occur on the slope.

Figure 8 - 7 illustrates an example of a bench-scale failure from Diavik. Conventional mapping techniques may simplify the failure as a planar sliding mechanism, or as a simplified wedge with one or two release surfaces. However, upon closer inspection using $f = 200$ mm photogrammetry (taken from distance $\approx 400$ m), more precise detail of the release surfaces becomes apparent. The highlighted lateral constraint shows that release on the right side of the wedge is provided by at least four major planar tension fractures, connected by an irregular zone of intact rock fracture.

The planar tension fractures show fractographic features including hackle marks and plumes. The formation of the complex lateral release surface was likely induced by blasting damage, exploiting pre-existing microfabric produced by previous tectonic influence and potentially quaternary glaciation. Although the failure was certainly influenced by blasting, the time of failure is unknown. The final release may have occurred after blasting, due to destruction of intact rock bridges, extension of pre-existing discontinuities, and gravity-driven crack growth. The figure illustrates that even ostensibly simple bench-scale planar sliding failures may involve complex lateral release mechanisms where interaction occurs between low-persistence discontinuities or a proto-joint network, blasting-induced damage, and brittle failure of intact rock by gravity-induced stresses.
Figure 8-7: Example of complex 3-D structural control, with blast-induced brittle fracture on a bench-scale sliding failure in the Diavik A418 pit, extracted from $f = 200$ mm photogrammetry model (RL + 390 – 420 m).
8.4.1. Development of 3-D Window Mapping Methods

Window mapping methods using 3-D remote sensing data require more optimisation, in order to provide reliable and standardized data collection methods for discontinuity persistence and intact rock bridge content. Compared with traditional 2-D window mapping methods, the three-dimensionality of remote sensing models represents an opportunity for more realistic characterisation of rock mass structural parameters including relief, in situ block size and shape distributions, discontinuity cross-cutting relationships, termination style, and 3-D intact rock bridge geometry.

Figure 8 - 8 presents an example application of a 3-D window map at the RL + 870 m level of the Jwaneng East wall, near the field window mapping study area. Locally the rock mass is characterised by three main discontinuity sets: the metasedimentary foliation, and two sets of release joints from sets R2 and R3. The 3-D terrain model allows for easy translation and rotation to reorient the viewer. Two alternate views are shown, looking head-on towards the major release joint sets R2 and R3 (i.e. view direction is perpendicular to strike). The surface of a major R3 joint shows evidence of extensive secondary brittle cracking induced by blasting, with incipient step-paths that under gravity loading may eventually result in the failure of cubic blocks with volumes in the order of 1 m$^3$. The rotated view of set R2 provides a useful preliminary assessment of local spacing of the set (approximately 1 m to 5 m). Close inspection of the 3-D window also reveals that although some major R2 joints are very persistent (i.e. > 20 m) and transect the window, many smaller R2 joints appear to terminate within intact rock.

Although this simplified example has not been used to specifically assign a characteristic value for intact rock bridge content, it illustrates the immediate utility of 3-D window maps, and the need for future development of 3-D mapping techniques. In this example, the true spacing of R2, and the occurrences of terminations in intact rock may be difficult to interpret with 2-D window methods. Additionally, the ability to pan and rotate the mapping view allows for undistorted measurement of trace lengths and discontinuity surface areas. As suggested by Sturzenegger et al. (2011), further development of high-resolution 3-D imaging is recommended in order to improve identification of low-persistence joints, brittle cracks and step-path features at outcrop scale.
Figure 8 - 8: Example application of a 3-D window for bench face mapping.
8.5. Role of Persistence in a Fractured Rock Slope

The current investigation has shown that jointed rock masses may contain multiple generations of discontinuities, each generation characterised by very different persistence. At Highland Valley, for example, the modern fracture network was interpreted as a product of at least 6 major damage-forming episodes, given the suggested nomenclature D1 to D6, spanning from the Mesozoic emplacement of the Guichon Creek batholith, up to modern mining-induced damage. To better understand the role of persistence in jointed rock slopes, geotechnical engineers and geologists must characterise the genesis of rare, outlier structures with the highest persistence (i.e. the top 1% of discontinuity sample populations), in order to understand their potential influence in forming critical slope failure surfaces.

