Numerical modelling of complex slope movements at Savage River Mine, Tasmania

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ABSTRACT: The Savage River Iron Ore Mine has a history of flexural toppling failures of pit slopes. This mechanism has been studied with the distinct element program *UDEC*, using a rock mass model including both explicit and ubiquitous joints and a Hoek-Brown yield criterion for the rock matrix, and simulation of progressive dewatering of the pit. Two previous failures in the North Pit were back analysed and then the same rock mass parameters were used to examine several options for extended mining of that pit. These options included different ultimate pit depths, dewatering strategies and cases with and without a ramp in the west wall. The models indicated the potential for continued wedge - flexural toppling failures in future west walls but showed that these could be managed by dewatering and monitoring.

1 INTRODUCTION

The Savage River Iron Ore Mine has been operating for over 30 years in the northwest of Tasmania, Australia. A series of flexural toppling failures has occurred in parts of the western walls of Centre Pit and North Pit, limiting overall slope angles to 37° with vertical slope heights of about 150 m.

The stress analysis program *UDEC* (Itasca 1996) has been used to back analyse observed failures in two sections of the North Pit walls and then to examine several options for future mining of that Pit.

A companion paper (Hutchison et al. 2000) provides more details of the geology and flexural toppling history, whilst this paper concentrates upon the numerical modelling strategy and results.

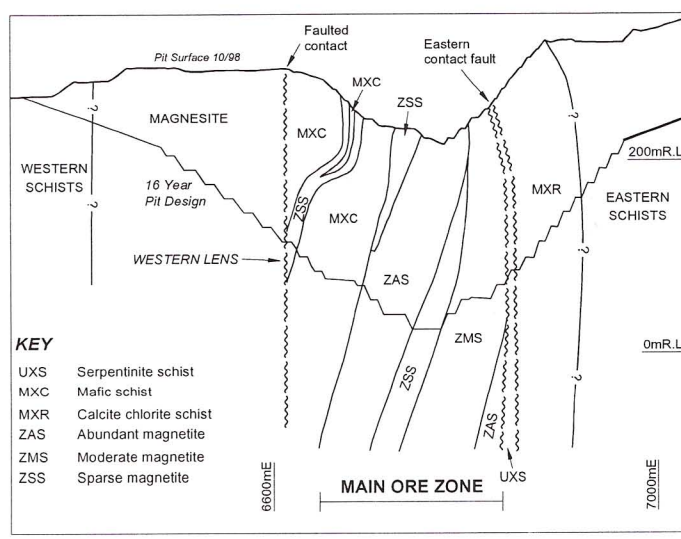

Figure 1. Typical geological section, North Pit.

2 GEOLOGY

A typical geological section within North Pit is shown in Figure 1. Rocks to the west of the orebodies consist of weak mafic schists (designated MXC or MXB) and moderately strong banded mylonites (MYF). The orebody in Figure 1 is shown based on magnetite content but for the modelling it was separated into ore hard (OH) and ore soft (OS) categories, based on perceived deformation characteristics.

The ore is bounded on the east by the near-vertical Eastern Contact Fault comprised of clayey gouge and soft serpentinite schist (UXS). East of the fault is competent and strong calcite chlorite schist (MXR).

Other dominant structural features in the west wall rocks of both pits are steeply dipping, N-S striking foliations and foliation shears, which have a major influence on toppling. Dolerite (in the Centre Pit) and magnesite intrusions also influence toppling failures, but usually by increasing stability. There are two sets of pervasive discontinuities in the west wall of the northern-most area that was modelled. These dip 50-55° to the north-east and south-east. Although they occasionally produce small wedge failures on 60° batter faces, large scale wedge failures have not formed due to the steepness of the potential intersection angle (46°) compared with the overall slope angle of 37°.

3 FLEXURAL TOPPLING HISTORY

Discontinuities in the west walls that strike semi-parallel to the pit walls and dip steeply (typically 60 – 85°) into the slope facilitate the development of toppling failures at Savage River. Most of the initial movement occurs on relatively thin foliation plane shears that are spaced 5 – 15 m apart.

The earliest such failure began in the west wall of Centre Pit in 1989, during excavation of a berm at the base of a slope that had been mined at about 45° over a vertical height of 107 m. Very slow horizontal movements of up to 13 m occurred. The slope was redesigned to an overall angle of 37° but failures became active again on several occasions as mining continued. Depressurisation of the slopes via horizontal drainholes and careful mining and monitoring eventually allowed all the ore to be extracted safely. Mapping of the final slope showed that left lateral vertical slip movement had occurred along discrete foliation shears, leading to development of reverse scarps up to 3-4 m high.

