Numerical Modelling of Conventional and Pre-Tensioned Roof Bolting Methods in an Underground Coal Mine

A. Jafari and V. S. Vutukuri

Department of Mining Engineering, University of Tehran, P. O. Box 11365 - 4563, Tehran, Iran
School of Mining Engineering, University of NSW, Sydney 2052, Australia

Abstract: In underground coal mines the rate of roadway drivage is of great importance. This can be affected by the method of roof support and the time spent for erecting supports. This paper presents a computer study on the effect of applying high pre-tension to roof bolts at installation. The results of modelling are compared with available field data. Roof stability improves by applying high pre-tension to the bolts.

Key Words: roof bolting, pre-tension bolts, computer modelling, roof stability

Introduction

Computer based numerical methods have been widely used to estimate stresses and movements of strata surrounding openings in underground mines. However, great caution must be taken in the implementation of any computer program so that the most suitable method of modelling for any given case of study can be selected. Results should be carefully interpreted, and where possible, compared and verified with field test data. Bearing these facts in mind numerical modelling can generally be a means to reduce the number of required field tests and to estimate those parameters of rock which might be difficult or impractical to measure directly.

In this paper conventional and high pre-tensioned roof bolting methods are modelled using numerical methods. The effect of an unsupported span is also assessed by modelling. Results are described and compared with field measurement data where available.

Model Used for Numerical Study

Any numerical model to be used for simulating headings in a coal mine which uses roof bolting as a means for roof support must be able to:

(i) Simulate the rock using an appropriate constitutive model.
(ii) Simulate bedding planes and joints, and
(iii) Simulate rock bolts.

A two dimensional explicit finite difference code, FLAC, (Cundall and Board. 1988) which meets the above conditions was used to simulate the behaviour of underground structures using different bolting methods. The program FLAC (Fast Lagrangian Analysis of Continua) is designed with particular reference to geotechnical problems and can consider most of the requirements for this study.

Since FLAC is a two dimensional code all modelings were carried out on a vertical section. A plane strain condition was assumed in the models.

Rock Mass Modelling

Since the rock mass surrounding an opening in a coal mine has plastic behaviour the Mohr-Coulomb plasticity model was selected to represent the rock behaviour in this
study. In this model, material may yield in shear and is considered to respond in elastic-perfectly plastic manner. The shear yield function \( f \) is given by:

\[
\beta = \sigma_1 - \sigma_3 N_0 + 2C\sqrt{N_0}
\]

(1)

where \( N_0 = \frac{(1 + \sin \phi)}{(1 - \sin \phi)} \);

\( \sigma_1 \) = Major principal stress;

\( \sigma_3 \) = Minor principal stress;

\( C \) = Cohesion; and

\( \phi \) = Friction angle.

Compressive stresses are considered negative and \( \sigma_1 \) is the major principal stress in tension positive conventional system.

Yield at each node is detected if \( f < 0 \), and the program will allow plastic flow to occur to make the condition of \( f = 0 \) valid (\( f > 0 \) in elastic range). For this purpose a non associated flow rule is used which is given by:

\[
\Delta e_{i}^p = \lambda \frac{\partial g}{\partial \sigma_1}
\]

(2)

where \( \Delta e_{i}^p \) = plastic strain;

\( \lambda \) = 1.3;

\( g \) = Plastic potential for shear yielding.

"g" is given by

\[
g = \sigma_1 - \sigma_3 N_0 + 2C\sqrt{N_0}
\]

(3)

where \( N_p = \frac{(1 + \sin \psi)}{(1 - \sin \psi)} \); and

\( \psi \) = Dilation angle.

It can be shown that \( \lambda \) is given by

\[
\lambda = \frac{f_1}{A - BN_0 - BN_p + AN_p N_0}
\]

(4)

where \( A = K + 4G/3 \);

\( B = K - 2G/3 \);

\( f_1 \) = Function \( f \) evaluated for the initial trial stresses;

\( K \) = Bulk modulus of material; and

\( G \) = Shear modulus of material.

