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Experimental and numerical study of double shearing of bolt under confinement

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Experimental and Numerical Study of Double Shearing of Bolt under Confinement

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ABSTRACT

The shear behaviour of reinforced rock joints is investigated for the bolt-grout-rock interaction and for failure mechanism. The effectiveness of the bolt application under lateral and axial loading conditions within surrounding materials is investigated in different medium strength. Double shearing testing of bolts were studied in concrete blocks of 20, 40, 50 and 100 MPa strengths, subjected to different pretension loads of 0, 5, 10, 20, 50 and 80 KN respectively. The experimental study was complemented with three-dimensional numerical analysis. Parameters examined include: shear resistance, shear displacement, induced strains and stresses during bolt bending process and its ultimate failure across the sheared joint planes. The conclusions drawn from the study were; the level of bolt resistance to shearing was influenced by the bolt profile configuration, the strength of the rock or medium influenced the level of load generated on the bolt, and the increase in bolt pretension has contributed to the increased shearing load of the bolted medium.

INTRODUCTION

Rock bolts are one of the most popular systems of support in underground mining, tunneling operations. Speed, effectiveness, minimum installation space and cost, are other factors have contributed to the increasing acceptance of the bolting as a favourable support system in underground structure stabilisation. Two schools of thoughts are emerging with differing views on bolt pretension; however, one issue that remains a great challenge to the geotechnical engineers is how to minimize the influence of shearing forces across joints and planes in the vicinity of an underground excavation. This challenge broadens with increasing recognition of the role played by high horizontal stress on ground condition particularly in Australian underground mining situation. Additionally, bolt surface profile has emerged as another important parameter, which has been found to have significant effect on load transfer mechanism of a bolt (Aziz et al., 2003). The study was conducted experimentally by double shear testing of a fully encapsulated bolt installed in a three-piece concrete blocks. With the middle concrete block section being sheared relative to side blocks, the paper highlighted the effective role that the bolt surface profile configuration and pretensioning played in rock/ concrete reinforcement.

Research on sheared surface reinforcement has been pursued, with increasing vigor in recent years, since the benefits of full encapsulation was realized and appreciated. Bjurstrom (1974) conducted a series of shear tests on fully cement bonded rock bolts embedded in blocks of granite. According to Bjurstrom, the failure mode, the strength and deformational stiffness of a bolted joint was dependent on the angle of bolt installation across the joint. Azuar (1977) carried out experimental tests with resin-grouted bolts embedded in concrete. Azuar reported that the maximum contribution of the bolt to the shear resistance of the jointed surfaces was influenced by between 60 to 80 % of the ultimate tensile load installed perpendicular to the joint surface and 90 % at the inclined state. Hibino and Motojima's (1981) experimental tests with 2 mm diameter bolts in concrete blocks showed that the fully bonded bolts had higher shear resistance than that of point anchored ones. Also the inclination of the bolt did not increase the shear resistance, which is in contradiction to other researcher's findings. Dight's (1982) study found that the effect of dilatancy was similar to the bolt inclination. Bolt deformability was related to the deformability of the rock, and that the bolt experienced a combination of both the shear and tension stresses. Ferrero (1995) conducted laboratory tests on reinforced rock joints with different medium materials and different steel bars. He found that, bolt pretensioning influenced the stress-strain behaviour of the reinforced joint. Pellet and Egger (1996) reported that in hard rock environment, bolt failure occurred at smaller displacement. Kharchafi and et al (1999) carried out large scale (1:1) laboratory tests of fully grouted frictional bolts using double shearing technique and found that the bolt failure was principally caused by traction concentrated between the two hinges.

Clearly, and based on the past research studies in the field, there remains a considerable scope for further improvement in understanding of the interaction between the bolt, grout and rock, particularly in relation to shearing across joints, and also the importance of bolt surface profile configurations. Accordingly, this paper is aimed to address these issues both experimentally as well as by 3D numerical simulation.

EXPERIMENTAL STUDY

Figure 1 show the general set up of the double shear box unit in a testing machine. Double-jointed concrete blocks were cast for each double shearing test. Three different strengths of concrete blocks at 20MPa, 40MPa and 100 MPa were cast to simulate three different rocks strength. One thousand four hundred mm long bolt, threaded 100 mm on both ends was then installed in the concrete mould specimen using Minova PB1 Mix and Pour grout resin. Care was taken to install all the bolts in their respective concrete blocks with uniform profile /flash orientation.