Similar failures at section N9400 in North Pit developed recently and have been managed in the same way. The geological profile was similar to that shown in Figure 1, and accelerated movement began when the 37° slope had been mined to a vertical height of about 90 m. Dewatering led to temporary stabilisation of the slope, but movements increased again when the slope was about 120 m high. In addition to reverse scarps along foliation shears, a large block at the top of the slope dropped about 0.6 m along a fault bounding the magnesite lens to the west. The area of toppling was bounded by two pervasive discontinuities of the previously mentioned SE and NE dipping discontinuity sets.

The influence of similarly oriented but less pervasive discontinuities and the strength of the more competent band of mylonite located at mid slope height was not known. A section further south, at N9250, which did not have these discontinuities, experienced a small planar slip / toppling failure at the top of the slope above the mylonite. This section was therefore analysed first, to obtain the minimum strength of the mylonite that prevented toppling in the modelled upper slope from progressing.

4 ROCK MASS MODEL

Recognition of the critical role of foliation shears in the development of a flexural toppling mechanism at the mine led to a decision to use the 2D distinct element program *UDEC* (Itasca 1996) to model the pit slopes at Savage River. This program allows shear and separation to occur on discrete discontinuities and also permits yield of rock block material, in shear or in tension, via a range of nonlinear constitutive models. In addition, it can perform effective stress analyses based on groundwater pressures acting within joints and within block material.

The 3D equivalent program was to be used if *UDEC* could not emulate potential effects of the two discontinuity sets. However the 2D simulation adequately modelled observed behaviour.

A number of foliation shears were to be represented as explicit joints, then the other joint sets could be simulated with the ubiquitous joint rock material constitutive model and/or by treating rock mass yield by using the modified Hoek-Brown yield criterion (Hoek et al. 1995):

$$\sigma_1 = \sigma_3 + \sigma_c(m_b\,\sigma_3/\sigma_c + s)^a \qquad (1)$$

where σ_1 and σ_3 are the major and minor principal stresses respectively (compressive stresses positive), σ_c is the uniaxial compressive strength of intact rock pieces, and m_b, s and a are rock mass parameters.

The Hoek-Brown parameters m_b, s and a for each material were computed from the equations of Hoek et al. (1995), using values of Geological Strength Index (GSI) estimated by relating field observations of each rock unit to Hoek's table (1997).

Itasca (1995) adapted the Mohr-Coulomb material model to represent the behaviour of uniform Hoek-Brown material with a=0.5. This was done by computing an effective cohesion and friction angle for each zone within the model from the tangent to (1) for the current confining stress, and then updating these parameters as the calculation progressed. Coulthard and Little (1999) generalised their code to treat the modified Hoek-Brown yield criterion and to handle multiple materials in *FLAC* or *UDEC* models. That work was generalised here to function with the ubiquitous joint constitutive model in *UDEC*.

Initial estimates of material parameters for the rock units in the *UDEC* models are given in Table 1. The GSI values and ubiquitous joint dips were obtained from field mapping whilst the density, elastic parameters and UCS values came from laboratory testing for the feasibility study (ABM 1996). Shear strengths were estimated from previous direct shear tests or from back analysis of previous toppling failures on the west wall of Centre Pit using a simple limiting equilibrium method. As shown in the Table, some of the parameters were modified slightly during the back analysis phase of this study.

Active discrete joints in the N9250 model (Figure 2) were those on either side of MXCc and a vertical fault behind the crest. These had cohesion of 50 kPa, friction angle 20° and zero tensile strength. All other block interfaces had high strength, so that any yield would develop through block material, on ubiquitous joints or via rock matrix shear or tensile yield.

Groundwater pressures at each stage were computed in *UDEC* from estimated phreatic surfaces that ware based on standpipe readings from the mine. These pressures were then used in effective stress analysis of the response of joints and block material.

