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Behavior of Fiber Glass Bolts, Rock Bolts and Cable Bolts in Shear

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Abstract This paper experimentally compares the shear behavior of fiber glass (FG) bolt, rock bolt (steel rebar bolt) and cable bolt for the bolt contribution to bolted concrete surface shear strength, and bolt failure mode. Two double shear apparatuses of different size were used for the study. The tensile strength, the shear strength and the deformation modulus of bolt control the shear behavior of a sheared bolted joint. Since the strength and deformation modulus of FG bolt, rock bolt and cable bolt obtained from uniaxial tensile tests are different, their shear behavior in reinforcing joints is accordingly different. Test results showed that the shear stiffness of FG bolted joints decreased gradually from the beginning to end, while the shear stiffness of joints reinforced by rock bolt and cable bolt decreased bilinearly, which is clearly consistent with their tensile deformation modulus. The bolted joint shear stiffness was highly influenced by bolt pretension in the high stiffness stage for both rock bolt and cable bolt, but not in the low stiffness stage. The rock bolt contribution to joint shear strength standardised by the bolt tensile strength was the largest, followed by cable bolts, then FG bolts. Both the rock bolts and cable bolts tended to fail in tension, while FG bolts in shear due to their low shear strength and constant deformation modulus.

Keywords FG bolt · Rock bolt · Cable bolt · Joint shear strength · Joint shear stiffness · Bolt failure mode

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1 Introduction

In civil and mining engineering projects, the presence of joints/fractures in rock masses weakens the rock strength and makes rock masses more deformable. Rock masses may open and slide along joints, causing both parallel and perpendicular movements. To control the relative movement and deformation of rock masses, bolting is a common and basic reinforcement technique. A variety of bolts have been used in rock reinforcement, which can be normally divided into fiber glass (FG) bolts, rock bolts and cable bolts according to their material composition and basic structures. FG bolts and rock bolts are both single solid tendons, while cable bolts are flexible tendons composed of multi-wire strand (Hutchinson and Diederichs 1996). Unlike rock bolts and cable bolts which are made of steel, FG bolts are made of inorganic nonmetallic glass fibers. Each of them also contains a variety of bolt types differing primarily in structures and dimensions, such as bolt diameter, rib spacing, rib height and surface profile. All these designs are aimed to improve the bolt strength and load transfer between bolts and rock masses when bolts are loaded in tension.

In field condition, bolts suffer from a combination of both tensile and shear forces. This is a more complex situation than that of pure tension. To better understand the load transfer mechanism and shear strength of the sheared bolted joint, a number of experimental tests were carried out by researchers on joints mostly reinforced with rock bolts (Bjurstrom 1974; Dight 1983; Dulacka 1972; Egger and Fernandes 1983; Ferrero 1995; Fuller and Cox 1978; Grasselli 2005; Haas 1981; Jalalifar 2006; Kharchafi et al. 1999; Pellet et al. 1995; Spang and Egger 1990). These studies investigated the key influencing factors, such as properties of rock and grout, bolt strength and dimension,

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bolt structure, bolt installation angle, bolt pretension. Recently, experimental studies were also conducted on the shear behavior of joints reinforced by cable bolts and FG bolts (Aziz et al. 2015, 2014; Craig and Aziz 2010; David et al. 2015), and similar influencing factors were examined. Due to the difference of FG bolts, rock bolts and cable bolts in strength, deformation modulus and structure, their shear behavior is different as well. This paper focuses on the influence of bolt strength and bolt deformation modulus on the bolted joint shear strength, joint shear stiffness and bolt failure mode for FG bolts, rock bolts and cable bolts.

2 Tensile Behavior of Different Bolts

Tensile properties, including the strength and deformation modulus, are the important characteristics of bolts in ground support, and they influence the bolt performance in reinforcing rock strata. From the simple uniaxial tensile test, elastic modulus, plastic modulus (modulus in the strain hardening stage), yielding strength and tensile strength can be calculated. When it comes to cable bolts, however, it is practically not easy to determine accurately the deformation modulus, which is mainly due to the high failure load and structural character of cable bolts. It is impossible to grip cable bolt end tightly to avoid cable end slippage at high load. Thus, the deformation modulus of cable bolts is not perfectly accurate. Nevertheless, the deformation modulus of cable bolts obtained from tensile tests can represent and demonstrate the basic feature of cable bolts with reasonable accuracy.

Figure 1 shows the setup of a tensile test of FG bolts, and the stress–strain relationships of FG bolts, rock bolts, and cable bolts are given in Fig. 2 (Faulkner 2012; Faulkner et al. 2013).

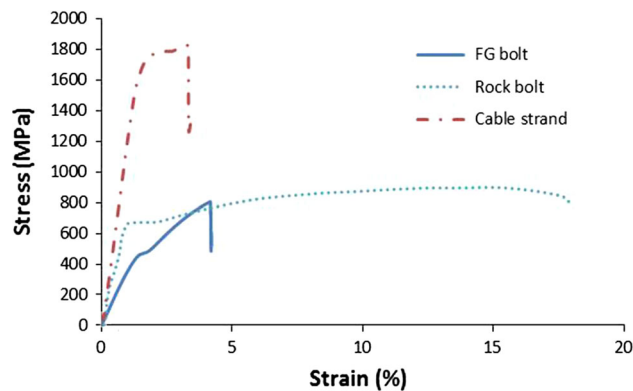**Fig. 1** Tensile test of FG bolt

Fig. 2 Stress-strain relationship of different bolts (relationship of cable bolt is from Faulkner 2012, all the others are from tests conducted in the CME laboratory of UOW)

From Fig. 2 it can be seen that all the stress–strain curves were different. For FG bolts, the stress–strain relationship was very close to a line at the beginning and then its stiffness decreased slightly with the increase of elongation until the final failure. Overall, its tensile modulus did not change much before failure. When it comes to rock bolts, there were obviously two different stages during the tensioning process, the first elastic stage and the second plastic stage. The tensile modulus was much larger in the elastic stage than in the plastic stage. The failure strain of rock bolts was normally at the range of 10–20 %. In comparison, a cable strand had similar deformation moduli in elastic stage as rock bolt, but was higher in plastic stage. Cable bolts failed at a small strain less than 5 %. In general, cable bolts have higher tensile strength than rock bolts and FG bolts.