Stability assessments should also aim to characterise the interactions between major outlier structures that may dominate failure kinematics, and the low persistence discontinuities that are exponentially more frequent. Figure 8 - 9 presents a conceptual example of a potential slope instability influenced by the persistence of two major structures: (1) a major discontinuity forming a non-daylighting failure surface, and (2) a low persistence Voronoi joint network forming a stockwork structure. The major discontinuity terminates within the rock mass towards the toe of the slope, and toe breakout through a rock mass bridge is required in order for slope failure to occur.

![Figure 8 - 9: Conceptual influence of rock mass structure on propagation of persistent structural controls.](image)

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Three cases are shown, each with the same geometry for the major discontinuity, but different Voronoi polygon sizes. In Case A the rock mass is massive and the Voronoi polygons are large. In Case B and C, the size of the Voronoi polygons is decreased, depicting a transition to blocky and highly disturbed rock mass. Although the persistence of the major failure surface structure is the same for all cases, the resulting slope behaviour will differ according to the structure of the rock mass bridge at the slope toe.

In the change from massive structure in (A) to highly disturbed/damaged in (C), the rock mass will transition towards weaker behaviour, that may be better characterised by continuum mechanics than discontinuum mechanics. Decreasing block size corresponds with an increase in the number of blocks and the total contact length between them. Consequently, kinematic freedom increases from Case A to C, and the overall rock mass strength transitions from a cohesion-dominated condition in Case A (where rock mass shear strength is dominated by the intra-block cohesive strength and tensile strength), to a friction-dominated condition in Case C (where rock mass shear strength is dominated by Voronoi contact shear strength, which results from their micro-parameters including tensile strength, cohesion and friction; Gao, 2013; Kazerani and Zhao, 2010).

Whereas toe breakout in Case A requires rupture to occur through large, discrete intact rock bridges (i.e. intra-block failure), breakout in Case C may be accommodated more by linkage between interconnected low-persistence discontinuities, with a smaller expected component of intact rock bridge failure. From Case A to C the size and the absolute quantity (cumulative length % along the failure surface) of intact rock bridges decreases.

As rock masses become more fractured, discontinuity propagation is increasingly influenced by localized stress concentrations around existing fractures. Consequently, discontinuity propagation in highly fractured rock masses tends to occur in short segments, with frequent deflection and arrest of propagating cracks on pre-existing discontinuities (Gudmundsson et al., 2010). Complex slope failure surfaces in heavily fractured rock masses are thus expected to be dominated by linkage of pre-existing fractures, and reduced failure of intact rock bridges.
Studies into fault zone damage evolution confirm this assumption, by showing that discontinuities will propagate more easily from stiff zones of rock mass surrounding a fault zone, into softer zones within the fault core (Gudmundsson et al., 2010; Shipton and Cowie, 2003). Figure 8 - 10 conceptually illustrates the gradational reduction in rock mass stiffness that occurs towards the core of a fault zone. The result is a feedback-loop effect where fractures will preferentially propagate into the softer damage zone, but not outward into the host rock.

**Figure 8 - 10:** Decrease in stiffness towards the fault core causes fractures to propagate more easily from the stiff host rock into the core, than vice versa (Reproduced and modified after Gudmundson, 2011).

Field investigation for open pit slope stability assessment should thus aim to identify all length scales of discontinuities that may interact to influence the propagation of potential sliding or release surfaces. Field mapping must include discrete treatment of major structures, such as the dominant failure surface in Figure 8 - 9, and at least qualitative assessment of minor block-forming “intrablock” structures (Day et al., 2012), depicted in Figure 8 - 9 by the Voronoi polygons. Dight and Baczynski (2009) recommend that for intensely jointed rock masses where average spacing is between approximately 1 cm and 10 cm, only medium persistence or larger structures should be mapped, with a cut-
off length of 2 m or 3 m, with the intervening bridges between major features being assigned a scaled rock mass strength according to overall fracture intensity, rock quality and discontinuity surface conditions.

