Rigid or Yielding Roof Bolts: At the Face or away from the Face

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ABSTRACT

The concept of rigid or yielding bolts is discussed based upon support requirements for excavations of equivalent geomechanic behaviour. The concept of equivalent geomechanical behaviour is introduced for comparison purpose where the rock mass properties are varied for a given depth (stress) so that the deformation of the roof without any support remains constant ~ 300mm. A simple experimental theoretical analysis for a uniform stress field acting on an axisymmetric excavation is conducted to establish roof bolt requirements under different conditions such as width of a given excavation, time of setting of support etc. Results are presented in graphical forms.

INTRODUCTION

Bolting is the major method of roof support in coal mines in Australia, USA and South Africa. This method has been very effective because of the specific conditions occurring in the mines, particularly low - medium depth and comparatively stronger roof strata. The speed of erection of roof bolting as a support system also permits high rates of advance of headings. At the prevailing depths in these countries (rarely exceeding 500m), this method of support has been effective in retreat systems of longwall mining.

The effectiveness of this method is based upon providing active support systems whereby the immediate roof is strengthened as a result of formation of a competent beam and/or reduction in zone of failure due to reaction of anchors. This results in lower deformation and thereby provides resistance to further deformation of the rocks away from the immediate zone of disturbance and provides stability.

A number of theories have been developed for the calculation of bolting requirements for laminated and non-laminated strata (Parec, 1984). Theories are based upon suspension and friction effects of reinforcement.

In actual practice, conditions are quite different and these simplified theories are not applicable. The design of roof bolting systems is based upon experimental observations using information based upon bolt loads, roof displacement and roof failure (Gale and Fabiszek, 1988, 1989).

The prevailing concept is to install roof support as early as possible and use as strong bolts as possible to maintain the integrity of the roof. As a result, developments in roof bolting have been directed to achieve greater and greater rigidity, higher and higher strength, and greater density of bolts. In deeper mines the effect of this concept is that the support must be installed immediately at the face, resulting in the use of support densities approaching 3 high strength rigid bolts per metre square.

With increased depths or high stress even this density of support does not provide reliable roof control. Failure of roof bolts occurs with excessive deformation particularly when rigid anchors are installed close to the face. The reason being that the deformation of rock is much higher than the deformation the roof bolt can undergo before failure. Because, these days, chemical anchors are most common, these undergo failure either due to debonding between rock and chemical interface, the chemical and steel interface or the failure of the steel bolt. This situation often occurs due to high deformation caused in the bolt if the point of installation is close to the face or under high stress conditions.

The concept of yielding bolts therefore has been developed and is being applied in actual practice. As a result a number of yielding bolts such as swellies, split sets etc have been developed and used.

It is therefore important to recognise the behaviour of the rock and design a roof bolting system to match rock deformation. A rigid roof bolt installed too early would be heavily loaded and may become ineffective while yielding bolt may continue to deform providing effective support.

DEFORMATION AND FRACTURE AROUND AN EXCAVATION

When a rock specimen is tested in a stiff testing machine, the deformational behaviour of a rock specimen can be represented generally by a shape of the curve shown in Fig 1. The shape of the deformation curve, the boundaries (range) of the various zone and the properties of each zone, depend upon the size and shape of the specimen, type of rock and lateral stress (δy). On drainage of an excavation, rock surrounding the excavations is stressed and deforms in the same fashion resulting in the formation of different zones which correspond to the deformational behaviour of the rock specimen as observed in the laboratory.

Let us take the case of an axisymmetric excavations, and examine stresses, displacements, formation of different zones around it under the effect of a uniform stress field and the effect of interaction of the support system. Fig 2 shows development of different zones around a circular excavation. These zones are analogous to the deformational behaviour of rock shown in Fig 1. Zone A₁ is the fracture zone, where the rock has undergone extensive displacement. The residual strength of this zone is basically due to the movement of broken fragments as a result of internal friction and the constraints provided by the support system. Zone A₂ is the zone where fracturing has occurred and increased deformation results in a drop in strength. Zone A₃ represents the zone of peak strength of the material, where an increase in deformation does not influence the strength. Zone B is the elastic zone where the stress acting is below the peak strength of the rock. As such, Zone A₁ corresponds to the residual strength of the rock; Zone A₂ is the zone of decreasing strength (strain softening); Zone A₃ is the zone of plastic deformation (limiting strength) and Zone B is the zone of visco-elastic deformation.