3 Failure Criterion of Bolt Under Combined Loads

A bolt used to reinforce jointed rock masses takes both tensile and shear forces due to the axial and lateral movement of rock masses. Thus the bolt failure is attributed to the combination of tensile and shear forces (Dight 1983; Jalalifar and Aziz 2010a; Pellet and Egger 1996). An equation was developed to predict the failure load which takes into account contributions of both tensile and shear forces (Dight 1983).

$$\left(\frac{N_o}{N_f}\right)^2 + \left(\frac{Q_o}{Q_f}\right)^2 = 1 \tag{1}$$

where N_o and Q_o are the tensile and shear forces, respectively, at failure of a bolt; N_f is the ultimate tensile strength of a bolt (equal to $A_b\sigma_f$); Q_f is the ultimate shear strength of a bolt (equal to $A_b\tau_f$); A_b is the bolt cross-section area; σ_f and τ_f are the failure strengths of bolt loaded in pure tension and pure shear, respectively.

Regarding Eq. (1), when τ_f is equal to $\frac{\sigma_f}{2}$, the equation represents the Tresca criterion in the plane stress state. When τ_f is equal to $\frac{\sigma_f}{\sqrt{3}}$, the equation becomes the Von Mises criterion in the plane stress state. In this paper, the Tresca criterion is to be used. For rock bolts and cable bolts, it is suitable to use Tresca criterion. However, there is currently no existing failure criterion for FG bolts. Nevertheless, the potential criterion for FG bolts should also be limited by the shear strength and tensile strength and thus has a shape similar to Tresca criterion or Von Mises criterion. Therefore, a similar criterion based on the tensile and shear strength of FG bolts is to be used to qualitatively describe the FG bolt failure behaviour.

From the form of the failure criterion in Eq. (1), when N_f is not equal to Q_f, the failure equation takes the form of an ellipse, which is the normal case for all currently used bolts. If both rock bolts and cable bolts are assumed as a Tresca material (Dight 1983), the relationship will exist between N_f and Q_f in the form of $Q_f = 0.5N_f$. However, there is not a constant relationship between N_f and Q_f for different FG bolts. Accordingly, punch tests were carried out to obtain the shear strength of FG bolts using a punch shear apparatus as shown in Fig. 3. Table 1 gives the punch test results of FG bolts (David et al. 2015). By considering the tensile strength of FG bolts shown in Fig. 2, then the relationship for FG bolts is roughly $Q_f = 0.13N_f$.

Based on Eq. (1), the failure criteria of FG bolts, rock bolts and cable bolts are shown in Fig. 4 in the form of $\frac{N_o}{N_f}$ and $\frac{Q_o}{N_f}$. If the bolt deflection is considered constant and joint friction is ignored in the calculation of bolted joint shear strength provisionally, clearly, the larger the summation of tensile force and shear force in bolts, the higher the joint shear strength. It can be seen from Fig. 4 that, in general, the summation of tensile force and shear force increases with the decrease of shear force. This is mainly due to the different change rates of tensile and shear forces
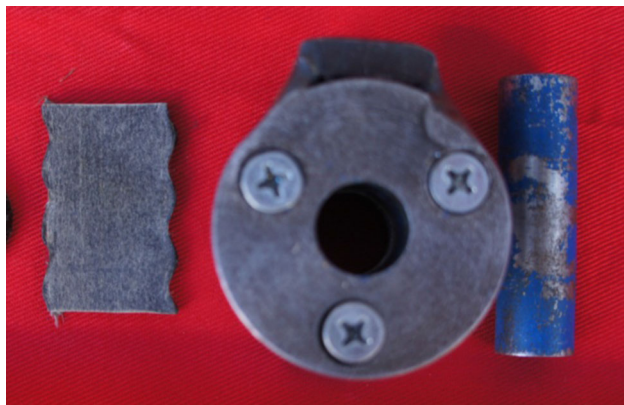
Table 1 Punch shear test results of FG bolt

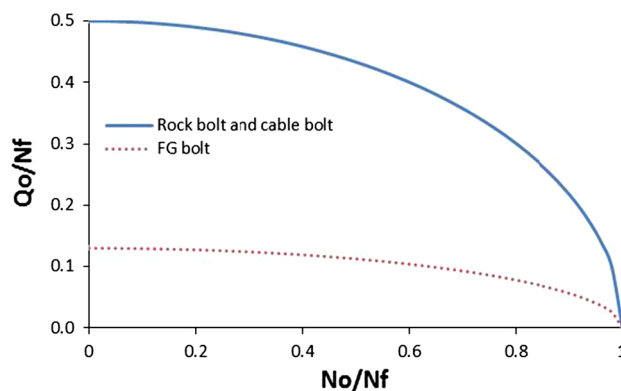| Sample	MN	T (m)	D (m)	MPa
A	0.012	0.00297	0.0128	102.22
B	0.012	0.00302	0.0127	102.30
C	0.013	0.00302	0.0129	107.50
Average				104.01

Fig. 4 Relationship between tensile force and shear force of FG bolts, rock bolts and cable bolts

as shown in Fig. 4. Since the tensile force increases much more rapidly than the shear force decreases, the summation of tensile and shear forces at bolt failure increases with the decrease of shear force. Besides, the influence of shear force is greater in FG bolts than in rock bolts and cable bolts due to the small shear strength of FG bolts. That is to say the increase of shear force in FG bolts is more detrimental than in rock bolts and cable bolts. Considering the deformation moduli of three different bolts in Fig. 2, the shear force in rock bolts and cable bolts loaded laterally increases faster than in FG bolts prior to yielding. However, the shear force in FG bolts increases much faster after the initial elastic stage of rock bolts and cable bolts since FG bolts do not have an obvious plastic stage. Thus at increased shear displacement FG bolts tend to fail at small tensile force due to their constant deformation modulus and low shear strength.

