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# MODELLING SIMULATION OF THE PERFORMANCE OF CABLE BOLTS IN SHEAR

Haleh Rasekh, Ali Mirza, Naj Aziz, Jan Nemcik and Xuwei Li

**ABSTRACT:** The application of cable bolts as a secondary support system in coal mines has substantially increased over the last decades. This development emphasises a prerequisite for a better understanding of the mechanical behaviour of cable bolts in combination of shear and axial loads, which is of interest to geotechnical engineers and designers of underground structures. Based on various research studies undertaken, analytical models have been proposed to assess the shear performance of cable bolts with the aim of designing safe and sustainable rock structures. Recently, a shear strength model for cable bolts was proposed by Aziz *et al.*, (2015). The model was based on Mohr-Coulomb failure criterion and the Fourier series concept. The proposed analytical model was associated with a set of systematic experimental studies by which model coefficients were calibrated. This paper compares the performance of a number of proposed mathematical models against the experimental data and presents a model that simulates the shear behaviour of cable bolts, which is in agreement with the experimental results.

## INTRODUCTION

Since the 1960s, the use of cable bolts as long fully grouted elements in the mining industry was started with the advantageous for rock mass to support itself (Fuller and O'Grady 1993). Cable bolts are flexible tendons with high tensile strength that contain a group of steel wires, which are twisted into strands. It provides reinforcement and support in mining and civil engineering excavations. Cable bolts are applicable in various areas of walls, roof and floor of underground and surface openings, including: drifts and intersections, they are also used for metalliferous mining open stope backs, open stope walls, cut and fill stopes, draw points and permanent openings (Hutchinson and Diederichs 1996; Windsor 1992; Fuller 1983 and Puhakka 1997).

Cable bolt profiles come in three types: plain, spiral and indented as shown in Figure 1. The performance of cable bolt changes, depending on profile.



Figure 1: Cable bolts profile: plain, spiral and indented

Pull out tests and shear tests are different methodologies to evaluate the behaviour of cable bolts. Assessing the tensile failure and load transfer capacity of cable bolts by using the pull out test methodology was reported by Hawkes and Evans (1951), Fuller *et al.*, (1978), Diederichs (1993), Bouteldja (2000) and Morsy and Han (2004). Nowadays, the performance of cable bolts under shearing

is a topic of interest to many researchers. Cable bolt failure in a field is a combination of shear and tensile loads. Shear test can be conducted in the laboratory by using either the single shear or double shear tests. The single shear test as an approach to determine the shear strength of cable bolts has been conducted by a number of researchers while limited studies have been carried out using double shear tests. A number of mathematical models have been proposed to calculate the shear strength of fully grouted cable bolts. Six analytical models will be summarised in the following part and compared with measured test data at the UOW laboratory.

### LITERATURE REVIEW

Different analytical models are available to determine the shear strength of fully grouted cable bolts. Details of six of these are set out below and it should be noted that most of these involved single shear testing. Only the tests by Aziz *et al.*, (2015) involve double shear testing.

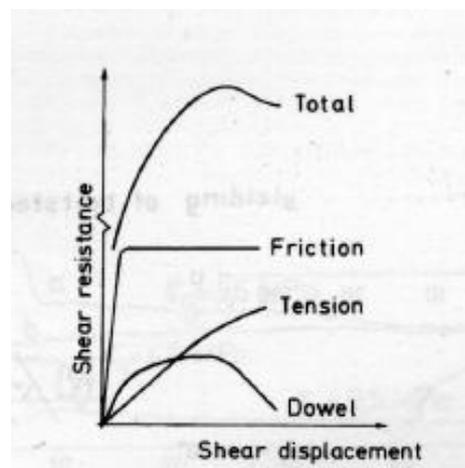
Dulacska (1972) conducted 15 single shear tests by using different concrete grades, three steel sizes and four different angles for stirrup to examine the action of dowel in cracked concrete to establish theoretical load-deformation relationships. The steel bars near the cracks and the friction across the concrete surfaces counteract the slipping of the cracks.

The shear force was modelled by the following equation:

$$T = 0.2d_b^2 \sigma_y \sin \delta \left[ \sqrt{1 + \frac{\sigma_c}{0.03\sigma_y \sin^2 \delta}} - 1 \right] \quad (1)$$

where,  $T$  is the shear force carried by the bolt,  $d_b$  is bolt diameter,  $\sigma_c$  is Uniaxial Compression Strength (UCS) of rock,  $\sigma_y$  is yield stress of bolt, and  $\delta$  is angle of stirrups.