Figure 1. General set-up of the assembled double shear box unit in the testing machine

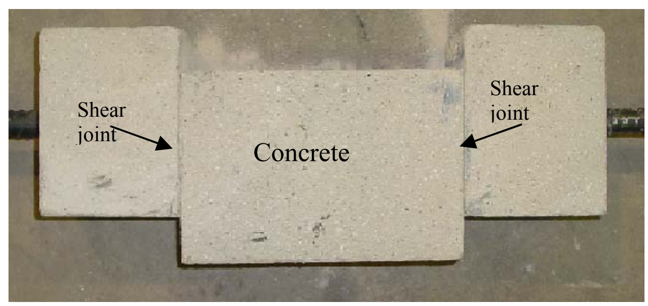
The concrete/bolt assembly was then mounted in a steel frame shear box fabricated for this purpose. A base platform, that fitted into the bottom ram of both the 500 kN capacity Instron Universal Testing Machine and 5000 kN Avery machine, capacity 500 kN and 1500 kN respectively, was used to hold the shear box in place. The two outer ends of the shear box were then clamped tightly with the base platform to avoid toppling of the blocks during shearing process. A predetermined tensile load was applied to the bolt prior to shear loading. This pretension force acted as a compressive / confining pressure to simulate different forces on the joints within the concrete. The pretension loads applied were 0, 20kN, 50kN and 80kN respectively. Axial tensioning of the bolt was accomplished by tightening simultaneously the nuts on both ends of the bolt. The applied axial loads were monitored by two 300 kN capacity hollow load cells mounted on the bolt on either side of the block. The outer sections of the shear box remained fixed as the central block was pushed down. Five bolt types were used for the study with their specifications shown in Table 1. Only Bolt Type T1 was tested in 100 MPa concrete.

Table 1: Bolt profile details

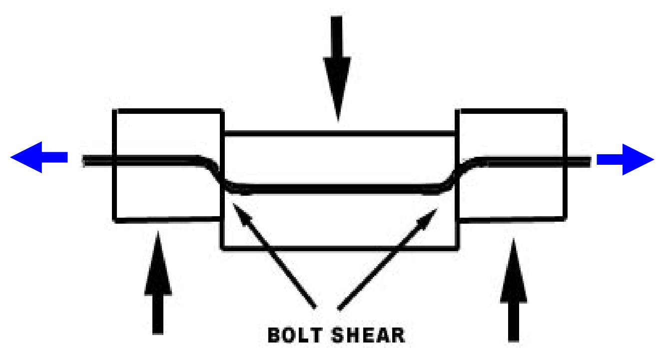
Specifications \ Bolt type	T1	T2	T3	T4	T5
Bolt core dia. (mm)	21.7	21.7	21.7	10.7	12
Profile centres	12.00	12.50	25	-	-
UTS (kN)	330	340	340	44	67
Yield point load (kN)	250	256	247	38	57
Profile height (mm)	0.65	1.40	1.25	-	-
Profile angle (^{0})	21.5	21.5	21.5	-	-
Profile top width (mm)	1.50	2.00	2.50	-	-
Profile base width (mm)	3.00	4.00	5.00	-	-

EXPERIMENTAL RESULTS AND DISCUSSION

Figures 2 a, b and 3 show the photographs of shear reinforced concrete blocks and stripped deformed bolts respectively. The resin encapsulation layer along the length of the double sheared bolts clearly shows the zones of tension and compression. The resin layer remained adhered to the surface of the bolt section that undergone compression during bending, while it had broken off the surface that were in tension.