4 Double Shear Test Process and Tested Bolt Types

Double shear test apparatuses of two different dimensions were used in double shear testing of FG bolts, rock bolts and cable bolts. The small shear apparatus consists of two 150 mm side cubes and a central prism of

Fig. 3 FG plate and punch shear test device

$150 \times 150 \times 150$ mm. The large shear apparatus is 300 and 450 mm in length for the side and central concrete blocks, respectively, with the cross section of 300×300 mm. FG and rock bolts were tested using the small apparatus and the large one was used for cable bolts. Since the preparation and testing process using two different apparatuses were similar, only shear tests on cable bolts using the large apparatus were illustrated as an example in this section.

4.1 Concrete Block Casting

Figure 5 shows the concrete blocks cast in steel moulds. Concrete blocks used for double shear testing were cast in the steel frame of the double shear test apparatus, which was also used for testing later. Prior to casting, a greased plastic/steel conduit of suitable diameter was placed through the center of the mould lengthways to create a hole for bolt installation in the concrete blocks. The conduit was wrapped with plastic wire to create a rifled surface in the borehole for bolt anchorage. Mixed concrete was poured into each section of the 20 mm thick steel moulds. The conduit and the plastic wire were removed when the concrete set. The concrete was left to set in the mould for 24 h, and then removed and stored for future use.

In addition, immediately after concrete casting plastic tubes were inserted into the top part of the concrete blocks to make inlet holes for grouting. If no inlet holes were made when casting concrete, grout inlet holes would be drilled later for future grouting.

4.2 Bolt Installation

When preparing to test, the concrete blocks were placed back in the steel moulds. Then a cable with desired length was inserted into the central hole, tensioned and anchored in place using a hydraulic tensioner and two barrel and wedge systems. To monitor the variation of axial loads at cable ends during the initial cable pre-tensioning and the subsequent shearing stage, two 60 t load cells were installed at the cable ends. Masking tape and silicon gel were used to seal grout from escaping out of the hole during grout injection and bolt encapsulation. The assembly was left for at least the manufacture-recommended cure time to allow the grout to cure. Most specimens were left for roughly 7 days before being tested.

4.3 Shear Test Procedure

The process of double shear testing consisted of loading the middle block vertically in a 500 t compression testing machine. The 450 mm long middle section of the double shear apparatus was vertically loaded at the rate of 1 mm/min for the maximum 100 mm vertical displacement. The rate of loading, shear-induced loads and displacement was monitored and simultaneously displayed visually on a PC monitor. Figure 6 shows a post-test view of the sheared central block.

The shear block was then dismantled manually after shear testing was completed. An example of the broken cables and blocks after testing is shown in Fig. 7.

4.4 Bolt Types

Different types of FG bolts, rock bolts and cable bolts were tested using the above mentioned shear test apparatus. In physical appearance these bolts differ mainly in diameter and surface profile as detailed in Table 2 and in Fig. 8. Since the performance of cable bolts with spirally ribbed wires and smooth wires are different, cable bolts having spirally ribbed wires are asterisked as shown in Table 2 and other tables in this paper.

Fig. 5 Concrete block casting

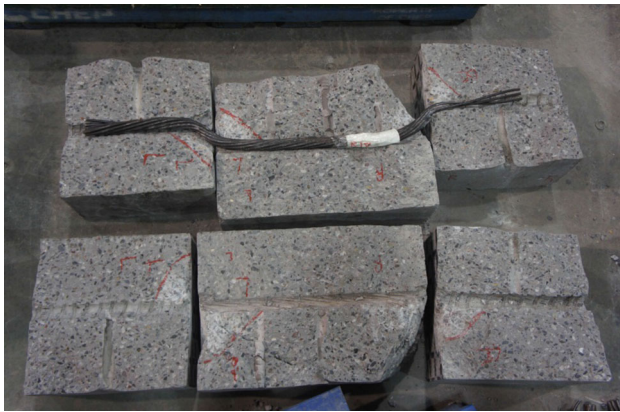**Fig. 6** Sheared central block of the tested cable bolt

Fig. 7 Dismantled test sample after testing

5 Test Results

In total 31 bolts were studied, including 5 FG bolts, 9 cable bolts and 18 rock bolts. Except for 11 rock bolts, all the other 20 bolts were shear-loaded to failure. Table 3 summarizes the test results of 20 bolts which were loaded to failure. From Table 3, it can be seen that FG bolted joints failed at small shear displacement and shear force. The joint shear displacement at rock bolt failure was similar to that at cable bolt failure in general, but the shear strength of rock bolted joints was much lower than the cable bolt counterpart. In addition, bolt pretension had a positive effect on the peak shear force, but a negative effect on the joint shear displacement.

