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CALIBRATED PARAMETERS FOR THE PREDICTION OF SUBSIDENCE AT MANDALONG MINE

Ross Seedsman

ABSTRACT: The consent conditions at Mandalong Mine require that subsidence deformations must not change the flood hazard category or subject a dwelling to deformation beyond safe surface and repairable (SSR) unless permission is granted by the effected landholder. The subsidence prediction in 2003 utilised an analysis of sag based on voussoir beams and of pillar compression based on foundation engineering principles. The model uncertainty for the sag analysis was assessed to be relatively high with a low parameter uncertainty, while for the pillar compression the model uncertainty was low but the parameter uncertainty was high. Up to June 2009, seven longwalls have been extracted. The consent conditions have not been breached. Both the voussoir beam and pillar compression models have been demonstrated to be valid. There have been changes in the way in which key input parameters are estimated.

INTRODUCTION

When Centennial Coal purchased the mine in 2003 the mine plan proposed panels of up to 250 m width and maximum subsidence of 2.98 m. Their review of the consent conditions raised concerns about risks to continuity of operations. The standard subsidence predictions methods available at the time indicated that panel widths of approximately 80 m would be required to bring the continued operations risk down to acceptable levels. Seedsman (2006) proposed an alternative prediction methodology that factored in the geotechnical conditions in the overburden and identified the likelihood that panels up to 175 m could be possible. The initial longwall panels were designed at 125 m and currently the panel width is 160 m. Whilst the panels are relatively narrow, the viability of the operation is underpinned by the thick seam extraction – up to 5 m.

The decision to start the mine with 125 m panels was based on the need to validate and calibrate the prediction methodology. A large number of survey lines have been monitored (Figure 1) and the data used to check key parts of the prediction. Figure 1 presents an interpretation of the subsidence bowls that was calculated using Surfer with an anisotropy factor of 3 aligned parallel to the panels. The maximum subsidence to the end of LW7 was about 1.2 m and this is located under the highest elevation which also corresponds to the greatest depth of cover of 360m. At the outbye ends of the panels (depths of about 160 m – 180 m) there are some variations to the overall patterns and these provide the basis for some of the discussion in this paper.

DESIGN IN 2003

Derivation of allowable subsidence

Currently, and also in 2003, the prediction of all surface subsidence deformations starts with a prediction of the vertical movement induced at the surface. The change in flood hazard category was relatively simple to define in terms of vertical subsidence (500 mm was selected as the maximum allowable).

The SSR criterion was not quantified in the consent conditions, and after an review of various reports and an inspection of the surface, the target values were set at 7 mm/m tilt and 4 mm/m strain. The step to vertical subsidence was still required. Noting that the panel width/depth ratios would be low, it was concluded that the K1, K2, and K3 curves (Holla, 1987) could not be used. Constant values of 0.65, 2.0 and 2.5 respectively were used for subsidence less than about 500 mm. It was assessed that a maximum vertical subsidence of 500 mm would apply at the SSR. Most of the dwellings are located on the flood plain so the vertical subsidence constraints applied simultaneously.

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1 Director, Seedsman Geotechnics Pty Ltd
Sag

The geotechnical model for the spanning of massive units is shown in Figure 2. The data input requirements for the model are:

- Panel width – the rib to rib distance of the extraction panel.
- Interburden distance – the distance from the roof of the seam to the base of the massive unit. This is determined from borehole data.
- Goaf angle – the angle by which the panel width is reduced at the base of the massive unit, and by which the surcharge is also reduced. The design utilised a 12° angle, as determined by a back analysis of other subsidence events in the coalfield (Seedsman 2004). A standard deviation of 8° was identified in the back analysis.
- The thickness of the massive unit. This was determined from the core and geophysical logs, based on the presence of a continuous coarse sandstone/conglomerate with no mudstone band thicker than about 10mm (these being interpreted to be mudstone pebbles).
- The surcharge on the beam, as given by the depth to the top of the massive unit
- Uniaxial compressive strength (67 MPa) and Youngs modulus (18.8 GPa) – laboratory values not corrected for the rock mass given the requirement for the unit to be a massive unit without discontinuities.
- The model assumes that the goaf below the spanning unit does not provide any support to the beam.

Figure 2 - Components of a model for assessing spanning

Pillar compression

The pillars were designed with factors of safety greater than unity, and greater than 2.23 under the flood plain. Pillar stress was estimated using a simple inverted pyramid model and a loading angle of 21°.

Pillar subsidence is a function of the stresses that are developed, the width of the pillar, and the deformation properties of the coal, roof and floor strata (Figure 3).