8.6. Blast Damage Intensity Factor $B_{21}$

A blast damage intensity parameter $B_{21}$ was introduced, that considers the total length of blast-induced fractures observed in a bench face window map, divided by the surface area of the window. Additional field-based studies are required to improve methods for identifying blast-induced damage, and differentiating between the role of blasting in (1) causing destruction of joint cohesion (rock bridges), extension, and dilation of pre-existing discontinuities, (2) creating new damage in the form of non-systematic meso-scale brittle fractures in intact rock, and (3) degrading the strength of intact rock by creating a dense micro-fabric of blast-induced cracks resulting from strain-wave induced damage (Dehghan Banadaki, 2010).

Conceptual numerical models of slopes with heights of 500 m and 1000 m, constructed using Phase², UDEC and Slope Model illustrated two approaches to incorporating rock bridges into slope models:

1. **Failure-surface specific**: intact rock bridges or rock mass bridges occur as intervals along a partially-formed critical failure surface; the influence of the rock bridges is most expected to yield an increased shear resistance along the critical surface; this method originates with the weighted shear strength, limit equilibrium method of Jennings (1970)

2. **Volumetric or dispersed**: intact rock bridges or rock mass bridges occur as non-coplanar intervals dispersed throughout a volume of rock mass; the influence of the rock bridges is expected to yield a greater peak strength and stiffness for the entire volume of rock mass

Numerical models of an overall slope failure mechanism requiring rock mass dilation showed that volumetric or dispersed rock bridges inside a pre-defined failure volume can have a significant impact on slope behaviour. Distinct element modelling with UDEC and lattice-spring modelling with Slope Model showed that rock mass bridge content inside
the slide mass influences the formation of the internal shear surfaces that accommodate
dilation. Given equal conditions for a pre-existing critical failure surfaces, the contact of
intact rock bridges or rock mass bridges inside the slide volume can be a very important
determinant of stability. Therefore, stability assessments for large open pits should
consider the potential for stress-induced fracturing of intact rock bridge or rock mass
bridges, contributing to the formation of complex failure surfaces or the development of
kinematic freedom through internal shearing and dilation.

8.7. Length Scale for Intact Rock Bridges

Rock bridges can only be meaningfully characterised with reference to the pre-existing
discontinuities that they interrupt. Field characterization methods should focus on
identification of discrete rock bridges which relate to specific instances of potential slope
instability, such as evidence of non-daylighting wedge structures, or potential sliding
failures that may be laterally constrained by intact rock (e.g. Hungr and Amann, 2011).
Field descriptions should always include explicit details of rock bridge size, geometry,
location, and composition, especially whether the rock bridge comprises intact,
unfractured rock or instead represents a fractured rock mass bridge.

Discontinuity persistence values from photogrammetry and LiDAR at all three mine sites
showed distributions that appear to conform to negative exponential functions. Each
sample of fitted discontinuities showed different peak values, influenced by the survey
resolution, window size and sampling bias:

At Jwaneng, peak frequencies for discontinuity persistence occurred for values of 15
m for the long-range survey \((f = 100 \text{ mm}, \text{ distance } = 1 \text{ km, ground-point spacing} \approx 6.4 \text{ cm})\), and 2 m for the close-range survey \((f = 100 \text{ mm}, \text{ distance } = 50 \text{ m, ground-point spacing } \approx 3.2 \text{ mm})\);
At Diavik, peak frequencies for discontinuity persistence occurred for values of
approximately 8 m to 9 m for the \(f = 100 \text{ mm} \) survey (ground-point spacing \( \approx 3.2 \text{ cm} \)); 4 m to 5 m for the \(f = 200 \text{ mm} \) survey (ground-point spacing \( \approx 1.6 \text{ cm} \)); and
3 m to 4 m for the \(f = 300 \text{ mm} \) survey (ground-point spacing \( \approx 1.1 \text{ cm} \)).
At Highland Valley, the lower-resolution LiDAR survey (spot spacing = 15; resolution = 5 cm to 20 cm) showed a peak in discontinuity persistence between 1 m and 2 m; and the higher resolution survey (spot spacing = 7; resolution = 1 cm to 10 cm) showed a peak at approximately 0.8 m.