6 Joint Shear Stiffness with Bolt Reinforcement

Figure 9 shows typical shear load–displacement curves of three double shear tests on FG bolts, rock bolts and cable bolts and the schematic relationship as well. The shear load–displacement curves of rock bolts and cable bolts were similar, but they were different from the FG bolts. For rock bolts and cable bolts, obviously, the shear process could be divided into two stages before bolt failure, the high stiffness stage and the low stiffness stage. In a specific shear test, the shear stiffness was controlled by a variety of influencing factors, such as the bolt modulus, bolt deformation, concrete strength and variation of joint friction coefficient. Normally, the variation of bolted joint shear stiffness was mainly attributed to the variation of bolt modulus (Jalalifar 2006; Pellet and Egger 1996). So for rock bolts and cable bolts, the high stiffness stage was normally related to the elastic state and the low stiffness stage was related to the plastic state of bolts. Unlike rock bolts and cable bolts, FG bolts did not obviously exhibit two different stages. Instead, similar as FG bolts loaded in tension, their shear stiffness decreases gradually with the increase of joint shear displacement until bolt failure.

Based on the shear behavior of joints reinforced by FG bolts, rock bolts and cable bolts, they can be represented by a linear and a bilinear relationship. Tables 4 and 5 summarize the joint shear stiffness of rock bolts and cable bolts, respectively. Both high stiffness and low stiffness stages were calculated and listed in tables. Since rock bolts listed in Table 5 were not loaded to failure, their average shear stiffness cannot be obtained to compare with cable bolts. Therefore, the average shear stiffness of rock and cable bolts was not summarized in Tables 4 and 5. Table 6 shows the average shear stiffness of FG bolted joint before the peak load. From Tables 4, 5, 6 the following conclusions can be drawn:

In the high joint shear stiffness stage, the joint shear stiffness was influenced by pretension for both cable bolts and rock bolts. The higher the bolt pretension, the larger

Table 2 Specification of tested bolts

Bolt type	Diameter (mm)	Surface profile		Cross section	Structure feature	Tensile strength (kN)
		Thread or rib spacing	Profile			
FG	22	10	\	Solid	\	307
RB1	22	11.5	\	Solid	\	328
RB2	22	12	\	Solid	\	342
RB3	22	24	\	Solid	\	358
CB1*	22	\	Spirally ribbed	Solid	\	531
CB1	22	\	Smooth	Solid	\	590
CB2*	28	\	Spirally ribbed	Hollow	\	558
CB3*	28	\	Spirally ribbed	Hollow	Bird cage	573
CB3	28	\	Smooth	Hollow	Bird cage	637
CB4	28	\	Smooth + Spirally ribbed	Hollow	\	673

* Spirally ribbed cable wires

Fig. 8 Bolt types tested in double shear tests

CB1*

CB1

CB3*

CB3

CB4

CB2

RB1

RB2

RB3

FG
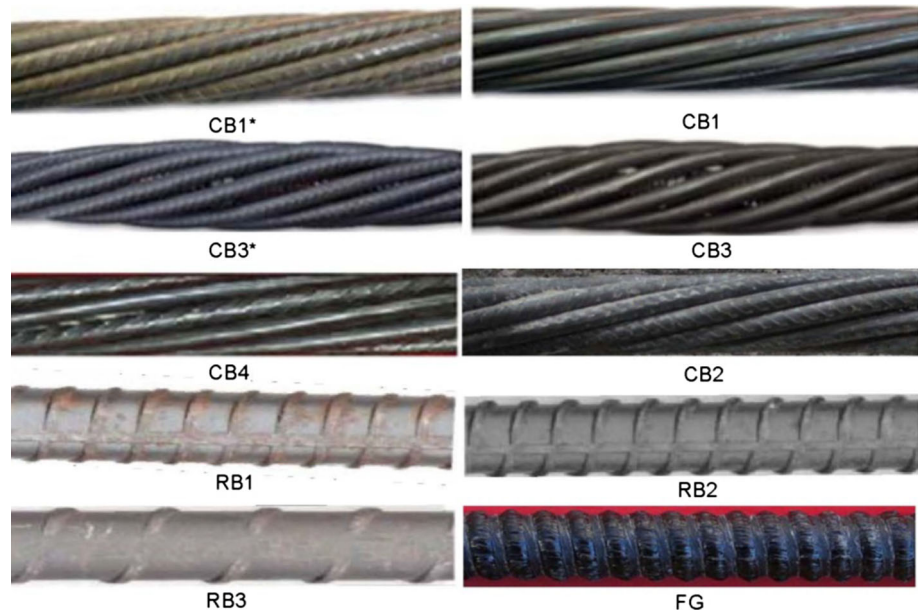
Table 3 Test results of 20 bolts shear-loaded to failure

| Bolt type	Concrete strength (MPa)	Pretension (kN)	Joint shear displacement at bolt failure (mm)	Peak shear force (kN)
FG	40	2.5	22	164
FG	40	4.5	22	183
FG	40	5	36	205
FG	40	15	18	220
FG	40	20	16	258
Average			23	/
CB1*	40	250	74	1115
CB1	40	250	65	1259
CB2*	40	250	73	1210
CB3*	40	250	33	829
CB3*	40	100	46	933
CB3	40	250	59	1424
CB3	40	100	79	1318
CB4	40	250	85	1601
CB4	40	100	98	1544
Average			68	/
RB1	20	20	92	762
RB1	20	20	81	813
RB1	20	20	86	821
RB1	20	20	75	756
Average			84	/
RB1	100	20	70	770
RB1	100	80	54	799
Average			62	/