Figure 3 - Factors in pillar subsidence model

The compression of the pillar itself was calculated with simple elastic theory and a modulus of the coal being set at 1.5 GPa, this value being at the low end of the range for large sized coal samples quoted by Medhurst and Brown (1998). The compression of the roof and floor was assumed to be the result of
the settlement of a rigid footing (Poulos and Davis, 1976), with the roof modulus being assumed to 15 GPa. The modulus values were based on laboratory values as at the time (2003) there was no appropriate way to estimate the deformation modulus of ‘soft rock’ masses. At that time, the state of the art was the 1999 paper by Hoek and Brown that proposed that the modulus could be obtained from:

\[ E_{rm} (\text{GPa}) = \sqrt{\frac{\text{UCS}}{100}} \times 10^{\left(\frac{\text{GSI}-10}{40}\right)} \]

Where the GSI is the Geological Strength Index and the UCS is in units of MPa. This gives 125 GPa for an intact 50 MPa rock (compared to a typical laboratory value of 15 GPa). For a GSI of 50, a modulus of 22 GPa is obtained. For the floor the calculations were modified to account for the finite thickness and presumed drained modulus of low strength claystones of the Awaba Tuff.

**PROGRESSIVE IMPLEMENTATION**

In order to manage the risks inherent when introducing a new subsidence prediction method in a highly charged environment, a conservative strategy was recommended and adopted. Approval was sought for the first 2 longwalls, each 125 m wide with a 41 m chain pillar. The prediction for maximum vertical subsidence at the LW2 was 250 mm, 50% of what was believed to be the maximum allowable for SSR and flood damage. This was composed of 50mm of sag, an immediate pillar compression of 30 mm – 50 mm, and a longer term consolidation of the Awaba Tuff of about 150mm.

At the end of LW1 and prior to the extraction of LW2, when LW1 can be considered to be an isolated panel, the maximum subsidence in the inbye areas was 183 mm and outbye the maximum subsidence without fault influence was 70mm. At the end of LW2 in areas from known faulting, the maximum vertical subsidence recorded was 282 mm in the elevated ground and 160 mm under the flood plain. Table 1 compares the outcomes for LW1 and LW2 with the allowable levels interpreted from the consent conditions. It can be seen that the performance of the mine layout is well within the consent.

The behaviour around the thrust faults outbye was predicted but the location of the subsidence was about 100 m further outbye than predicted. The immediate pillar compression was higher than predicted but the longer-term compression did not develop. This result was not surprising given the recognition of the limitations in determining the deformation modulus values, and provided justification for the conservative implementation.

**Table 1 Performance against consent conditions for the 125m panels**

<table>
<thead>
<tr>
<th>Consent condition</th>
<th>Interpretation</th>
<th>Allowable</th>
<th>LW1 and LW2</th>
<th>End LW4</th>
</tr>
</thead>
<tbody>
<tr>
<td>SSR</td>
<td>Tilt at dwelling</td>
<td>5-7 mm/m</td>
<td>3.9 mm/m</td>
<td>2.6 mm/m</td>
</tr>
<tr>
<td></td>
<td>Tensile strains at dwelling</td>
<td>3-4 mm/m</td>
<td>0.8 mm/m</td>
<td>1.3 mm/m</td>
</tr>
<tr>
<td></td>
<td>Compressive strains at dwelling</td>
<td>3-4 mm/m</td>
<td>1.6 mm/m</td>
<td>1.8 mm/m</td>
</tr>
<tr>
<td>Flood category</td>
<td>Vertical subsidence under the flood</td>
<td>500 mm</td>
<td>160 mm</td>
<td>225 mm</td>
</tr>
<tr>
<td></td>
<td>plain</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Because of the timing of approvals, LW3 and LW 4 were also extracted at 125 m width. Up to LW4, the subsidence deformations had been less than the 500mm and SSR constraints set by Centennial (Figure 1), so the decision was made to increase the face width. LW5 onwards have been 160 m wide. At the end of LW7, the maximum subsidence is in the order of 1.2m (Figure 1).

The differences in the subsidence patterns for the shallow and deeper areas of the mine and with 125 m and 160 m wide panels are dramatic. In the deeper areas (Figure 5), the pillar compression component dominates and the sag between the panels is a secondary feature. In the shallow areas (Figure 6) the sag component dominates and the difference between the 125 m panels and the 160 m panels is clear.
As mining has progressed, the opportunity has been taken to progressively improve the predictions. The basic models of sag and pillar compression have remained unchanged, but there has been a change in the way some of the key parameters are estimated. The model uncertainty is now considered to be low, and the parameter uncertainty has reduced such that the mine operates much closer to the 500 mm allowable limit.