Table 1. Rock material parameters for Savage River Mine *UDEC* models

Rock unit	Density (kg/m^3)	Young's modulus (GPa)	Poisson's ratio	GSI	UCS (MPa)	m_i	Ubiquitous joints			
							Dip (°)	Cohesion (kPa)	Friction angle (°)	Tensile strength (kPa)
MXR	2850	32	0.24	70	125	20	-	-	-	-
Fault1	2500	0.7	0.25	25	10	4	85 [90]	6	26	8
OH	3350	2.9	0.25	30	87	15	-	-	-	-
OS	3350	1.7	0.25	25	53 [20]	15 [25]	-	-	-	-
MXB	2830	2.8	0.25	40	25	8	55 (45)	6 (10)	26	3
MYF	2830	10.3	0.25	55	55	6	80 [75]	20	30	10
MXCa	2830	5.2	0.26	40 (50)	15 (25)	8	70 [80]	6 [50]	26	3
MXCb	2830	5.2	0.26	40 (50)	15 (25)	8	45	6 [50]	26	3
MXCc	2830	5.2	0.26	40 (50)	15 (25)	8	90 [75]	6 [50]	26	3
Magnesite	3000	22	0.25	60	100	20	-	-	-	-
Clay	2000	0.01	0.40	-	-	-	-	50	15	50

Note: Initial estimates for N9250 model given, with adjusted values from back analysis in parentheses. Final values for N9400 models, where different, are shown in square brackets. Strength parameters for clay are for isotropic Mohr-Coulomb model.

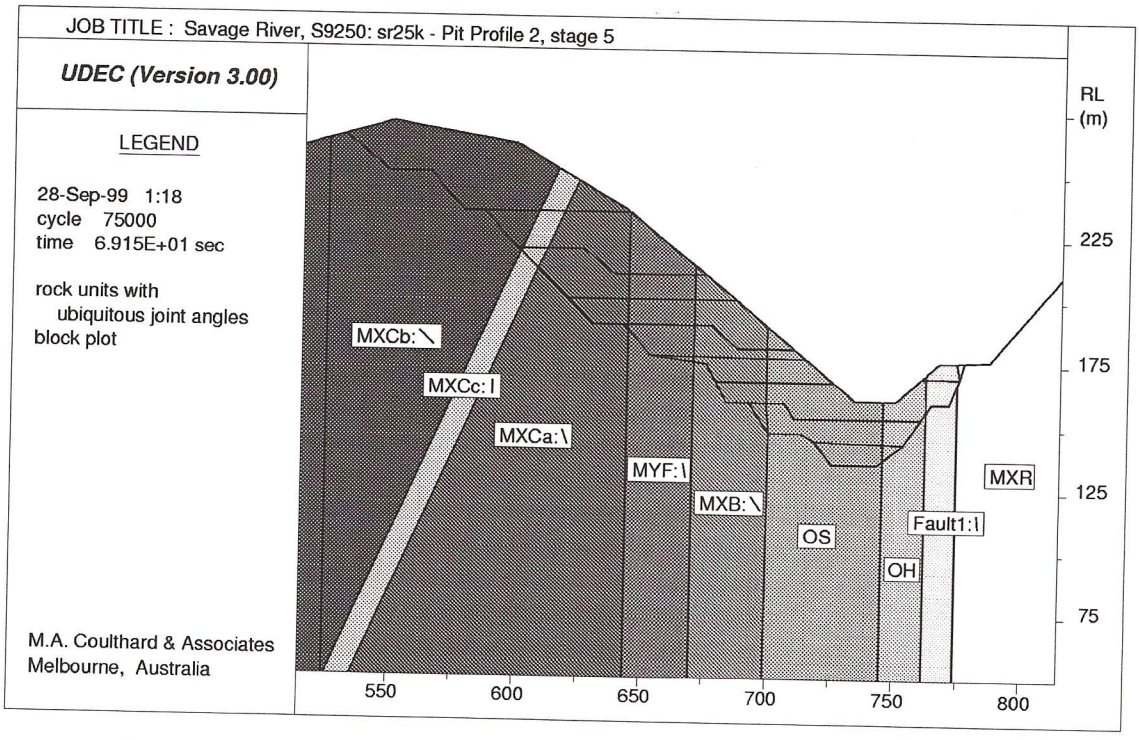

Figure 2. *UDEC* model of North Pit, section N9250. Rock units, with orientation of ubiquitous joints, where modelled, given by slope of lines after rock unit names. Block structure shows future excavation stages from February 1998 to April 1999 pit profiles.

5 NUMERICAL MODELLING

The combined planar slip / toppling failure that had been observed in the upper slopes at Section N9250 of North Pit (see section 3 above) was back analysed by comparison with field observations and trends from surface prism monitoring. The material properties were further adjusted by back analysis of the lower part of that pit slope. A model of the upper parts of the west wall of the pit at N9400 was then compared with observations, to verify the rock parameters before they were used for analyses of future mining options for the extension of North Pit.