Bjurstrom (1974) directed a series of single shear tests to find out the shear forces transfer in grouted untensioned bolted joints. The granite blocks with natural and artificial joints and different normal pressures were tested. The joint surface was smooth and the total shear displacement was less than 50 mm. 50 tests were carried out without bolt installation for verification of the friction and 60 tests were with bolt installation. The shear displacement was measured by differential transformer (LVDT). As shown in Figure 2, the shear strength was determined by three components: friction, tension of the bolt and the dowel effect. Also, the results demonstrated that when the angle between the bolt and joint is less than 35°, the failure occurs in tension. By increasing this angle to 40-45°, the failure will be a combination of shear and tension.



**Figure 2: Relationships between forces caused by friction, tension of bolt and dowel effect and shear displacement for a joint reinforced by grouted untensioned bolts (Bjurstrom 1974)**

The shear strength is calculated by following equations:

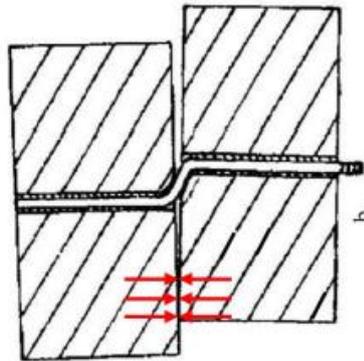
$$\text{Reinforcement effect } T_b = p(\cos \beta + \sin \beta \tan \varphi) \quad (2)$$

$$\text{Dowel effect } T_d = 0.67 d_b^2 \sqrt{\sigma_c \sigma_y} \quad (3)$$

$$\text{Friction of joint } T_f = A_j \sigma_n \tan \varphi \quad (4)$$

where,  $p$  is the axial load corresponding to shear displacement,  $\beta$  is the initial angle between bolt and the joint,  $\varphi$  is friction angle,  $d_b$  is bolt diameter,  $\sigma_y$  is yield strength of bolt,  $\sigma_c$  is UCS of the rock,  $A_j$  is joint area,  $\sigma_n$  is normal stress on joint.

Haas (1976) conducted single shear tests on blocks of chalk and limestone. There were different variables in tests such as: type of bolt, normal pressure on interfaces and different orientations of bolts due to the shear surface ( $0^\circ$ ,  $+45^\circ$  and  $-45^\circ$ ). The effect of using rock bolts was investigated by increasing the resistance of rock in shear displacement along the fractured surfaces. Results indicated that increasing the shear resistance is the result of applying 170 kPa of normal compressive stress on the plane and also orienting the bolt in the direction of joint shear progress. However, the single shear test is due to non-equilibrium load distribution along the joint plane (Figure 3).



**Figure 3: Non-equilibrium in vicinity of the shear joint (Haas 1976)**

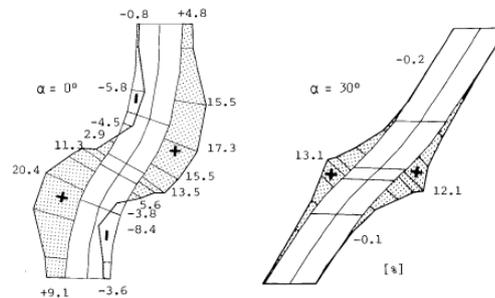
Haas (1976) also proposed a mathematical model for calculating the shear resistance of bolted rock. The average shear stress is determined by:

$$\tau_{ave} = \tau_0 + \frac{\mu T_0 \cos \theta + T_0 \sin \theta}{A_s} \quad (5)$$

where,  $\tau_{ave}$  is the average shear stress with bolt for impending movement,  $\tau_0$  is the average shear stress on the interface without a bolt,  $\mu$  is coefficient of friction,  $\theta$  is initial orientation of bolt,  $A_s$  is area in shear, and  $T_0$  is tension in the installed bolt.