**Goaf angle**

During the retreat of LW5, greater than predicted subsidence developed in a restricted area. Both
inbye and outbye of this area, the vertical subsidence along the panel centreline was within the predicted range. A fully cored borehole was in close proximity and this showed that the interpretation of the conglomerate thickness was valid. Underground, the area coincided with a pronounced roll in the seam (Figure 4) which had already been implicated in a number of ground control difficulties – at the face the overburden was noted to cave more readily. There was only one other subsidence line that crossed the trend of the roll and with hindsight it was possible to identify some atypical deformations.

It is proposed that the roll is characterised by greater jointing in the overburden such that the goaf angle would be reduced. It is noted that the back analysis had indicated that the goaf angle varied between $-20^\circ$ and $+20^\circ$, with the $-20^\circ$ value being an outlier. Omitting the outlier, the average goaf angle was found to be $12^\circ$ with a standard deviation of $8^\circ$.

The impact of reducing the goaf angle is to increase the span at the base of the spanning unit. This may lead to increased deflection or in the worst case failure of the beam. The higher subsidence developed at a depth of cover of approximately 180 m and the beam thickness was confirmed to be 39 m. Figure 7 presents plots that show how the stability and deflection change progressively. Note that for typical conditions, this change of goaf angle represents an increase in effective span at the base of the conglomerate of about 40 m.

![Figure 7 - Stability and deflections changes with reducing goaf angle](image)

It is interesting to note that the author has applied the model to other coal fields and has found that goaf angles of $20^\circ$-$25^\circ$ may apply to longwall layouts that are aligned at much higher angles (say 40-45$^\circ$) to the dominant joint direction. At Mandalong the orientation is within 10$^\circ$.

**Rock mass deformation modulus**

After LW2 it was noted that the immediate pillar compression was much higher than anticipated and there were no signs of further movements that had been suspected due to the consolidation the Awaba Tuff. The total deformation was within the anticipated range.

In 2006, 3 years after the initial designs, a method for the estimation of the rock mass modulus based on the reduction of laboratory values was published. Reducing laboratory modulus values to represent field behaviour is standard practice in rock engineering. Hoek and Diederichs (2006) provide the following equation to estimate the deformation modulus of rock masses from the laboratory values (Ei):

$$E_{rm} = E_i \left(0.02 + \frac{1 - \frac{D}{2}}{1 + e^{\left(\frac{60 + 15D - GSI}{11}\right)}}\right)$$

Where D is a disturbance factor to account for excavation blasting damage (set at 0 for this application).

The reduction factor is of an S shape with little change in modulus for very blocky rock masses (such as the Mandalong Conglomerate) and large changes for rock masses with intermediate values of GSI.
(bedded and laminated roof and floor strata). Note that a change in GSI of 4 units can, for intermediate values of the GSI, lead to a change of 10% in the rock mass modulus.

For the Mandalong project, allocation of GSI values has been based on coal joints being rough and stone joints being smooth, and the West Wallarah coal and the roof sandstone being considered blocky and the other materials being very blocky. It is noted that these selections are based, in part, on a calibration to the subsidence outcomes to date. GSI values and rock mass deformation moduli for the key materials in the design are presented in Table 2.

<table>
<thead>
<tr>
<th>Material</th>
<th>Ei</th>
<th>RSI</th>
<th>Erm</th>
</tr>
</thead>
<tbody>
<tr>
<td>West Wallarah Seam</td>
<td>3</td>
<td>60</td>
<td>1.5</td>
</tr>
<tr>
<td>Fassifern and Pilot Seams</td>
<td>2</td>
<td>45</td>
<td>0.4</td>
</tr>
<tr>
<td>Floor stone</td>
<td>10</td>
<td>43</td>
<td>1.8</td>
</tr>
<tr>
<td>Roof sandstone</td>
<td>15</td>
<td>60</td>
<td>7.5</td>
</tr>
<tr>
<td>Roof mudstones</td>
<td>15</td>
<td>49</td>
<td>4</td>
</tr>
<tr>
<td>Mandalong Conglomerate</td>
<td>22.8</td>
<td>95</td>
<td>22.8</td>
</tr>
</tbody>
</table>

**CONCLUSIONS**

The prediction methods have performed well and the outcomes are consistent with the consent conditions. The engineering behaviour models on which the predictions are based are well established and details of the various calculations involved can be readily found in the engineering literature. Mandalong has provided a well documented case study their application.

The approach to subsidence prediction used at Mandalong can be readily transferred to other coal fields. Early recognition of the spanning capability of thick beams came from work on the Bulgo Sandstone in the Southern Coalfield. The author has applied voussoir beam theory to the Triassic sandstone in the Western Coalfield and also the Tertiary basalts in the Bowen Basin. In the Southern coalfield, mine design usually incorporates the onset of pillar yield at the tailgate and hence failure when fully goafed. There is a need to incorporate the post failure deformation of the pillars in the pillar compression calculation.

**REFERENCES**