* Spirally ribbed cable wires
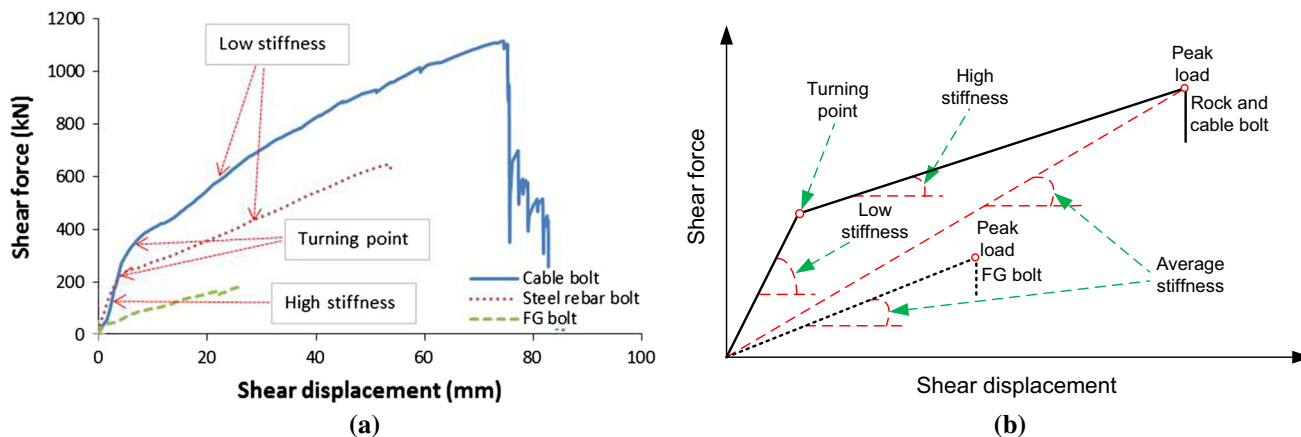
Fig. 9 Shear force vs shear displacement of FG bolts, rock bolts and cable bolts: **a** real test results; **b** schematic

Table 4 Shear stiffness of cable bolted joints (bolts were loaded to failure)

Bolt type	Pretension (kN)	Average shear stiffness (kN/mm)		Low/high
		High	Low	
CB1*	250	25.7	5.7	0.22
CB1	250	22.6	7.8	0.35
CB2*	250	19.9	6.8	0.34
CB3*	250	29.9	8.6	0.29
CB3*	100	20.0	8.7	0.44
CB3	250	23.7	8.2	0.35
CB3	100	15.3	7.1	0.46
CB4	250	19.4	7.7	0.39
CB4	100	13.2	7.2	0.54
Average		21.1	7.5	0.38

* Spirally ribbed cable wires

Table 5 Shear stiffness of rock bolted joints (bolts were not loaded to failure)

Bolt type	Pretension (kN)	Average shear stiffness (kN/mm)		Low/high
		High	Low	
RB1	0	12.6	1.7	0.13
RB1	20	26.2	3.9	0.15
RB1	50	24.4	4.7	0.19
RB1	80	32.6	5.7	0.18
RB2	0	15.7	1.4	0.09
RB2	20	20.7	5.2	0.25
RB2	50	24.5	4.6	0.19
RB2	80	25.0	5.4	0.21
RB3	0	14.5	1.0	0.07
RB3	20	32.4	4.7	0.15
RB3	50	38.5	3.6	0.09
Average		24.3	3.8	0.15

the joint shear stiffness. In addition, rock bolted joints had similar average shear stiffness to cable bolted joints in this stage.

In the low joint shear stiffness stage, the joint shear stiffness of cable bolted joints was not noticeably influenced by pretension. The influence of pretension on rock

Table 6 Shear stiffness of FG bolted joints (bolts were loaded to failure)

Bolt type	Pretension (kN)	Average shear stiffness (kN/mm)
FG	2.5	7.5
FG	4.5	8.3
FG	5	5.7
FG	15	12.6
FG	20	15.7

bolted joints was not very consistent. The joint shear stiffness of rock bolted joints was similar with different pretension loads, while it was very small without pretension. Additionally, the average shear stiffness of cable bolted joints doubled that of rock bolted joint. This was consistent with the larger plastic modulus of cable bolts compared to rock bolts as shown in Fig. 2.

The low–high stiffness ratio was also different between rock bolt and cable bolted joints. The average ratio of cable bolted joint was 0.38, while the rock bolted joint was only 0.15.

For FG bolted joints, the joint shear stiffness also increased with the increase of bolt pretension.

7 Bolt Contribution to Joint Shear Strength

There have been various analytical and experimental investigations undertaken looking at bolts and their contribution to joint shear strength (Ferrero 1995; Grasselli 2005; Jalalifar and Aziz 2010b; Pellet and Egger 1996). These studies suggest that two types of contribution, the frictional effect and the dowel effect, are made by a bolt to the joint shear strength. Figure 10 shows the typical loading state of a bolt-reinforced joint.

The bolt contribution to joint is:

$$R = N_o \sin\theta + Q_o\cos\theta + [N_o\cos\theta - Q_o\sin\theta]\tan\emptyset \qquad (2)$$

where, R is the bolt contribution to joint shear strength; N_o and Q_o are the tensile and shear forces, respectively, at the failure location of a bolt; θ and $\emptyset$ are the bolt deflection angle and joint friction angle, respectively.

In experimental analysis, there is another definition of the bolt contribution to joint shear strength based on recorded loads (Grasselli 2005; Jalalifar and Aziz 2010b)

$$R = F_s - 2N\tan\varphi \qquad (3)$$

where F_s is joint shear force; N is the externally applied normal force at the joint surface.