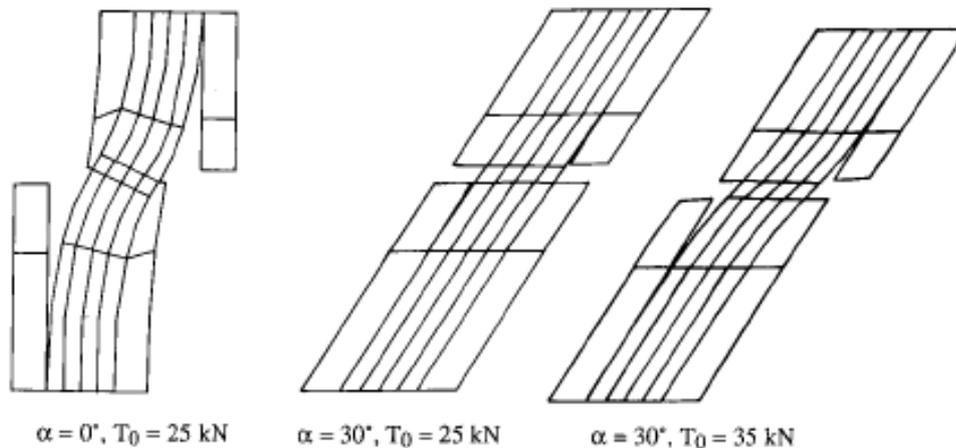
Spang and Egger (1990) studied the shear performance of a fully grouted untensioned bolts in jointed rock by single shear test methodology. They used numerical studies by Finite Element Method for a three dimensional model. Bolt inclination to the joint surface was considered at  $30^\circ$  and  $90^\circ$ . Three stages of behaviour for a fully grouted bolt in the jointed rock mass were defined as elastic, yield and plastic stages. The behaviour in the elastic zone followed the Mohr-Coulomb relationship. It was monitored that the yield stage in the bolt and mortar occurs at the very first stages of loading when the shear displacement was 1 mm and the bolt only reached 10% of its ultimate strength. Therefore, the behaviour was mostly governed in the plastic zone. The Drucker-Prager and von Mises failure criteria were used to define the elastic-plastic behaviour of materials. The joint-friction of blocks was neglected for simplification in the model with blocks being held together by the grouted bolt.

Results from tests showed the higher shear resistance for an inclined bolt compared to the perpendicular one. Also, the deformation amount in the inclined bolt was less than the bolt without inclination. The failure in the perpendicular bolt was attributed to bending that combined shear and tension across the joint surface. On the other hand, the failure for the inclined bolt was due to the tension near the shear surface. As shown in Figure 4, the plastic strain in the inclined bolt (30°) is less than the perpendicular one.



**Figure 4: Plastic strains in the bolt for  $T=30\text{kN}$  and  $\alpha = 0$  and  $30$  (Spang and Egger 1990)**

The gap created due to the shear force and failure of the bond between steel and mortar for the perpendicular bolt, was more than the inclined bolt (Figure 5).



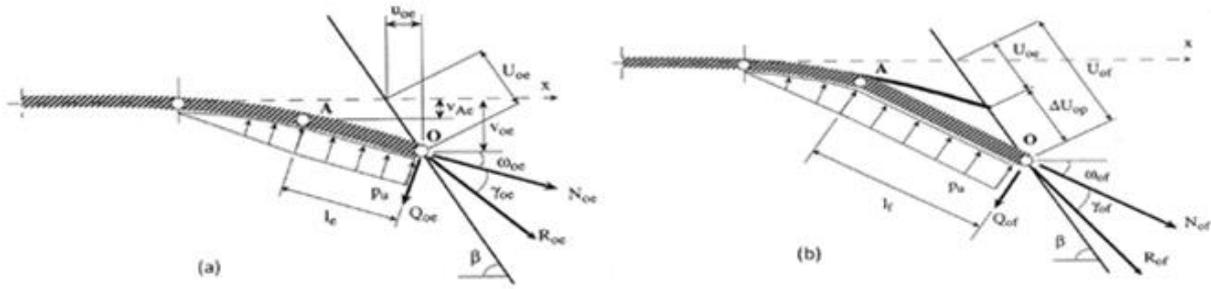
**Figure 5: Gap between bolt and mortar for  $\alpha = 0^\circ$  ( $T = 25\text{kN}$ ) and  $30^\circ$  ( $T = 25, 35\text{kN}$ ) (Spang and Egger 1990)**

The proposed equation to calculate the shear resistance of a bolted joint is:

$$T_0 = P_t \cdot [1.55 + 0.011 \cdot \sigma_c^{1.07} \cdot \sin^2(\alpha+i)] \sigma_c^{-0.14} (0.85 + 0.45 \tan \phi) \quad (6)$$

where,  $T_0$  is maximum shear resistance,  $P_t$  is maximum tensile load of the bolt,  $\phi$  is angle of friction, and  $\alpha$  is bolt inclination.

