Assessing Geotechnical Risk of Instability between Adjacent Mining Operations – A Case Study

M. DUNBAVAN
Ph.D., M.A.G.S.
Supervising Engineer, In-Situ Processes, BHP Engineering
G. BOYD
M.Sc., M.A.I.M.M., M.A.G.S., M.G.S.A.
Managing Director, Boyd Mining Pty Ltd

1. INTRODUCTION

Instability of a 60 m highwall excavated to a common boundary between two adjoining but separately owned coal mining operations in the NSW northern coalfields represented potential for significant economic loss for both parties. On the one hand, the failure threatened an arterial coal haulroad located adjacent to the boundary in one lease, whilst on the other, the possibility of reserves sterilisation due to a restriction of mining limits became apparent.

A lack of appreciation of the geological structure along the boundary at the time of establishing a boundary mining agreement resulted in failure of the first of 4 x 300 m long mining blocks located against the boundary. To develop acceptable guidelines for establishing safe mining limits, a joint investigation into the cause(s) of failure was commissioned.

To more usefully define mining limits, it was agreed a probabilistic approach to stability analyses be adopted from which the risk of boundary highwall failure was to be assessed.

This approach allowed operators to better understand, in relative terms, the magnitude of risk embodied in the mining limits analysed. The quantification of risk also represented an opportunity to value the cost of insuring against losses incurred by inadvertent failure.

2. BOUNDARY HIGHWALL INSTABILITY

2.1 History of Failures

Instability first occurred in March 1989 during stripping to the base of the upper overburden, exposing P Seam. Pit limits at the time of first instability are shown in Figure 1, with the toe of the (basal) B Seam 110 m west of the lease boundary, and the toe of the P Seam layer 90 m west of the same boundary. Movement was concentrated at the roof of T coal horizon or above, with a scarp ultimately forming behind the planned highwall crest.

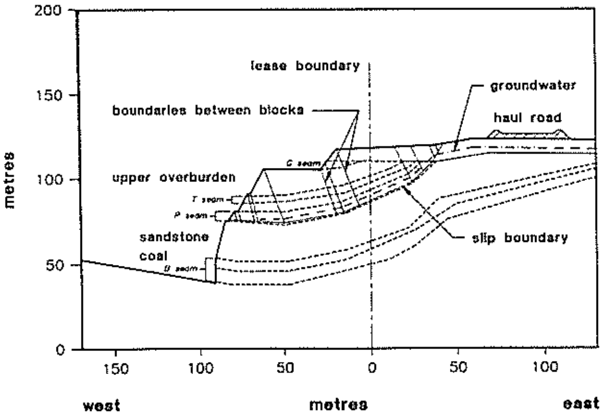

Figure 1 Lease boundary geology and structure

With further extraction along the boundary, slip of the upper overburden occurred during excavation of a bench to a level just above the roof to the T Seam. To reduce the hazard to mining operations from continued rock movement, the slope above bench level was dozed parallel to bedding at an angle of approximately 35°. During excavation of the next bench, movement occurred along the base of the P Seam initially in the northern portion of the panel when the toe of the slope was only 20 m west of the lease boundary (see Figure 1). Movement was accompanied by significant floor heave indicating the failure surface lay in the floor beneath P Seam. Movement in the slope extended over the full length of the panel, but movement along the base of P Seam diminished to zero at the southern panel limit, where the toe of the seam was located at 50 m west of the boundary. The scarp of the failure developed at its current day position at 90 m east of the lease boundary (see Figure 1).

2.2 Failure Path

Measurement and observation of the slope deformation suggests a non-linear displacement at the base of the P Seam. Exposure of the P Seam floor sequence revealed the failure path corresponded to a mylonite of crushed carbonaceous siltstone with subordinate silt and clay some 100 mm in thickness.

The failure path, confined to the mylonite followed the dip of the beds at the base of the P Seam horizon. Variation in dip along the failure path caused discrete blocks to form, with subsequent displacement and rotation resulting in the deformation clearly observable in the failed slope.

At the head of the failure, a sub-vertical scarp developed over a visible depth of 3 m to 4 m. Subsidence of blocks in front of the scarp showed only minor rotation, suggesting the final (i.e. current) scarp location resulted from local instability at the head of the failure, and not due to the primary failure mechanism.

The mylonite at the base of the P Seam occurs commonly in exploration drilling records from both sides of the boundary. It is a natural geological feature associated with past tectonism, and is considered a feature of sub-regional extent, and certainly occurs throughout the high dip zone along the common lease boundary.