(a)



(b)

Figure 2. Deformed shape of the blocks after the test (a) real sample (b) the schematic with applied and induced forces

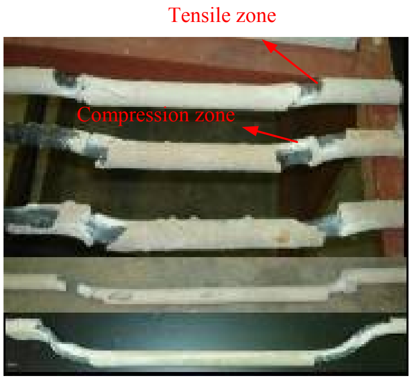
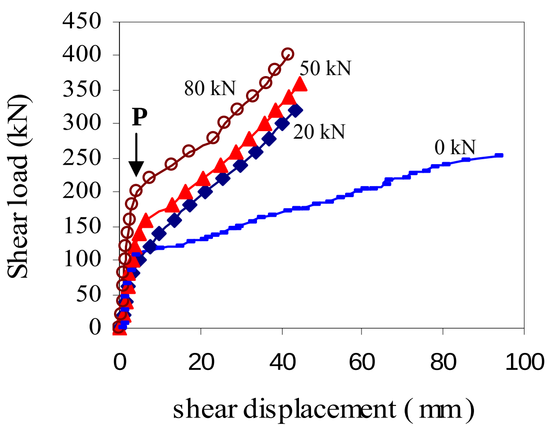
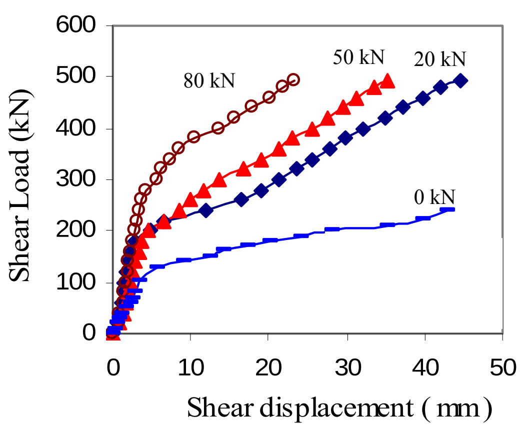


Figure 3. Bolt deformation with tension and compression zones after the test

Figure 4 to 6 show graphs of shear load and displacement of bolts tested in both 20 and 40 MPa strength concrete and under different pretension loads of 0, 20, 50 and 80 kN respectively. Each graph comprises two parts; The pre-yield point (P) with high stiffness section indicating the increased resistance to shear load and post yield point load-displacement section with increased rate of bolt bending, which is known as plastic section (Figure 4). The peak shear load is determined at point "P".

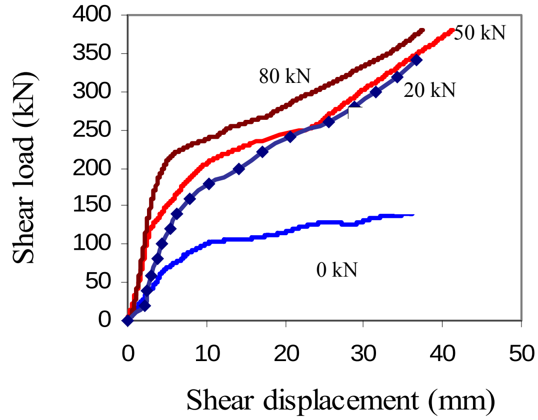


(a) Bolt Type T1 in 20 MPa concrete

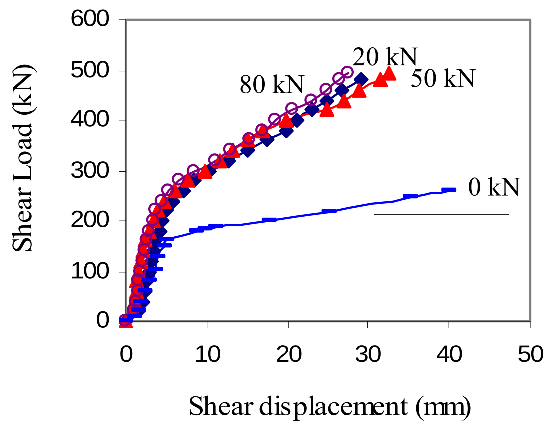


(b) Bolt Type T1 in 40 MPa concrete

Figure 4. shear load and vertical displacement profiles in both (a) 20 and (b) 40 MPa concrete medium, Bolt Type T1