Rock bridge trace lengths measured in window mapping at Jwaneng and Diavik may also conform to negative exponential functions. Window mapping at both sites considered only bridges of apparent intact rock identified in high-resolution digital photographs. In reality, however, some bridges of intact rock between co-planar discontinuity tips may reflect tightly-closed, tight or “healed” discontinuities (i.e. cemented via pressure solution or thermal “over-closure”, Barton, 2007) with small aperture that appears at unmagnified scale to comprise intact rock.

Figure 8 - 11 shows a histogram of all rock bridge trace segment lengths measured using Method 1, where bridges are traced only between discontinuities of similar orientation.

![Histogram of Rock Bridge Trace Segment Lengths](image)

**Figure 8 - 11:** Rock bridge trace segment lengths acquired using Method 1.

The peak frequency occurs for rock bridge trace lengths in the range of 0.5 m to 1 m, which is 1 to 2 orders of magnitude smaller than the mean and maximum values of
discontinuity persistence for each site. The negative exponential distribution is corrected with a constant factor of +3 in order to avoid convergence to zero at trace lengths exceeding 2 m. The correction factor of +3 is a function of sample size \( n \); expressed independently of \( n \), the correction amounts to +1% of the peak frequency value.

Figure 8 - 12 presents a histogram of all rock bridge trace segment lengths measured using Method 2, where bridges are traced between any nearest adjacent discontinuity, in order to divide the window into an assembly of discrete blocks with complete perimeters.

\[
\text{frequency} = e^{-5.5(x-1.95)} + 25
\]

Steeper curve / tighter hinge;

exponential growth is more abrupt than Method 1 results

Figure 8 - 12: Rock bridge trace segment lengths acquired with Method 2.

The peak frequency for Method 2 also occurs for rock bridge trace lengths in the range of 0.5 to 1 m; however, the curve is “steeper” than the Method 1 suggested best-fit line, such that the transition to exponential growth in the frequency of small rock bridge trace lengths is more abrupt. The suggested negative exponential distribution curve is corrected with a constant addition of +25 in order to avoid convergence to zero for trace lengths greater than approximately 2 m. The correction factor of +25 is a function of sample size; expressed in terms of sample size \( n \); expressed independently of \( n \), the correction amounts to approximately +1% of the peak frequency value.
Figure 8 - 13: Cumulative percentage plots for discontinuity persistence (i.e. fitted discontinuity disc diameters) and intact rock bridge traces for Diavik and Jwaneng.

**Diavik**

**Jwaneng**

Figure 8 - 13: Cumulative percentage plots for discontinuity diameters (persistence) and intact rock bridge traces.
Persistence curves from Diavik show increased skewness towards lower-persistence with increased focal lengths. However, even for the \( f = 300 \text{ mm} \) results, approximately 15% of discontinuity measurements exceed the “high persistence” threshold of 10 m suggested by ISRM (1978), and approximately 5% exceed the “very high persistence” threshold of 20 m.

Persistence curves from the Jwaneng plot indicate a major difference in the results from the long-range survey (distance \( \approx 1 \text{ km} \); ground-point spacing \( \approx 6.4 \text{ cm} \)) to the close range survey (distance \( \approx 50 \text{ m} \); ground-point spacing \( \approx 3.2 \text{ mm} \)). The long-range survey focused on characterising the persistence of major tectonic structures and the upper bounds of foliation discontinuity persistence. Consequently, approximately 80% of discontinuity measurements exceed the “high persistence” threshold of 10 m suggested by ISRM (1978), and about 50% exceed the “very high persistence” threshold of 20 m. In contrast, the close-range survey focused on characterisation of low-persistence step-path features including blast-induced fractures. As a result, the close-range survey indicates only 5% of discontinuity measurements exceeding the “high persistence” threshold of 10 m.

