Slope Stability During Blasting: A Case History

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SUMMARY The paper presents a case history of the management of risk associated with blasting-induced slope instability based on blast acceleration monitoring and back analysis. Design curves showing charge weight versus distance for sensitive slopes result from the analysis.

1. INTRODUCTION

Newcrest Mining Ltd (NML) owns and operates the Telfer Gold Mine, which is located in the Great Sandy Desert of Western Australia, approximately 400 km east of Port Hedland. In the part of the operation relevant to this study, a sequence of argillites and arenites dips to the east at 32° to 37° on the eastern flank of the Telfer Main Dome structure. Open pits 1A and 1B are located here. Mining in pit 1A concentrated on extraction of the Middle Vale Reef (MVR) orebody and reached an ultimate pit depth of 125 m during the fourth quarter of 1990. Pit 1B produces millfeed and ore for heap-leach extraction from the mineralised sheeted-vein stockwork occurring beneath the MVR.

NML are currently mining pit 1B to a final pit depth of 95 m, which involves the "push back" of the existing pit 1A footwall slope. Blasting operations in pit 1B gave rise to concern for the stability of that part of the pit 1A footwall slope which was unsupported and occurred above the haul road (Figure 1). The integrity of the haul road is extremely important as this provides the only access to both the underground decline portal and the then operating pit 1A, both of which provide high grade millfeed.

2. GEOMETRY OF THE PROBLEM

The Telfer Main Dome structure produces an increase in bedding plane dip with depth as shown on Figure 1, which also shows the geometry of the footwall slope circa May 1991. It can be seen from the figure that the pit 1A footwall slope above the haul road had undercut bedding planes over its entire height. A slab of sandstone called the "top-slice", approximately 3 m thick, had been left unboiled on the slope face, and the mass failure of this slab onto the haul road was of serious concern to the mine management as mining in pit 1B progressed downwards. In addition, there were two other major areas of concern, the first being the section of the undercut slope above the decline portal, and the second (approximately 200 m from the portal) being where a ventilation raise occurred at the toe of the slope.

3. ENGINEERING GEOLOGY

3.1 Lithology and Structure

The Footwall Sandstone in which the slope was cut consists of interbedded sandstones, siltstones and claystones, with kaolinitised silt alteration products occurring throughout the rock mass and preferentially along bedding planes.

The dominant structural feature is the planar bedding. Other defect sets in the form of joints and veins are clearly visible in the Footwall Sandstone, however they tend to form release boundary surfaces to blocks sliding along bedding planes. For the purposes of this analysis, we assumed these defects are sufficiently persistent and frequent to allow sliding to occur freely.

3.2 Density and Shear Strength Parameters

The mean density of the Footwall Sandstone is 2.34 g/cm³.

Clayey and/or silty bedding planes in the footwall sandstone are known by back analysis to have very low friction (6°) and cohesion (2 kPa) properties. However, the bedding plane beneath the 3 m thick top slice shown on Figure 1 could not have such properties, otherwise it would have been dislodged shortly after (or during) the formation of the undercut slope.

Table 1 is a summary of the results of 8 tests using blocks of rock, which show a significant variation in basic friction angle. Tests 1, 2 and 8 are considered invalid due to the poor contact reported between test blocks. The remaining tests have a mean of 32° with a standard deviation of 8°.
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Profiling of the bedding planes indicated that a mean incremental friction angle of about 7° was available, while back-analysis of a footwall slope failure yielded an incremental friction angle of 9°.

For the purposes of this study, therefore, a mean basic friction angle of 32° plus a mean incremental friction angle of 9° were assumed. We also assumed that the cohesion on the plane was zero.

4. BLASTING PRACTICE

4.1 Designs

Blast patterns in the floor of pit 1B are usually drilled either on a 3m by 6m pattern, or a 4m by 4.5m pattern, using 146mm diameter blast holes, to a depth of 5.5m. A typical pattern is shown on Figure 2, and shows the centre-lift design.

Most holes are charged with 24kg of ANFO, exceptions being the central line of holes, which normally contains 20kg of ANFO per hole, and the row of holes which is drilled along the crest of the pit 1A footwall slope, which is usually charged with 18kg of ANFO per hole.

Holes are primed with 400g cast primers into which down-the-hole delay detonators are inserted. Delay number normally increases away from the central line of holes (for example, Figure 2). Holes are tied in on a V pattern, with 35ms or 42ms surface delays between tied-in rows. The combination of down-the-hole and surface delays gives rise to timing contours which have smaller apical angles than that of the V tie-in.

4.2 Hole Timing

Figure 3 shows the frequency and cumulative frequency distributions of time between consecutive hole detonations in the blast design presented in Figure 2. This figure assumes that no scatter occurs in delay timing, and shows that approximately 40% of holes detonate within 2ms of another hole, while almost 90% of holes detonate within 10ms of another hole. Clearly a significant number of more or less instantaneous detonation events occur.

![Figure 2 Typical Pit 1B Production Blast Design](image1)

![Figure 3 Time Between Detonating Holes](image2)
5. BLAST MONITORING

5.1 Equipment

Monitoring stations (at 9 locations along the footwall slope) were constructed at angles of mild steel sheet at 90° and welded onto the face plates of tensioned cable bolts near the toe of the footwall ramp batters.

Bruel and Kjaer type 4370 accelerometers were mounted on studs bonded to the mounting brackets and orientated to measure vertical and horizontal components of acceleration. The accelerometers were connected to Rion Vibration meters and data recorded on a TEAC R71, 7-channel data recorder. The system was calibrated to Australian Standards before and after each measurement, limiting the estimated error to 2%.