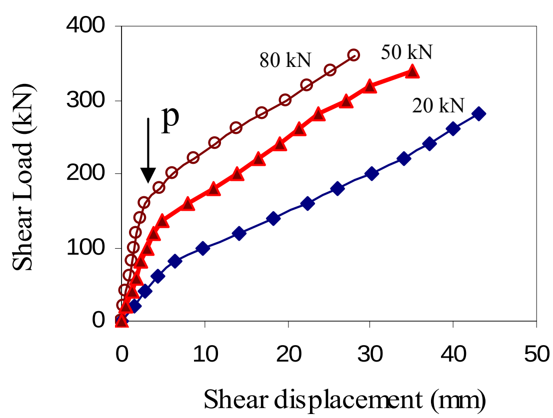


(a) Bolt Type T2 in 20 MPa concrete

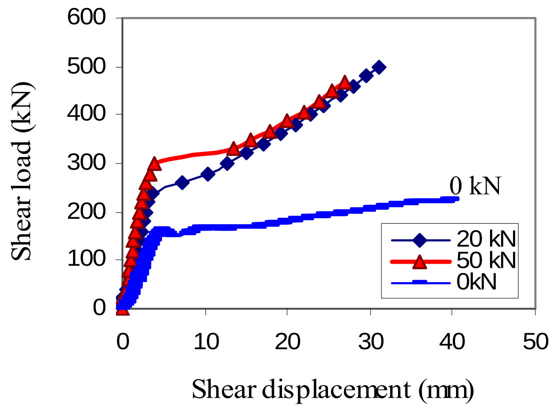


(b) Bolt Type T2 in 40 MPa concrete

Figure 5. Shear load and vertical displacement profiles in both (a) 20 and (b) 40 MPa concrete medium, Bolt Type T2



(a) Bolt Type T3 in 20 MPa concrete



(b) Bolt Type T3 in 40 MPa concrete

Figure 6. Shear load and vertical displacement profiles in both (a) 20 and (b) 40 MPa concrete medium, Bolt Type T3

Figure 7 shows the comparative results in different bolt types and concrete medium (20 and 40 MPa).

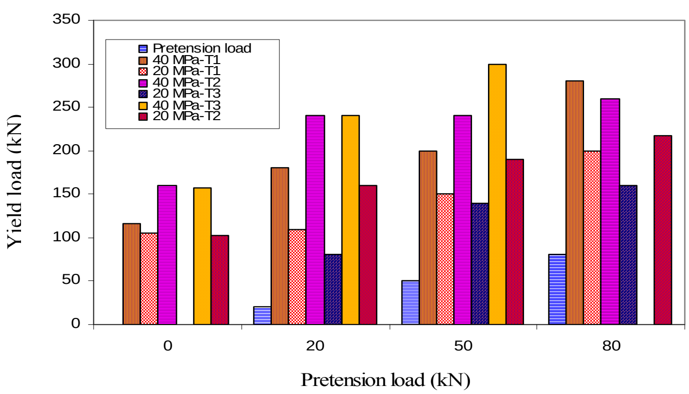


Figure 7. Comparative results in different bolt type and concrete medium

Figures 8 shows the load displacement profiles of the bolts T4 and T5 tested under different axial load conditions. The level of maximum shear load and displacement were different because of different pretension loads. The relationship between shear yield load and pretensioning in Bolt Type T5 and failed sheared bolt Type T5 are shown in Figure 9. α in Figure 9 a is the slope angle of the rate of failure over the maximum tensile load with respect to increased pretension loads. β is the angle between bolt failure surface and the bolt cross section line and γ is the angle between the bolt failure surface and applied shear load direction, or sheared joint plane.

Figure 10 shows the load displacement profiles of the bolt Type T1 in different pretensioning in 100 MPa concrete. The excessive bolt necking in 100 MPa concrete is shown in Figure 11. Points A and O are known as the hinge points, which are the points of maximum bending moment locations. Figure 12 shows the failed bolt across the joint planes and the crushed zones within the vicinity of the sheared planes in Bolt Type T1 in 100 MPa concrete with 80 kN pretension load.