3. GEOTECHNICAL CONDITIONS - LEASE BOUNDARY

3.1 Geology

The upper horizon (above the base of the P Seam) consists of thinly interbedded argillaceous sandstone, siltstone and carbonaceous siltstone and claystone carrying up to three mineable coal seams (P, T and G in ascending order).

Low grade regional metamorphism associated with tectonic folding and faulting has developed a shale sequence in fine grained sequences. The induration arising from metamorphism has left all fine grained sediments as hard, brittle and highly competent rocks with slaking potential, but not prone to dispersion.

Due to rock competence, the depth of weathering is shallow, with the residual soil zone confined to a depth of 2 m or less. The moderate to slightly weathered zone, comprising iron stained joint surfaces, extends to a maximum depth of 15 m.

Significant intrusion of the T and P Seams by dolerite has affected coal quality, and on occasion has formed minor clay seams representing decomposed sill material. However, the mylonite below the P Seam horizon appears free of intrusive, and development of silt and clay in the mylonite is thought to represent products of degradation of the mylonite, probably due to groundwater percolation.

3.2 Structure

Detailed mapping about the failed highwall revealed a total of eight parallel reverse faults dipping 80° west, and resulting in a total displacement of approximately 2 m (west blocks up). The largest displacement on any one fault measured 0.8 m.

The faults lie in the western limb of an anticlinal structure, the axis of which occurs 150 m east of the common boundary. Bedding dips were measured to peak at 45°, with the interval of steepest dips averaging 31°. This compares to an average of 16° measured from a computer model of the anticline.

Seepage was observed to originate from bedding structures, suggesting a degree of openness along both bedding and jointing. However, no slickensided nor polished surfaces were ascribed to bedding, partings or joints, which conforms with general observations in both pits.

Jointing is close spaced (0.5 m to 1.5 m) and from past mining experience, has caused little to nil instability in excavated highwalls along the boundary.

3.3 Groundwater/Rainfall Observations

Four geotechnical holes were completed as single or multi-level piezometers.

Groundwater was recorded in two piezometers, the first completed at the base of P Seam where a level of 0.5 m above base was observed, and the second completed at the base of the deeper B Seams where a level of 1 m above base was recorded.

The groundwater observed at the base of the P Seam is perched, indicating incomplete hydraulic connection down through the sequence.

Rainfall records indicate that during March 1989 when first failure of the highwall occurred, a total of 115.6 mm of rain fell, 50 mm of which occurred in the one wet weather spell over three days, with 30 mm recorded in one of the days. April, May and June of 1989 recorded high non-seasonal rainfalls of 139.2 mm, 122.4 mm and 113.5 mm. These compare to a 10 year average for the same months of 72.24 mm, 46.62 mm and 36.76 mm.

The mining of the next panel and subsequent failure during August/September 1989 was preceded and accompanied by very low rainfall, 5.4 mm and 12.4 mm respectively compared to a ten year average of 44.91 mm and 45.07 mm respectively for the same months. It is speculated that the highwall rainfall in March, April, May and June caused a significant rise in groundwater levels, declining but still significant during the mining of the next panel. Observations at the face during overburden excavation indicated dry conditions and the slope after failure remained dry.

3.4 Material Properties

The final selection of design parameters was based on back analysis of the major highwall failure and independent laboratory testing of rock mass defects, including the P Seam mylonite, by respective consultants.

3.4.1 Back analysis

Back analysis of a reconstruction of the failure in the previous panel suggested with zero groundwater loading, and zero cohesion, a friction angle of 17.5° equated to a unit factor of safety. Computations were completed assuming the following parameters:

. Upper overburden: $C = 0$, $\phi = 25°$, $\delta = 25$ kN/m^3
. Puxtress mylonite: $\delta = 20$ kN/m^3
. Fractured coal: $C = 15$ kPa, $\phi = 36°$, $\delta = 15$ kN/m^3

The Sarma method of analysis was used, which permits use of non-linear and user defined surfaces of deformation. Surfaces observed and reconstructed from all information available were thus included and modelled.

3.4.2 Laboratory testing

Independent laboratory strength testing of the Puxtrees mylonite indicated:

- peak shear strength corresponding to C = 0 to 28 kPa, friction = 28°;
- residual shear strength corresponding to C = 0 to 27 kPa, friction = 17.5° to 19.0°.

