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 400km 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 125m 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 95m, 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 haulroad (Figure 1). The integrity of the haulroad 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.

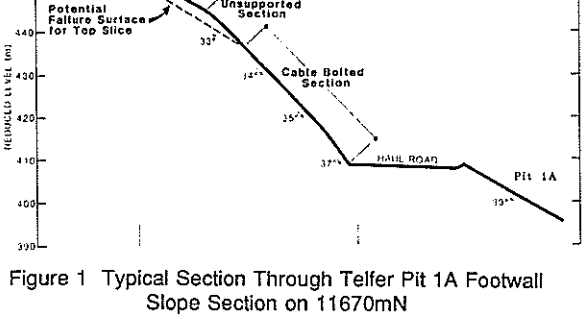


Figure 1 Typical Section Through Telfer Pit 1A Footwall Slope Section on 11670mN

Previous planar bedding plane failures onto the haul road had occurred in the pit 1A footwall slope during and after production blasts in pit 1B, although these localised failures had occurred along the northern part of the haulroad and had not presented any serious problems.

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 haulroad had undercut bedding planes over its entire height. A slab of sandstone called the "top-slice", approximately 3m thick, had been left unbolted 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 200m 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 that 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 2.34t/m^3.

Clayey and/or silty bedding planes in the footwall sandstone are known by back analysis to have very low friction (8°) and cohesion (21kPa) properties. However, the bedding plane beneath the 3m 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 I is a summary of the results of tilt 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°.

TABLE I

RESULTS OF TILT TABLE TESTS

Test No.	Number of Trials	Mean Angle Required to Cause Sliding	Comments
1	4	34	POOR PLANE CONTACT (20-30%) smooth cored, silty plane. SILTSTONE.
2	4	31	
3	3	36	75% CONTACT. Undulating, smooth to rough. SILTSTONE.
4	3	39	80% CONTACT. Blocks well matched. SILTSTONE.
5	3	38	95% CONTACT. Bedding rough planar. SILTSTONE.
6	3	43	90% CONTACT. Curved, smooth, planar no infill SANDY SILTSTONE.
7	4	39	100% CONTACT. Silt infill <1mm on plane. SILTSTONE.
8	3	20	POOR CONTACT. Extremely weak. SILTSTONE.
9	3	33	Irregular, smooth plane. SILTSTONE.
10	3	27	smooth, planar bedding. SILTSTONE.
11	1	24	smooth, planar bedding. SILTSTONE.
12	3	23	smooth, planar bedding. SILTSTONE.
13	3	24	planar rough, weak. SILTSTONE.
14	3	20	Irregular rough, weak. SILTSTONE.
15	3	25	smooth, planar weak. SILTSTONE.

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 8° were assumed. We also assumed that the cohesion on the plane was zero.

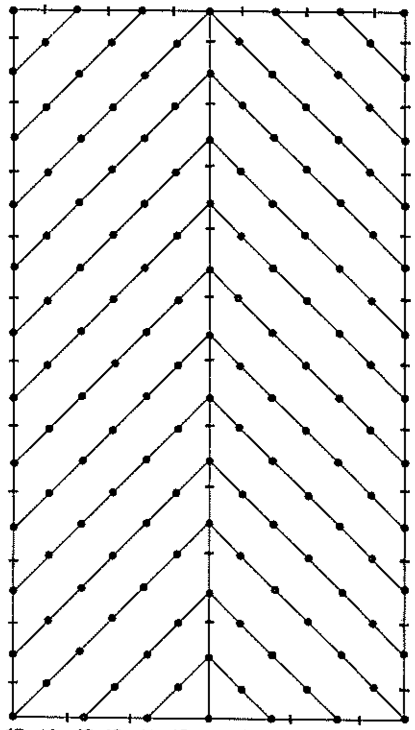
4. BLASTING PRACTICE

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.



• — BLAST LOCATION
— — SURFACE DELAY
8 — DOWN-HOLE DELAY NUMBER

Figure 2 Typical Pit 1B Production Blast Design

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.

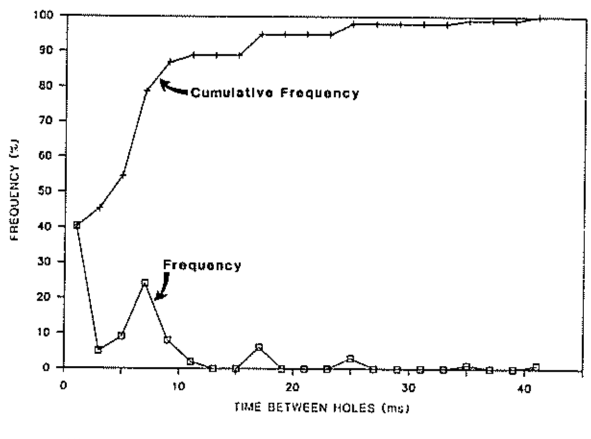


Figure 3 Time Between Detonating Holes

5. BLAST MONITORING

5.1 Equipment

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

Bruel and Kjaer type 4370 accelerometers were mounted on studs bonded to the mounting brackets and oriented 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:

$$SD = \text{distance} \;.\; \text{total charge weight}^{-0.5}$$

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:

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

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

$$\text{Acceleration (g)} = 1.1\ SD^{-1.671}$$

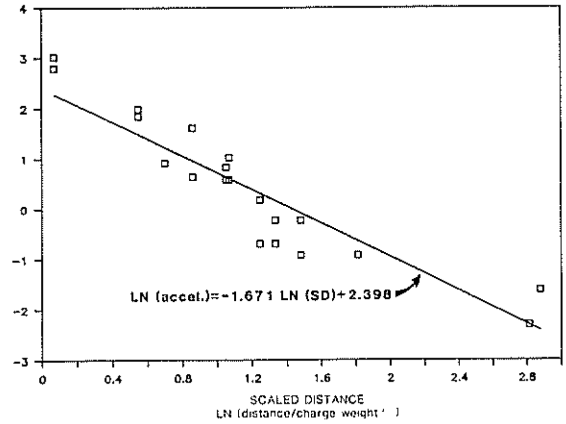


Figure 4 Production Blast Acceleration Monitoring Results

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:

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

but greater than:

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

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 unbolted 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 374 blast holes was fired in pit 1B. During the blast, a small section of the unbolted top-slice became dislodged from the face and slid onto the haul road. The distance from the blast to the brow 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 1600bcm of material slid from the slope. The distance from the blast to the brow of the undercut portion which failed was approximately 22m.

The scaled distance relationships developed in Section 5.2 was 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 Telfer 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 unbolted 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.

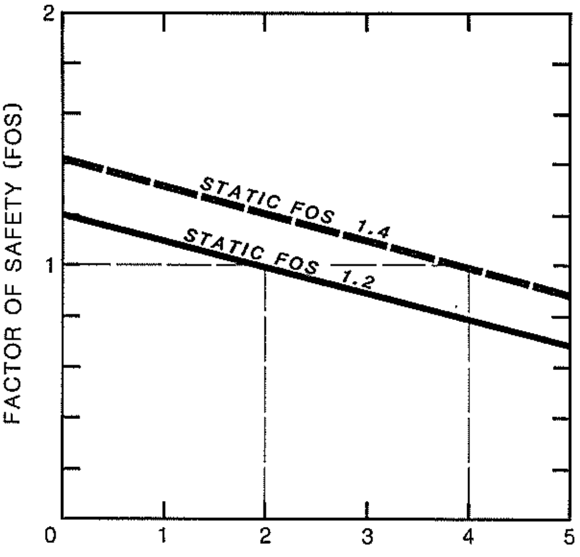


Figure 5 Assumed Factor of Safety Vs. Acceleration Relationship for Blasts

6.2.3 Portal and vent raise slopes

Based on our preliminary findings from the portal slope stability investigation, we estimated the minimum factor of safety of the undercut slope above the portal to be 1.2. Figure 5 shows the line relating factor of safety to blast acceleration at the brow for a slope having a static factor of safety of 1.2. This line indicates that, conservatively, a peak acceleration of 2g is required to cause failure in the slope. In order to reduce the risk and increase reliability, we halved this value; that is, we did not allow accelerations to exceed 1.0g in this part of the slope.

Using the upper 95% confidence limit relationship derived in Section 5.2 from measured blast accelerations we could estimate the maximum accelerations which could be expected during a blast. By re-arranging the equation, we obtained the charge weight (or number of blast holes) conservatively required a produce an acceleration of 1g for a variety of distances. Table II shows the resulting information.

TABLE II

BLAST SIZE AND MINIMUM DISTANCE ABOVE PORTAL

Charge Weight (kg)	Number of Holes	Minimum Stand-Off Distance (m)
102	4	20
410	17	40
920	38	60
1640	68	80
2550	106	100

A similar analysis was conducted for the slope above the ventilation raise. The slope was expected to have a static factor of safety of approximately 1.4 which, from Figure 5, suggests that a blast-induced acceleration of 4g could lead to failure. Thus if 2g was the maximum acceleration allowed at the brow of the undercut, then the blast sizes are conservatively estimated in Table III for a variety of distances.

TABLE III

BLAST SIZE AND MINIMUM DISTANCE ABOVE VENT RAISE

Charge Weight (kg)	Number of Holes	Minimum Stand-off Distance (m)
235	10	20
940	39	40
2120	88	60
3760	157	80
5900	246	100

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:

$$\text{Acceleration (g)} = K.SD^{-1.67}$$

applied, where SD = distance. Total charge weight $^{-0.5}$ and K ranged from 0.4 to 3.1 with a mean of 1.1;

Back analysis of the original, unbolted top-slice on the pit 1A footwall slope suggested that blast-induced accelerations of at least 2g were required to cause significant block failures in unbolted 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 a section of slope to failure during blasting could be estimated.

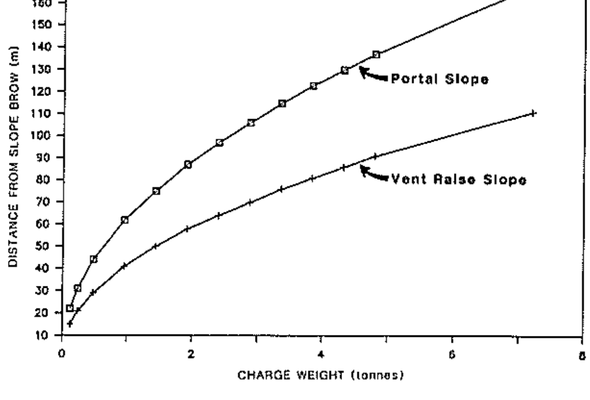


Figure 6 Charge Weight Vs. Minimum Distance for Production Blasts

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.

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