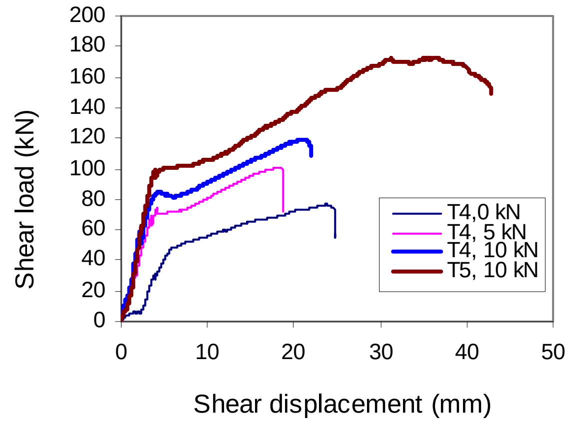
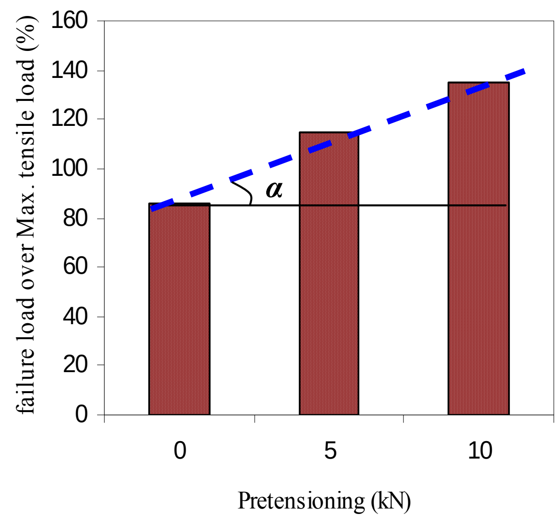
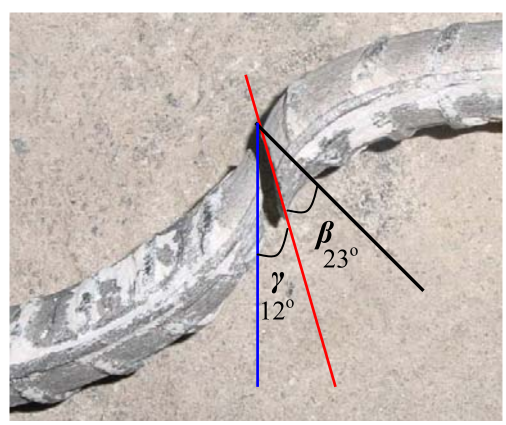


Figure 8. Shear load versus shear displacement in 0, 5 and 10 kN pretension load in Bolt Types T4 and T5 in 40 MPa concrete



a



b

Figure 9 (a) Relationship between failure load and maximum tensile strength of the single shear on bolt type T5, (b) bolt failure angle

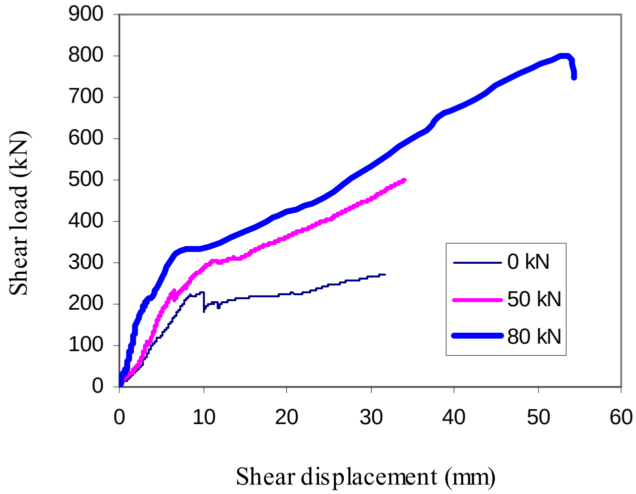


Figure 10. Shear load versus shear displacement in 100 MPa concrete and different pretensioning, Bolt Type T1

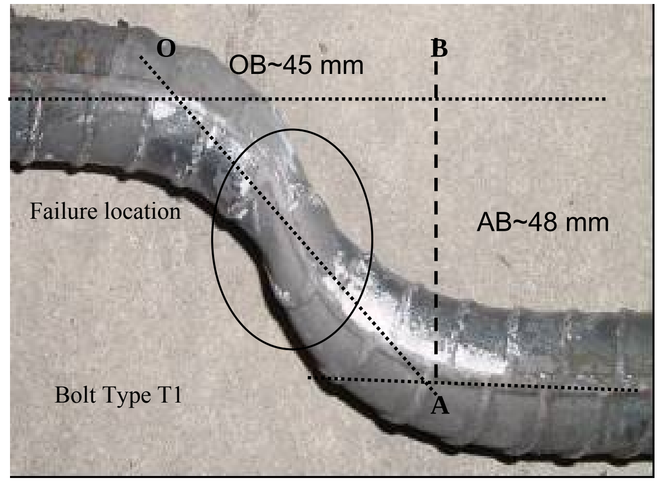


Figure 11. Excessive bolt necking in concrete 100 MPa in 80 kN pretension load, Bolt Type T1