The Diavik results indicate that approximately 90% of intact rock bridges traced using Method 1 (i.e. only traced between similar discontinuity sets) are smaller than 3 m in length. The Jwaneng results indicate that approximately 90% of intact rock bridges traced with Method 1 are smaller than 1 m in length. Together, the results suggest that proto-joints which have not fully formed by coalescence and extension of microcracks (Hencher, 2012) may be separated by intact rock bridges in the order of 1 m in length. However, a conservative assumption for slope design would be to treat depositional discontinuity structures such as sedimentary bedding as continuous, not containing co-planar intact rock bridges.

Both Diavik and Jwaneng results indicate that intact rock bridge traces constructed using Method 2 tend towards smaller mean length than Method 1. The Diavik results indicate that 90% of intact rock bridge traces created with Method 2 are less than 1.5 m in length. The Jwaneng results indicate that 99% of intact rock bridges traced using Method 2 are less than 0.5 m in length. The result seems intuitive: in Method 2, rock bridges are traced between any nearest adjacent discontinuity tips, intending to simulate the natural
tendency for rock mass failure to preferentially occur, where possible, along pre-existing discontinuities before intact rock failure occurs.

The results appear to support the preliminary conclusion that intact rock bridges must generally be limited to smaller dimensions, between 1 and 2 orders of magnitude less than the dominant discontinuity persistence values. The range of surface area or length proportion values attributed to intact rock bridges in forensic investigations of slope failures (~ 0.2% to 7.3%, see Chapter 2) further supports the concept of an upper limit on intact rock bridge content in discontinuity-controlled slope failures.

8.7.1. Influence of Blasting Damage on Roughness

Blasting is usually attributed only to the destruction of intact rock bridges by extension of pre-existing discontinuities and creation of new cracks (e.g. Diederichs, 1999). Although blasting may destroy bridges of intact rock in the near-surface blast-damage zone, the resulting discontinuities may be rough and irregular, with secondary brittle fracture surfaces that contribute to frictional interlocking between blocks.

Figure 8 - 14 shows examples from Diavik and Jwaneng where blast-induced damage has produced a rough secondary hackle fringe in one case and undulating partial arrest marks in another. Both of the blast-induced fractures occur on larger, pre-existing joint surfaces exposed in bench faces. In low-confinement conditions as exist for bench-scale failures, blast-induced roughness may contribute to the dilation component of discontinuity shear strength; normal stresses in near-surface excavations are likely insufficient to cause asperity destruction if the intact rock strength is sufficiently high (~R3 + corresponding to UCS > 25 MPa, ISRM, 1978).
Figure 8 - 14: (A) Blast-induced fractures with rough hackle fringe (h), and en echelon cracks (e.e.) at Diavik; (B) fringe undulations (u) and arrest marks (a) at Jwaneng.
8.8. Recommendations for Future Work

Despite the increasing adoption by geotechnical professionals of remote sensing techniques and advanced numerical methods incorporating the influences of discontinuity persistence and intact rock bridges into slope stability analysis, there still remains a clear need for further research. Field mapping and numerical modelling guidelines specifically modified for assessment of large open pit slopes are needed in order to (1) reliably account for the role of discontinuity persistence in influencing failure mechanics, and (2) address the potential magnitude and extent of brittle fracture in large pit slopes.

Some recommendations for future research are presented below:

- **Increased research into risk-based slope design methods**

Mine designers are increasingly changing from deterministic hazard assessment to risk-based methods. Slope stability assessment for large open pits should make rigorous use of probabilistic methods for addressing the influence of discontinuity persistence and intact rock bridges. As shown by Langford and Diedrichs, (2012), probabilistic risk-based design methods provide a systematic methodology for addressing the influence of geological parameter uncertainty (describing unknown natural variation in physical properties of geological materials) and also model uncertainty (describing possible unknown shortcomings arising from modelling assumptions or constitutive laws assumed for simulating rock mass behaviour) on risk of instability or unsatisfactory excavation performance.

- **Increased application of Fracture Network Engineering**

Advances in microseismic monitoring and DFN simulation of persistence and intact rock bridges show promise for development of real-time monitoring of progressive destruction of rock mass bridges (Pettitt et al. 2012, 2011; Reyes-Montes et al., 2009). Researchers will benefit from collaboration between DFN investigations into fracture flow and geomechanics. Ongoing work in DFN methods for fluid percolation applications (Baghbanan and Jing, 2007; Xu et al., 2006; Wang, 2005; Ozkaya and Mattner, 2003) will continue to bridge the gaps between characterisation of fracture intensity, which
considers distribution of discontinuities throughout an area or a volume, and rock bridge content, which requires understanding of the discontinuity termination and connectivity.