5.1 *Back Analyses*

The *UDEC* model of section N9250 was mined in five relatively large lifts from the pre-mining ground profile to the February 1998 pit slope geometry. The rock units represented in that model are shown in Figure 2, which also shows the various excavation stages that were used to simulate mining through to the April 1999 profile. The observed planar failure occurred along 45° outward-dipping joints in the MXCb; these were modelled as ubiquitous joints.

With the rock properties as in Table 1 the upper slope failed even without groundwater pressures, which was clearly contrary to field observations.

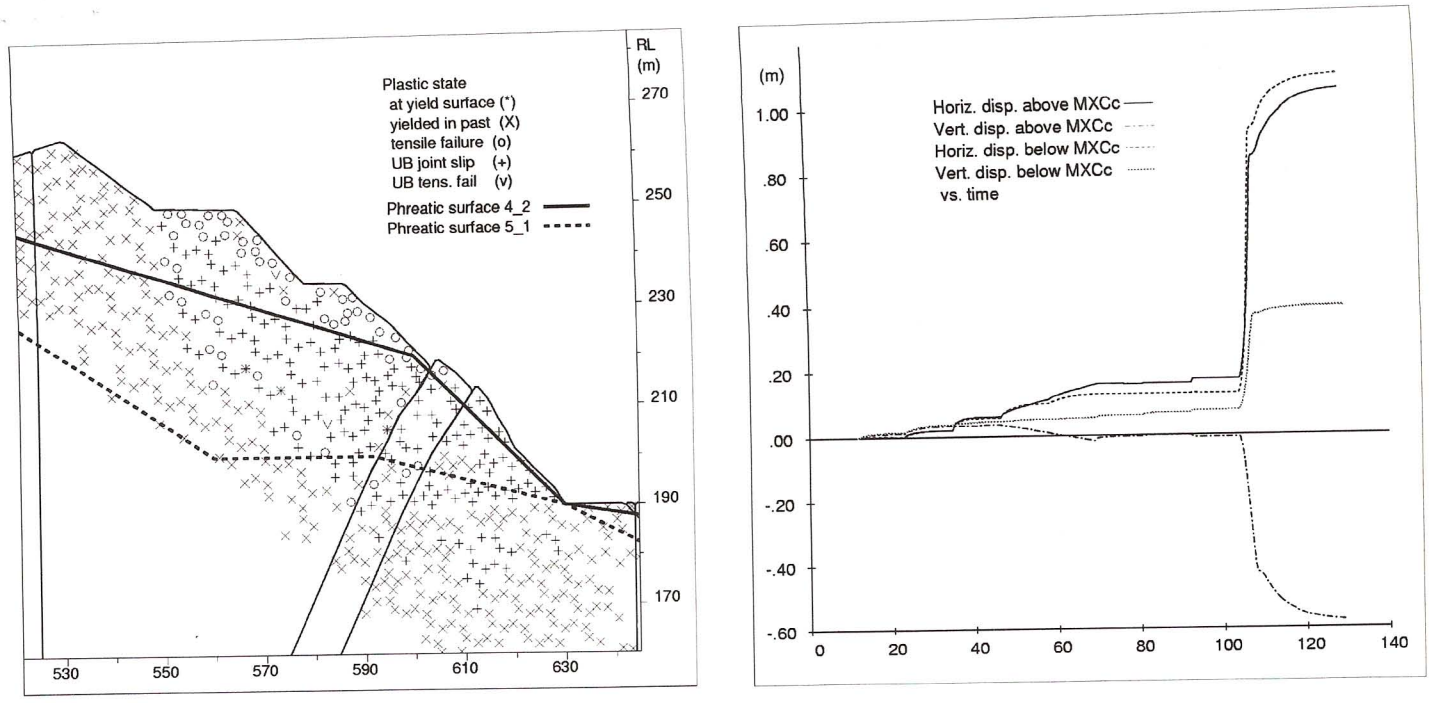

Figure 3. Combined planar slip / toppling failure in upper slope of North Pit, section N9250, at August 1998 stage of mining, stabilises when drainholes lower phreatic surface: left: deformed mesh, plastic yield and phreatic surfaces (heavy solid and dashed lines); right: histories of vertical and horizontal displacement of points on slope face above and below MXCc.