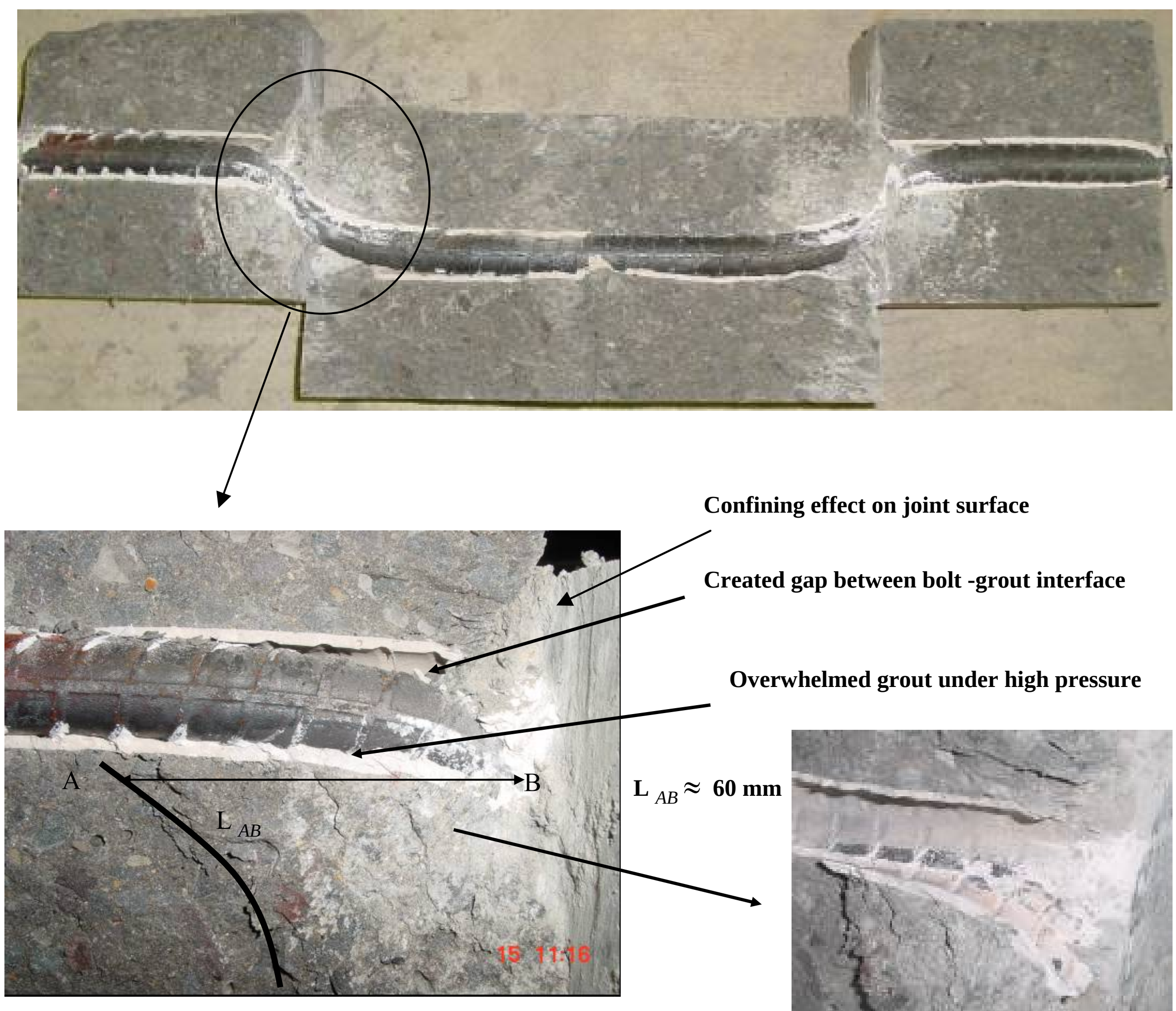


Figure 12. Bolt/ joint concrete interaction at shear joint in concrete 100 MPa and 80 kN pretension load