To compare bolts with different dimensions, the bolt contribution is normally standardized by the maximum tensile failure force of bolts as follows:
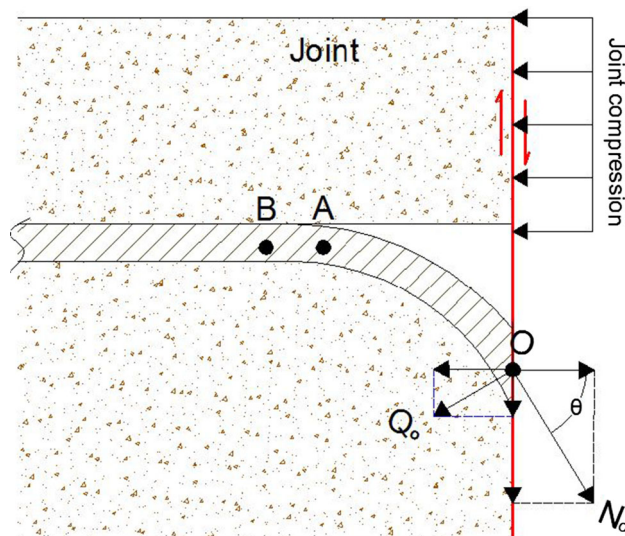

Fig. 10 Loads induced on joint and in tendon (adjusted from Xuwei et al. 2015)

$$T_c = \frac{R}{2F_{max}} \qquad (4)$$

where T_c is the normalized bolt contribution to joint shear strength; F_{max} is the bolt tensile strength.

Figure 11 shows the loading state of the bolt-reinforced jointed rock mass in the field after the bolt installation. Researchers simplified this field condition in the laboratory to study the reinforcing effect of bolts. Figure 12 shows the overall loading state of a typical shear system in the normal direction. Primarily two stages are included, the initial stage and the shear loading stage. In the initial stage, the normal force at the joint contains the bolt pretension and the initial external confining force:
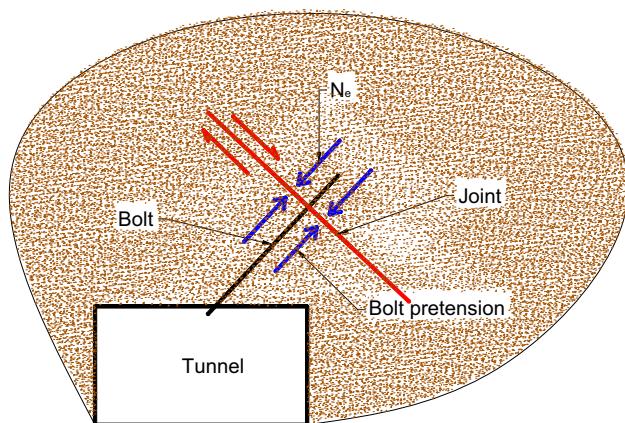
$$N_{joint} = N_{pretension} + N_e \qquad (5)$$

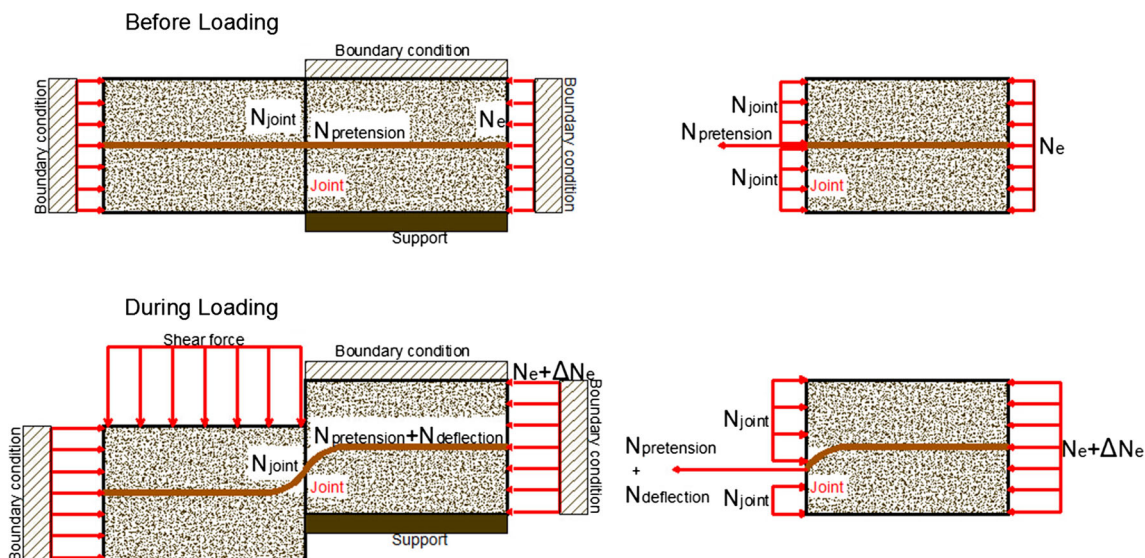**Fig. 11** Loading state of a bolt-reinforced jointed rock mass in the field

Fig. 12 Loading state of a simplified laboratory shear system in the normal direction

In the loading stage, besides the bolt pretension and the initial external normal force, the normal force at joint contains the normal force induced from bolt deflection as well ($N_{\text{deflection}} + \Delta N_e$). The induced normal force is attributed to the additional normal component of bolt force at joint ($N_{\text{deflection}}$) and the additional external normal force (ΔN_e). The additional external normal force is dominated by the boundary condition of the shear system. Thus, the total normal force at the joint during the shearing process is:

$$N_{\text{joint}} = N_{\text{pretension}} + N_{\text{deflection}} + N_e + \Delta N_e \qquad (6)$$

Regarding the normal confining force (N) at joints in Eq. (3), there are two types as shown in Fig. 12. One is related to the initial normal confining force applied on the concrete end surface before testing (Srivastava and Singh 2015), N_e, and the other is related to the additional normal force induced from bolt deflection and boundary confinement, ΔN_e. They can be simply named the initial external normal force and the induced external normal force, respectively. To attain the bolt contribution to joint shear strength, the bolt pretension ($N_{\text{pretension}}$) and the induced external normal force (ΔN_e) should be considered as part of this contribution. Thus in Eq. (3), N refers to the initial external normal force at joints, N_e, rather than the combination of the initial (N_e) and induced (ΔN_e) external normal forces and the bolt pretension ($N_{\text{pretension}}$). If no initial external normal force (N_e) is applied, N is equal to zero in Eq. (3). For all the tests presented in this paper, there was no initial external normal force, thus N is equal to zero.