Pellet and Egger (1996) developed an analytical model to investigate the shear strength of an untensioned fully grouted rock bolts. The bolt's behaviour in interaction between axial and shear forces and its large plastic displacements were measured. The behaviour of rock bolt in elastic and plastic zones are as shown in Figure 6. The Tresca criterion was used to model the bolt in failure under the combination of shear and axial loads. The single shear test was the methodology for this study, and the shear force acting at the shear plane can be determined by:



**Figure 6: Force components and deformation of a bolt (a) in elastic zone, and (b) in plastic zone (Pellet and Egger 1996)**

$$Q_{of} = \frac{\pi D_b^2}{8} \sigma_{ec} \sqrt{1 - 16 \left( \frac{N_{of}}{\pi D_b^2 \sigma_{ec}} \right)^2} \tag{7}$$

where,  $Q_{of}$  is shear force acting at shear plane at failure of the bolt,  $P_u$  is the bearing capacity of the grout or rock,  $D_b$  is diameter of the bolt,  $\sigma_{el}$  is the yield stress of the bolt,  $N_{oe}$  is axial force acting at shear plane at the yield stress of the bolt,  $\sigma_{ec}$  is failure stress of the bolt and  $N_{of}$  is axial force acting at shear plane at failure of the bolt.

Aziz *et al.*, (2015) conducted a series of double shear tests on seven different types of cable bolt subjected to different amount of pre-tension load to determine the shear strength of pre-tensioned fully grouted cable bolts (Figure 7). The total shear strength was measured as the combination of shear strength of cable bolts and the friction between concrete blocks.



**Figure 7: Arrangement of the cabled concrete blocks in 500t machine (Aziz *et al.*, 2015)**

Aziz *et al.*, (2015) also proposed a mathematical model to determine the shear strength of cable bolts using the combination of Mohr Coulomb criterion and Fourier series scheme. This model is capable of determining the peak shear strength of the cable bolt (Aziz *et al.*, 2015). The proposed model is:

$$\tau_p = \left( \frac{a_0}{2} + \sum_{n=1}^3 a_n \cos \left( \frac{2n\pi}{2\pi} \cos^{-1} \left[ \frac{-4a_2 + \sqrt{16a_2^2 - 48a_1a_3 + 144a_3^2}}{24a_3} \right] \right) \right) \tan(\varphi) + c \tag{8}$$

where,  $\tau$  is the shear stress,  $S$  is the shear load,  $C$  is cohesion,  $a_n$  is Fourier Coefficient,  $n$  is the number of Fourier Coefficient, which is considered between 0 and 3,  $u$  is the shear displacement and  $T$  is the shearing length.

The test apparatus, sample preparation and experimental plan use by Aziz *et al.*, (2015) were as follows:

Three concrete blocks with cross sectional area of 300 x 300 mm<sup>2</sup> made the double shear assembly. The length of the corner concrete blocks was 300 mm while it was 450 mm for the middle one. After curing, the concrete blocks had a compressive strength of 40 MPa and they were kept wet for 28 days for curing purposes. The concrete blocks were positioned in the steel moulds. During pre-tensioning and shearing test the axial loads were applied on the cable bolt. Therefore, two 60 t load cells were incorporated in order to monitor the axial loads. Two sets of barrels and wedge were installed to retain the cable bolt in tension. Depending on the test requirements, chemical resin or cementitious grouts were injected to the holes on top of the concrete blocks. Seven days after pre-tensioning the cable bolt and the grout injection, double shear tests were conducted with the 500 t machine at the rock mechanics laboratory of University of Wollongong. The rate of shear displacement was set by the digital controller as 1 mm/min. The shear load was applied to the middle concrete block to moving in a vertical direction by using a hydraulic jack located on top of the instrument. The data taker recorded the amount of shear and normal loads and shear displacement.

More than 10 double shear tests were conducted but in this paper the result of two tests will be analysed and compared with the proposed models. Two tests were conducted on the plain superstrand and spiral superstrand cable bolts with pre-tension loads of 25 t. Table 1 represents the test plans.

**Table 1: Detail of double shear tests carried out on plain wire superstrand and spiral wire Superstrand cable bolts**

Cable Bolt Properties					Drill bit (mm)	Bonding agent	Pre-tension load (t)
Product name	Cable $\phi$ (mm)	Wire geometry	Cable cross-section	Cable geometry			
Spiral superstrand	21.8	Spiral	19 wire, PC strand	Non-birdcaged	28	Oil based resin	25
Plain superstrand	21.8	Plain	19 wire, PC strand	Non-birdcaged	28	Oil based resin	25