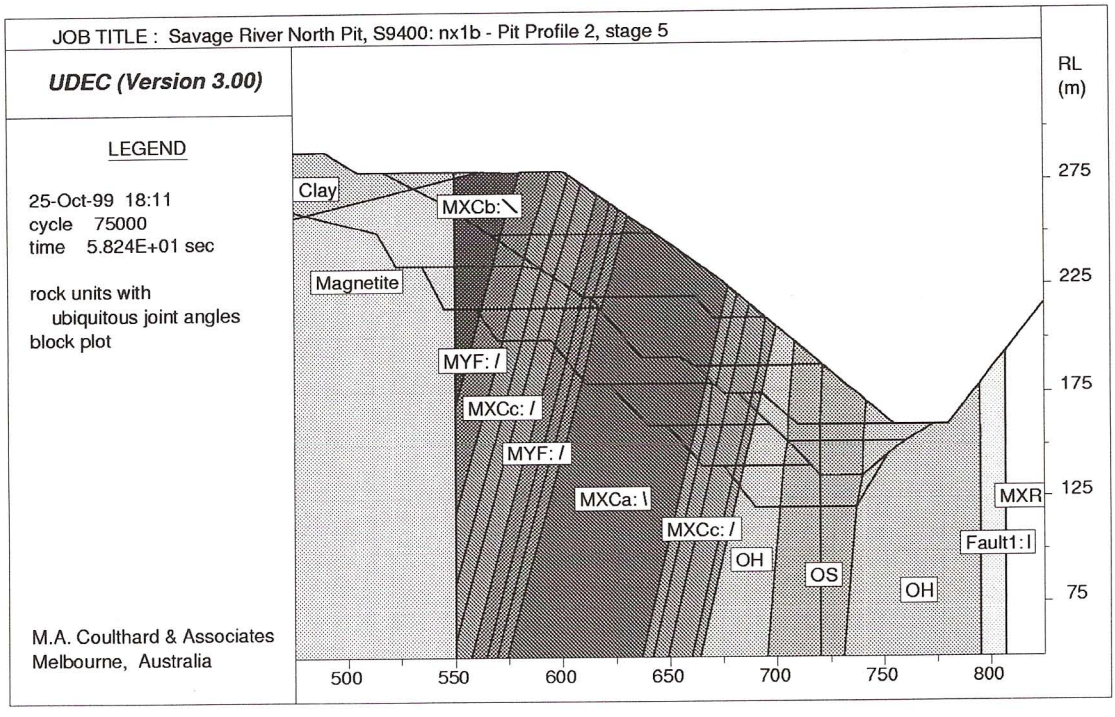

Figure 4. *UDEC* model of North Pit, section N9400. Rock units: ubiquitous joint orientations. Block structure showing future excavation stages from February 1998 to April 1999 pit profile and then on to one possible future pit configuration.

The strength of the mid-slope MXC units was increased until the upper slope was stable with no groundwater (to UCS = 25 MPa and GSI = 50). Next, that model was re-run with estimated phreatic surfaces at each stage. Major instability developed over the range RL 180 – 265 for the pit profile as at August 1998, but this stabilised as soon as a drawn-down phreatic surface was included in the model (Figure 3). Both these aspects were consistent with observations at the mine.

A localised slumping type of failure occurred over RL 150-175 m in the lower west wall of the actual pit. The ubiquitous joint orientation and cohesion of the relevant rock unit (MXB) were adjusted in the numerical model until a similar response was obtained. The final values used in MXB were ubiquitous joint dip of 45° and cohesion (ucoh) of 10 kPa, which produced a somewhat shallower failure than had been observed. With ucoh ≥ 20 kPa that part of the slope was predicted to be stable.

5.2 *Verification Analysis*

The back analysed material parameters were used in a similar model of section N9400 of North Pit (Figure 4). This model contained more discrete joints within the MXC units, to represent extra foliation shears mapped in the field. These joints, the vertical fault at the east boundary of magnetite and the sub-vertical joints in and at the boundaries of the OH and Fault1 units, were all given cohesion of 5 kPa, friction angle 20° and zero tensile strength. As shown in Table 1, some changes were made to the ubiquitous joint orientations in MXC, and the ubiquitous joint cohesion in MXC was increased, to encourage any shearing to localise on the explicit joints rather than being distributed through the rock mass. The clay was modelled as a simple Mohr-Coulomb material, i.e. no ubiquitous joints or Hoek-Brown criterion.