The contribution of FG bolts, rock bolts and cable bolts was normalized by their tensile strength and listed in

Tables 7, 8, 9, respectively. All bolts listed in Tables 7, 8, 9 were loaded to failure during the test.

From Tables 7, 8, 9, it can be seen that the bolt contribution to joint shear strength was different for FG bolts, rock bolts and cable bolts. Firstly, the contribution of FG bolts was extremely small, and the average contribution was just 0.33 times its tensile strength. In contrast, the contribution of a rock bolt was higher than its tensile strength and the average contribution was 1.20 times its tensile strength. Similar to the rock bolt, the cable bolt contribution was normally larger than its tensile strength as well, but slightly lower than rock bolts.

In addition, for FG bolts, rock bolts and cable bolts, their contribution to joint shear strength all increased with the increase of bolt pretension except the hollow cable bolt T3*. T3* is a cable bolt with spirally ribbed cable wires, which exhibits a different pretension effect compared to other FG bolts, rock bolts and cable bolts.

Looking back at the calculation of bolt contribution to joint shear strength in Eq. (2), it is known that the bolt contribution is attributed to two parts, the direct contribution and the indirect contribution (friction effect) of bolt axial and shear forces. When the loading state of a bolt remains stable, the larger the bolt force component perpendicular to a joint, the lower the joint shear capacity. This is due to the contribution of perpendicular force component being weakened by the friction coefficient which is less than one in these tests.

For FG bolts, rock bolts or cable bolts, their shear strength is much lower than their tensile strength. So, in order to achieve higher shear strength of a bolted joint, it is better to reach bolt failure at higher axial force rather than

Table 7 Normalized FG bolt contribution to joint shear strength (bolts were loaded to failure)

Concrete strength (MPa)	Pretension (kN)	Tensile strength (kN)	Failure shear force (kN)	Normalized bolt contribution
40	2.5	307	164	0.27
40	4.5	307	183	0.30
40	5	307	205	0.33
40	15	307	220	0.36
40	20	307	258	0.42
Average				0.34

Table 8 Normalized rock bolt contribution to joint shear strength (RB1) (bolts were loaded to failure)

Concrete strength (MPa)	Pretension (kN)	Tensile strength (kN)	Failure shear force (kN)	Normalized bolt contribution
20	20	328	762	1.16
20	20	328	813	1.24
20	20	328	821	1.25
20	20	328	756	1.15
100	20	328	770	1.17
100	80	328	799	1.22
Average				1.20

Table 9 Normalized cable bolt contribution to joint shear strength (bolts were loaded to failure)

Cable type	Concrete strength (MPa)	Pretension (kN)	Tensile strength (kN)	Failure shear force (kN)	Normalized bolt contribution
CB1*	40	250	531	1115	1.05
CB1	40	250	590	1259	1.07
CB2*	40	250	558	1197	1.07
CB3*	40	250	573	829	0.72
CB3*	40	100	573	933	0.81
CB3	40	250	637	1424	1.12
CB3	40	100	637	1318	1.03
CB4	40	250	673	1601	1.19
CB4	40	100	673	1544	1.15
Average					1.02

* Spirally ribbed cable wires

higher shear force. In a perpendicularly bolted joint, the larger the joint shear displacement at bolt failure, the higher the axial force in the bolt. Therefore, the joint shear strength is higher as well. In addition, the deformation modulus of a bolt also influences the loading state of the bolt. Bolts with higher deformation modulus have higher capacity to take both shear and axial forces at a specific deformation.

Considering the tensile behavior of FG bolts, rock bolts and cable bolts as shown in Fig. 2, it is clear why the FG bolt contribution to joint shear strength was so small. Due to the approximate invariability of FG bolt modulus before final failure, force increment was almost constantly proportional to bolt deformation, thus the shear force in FG bolts was comparably large at bolt failure while axial force was far from its tensile strength compared to rock bolts and cable bolts. Thus FG bolt failure was primarily due to the approach of bolt shear force to the shear strength rather than the axial tensile force compared with rock bolt and cable bolt as shown in Fig. 13. FG bolts tended to fail at
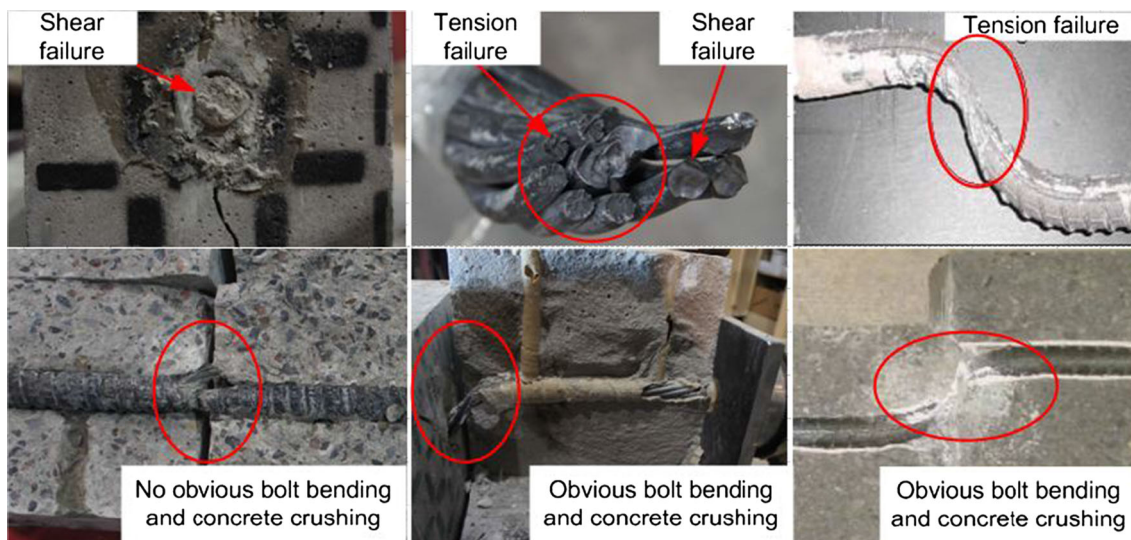
Fig. 13 Failure modes of FG bolt, rock bolt and cable bolt in reinforcing 40 MPa concrete joints