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Fig. 1. Pre and post failure behaviour of rocks

Fig. 2. Calculation scheme of the problem
Naturally, laboratory strength data obtained on small rock samples, cannot be directly transferred to the field values for the above zones. Empirical relationships have been developed for the rock mass strength. The traditional Mohr-Coulomb strength criterion can be represented by:

$$\text{UCS}_{\text{m}} = \sigma_1 - \sigma_3 \beta$$  \(\text{(1)}\)

$$\beta = \frac{1 + \sin \phi}{1 - \sin \phi}$$  \(\text{(2)}\)

Where

- \(\sigma_1, \sigma_3\) = Principal stress
- \(\phi\) = Angle of internal friction
- \(\text{UCS}_{\text{m}}\) = Uniaxial compressive strength of rock mass

The application of this criterion for time dependent failure can be done by representing \(\text{UCS}_{\text{m}}\) as a function of certain factors as follows:

$$\text{UCS}_{\text{m}} = \text{UCS}_{\text{lab}} \cdot f_m \cdot f_t \cdot f_d$$  \(\text{(3)}\)

Where

- \(\text{UCS}_{\text{lab}}\) = Laboratory strength of rock
- \(f_m\) = Factor depending upon the dimensions of the heading and the distance between the joints
- \(f_d\) = Dilatation factor
- \(f_t\) = Long time strength factor

The deformation of an excavation (Fig 2) can be given by a function represented by the various zones and can be represented by Eqn (4) as follows:

$$U = C \cdot Z_1^{0.5} \cdot Z_2^{-1} \cdot Z_3^{-2}$$  \(\text{(4)}\)

Where

- \(C = \frac{Q \cdot f_m \cdot \sin \phi_3}{Z_2}\)
- \(Z_1 = \frac{f_3}{f_r}\)
- \(Z_2 = \frac{r_2}{r_1}\)
- \(Z_3 = \frac{r_1}{r_0}\)
- \(\phi_3\) = Time variable modulus of Shear
- \(Q\) = Tunnel area
- \(r_0\) = Support resistance

The value of \(Z_1, Z_2\) and \(Z_3\) depends upon Po (Fig 2), Q, rock strength, time etc.

Depending upon the rock type and stress field, all the failure zones may not develop. For example excavations driven in salt zones 01 and 03 may not form. There is only plastic zone where strength is maintained over sufficiently large deformations even when the stress far exceeds the strength of rock. In such situations there is no need of support. Support however is in fact needed to guard against the collapse of the broken material as in brittle rocks, coal, sandstone, strong mudstone etc.

The solution of Eqn (4) as a function of time can be obtained using iterative procedures. At each step of time, the corresponding strength of the rock mass in relation to the deformation is introduced. The solution, though approximate, agrees fairly well with the exact solution (Linkov, 1977).

The above simple solution is presented for a circular excavation in a homogeneous stress field. The important thing to remember is that the field deformation must be compared with theoretically calculated deformation. For example, in coal mines, where the roadways are rectangular the deformation of the roof at the centre will be more compared to a circular excavation. In such a case the size of the width of the excavation (2m) may be increased by some factor (say 1.2) to match field values. Increas or decrease in the width virtually changes the values of all the parameters.

Also, because in reality, rock in the roof, coal and floor have different properties, therefore the deformation in the roof, coal and floor must be calculated separately. Similarly, if the horizontal stress value in the roof, floor and seam are different, then the respective values should be used for the calculation of deformation.

Equation (4) has been used for analysis of deformation of rectangular excavations at West Cliff Mine. Fig 3 shows the results. Roof and floor displacements in isolated headings at Stations A and C are fairly close to the predicted values using Eq (4).