Increased use and availability of alternative DFN codes for stochastic simulation of fracture network and block assemblies in jointed rock masses will stimulate development of new methods for:

- Assessing in situ block size distribution (Elmouttie and Poropat, 2012)
- Accounting for spatial correlation between different discontinuity sets, and thus improving simulation of the mutual influences of pre-existing discontinuities on the formation of new fractures (Dowd et al., 2007)
- Carrying out stability assessment based on key block analysis and progressive ravelling failure of blocks in jointed rock slopes (Merrien-Soukatchoff et al., 2012)
- Assessing slope failure kinematics and spatial variability of rockfall risk (Lambert et al., 2012)

Sturzenegger (2010) found that DFN models based on field measurements of discontinuity trace intensity could indicate 1% to 14% of the simulated rock mass volume comprising fully-formed blocks. Rogers et al. (2009) found that increasing volumetric fracture intensity $P_{32}$ could increase the volume of discrete blocks to nearly 100%, and that the transition from rock bridge-dominated rock mass, to a kinematically free rock mass of fully formed blocks, occurs over a relatively small change in $P_{32}$. Further study into intensity-based methods for characterisation of discontinuity persistence, interconnectivity and intact rock bridge content would help better understanding of the relationships governing block geometry and failure kinematics in large rock slopes. The rock mechanics interaction matrix method proposed by Harrison and Hudson (2006) and Hudson and Harrison (2000) may provide a helpful technique for formulating studies into non-conventional parametric interactions between parameters such as discontinuity persistence, termination style, interconnectivity, and intact rock bridge content.

- Continued numerical modelling research on dynamic rock failure mechanisms, and field investigations into the role of blasting and the blast damage damage zone in open pit slopes
Although increasingly sophisticated numerical codes are being developed to simulate dynamic blasting-induced damage both at the laboratory scale (Dehghan Banadaki and Mohanty, 2012; Mohanty et al., 2005) and at the open pit production bench scale (Furtney et al., 2011), slope designers still often rely on empirical methods for assessing the influence of blasting damage in open pit slopes (Hoek and Karzulovic, 2000; Little et al., 1999).

Future research should aim to investigate field-based methods for assessing blast-induced damage using techniques such as the newly-proposed $B_{21}$ parameter for quantifying intensity of blasting-induced damage in bench face window maps, and use of remote sensing data to characterise bench face roughness (Lee, 2011). Field assessment should be used as a validation tool, used in concert with empirically- and numerically-derived estimates of the thickness of the “blast-damage zone” and the strength and stiffness properties of blast-damaged rock masses.

- **Application of novel constitutive models for use in simulating very large slopes with high stresses, where confinement-dependent failure processes may occur**

Langford and Diederichs (2012) applied the S-curve approach to a risk-based analysis of probable depth of failure for tunnels in highly stressed, hard rock. The same principles may be useful in characterising potential slope instability in large open pits. By using an S-curve type shear strength envelope, the influence of confinement-dependent failure mechanisms may be better simulated, including the transition from cohesion dominated rock mass shear strength at low confining stress, to friction-dominated shear strength at high confining stress.

In an investigation of stress development and brittle fracturing in the evolution of alpine valleys, Leith (2012) applied the S-curve approach with a tri-linear shear strength envelope. His results suggest that accounting for confinement-dependent rock mass shear strength may be very important for characterising intact rock fracture processes in large slopes.
• Continued forensic field investigations of failed rock slopes, with specific emphasis on quantifying the component of intact rock rupture that occurred during failure

Although the influence of rock bridges has been recognized for over 40 years, the body of published forensic studies that quantify intact rock failure in slope instabilities is sparse. Future research should aim to expand the library of case studies that quantify rock bridge content in slope failures, in addition to incorporating additional information on the persistence of major discontinuities controlling the instability mechanism.
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