The mechanism here is slightly different in that the upper wedge of MXCb slips downward on the fault at the east of the magnetite unit and so exacerbates the tendency to flexural toppling in the upper foliated units (Figure 5). At later stages of mining slip on foliation shears and toppling begin to develop in the lower slope also, as observed. The response to groundwater drainage again matched well the site behaviour. However, the strength of unit OS had to be adjusted (see Table 1) before the relative movement on the upper and lower shears, originally computed as about 10:1, came closer to the observed ratio of 2:1 (although still computed as 5:1).

This was considered to be a sufficiently good representation of the observed slope response to form the basis for further models of possible future mining of that part of the pit.

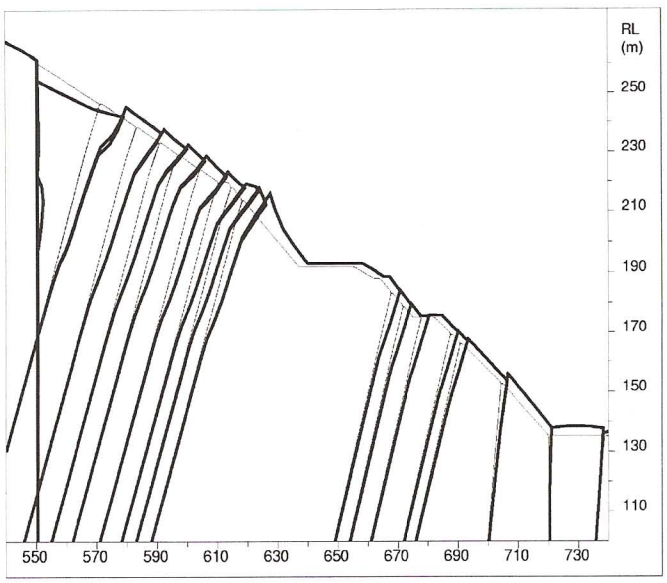

Figure 5. Sliding wedge and toppling mechanism of failure in west wall of North Pit, section N9400, at April 1999 stage of mining. Heavy lines show deformations magnified by factor of 10, with reverse scarps in upper and lower parts of slope.

5.3 *Assessment of Future Mining Options*

Two options for extended mining at section N9400 of North Pit were also analysed in *UDEC* models, using the same material parameters as previously:
Extension 1. incorporating a ramp in the west wall and the pit floor at RL120m; and
Extension 2. without a west wall ramp and with the east wall cut back and the pit floor at RL80m.

Each model was mined to the April 1999 level, then the extension of mining was represented by additional excavation stages. The stability of the west wall was assessed, with and without dewatering.

The Extension 1 model indicated that a wedge-toppling mechanism like that shown in Figure 5 would begin to develop but would probably remain stable without dewatering of the lower west wall. However some further dewatering was recommended to ensure slope stability.

With Extension 2, similar wedge-toppling failures developed at most of stages of mining. Dewatering stabilised the slope when the pit floor was at and above RL120, but movements continued slowly as later stages of the computation proceeded, even with extensive dewatering.

6 CONCLUSIONS

UDEC models of the response of the west wall of North Pit at sections N9250 and N9400 reproduced both the observed toppling failures and sensitivity of those failures to groundwater depressurisation. In each case the flexural toppling mechanism was driven by a wedge in the upper slope. At N9250 this "wedge" was formed by slip on the 45° dipping ubiquitous joints in unit MXCb, whilst at N9400 the predominant movement was slip on the vertical fault at the edge of the magnesite unit. The rock mass and joint properties derived from one back analysis were in close agreement with those from the other. This indicated that they provided a consistent model of this complex rock mass, which could then be used to predict the likely response to future mining options at the pit.

Although a similar mechanism begins to develop in the first mining extension option considered at N9400, the pit slope is likely to be stable without dewatering. In contrast, the steeper and deeper Extension 2 was found to be unstable even with extensive dewatering. However, the *UDEC* model and previous experience at the mine both suggest that these failures are likely to progress very slowly, particularly on the relatively flat slopes proposed, and give obvious early warning signs on the surface, such as reverse scarps. Consequently mining to this depth should be achievable with good on-site management, supplemented by monitoring.

ACKNOWLEDGEMENTS

The authors wish to thank Australian Bulk Minerals, the operators of Savage River Mine, for permission to publish this paper.

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