Using Eqs. 2 and 4 the parameter \( \lambda \) and flow rule can be found. It can be shown that to restore the state of stresses back to the yield surface, the following equations give the corrected principal stresses:

\[
\sigma'_1 = \sigma_1^1 - \lambda (A - BN_0)
\]

(5)

\[
\sigma'_2 = \sigma_2^1 = B\lambda (1 - N_0)
\]

\[
\sigma'_3 = \sigma_3^1 - \lambda (B - A N_0)
\]

where \( \sigma'_1, \sigma'_2, \sigma'_3 \) = Corrected principal stresses; and

\( \sigma_1^1, \sigma_2^1, \sigma_3^1 \) = Initial principal stresses.
Modelling of Bedding Planes

Bedding planes are usually the area of weakness within the rock mass. Shearing along bedding planes and their failing in tension, i.e. separation of beds is expected in a bedded roof. Therefore such planes of weakness should be simulated in a different way from the rock mass.

Bedding planes in this study are simulated by interface between two sets of elements which represent two rock beds.

Fig. 1 shows an interface element representing a bedding plane. The interface is an element with certain shear and normal stiffnesses. Each node along the bedding plane is checked for contact with its neighbouring points on the opposite side of the interface. If contact is found a length, L, equal to the sum of half the distance from nodes on both sides is assigned to that node. For example in Fig. 1 L-N is the length assigned to node N which is from the midpoint of NP up to the midpoint of MN. So the entire length of bedding plane is divided into contiguous segments, each controlled by one node.

After each time step of solution the corrected normal and shear forces, \( F_n \) and \( F_s \), at each point along the interface are calculated using the incremental relative displacement vectors at the contact points as follows:

\[
\begin{align*}
F_n^{(1+M)} &= F_n^{(1)} - K_n \Delta U_n^{(1+(1/2)M)}L \\
F_s^{(1+M)} &= F_s^{(1)} - K_s \Delta U_s^{(1+(1/2)M)}L
\end{align*}
\]

where
\( F_n^{(1+M)}, F_s^{(1+M)} \) = Corrected normal and shear forces;
\( F_n^{(1)}, F_s^{(1)} \) = Initial normal and shear forces;
\( K_n, K_s \) = Normal and shear stiffnesses of bedding planes;
L = Length associated with any node; and
\( t \) = Time.

![Diagram of an interface element representing shear and normal stiffnesses of bedding plane](image)

Fig. 1 An interface element representing shear and normal stiffnesses of bedding plane (after Itasca Consulting Group Inc., 1991)

Normal and shear forces are then checked for yield using the Coulomb shear strength criterion which is given by:
\[ F_{\text{Smax}} = C \cdot L + F_{N \cdot \tan \phi} \]  
(7)

where \( C \) = Cohesion along the interface; 
\( L \) = Effective contact length for the point of interest; and 
\( \phi \) = Friction angle of interface.

If the absolute value of shear force is found to be higher than the maximum shear strength, i.e. \( |F_s| > F_{\text{Smax}} \) then the value of \( F_{\text{Smax}} \) is given to \( F_s \) (original sign of shear is preserved). If tension across the interface exceeds its tensile strength, then the interface breaks and both normal and shear forces are set to zero.

**Modelling of Roof Bolts**

Roof bolts are modelled by one dimensional axial elements which can undergo axial tension. Compression is not allowed in this element.

In a two dimensional computer model bolts are inevitably assumed as a continuous sheet of metal along the opening, i.e. the spacing between rows of bolts is neglected. This discrete effect of bolts over the distance between rows of bolts can be simulated by linear scaling of material properties (Itasca Consulting Group Inc., 1991).

A roof bolt element may be defined inside the model and will automatically interact with surrounding materials when it is connected to common nodes with other elements. Fig. 2a shows the element used for modelling of a bolt and surrounding grout. Relative displacement of the nodes along the bolt element is computed which yields the axial force developed along the bolt.

Shear forces along the grout (grout-rock interface and grout-bolt interface) are also calculated based on relative nodal displacement. Fig. 2b shows the axial and shear behaviour of the bolting system. Out of balance forces at each point are computed from axial force in the reinforcement element and shear force along the grouted section. A one dimensional constitutive model is used for describing the axial behaviour of a bolt element and is given by:

\[ \Delta F_s = \frac{EA}{I} \cdot \Delta U_s \]  
(8)

where \( E \) = Young’s modulus of bolt; 
\( A \) = Cross-sectional area of bolt; 
\( I \) = Length of bolt between two nodal points; and 
\( \Delta U_s = (U_x^{(b)} - U_x^{(a)}) t_1 + (U_y^{(b)} - U_y^{(a)}) t_2 \)

and \( U_x^{(a)} \), \( U_x^{(b)} \) are displacement of nodal points (two ends of bolts) etc. \( t_1 \) and \( t_2 \) are direction cosines with reference to bolt axial direction.