The laboratory results thus agreed well with lower bound strength parameters determined by back analysis. However, it must be appreciated that back analysis yields parameters representing the effect of all processes acting in the slope affecting stability, and not just failure plane strength.

3.4.3 Expected parameters

Laboratory strength results show the frictional strength of the P Seam mylonite varied between 17.5° and 28°. The roughness of the surfaces in the mylonite zone and undulating trend of bedding surfaces, interrupted by small displacement faults, render it unlikely that a state of residual strength existed throughout the mylonite at the time of failure.

Further manipulation of the back analysis model indicated a most likely average frictional strength of approximately 21° (and zero cohesion) under an average groundwater head of 2 m.

Agreed strength parameters of other rock mass zones and structures for the design of Panel E8 are summarised in Table I. Mean parameters (as tabulated) were used in back analysis of the major highwall failure.

TABLE I
DESIGN VALUES FOR GEOMECHANICAL PROPERTIES

Rock Group	Unit Weight (kN/m^3)	Cohesion (kPa)		Friction Angle (deg)	
		Mean	Std. Dev.	Mean	Std. Dev.
Upper overburden (above P seam)	25	0	0	25	2
P seam mylonite	20	0	0	21	2
Sandstone (below P seam)	25	0	0	35	2.5
B seam shale	25	0	0	30	2.5

4. ANALYTICAL METHODS

4.1 Approach

The development of a slip mechanism in the overburden is dominated by geological structure. Variations in structure, mainly changes in dip of discontinuities, will result in different factors of safety at different locations in the slope. Any design approach has to address this variation and the method of stability analysis should be able to represent the effects of structure (joints, bedding planes and shear zones) on the slip mechanism.

Variation of structure was addressed by analysing six highwall cross sections for a slope strike length of approximately 450 m. The locations of cross sections were chosen to represent different structural regimes and were placed as close as practicable to boreholes to reduce reliance on interpolated geological data.

For stability analysis, a two dimensional method was adequate in representing broad geological features and the proposed terrace mining method for overburden removal. Sarma's limit equilibrium method was selected from a range of methods applicable to non-circular slip mechanisms. The main advantages of Sarma's method for this case were use of a non-circular slip boundary and specification of non-vertical block interfaces within the sliding mass together with shear resistance on those interfaces. Figure 1 illustrates a typical cross section with the slip mechanism used for slope design.

As well as significant changes in geological structure between cross sections, ground conditions also vary within each cross section. Even if detailed measurements were obtained from field or laboratory tests, data represent only point values from a large three dimensional volume. A probabilistic approach was adopted to represent variation in ground conditions. The Point Estimate Method was used in conjunction with Sarma's method of stability analysis and estimated probability distributions for ground conditions to produce a probability distribution for the factor of safety at each cross section.

4.2 Design Assumptions

The main assumptions required for the slope design were:

- existing geological data were representative of the major structural features in the area under study;
- geomechanical property values were normally distributed and the selected values were representative of ground conditions;
- the location of the slip boundary was dominated by the mylonite at the base of P seam for one case and by the fractured zone in B seam for the second case;
- the presence of groundwater in P seam is transient and dependent on infiltration from storms. The six hour storm produced the worst conditions due to slope drainage for longer storm durations. Similar conditions were assumed for slip through B seam;
- two dimensional slope stability analysis was representative of field conditions and three dimensional wedge slips did not affect slope stability.

4.3 Ground Condition

4.3.1 Geomechanical properties

The drilling records and existing excavations showed a complex series of coal seams, interburden rocks and dolerite intrusions comprised the strata exposed during mining. Exact replication of these strata in a numerical model for stability was not feasible. Thus, the strata were grouped into four major rock types for the purposes of stability analysis. The selection of values for geomechanical properties was based on the application of those values in Sarma's method of stability analysis in conjunction with the point estimate method. Table I lists the rock groups and geomechanical properties used in design calculations.

4.3.2 Groundwater levels

No significant seepage or water levels in piezometers had been observed in the upper overburden. B seam at the base of the slope appeared to be the main aquifer in the region. Seepage from the coal was steady, but no major flows were observed which might indicate elevated groundwater conditions in the pit slope. The assumption of transient groundwater conditions in the upper overburden was supported by some observations of seepage from excavated faces, but only after rain had recently fallen.

The highly fractured upper rock mass could allow rainfall infiltration to move rapidly through the mass to build a saturated zone starting from the base of P seam above the competent sandstone unit. Because of the likely rapid infiltration rate and the many discontinuities along which water could exit from the rock mass, a six hour duration storm was selected as producing the worst groundwater conditions. The derivation of relevant rainfall characteristics is described in a later section.