**CALCULATION OF ROOF BOLTING PARAMETERS**

Most of the roof support calculations are done for concrete or steel arches taking into account the reaction of the support (P0). By introducing different value of the rock-support interaction curve for different times (t) can be calculated. The load on the support depends upon the time elapsed between excavation and setting of support and the deformation characteristics of the support. A traditional relationship representing the support and rock interaction is given by (Fig 4), where,

$$U_b = U_m(Q,t) - U_m(Q,t) - U_m(P_0,t)$$  \(\text{(5)}\)

Where

$$U_m(Q,t) = \text{Displacement of the tunnel due to elastic(t)}$$
$$U_b(P_0,t) = \text{Displacement of the tunnel due to elastic after time(t)}$$
$$U_m(Q,t) = \text{Displacement of the tunnel due to elastic at time(t)}$$
$$U_m(P_0,t) = \text{Drop in deformation of the tunnel as a result of introduction of support with resistance P_0 at time(t)}$$

In the case of roof bolts, the displacement value \(U_m(P_0,t)\) is not due to deformation of the roof bolts as this represents a load of the total displacement of the excavation. The deformation of the bolt (\(U_b\)) is smaller than deformation of the excavation and depends upon the length of the bolt and can be given by the load developed.

$$U_b = U_m(P_0,t) - U_b(P_0,t)$$  \(\text{(6)}\)
Fig. 3 Comparison of calculated and experimental roof (a) and floor (b) displacement
Fig. 4 Traditional support - rock interaction curve

Where

\[ U_\xi (P_{\phi}) = \text{displacement of the point at the top end of the roof bolt of length } \xi \]

Reaction of the bolt is given by

\[ P_o = kU_b \]  

(7)

In a simplified analysis where the roof bolt support is assumed to have constant resistance \( P_o \) till the limit deformation of the bolt = \( \Delta = [(U_{1} (P_{\phi}) - U_{\xi} (P_{\phi})) / \xi] \) (Fig. 4) at failure, total displacement of the roof at a given time after installation of support is given by (Fig. 4).

\[ U_{P} = U_{m} (Q_{c}) + U_{a} (P_{\phi}) \]  

(8)

The value of \( U_{m} (Q_{c}) \) can be calculated from Eq. (4) by putting \( P_{\phi} = 0 \). The value of \( U_{\xi} (P_{\phi}) \) is given by:

\[ U_{\xi} (P_{\phi}) = U_{P} \left( \frac{\xi}{(\xi_{0})} \right) \]  

(9)

Where \( \xi \) = length of the bolt

Studies have shown that use of bolts increases the strength of rock mass. (Timofeev, 1940, Larson & Obision, 1983, Wallischlager and Nasau, 1983, Sayler and Krohn, 1982). Studies conducted by Timofeev (1980) show that increases in strength \( (kU_\xi) \) depend upon the capacity of the bolts (N) and the number \( (n_o) \) of bolts per m² (Fig. 5). The reaction of the bolts is given by

\[ P_o = N \times n_o \times kP_a \]  

(10)

By changing \( n_o \) to \( N \times n_o \) in Fig 5 and representing it by \( P_o \), Fig 6 is obtained.

**Estimation of roofbolting parameters**

Optimisation of roof bolting support must take into consideration the following parameters:

- Length of bolts, \( m \)
- Number of bolts/m²
- Capacity of bolts, kN
- Limit deformation of bolt, mm
- Limit deformation of heading, mm
- Time of installation, days
- Width of heading
- Strength of roof, seam, floor and rock
- Stress field

The length of bolts and number of bolts are the two most important parameters which influence support setting time. These two factors can be combined together in a single factor \( K \). The support system can then be optimised for the value of \( K \), such that

\[ K = n_o \times \xi \times \sigma \]  

(11)

where

- \( n_o \) = number of bolts/m²
- \( \xi \) = length of bolt, m
Fig. 5 Dependent of factor of strengthening on density and capacity of bolts (after Timofeev, 1980)