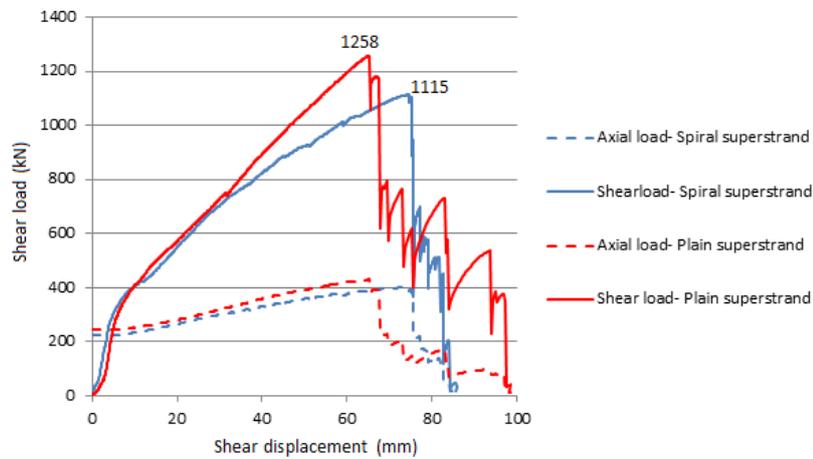
As Figure 8 shows the peak shear load for the plain and spiral superstrand cable bolts were 1258 kN and 1115 kN, respectively. The reason is that the cross sectional area of the spiral superstrand cable is reduced compared to the plain superstrand cable. It shows that the cable behaviour starts with the elastic stage up to the yield point. Then, the strain softening stage starts up to the peak shear load and after this stage, the cable wires start to break. The drop in the shear strength depends on the size of cable wires.

## ANALYSIS

This part consists of the comparison between the test data and model results. This result helps to identify the model which is in more agreement with experimental results in reality. The experimental results of shear test of two types of cable bolts, plain wire superstrand and spiral wire superstrand, will be compared with proposed analytical models. The properties of these two types of cable bolts subjected to shearing are summarised in Table 2. The  $\sigma_c$  of concrete blocks were 40 MPa and  $\sigma_y$  (yield stress) of bolts was 1677.3 MPa.

Tables 3 and 4 show the shear load measured by different models and the variation between their prediction results and the test result. The results demonstrate that the model developed by Aziz *et al.*, (2015) has the least amount of variation with test results. Therefore, it is concluded that this model is

significantly suitable to calculate the peak shear load generated on pre-tensioned fully grouted cable bolts. Additionally, the model by Hass (1976), Bjurstrom (1974) and Pellet and Egger (1990) calculates shear load with less than 20% variation while for Aziz *et al.*, (2015), it is far less.



**Figure 8: Test results of plain and spiral superstrand cable bolts**

**Table 2: Properties of plain wire Superstrand and spiral wire Superstrand cable bolts**

Performance Data	Unit	Plain wire Superstrand	Spiral wire Superstrand
Yield Load	kN	525	525
UTS	kN	590	573
Strand diameter	mm	21.8	21.8
C.S.A.	mm <sup>2</sup>	313	277

**Table 3 Measured shear load for different models for plain wire Superstrand cable bolts**

Model	Shear load (kN)	Variation (%)
Dulacska (1972)	54.16	91.38
Bjurstrom (1974)	520	17.19
Haas (1976)	682.65	8.6
Spang and Egger (1990)	553.87	13.38
Pellet and Egger (1996)	227.5	63.8
Aziz et al., (2015)	630.07	3.3
Test result (Plain superstrand cable bolt)	628	-

**Table 4 Measured shear load for different models for spiral wire Superstrand cable bolts**

Model	Shear load (kN)	Variation (%)
Dulacska (1972)	54.16	90.29
Bjurstrom (1974)	490.17	12.08
Haas (1976)	653.57	17.23
Spang and Egger (1990)	738.8	32.52
Pellet and Egger (1996)	125.71	77.45
Aziz et al., (2015)	551.22	1.12
Test result (Spiral superstrand cable bolt)	557.5	-

## CONCLUSIONS

The use of cable bolts as a secondary support system in the mining industry is on an increasing trend. Therefore, it is essential to calculate the performance of cable bolts subjected to shear load. This has been a topic of interest for many researchers and different mathematical models have been proposed to determine the shear strength of cable bolts. The comparison between a numbers of these models demonstrated that the model proposed by Aziz *et al.*, (2015) is in agreement with experimental result for plain and spiral superstrand cable bolts with 25 t pre-tension loads with less than 4% variation. Thus, it is rational to use this model to investigate the shear strength of a pre-tensioned fully grouted cable bolt. Moreover, Bjurstrom (1974) and Haas (1976) also proposed models that are useful but provide results which are less accurate compared to Aziz *et al.*, (2015).

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