For a six hour storm event, the infiltration was estimated to be 40 mm depth of water/m^2 for a 1,000 year average recurrence interval event. Using estimates of void ratio due to rock mass discontinuities and discounting absorption into the rock material, this infiltration corresponded to a 2 m head above the P seam mylonite. This level was adopted as an average value together with a standard deviation of 1 m head.

4.4 External Loads

Experience at the site had shown no measurable effects on slope stability from either overburden blasting or natural seismicity. The truck and shovel operations are based on successive excavation of 15 m to 20 m high terraces from west to east. Overburden shots are relatively small compared with those used in conventional dragline operations. Shots have a maximum burden of 20 m and are usually fixed with two free faces.

Loading applied by machinery (large trucks and occasionally a dragline) is small in comparison with the weight of the rock mass as well as being transient. The rock mass properties do not favour brittle failure. The main impact of a significant highwall slip would be loss of haul road access for machinery rather than danger to a particular machine or operator.

4.5 Probability Distribution for the Factor of Safety

The two cases, slip boundaries through P seam and B seam, were assessed independently. The P seam case was expected to be the one controlling final highwall design because of previous instability. However, the changing geological structure and more severe consequences of slip developing through B seam shale meant that this case could not be ignored. For each case the factor of safety was assumed to be a function of three variables.

For slip through P seam these were shear resistance of the P seam mylonite, shear resistance of the upper overburden and position of transient groundwater in P seam due to a 1,000 year ARI storm event; and for B seam were shear resistance of the B seam shale, shear resistance of the sandstone and position of transient groundwater in B seam due to a 1,000 year ARI storm event.

The point estimate method requires 2^n evaluations where n is the number of variables (eight for three variables), for estimation of the probability distribution of the dependent variable, in this exercise the factor of safety. From the eight values, the mean and standard deviation were calculated using standard statistical formulae. Thus, for a particular highwall profile and location, the average factor of safety could be calculated for the probability distributions of the three selected variables. The standard deviation of the factor of safety was used to calculate the probability that the factor of safety may be less than 1.0.

5. RISK ASSESSMENT

5.1 Time Dependency

The absence of any thick clay bands and the relatively shallow depth of weathering led to the assumption that geomechanical properties relevant to highwall design would be independent of time. Excavation of slopes certainly changes stress conditions in the remaining rock mass. Some relaxation (that is, a time dependent effect) may occur as stresses are redistributed, however this effect was expected to be minor. The majority of the discontinuities within the rock mass was rough and no significant deterioration within the mine life was expected.

The remaining variable considered in highwall design was groundwater level, which certainly varies with time. The highly fractured condition of the upper overburden and relatively thin soil cover may have made that part of the overburden susceptible to rapid infiltration of rainfall runoff with subsequent transient rises in groundwater level. The absence of persistent seepage from exposed rock faces and the lack of water in standpipes installed to different depths indicated negligible groundwater storage in the upper overburden. Thus, the groundwater levels were assumed to depend on storm events, the severity of which can be expressed as a function of time, namely the average recurrence interval (ARI).

Rainfall infiltration characteristics were estimated using a technique developed by the U.S. Department of Agriculture Soil Conservation Service. This technique is based on prediction of direct runoff from a specified depth of rainfall, given classification of the soil as one of four major hydrological groups together with an estimate of antecedent moisture condition (AMC).

The infiltration analysis was undertaken for a wide population of storm events. Design storm data were derived from information published in "Australian Rainfall and Runoff" (The Institution of Engineers, Australia, 1987). Values for precipitation as a function of storm duration were calculated for ARIs between 1 and 100 years. Because of the apparently high permeability of the rock mass, a storm duration of six hours was adopted as that creating the most adverse variation in groundwater conditions. This was a subjective assessment because no data were available for analysis. Extending the storm data to a 1,000 year ARI, the infiltration was equivalent to 40 mm depth of water per square metre surface area for a six hour duration storm.

Assuming that all infiltration moved rapidly down through the rock mass with no seepage, and that the material below the suspected slip boundaries was impervious, the 40 mm infiltration was estimated to produce a 2 m rise in groundwater level. The uncertainty in this estimate was high and a probability distribution for the height of the rise was chosen subjectively. The distribution was assumed normal with an average of 2 m and a standard deviation of 1 m. This distribution was not time dependent and the only time dependence resulted from the use of the 1,000 year ARI storm event.