smaller shear displacement with smaller axial tensile force and larger shear force, which accordingly yielded lower joint shear strength.

In addition, generally rock and cable bolts have high strength to carry loads both in tension and in shear, while FG bolts have only high tensile strength but considerably low shear strength. Even though FG bolts cannot carry high load in shear, they are still widely used in reinforcing coal ribs in underground coal mines because the shear displacement in coal ribs is not severe.

8 The Direct Bolt Contribution to Joint Shear Strength

From Eq. (2), it is known that the joint shear strength is attributed to two parts, the direct contribution and the indirect contribution of bolts. The direct contribution is related to the parallel component of axial and shear forces to the joint, while the indirect contribution is related to the perpendicular component of axial and shear forces to the joint. The bolt direct contribution corresponds to the bolted joint shear strength with perfectly smooth joint surface contact or without joint surface contact. From the direct contribution of a bolt to a rough joint, the bolt contribution to a smooth or a detached joint can be assessed as well to some degree.

Table 10 shows the bolt direct contribution to joint shear strength with reinforcement of FG bolts, rock bolts and cable bolts, respectively. Equation (3) was used in this calculation. In Table 10, the shear strength of FG bolts was obtained from punch tests on thin FG plates cut from FG bolts (David et al. 2015), and the half

tensile strength of rock bolt and cable bolt were used as their shear strength.

It should be noted that in the calculation of the bolt direct contribution in Table 10 the reading of load cells at cable end was considered as the same as the normal force at the joint. This was not very accurate because the consumption of axial force in bolts between joints and load cells was ignored. Since CB3 and CB3* were bird-caged cable bolts, the consumption of axial force was very large. Thus they were neglected in Table 10.

From Table 10, it can be seen that the direct contribution of FG bolts was different from rock bolts and cable bolts. The increase in shear strength of FG bolts ranged roughly between 0.73 and 1.8 times bolt shear strength. The shear strength increase of FG bolts was improved by bolt pretension. In comparison, the increase in shear strength of rock bolts and cable bolts both fell in the range of about 0.4–0.8 times their shear strength and their pretension effect was the same as FG bolts. So the increase in joint shear strength of FG bolts was much higher than the rock bolts and cable bolts counterparts. This was mainly due to the small shear strength of FG bolts.

9 Conclusions

The tensile strength and shear strength and deformation modulus of bolts are considered as important factors influencing the shear behavior of bolted rock joints. FG bolts, rock bolts and cable bolts are three commonly used bolts in reinforcing rock masses and their properties are different. Accordingly, they probably perform differently in reinforcing rock joints.

Table 10 Direct contribution of different bolts to joint shear strength (bolts were loaded to failure)

Bolt type	Pretension (kN)	Average peak axial force (kN)	Peak shear force (kN)	Joint friction coefficient	Direct contribution (kN)	Bolt shear strength (kN)	Increase in shear strength
FG	2.5	29	164	0.44	69	40	0.73
FG	4.5	43	183	0.44	73	40	0.81
FG	5	62	205	0.44	75	40	0.88
FG	15	31	220	0.44	96	40	1.41
FG	20	40	258	0.44	111	40	1.79
RB1	20	227	762	0.6	245	164	0.49
RB1	20	196	813	0.6	289	164	0.76
RB1	20	219	821	0.6	279	164	0.70
RB1	20	210	756	0.6	252	164	0.54
CB1*	250	400	1115	0.44	382	265.5	0.44
CB1	250	430	1259	0.44	440	295	0.49
CB2*	250	410	1197	0.44	418	279	0.50
CB4	250	462	1601	0.44	597	336.5	0.77
CB4	100	488	1544	0.44	557	336.5	0.66

* Spirally ribbed cable wires

By experimentally comparing the shear reinforcing effect of FG bolts, rock bolts and cable bolts with the consideration of their tensile properties, the following conclusions are drawn:

- The sheer force of an FG bolted joint increased gradually from the beginning to end, while the sheer force of joints reinforced by either rock bolt or cable bolt increased bi-linearly, which is clearly consistent with their tensile deformation modulus.
- The joint shear stiffness was highly influenced by bolt pretension in the high stiffness stage, but not in the low stiffness stage for both rock bolts and cable bolts.
- The rock bolt contribution to joint shear strength standardized by the tensile strength was the largest, followed by cable bolts, then FG bolts.
- Rock bolts and cable bolts tended to fail in tension at large shear displacements, while FG bolts in shear at small shear displacements due to its low shear strength and constant deformation modulus.
- The direct contribution of FG bolts to the joint shear strength ranged from 0.73 to 1.8 times its shear strength, while it was between 0.4 and 0.8 times their shear strength for rock bolts and cable bolts.

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