**Specification of Model used for Simulation**

A model was constructed using 2354 two dimensional elements. Model was limited to strata in roof, seam and floor only at one (right hand) side of centre line of heading as this line is an axis of symmetry. Seven bedding planes were introduced in the bolted roof horizon. Thickness of each bed was 0.3 m. No bedding plane was introduced in the floor as use of a large number of interface elements can cause error in other calculations. Thus, true floor displacements were not calculated. However a trial run of the model showed that if bedding planes are incorporated in floor as well as roof, calculated floor displacements are close to field measured values, but roof displacements remain unchanged.
Fig. 2  (a) Element used for reinforcement modelling which accounts for shear behaviour of grout annulus.  (b) Axial and shear behaviours of bolt element (after Brady and Lorig, 1988)

The model was 25 m wide and 30 m long. These dimensions ensured that boundaries of the model are far enough from the excavation and it can be assumed that it remains unaffected by drivage of other roadways and also initial stress values at boundaries remain unchanged. A 5 m wide and 2.2 m high opening was introduced. Mesh dimension is increased horizontally by factor of 1.2 as one moves away from the opening. This could allow use of a limited number of the elements to cover a larger area. The mesh used in the modelling of two bolting methods is shown in Fig. 3.

Four, 2.1 m long roof bolts were installed at 0.35 m, 1.25 m, 2.0 m and 2.5 m from the centre line with two near the rib being inclined. All bolts were fully encapsulated. Bolt pre-tensioning effect was simulated by compressing the bolted zone by equal forces but opposite in direction applied to top and bottom of the bolted zone.

Input Parameters for the Model

The material properties for roof and floor rocks and coal seam used in the modelling were taken from previous laboratory tests of samples of KCC’s mines and are given in Table 1.

Roof bolts input parameters are provided by the bolt manufacturer (Table 2). The bond stiffness and bond strength of grout were determined by pull out tests conducted at Tahmoor Mine, NSW (Lama and Jafari, 1994) and are given in Table 2. Properties of the bedding planes used in the modelling are given in Table 3.
Fig. 3 The mesh used in the modelling of two roof bolting methods

Initial Stress Conditions

Two sets of in situ stress measurement tests, using CSIRO hollow inclusion cell, were conducted in Tahmoor Mine, in an area where field experiments were performed. The mean values are used for purpose of this study and are given in Table 4.

Boundary Conditions

All nodes in the model along the top and bottom boundaries were assigned zero velocities in vertical direction, allowing them to move only horizontally as they are far enough from the excavation to be considered unaffected. Similarly all the nodes at left and right boundaries were assigned zero velocities in horizontal direction, allowing them to move only vertically. This also simulates vertical symmetric conditions about centre line of the heading.

Table 1 Material properties used in the modelling (after Lam and Shu, 1989)

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Density, kg/m³</th>
<th>UCS, MPa</th>
<th>Tensile strength, MPa</th>
<th>Young’s modulus, MPa</th>
<th>Poisson’s ratio</th>
<th>Cohesion, MPa</th>
<th>Angle of friction, degree</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>2530</td>
<td>94</td>
<td>8</td>
<td>18698</td>
<td>0.23</td>
<td>14</td>
<td>45</td>
</tr>
<tr>
<td>Laminite</td>
<td>2565</td>
<td>77</td>
<td>6.7</td>
<td>13402</td>
<td>0.23</td>
<td>5</td>
<td>43</td>
</tr>
<tr>
<td>Both coal</td>
<td>2392</td>
<td>22</td>
<td>0.9</td>
<td>3750</td>
<td>0.22</td>
<td>4</td>
<td>32</td>
</tr>
<tr>
<td>Floor rock</td>
<td>2530</td>
<td>67</td>
<td>7.6</td>
<td>12849</td>
<td>0.2</td>
<td>13</td>
<td>40</td>
</tr>
</tbody>
</table>