Fig. 6 Dependence $k_{sv}$ from $\nu_{sv}$
ANALYSIS OF INFLUENCE OF GEOMECHANICAL AND TECHNICAL PARAMETERS

A computer programme has been developed for estimation of roof bolting parameter based upon information of simplified data. The input parameters include:

- Laboratory strength
- Geological data of rock types
- Depth
- Dip of beds
- Width of excavation
- Stress field
- Direction of drawage
- Distance between parallel headings (interaction)
- Shape of heading (rectangular, arched)
- Capacity of roof bolts
- Number of roof bolts
- Yield limit of roof bolts

The programme calculates field parameters of the rock mass from back analysis based upon displacements measurements in the excavation.

The programme outputs load on support, length of roof bolts required, displacement of roof, sides and floor, density of bolting required.

Analysis has been done to establish the effect of depth, width of heading, time of setting, angle of internal friction of rock, yield limit of bolts or bolt density requirements.

Calculations have been made for different depths and rock properties with the assumption that maximum displacement of roof with minimum support (P_0 = 0) is 300mm. The rock strength for a given depth (stress values) has been calculated to meet the above condition. The rock assumed in each case is silstone with angle of friction of 30°.

Bolt capacity N is assumed = 200kN. For different depth and UCS (Mp) for rock, the value of K (number of bolts/m^3) as a function of yield limit of bolts is given in Fig 7. It is obvious that even though U (Q_t) is the same for the various rock conditions (UCS lab and depth), rigid bolts are optimum for low depth. As depth increases, yield limit of the bolts must increase otherwise much higher value of K will be needed.

If bolts are installed laser, rigid bolts can be used for higher depths Fig 7b. Similarly for smaller width of headings, at smaller depths (Fig 8a) one should use yield line bolts, but for higher widths, comparatively rigid bolts are recommended. In deeper mines, as every case yield line bolts must be installed (Fig 8b). The height of the curves for depth = 300m indicates that optimum value of yield limit is about 50-70mm.

Fig 9 shows that for a narrower heading, if rigid bolts are installed very quickly higher K is required; but for higher elapsed time, rigid or yielding bolts can be used, though rigid bolts will give lesser deformation and will be shorter. For a wider heading (5.5m), minimum value of K occurs with yielding bolts of 50-100mm.

It is clear that value of K can vary under wide variations (1-14) in spite of seemingly equivalent geomechanical condition. U_t (Q_t) = 300mm: Fig 10 shows change in K for different angles of internal friction (α) of rock and for different yield limit of bolt. For more plastic rocks optimal yielding limit of bolts is more than for brittle rock but 100mm yield limit will suit most rock types in coal mines.

CONCLUSIONS

A simplified approach using anisotropic excavations but incorporating field determination of an excavation show that roof bolting systems can be designed using conventional roof support-rock interaction concepts. A new concept which incorporates bolt length and bolt density as the criteria for optimisation of bolts is suggested although it is not the exact optimum cost. The theory developed in this study incorporates time dependent and post-failure behaviour of rock mass. Analysis has been done for the equivalent geomechanical conditions (i.e. the same deformation with minimum resistance of support for different geological and mining conditions with displacement of 300mm). Results of analysis show that rigid bolts are optimum for smaller depths and that the density of bolts can be reduced depending upon the acceptable displacement before the setting of bolts. Installation of rigid bolts at larger depths or for plastic rocks, could lead to overloading and failure of bolts or would require larger bolt density. Decreasing the width greatly reduces the bolt outline.

REFERENCES


Fig. 7 Determination of optimal bolt yielding for heading b = 5.5 m for different depth

(a) $U_b = 0.1 U_0$, (b) $U_b = 0.2 U_0$

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Fig. 8 Determination of optimal bolt yield for different heading widths
(a) depth $H = 300\text{m}$, (b) depth $H = 900\text{m}$
Fig. 9 Determination of optimal bolt yield for different initial displacements $U_l$. Depth = 300m.
(a) $b = 3m$ (b) $b = 5.5m$
Fig. 10 Determination of optimal bolt yield for different angles of internal friction ($\phi$) of rock