5.2 Results and Discussion

The monitored open-pit blasts were either production blasts, presplit blasts along the hangingwall batter or single hole shots designed to obtain information on individual waveforms. Presplit and single hole shots had a different acceleration response to the normal production blasts and were eliminated from statistical analysis on this basis.

Accelerations (both horizontal and vertical) from production blasts tend to correlate well with distance and total charge weight. This relationship is shown on Figure 4, in which the natural logarithm (ln) of the peak acceleration is plotted against the natural logarithm of the scaled distance, SD, where:

\[ SD = \text{distance} \cdot \text{total charge weight}^{0.85} \]

and where distance from the blast to the monitoring point is given in metres, and total charge weight is given in kilograms.

Linear regression of the data yields an equation of the form:

\[ \text{ln}(\text{acceleration}) = -1.671 \ln(SD) + 2.398 \]

which has an associated correlation coefficient of -0.94. For acceleration in the units of g, this regression line reduces to:

\[ \text{Acceleration (g)} = 1.1 SD^{-0.871} \]

The good correlation with total charge weight (rather than charge weight per delay) is probably due to the fact that a large proportion of holes detonate within a very short time of another hole (Figure 3).

Based on an analysis of the variance of the residuals from the regression line, we expect (with 95% confidence) that a measured acceleration result will be less than:

\[ \text{Acceleration (g)} = 3.1 \text{ SD}^{-1.671} \]

but greater than:

\[ \text{Acceleration (g)} = 0.4 \text{ SD}^{-1.671} \]

Using these relationships, limits can be placed on the expected accelerations from a production blast.

6. SLOPE STABILITY

6.1 Historical Stability

Several failures, mainly on a small scale, had occurred in the unbolstered top-slice of the footwall slope. Until May 1990, however, the locations of blasts relative to these events had not been recorded. Two subsequent events were therefore used for back analysis.

On 11 May 1990, a blast containing 372 blast holes was fired in pit 1B. During the blast, a small section of the unbolstered top-slice became dislodged from the face and slid onto the haul road. The distance from the blast to the bower of the undercut portion which failed was approximately 35m.

On 14 June 1990, a blast containing 198 blast holes was fired in pit 1B. Immediately after the blast, approximately 1600t of material slid from the slope. The distance from the blast to the bower of the undercut portion which failed was approximately 22m.

The scaled distance relationships developed in Section 5.2 were used to estimate (by way of back-analysis) the peak acceleration acting on the failed blocks during each blast. In order to ensure that predictions tended to err on the conservative side, we used the relationship derived for the lower 95% confidence limit. This relationship suggested that accelerations were at least 2.1g and 2.7g, respectively, for the two failures discussed above.

Half-way up the dip slope above an underground mine vent raise, at a depth of approximately 80m below surface, a 100m long section of undercut bedding, approximately 10m high, has been subjected to increasing blast-induced accelerations from pit 1B for months. We estimated that accelerations of up to 6g (with a mean of 2.4g) must have occurred during blasting. Nowhere had this slope failed. Clearly significant accelerations were required to cause failures in otherwise stable sections of slope.

6.2 STABILITY ANALYSIS

6.2.1 Introduction

The pseudo-dynamic analytical method normally used to analyse slope stability is more suited to the analysis of earthquake accelerations than those from blasting. During blasting, vibration frequencies in the range 20Hz to 40Hz (depending on distance from the blast) are typical for Teller production blasts in the footwall rocks. These are much higher than those associated with earthquakes (typically 0.5Hz to 2Hz) and, in addition, blast duration is relatively short. As a result, the use of blast acceleration data directly
In a conventional stability analysis leads to ridiculously conservative results. We therefore only used the slope stability analytical method to estimate the form of the relationship between factor of safety and acceleration, and did not use it as a predictive tool.

6.2.2 Factor of safety versus acceleration

The results of pseudo-dynamic slope stability analyses for low frequency, long-duration acceleration pulses show that the relationship between acceleration and factor of safety has a semi-logarithmic form. However, it can be conservatively approximated by a straight line, particularly in the area of interest close to a factor of safety of 1.0.

Based on the shear strength parameters discussed in Section 3.2, the factor of safety of the unbolted top slice was estimated to be at least 1.2 under static conditions. Based on the back-analyses presented in Section 6.1, we conservatively assumed that an acceleration of at least 2g was required to cause an unbolted block to slide. Assuming a straight-line relationship, therefore, the effective factor of safety for an unbolstered section of slope subjected to blast accelerations is shown on Figure 5, which also shows the equivalent curve for a slope with a static factor of safety of 1.4.

![Figure 5 Assumed Factor of Safety Vs. Acceleration Relationship for Blasts](image)

7. CONCLUSIONS

Normal production blasts fired in pit 1B had relatively short, effective inter-hole delays, with up to 90% of blast holes detonating within 10ms of another hole.

Blast acceleration monitoring indicated that the relationship:

\[ \text{Acceleration (g)} = K \times SD^{1.87} \]

applied, where \( SD = \) distance. Total charge weight \( q \) and \( K \) ranged from 0.4 to 3.1 with a mean of 1.1;

Back analysis of the original, unbolted top-slices on the pit 1A foottwall slope suggested that blast-induced accelerations of at least 2g were required to cause significant block failures in unbolstered sections of slope.

Based on a knowledge of blast size, distance from the brow of the undercut slope and static factor of safety, the susceptibility of the section of slope to failure during blasting could be estimated.

![Figure 6 Charge Weight Vs. Minimum Distance for Production Blasts](image)
Figure 6 shows the design curves used for managing the risk of blast-induced slope instability above the two most sensitive sections of the pit 1A footwall.

8.0 ACKNOWLEDGMENTS

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