Table 2 Bolt and bond parameters used in the modelling

<table>
<thead>
<tr>
<th>Young’s modulus, GPa</th>
<th>Diameter, mm</th>
<th>Yield load, kN</th>
<th>Bond stiffness, MN/m/m</th>
<th>Bond strength, kN/m/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>20t</td>
<td>21.7</td>
<td>230</td>
<td>0.0-0.7m 0.7-1.4m 1.4-2.1m</td>
<td>0.0-0.7m 0.7-1.4m 1.4-2.1m</td>
</tr>
<tr>
<td>20t</td>
<td>21.7</td>
<td>230</td>
<td>0.0-0.7m 0.7-1.4m 1.4-2.1m</td>
<td>0.0-0.7m 0.7-1.4m 1.4-2.1m</td>
</tr>
</tbody>
</table>

992
Table 3 Properties of the bedding planes used in the modelling (after ACIIRL, 1992)

<table>
<thead>
<tr>
<th>Normal stiffness, GPa</th>
<th>Shear stiffness, GPa</th>
<th>Cohesion, GPa</th>
<th>Angle of friction, degree</th>
<th>Tensile strength, GPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.1</td>
<td>0</td>
<td>15</td>
<td>0</td>
</tr>
</tbody>
</table>

Table 4 Stress values used in the numerical modelling (after Jafari, 1994)

<table>
<thead>
<tr>
<th>$\sigma_x$ (horizontal), MPa</th>
<th>$\sigma_y$ (vertical), MPa</th>
<th>$\sigma_z$ (horizontal), MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.15</td>
<td>10.0</td>
<td>15.35</td>
</tr>
</tbody>
</table>

Results of Modelling and Comparison with Field Data

The model was analysed to study the behaviour of strata and support system in the following cases: Conventional roof bolting method (case 1) and High pre-tensioned roof bolting method (case 2). The history of movement at selected points was recorded during the analysis as model was solved by time stepping towards the equilibrium. This history can not be compared with actual roof displacement curves as the step of solution can not be related to real time. However, the final values of displacement can be expected to represent the ultimate real displacements measured in the field.

The following parameters were examined in each of the above mentioned cases: roof and rib displacement at centre, bolt load and shear along bedding planes.

Roof and Rib Displacement

History of roof displacement at centre is shown in Fig. 4. The maximum roof displacement at the centre of the heading is 122.4 mm and 72.4 mm for cases 1 and 2 respectively. Displacements calculated by modelling and actual measured values in the field are given in Table 5. Actual measured roof displacement in conventional method (Lama and Jafari, 1994) is lower by 26% when compared with results of modelling. No attempt was made to adjust the roof displacement by changing the material properties. This was mainly due to two reasons: (i) The primary object of computer modelling was to determine the effect of pre-tensioning and compare two methods of roof bolting. (ii) The real value of roof displacement in the field could be slightly higher due to possibility of some roof displacement prior to installation of extensometers and/or displacement above the measurement horizon.

Table 5 Displacements calculated by modelling and actual measured values in the field for two methods of roof bolting

<table>
<thead>
<tr>
<th>Type of Measurement</th>
<th>Root displacement at centre, mm</th>
<th>Rib displacement at centre, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Results of modelling</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Conventional bolting method</td>
<td>122.4</td>
<td>18.9</td>
</tr>
<tr>
<td>Pre-tensioned bolting method</td>
<td>72.4</td>
<td>18.4</td>
</tr>
<tr>
<td>Reduction</td>
<td>41%</td>
<td>2.6%</td>
</tr>
<tr>
<td>Mean field measurement</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Conventional bolting method</td>
<td>90.5</td>
<td>Not measured</td>
</tr>
<tr>
<td>Pre-tensioned bolting method</td>
<td>44.1</td>
<td>Not measured</td>
</tr>
<tr>
<td>Reduction</td>
<td>51%</td>
<td></td>
</tr>
</tbody>
</table>

Results of modelling clearly show that roof displacement is reduced by 41% in pre-tensioned bolting method. This reduction is lower than the value found at field (51%). Surely the accuracy of the modelling is controlled by the accuracy of input data as well as the presence of all structures such as bedding planes, joints, etc. in the model.
Fig. 4 History of roof displacement at centre (a) conventional roof bolting method, (b) high pre-tensioned roof bolting method
The final movement of rib is 18.9 mm and 18.4 mm for cases 1 and 2 respectively. This suggests that rib displacement in the model is not affected by method of roof support. It also shows that displacement of rib is small which is similar to results of previous measurements conducted at North Cliff Mine.