5.2 Risk of Slope Failure

The probability distribution for the factor of safety was based on variables which were relevant for the occurrence of a 1,000 year ARI storm event. The risk of slope failure during mine operations is equal to the combined probability that the factor of safety will be less than unity and the probability of the 1,000 year ARI event occurring during the operating period (a value of 0.015 for 15 years operation). Values for this risk are given in the following section.

6. APPLICATION

6.1 Establishing Limits of Mining

The usual criteria for slope design include minimum values for the factor of safety under various loading conditions and combinations of material property values. The variability of the properties and conditions which influence the factor of safety significantly reduced confidence in the idea of using only the factor of safety as the design criterion. The adoption of a probabilistic approach was described earlier. The application of the results from the analyses is the topic of this section.

Engineers from both mines and the regulatory authority agreed that the minimum average factor of safety for slip in the upper overburden should be 1.3 and that for a slip in the entire overburden should be 1.5. This judgement was made before any information on the probability distribution of the factor of safety was available. For the selected cross sections within the design area, the highwall location to meet the above criteria was determined by trial and error using average geomechanical properties and groundwater conditions.

Slip in the upper overburden was the controlling factor. The use of average values in the first part of the calculation was convenient, but did not give a statistically acceptable average value for the factor of safety. This was obtained from the point estimate method. If the slope location did not meet the design criteria when this calculation was completed, the location was revised until criteria were met.

The risk of slope failure was then calculated using the time dependent probability of the occurrence of groundwater levels due to a 1,000 year ARI storm. The following table gives results of calculations for different cross sections.

TABLE II
RESULTS FROM PROBABILISTIC STABILITY ANALYSIS

Section	Upper Overburden			Entire Overburden		
	Ave FS	Pr FS<1.0	Risk	Ave FS	Pr FS<1.0	Risk
A	1.45	0.002	$3x10^{-5}$	1.84	10^{-6}	$1.5x10^{-8}$
B	1.36	0.004	$6x10^{-5}$	1.84	10^{-6}	$1.5x10^{-8}$
C	1.36	0.004	$6x10^{-5}$	1.78	10^{-7}	$1.5x10^{-9}$
D	1.36	0.004	$6x10^{-5}$	1.84	10^{-8}	$1.5x10^{-10}$
E,F	1.30	0.010	$1.5x10^{-4}$	1.76	10^{-7}	$1.5x10^{-9}$

The combination of values of factor of safety and the risk of failure were found by mine staff to be helpful in discussing the consequences of slope instability. For example, the consequences of instability of the entire overburden were much greater than those for the upper overburden, but the likelihood of the former occurring was less by at least 1,000 fold. The use of quantitative risk may be extended to estimating the financial risk (or liability) to which each mining operation was exposed. Slip in the upper overburden would render a haulroad on the adjacent lease unserviceable. The restoration cost would involve both earthworks to relocate the haulroad and the cost differential in producing coal until the haulroad was restored. The liability is the restoration cost times the risk of failure. With risks less than 10^{-4}, costs of tens of millions of dollars correspond to liabilities of thousands of dollars.

6.2 Monitoring Sub-surface Conditions and Displacements

Without resorting to complex (and expensive) instrumentation systems, the performance of the slope design and design assumptions may be assessed from groundwater levels and rock mass movement. Standpipe piezometers were installed in several boreholes. Screens were open to T, P and B seams and standing water levels were measured regularly. To correlate changes in water level with storm rainfall, an automatic data logger was installed in the field and connected to a tipping bucket rain gauge and water level sensors in the standpipes.

In the six months since installation, rainfall has been sparse and the district is a declared drought area. The water levels in the standpipes have consistently fallen during this period and showed no transient response to rainfall.

Monitoring of rock mass movement by conventional surveying methods has resulted in a very useful database for assessing design performance. Movement toward the excavation has been recorded since the commencement of overburden removal. Initially, the displacement rates increased and decreased sharply as a layer of overburden was removed adjacent to a monitoring station. As mining progressed, the displacement rates decreased in magnitude but persisted for a much longer period. This may be compared with the rate of change of momentum of the rock mass, in that deeper mining affected a larger rock mass which accelerated more slowly than smaller masses, but also decelerated more slowly.

The trend of displacement rate at the end of overburden removal was continuously decreasing. Although some extension cracks had appeared across the haulroad behind the highwall crest, the road remained serviceable. Displacement monitoring is being maintained.