**Bolt Load**

The final load in the bolts are determined by the program in 3 sections of bolts. Average load along the bolts is shown in Table 6 for two cases. Bolts are numbered from centre line of the heading to rib side. Mean load for all bolts is 164 kN and 130 kN for cases 1 and 2. This shows that greater load is developed in the bolts in the conventional method of roof bolting. The absolute bolt load predicted by the model is lower than maximum load measured in the field. This can be due to the existence of more or weaker bedding planes in the field which results in greater separation of beds and hence induces greater load in the bolts.

**Shear along the Bedding Planes**

Shear stresses along the bedding planes are also computed in the two models. Results showed that different beds in the bolted horizon of roof in pre-tensioned bolting were compacted and higher shear resistance along the bedding planes was produced which led to less bed separation. Shear stress in the first layer of roof is examined. In this layer mean shear stress along the upper side of bedding plane is 0.14 MPa in conventional bolting. This value for case of pre-tensioned bolting is 0.24 MPa. Shear stress is set to zero at the points of detachment by the program. Roof has detached along this bedding plane above the opening at 9 nodes for former case and at 3 nodes for latter case. Lower value of the shear stresses in case 1 is an indication of more failure along the bedding plane which is also clear from extent of detachment.

**Table 6** Average load along the bolts in conventional and high pre-tensioned roof bolting calculated by the model

<table>
<thead>
<tr>
<th>Bolt</th>
<th>Conventional bolting method</th>
<th>Pre-tensioned bolting method</th>
</tr>
</thead>
<tbody>
<tr>
<td>No.</td>
<td>184</td>
<td>178</td>
</tr>
<tr>
<td>1</td>
<td>206</td>
<td>173</td>
</tr>
<tr>
<td>2</td>
<td>175</td>
<td>107</td>
</tr>
<tr>
<td>3</td>
<td>92</td>
<td>61</td>
</tr>
</tbody>
</table>

**Effect of Roof Relaxation on Support Performance**

Roof relaxation resulting from large unsupported span has been found to be essential in obtaining better roof conditions and lower load on support system (Jafari, 1994). Advance of face and roof relaxation could not be directly modelled by a two dimensional program. However this, to some extent, can be done by allowing certain displacement in roof prior to introduction of support elements to the model. Based upon this procedure and using data obtained in the field model was solved without any bolt for 150 time steps, so that 10 mm roof displacement could happen. Then roof bolt elements were introduced and model was solved similar to case 1. Similar procedure was repeated for pre-tensioned bolting. Table 7 shows effect of roof relaxation on bolt load and roof displacement for two cases modelled, together with results of field measurement. Results showed that in the case of roof relaxation prior to bolt installation while ultimate roof displacement is slightly lower, bolt load significantly decreases. In both methods load in bolts decreased by 21%. Reduction of bolt load is quite in agreement with field observations (Lama and Jafari, 1994), where effect of large unsupported span was examined.
Table 7 Effect of roof relaxation on bolt load and roof displacement, results of modelling and field measurements

<table>
<thead>
<tr>
<th>Measured parameter</th>
<th>Results of Modelling</th>
<th>Mean field measurements</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Conventional bolting method</td>
<td>High pre-tensioned bolting method</td>
</tr>
<tr>
<td></td>
<td>Without relaxation</td>
<td>With relaxation</td>
</tr>
<tr>
<td>Average bolt load, kN</td>
<td>164</td>
<td>145</td>
</tr>
<tr>
<td>Roof displ. at centre, mm</td>
<td>122.4</td>
<td>120.8</td>
</tr>
</tbody>
</table>

Conclusions

A two dimensional finite difference program was used to model conventional and high pre-tensioned roof bolting methods. Certain simplification was involved in the modelling as the analysis was done in two dimensions which ignores some roof bolting parameters and also due to nature of the program itself (limitation in number of joints and bedding planes that can be used, etc.). Results qualitatively are in agreement with field observations (Tables 5 and 7). Roof displacement decreased by 41% in pre-tensioned bolting method when compared with conventional method. However pre-tensioning of bolts does not affect rib displacement. Ultimate load in the bolts is lower in case of pre-tensioned bolting. Relaxation of roof has also significant influence on load developed in the bolts.

This study showed that pre-tensioned roof bolting can significantly enhance roof support system and improve roof stability.

References