The following can be deduced from the load /displacement data and graphs by experimental results:

1. For the increase in pretension load from 20 kN to 80 kN, the peak elastic shear load (P) values for the three types of bolts increased by 81% for Bolt Type T1, 45% for Bolt Type T2 and 100% for Bolt Type T3. In 40 MPa concrete the respective values were 55 % increase in Bolt Type T1, and 9 % in Bolt Type T2. The peak value in Bolt Type T3 increased 25 % from 20 to 50 kN pretension load. This means that the tensioned bolt acts as an active support system and provided the confining pressure to the sheared joint surfaces.
2. The peak elastic shear load displacement level for the given axial pretension load was dependent on the bolt type. This displacement was more likely to decrease with increased pretension load.
3. The strength of the medium has influenced the shear load level but not the trend. Shear load values for all bolts were generally less in 20 MPa concrete medium in comparison to the shear load values of bolts tested in 40 MPa concrete.
4. Bolt Type T2 displayed consistent shear load/displacement profiles at all three levels of bolt pretension loads (i.e., 20, 50, and 80 kN) particularly in 40 MPa strength. This consistency was relatively less in 20 MPa concrete.
5. As shown in Figures 10, shearing of the bolt without bolt pretension can lead to an early loss of resin/bolt bonding and inward pulling and bending of the bolt, leading to excessive gap formation. This situation became worse when the bolt ends were not fitted with nuts and plates to hold against the concrete block ends. The presence of end plat plays importance role in providing better structure reinforcement.
6. The snapping or failure of the bolt across joint planes, were the results of both shearing and tensile loading. This is because the failed surfaces of the bolt were not vertical and parallel to the sheared vertical joint planes. The failed sheared bolt surface angle (α) was in the order of 12^{o} from the sheared joint plane (Bolt Types T4 and T5).
7. Bolt necking began around the peak elastic yield point. Noticeable necking was evident because of the predominately tensile load at the bolt joint plane intersection. When necking commences, the bolt diameter decreases at the effective length, which is between the hinge points in the vicinity of the shear joint plane.
8. For the pretension load of 80 kN, the shear displacement at failure for Bolt Type T5 was 40% higher than the corresponding shear displacement for Bolt Type T4. As Figure 9a shows the relationship between the failure load and the maximum tensile strength of the Bolt Type T5 in different pretension, indicating that the slope of the relationship is in the order of 18^{o}. These results contradicted Ferrero's result (1995), which stated that the pretension does not influence the maximum shear resistance of the system. Ferrero's tests were undertaken in a single shear test box, whereby the pretension loads were applied to one side of the bolt.
9. The displacement rate of the sheared bolted block in 100 MPa strength concrete was, as expected, lower than in both 20 and 40 MPa concrete respectively.
10. The crushed zones in 100 MPa concrete were less that those obtained in 40 MPa concrete. The length of the crushed zone was in the order of 60 mm on either side of the joint plane. This clearly demonstrated that during shearing there was significant resistance from the concrete and hence less vertical displacement.

3D NUMERICAL SIMULATION

Three Dimensional Finite Element modeling, (ANSYS, Version 9) was used to simulate the behaviour of bolted rock joints subjected to shearing. Induced stresses and strains along the bolt were evaluated and the results were compared with the experimental data. Three governing material (steel, grout, rock) with two interfaces (bolt-grout and grout-rock) were considered for the simulation.

The model bolt core diameter (D_b) of 22 mm and the grouted cylinder (D_h) of 27 mm had the same dimensions as those used in the laboratory test. Due to the symmetry of the problem, only one fourth of the system was considered here. Figure 13 shows the three-dimensional numerical model. The stress-strain relationship of the steel is assumed as a bilinear kinematics hardening model and the modulus of elasticity of strain hardening after yielding is taken as one hundredth of the original value.

The 3D solid elements (Solid 65 and solid 95) that have 8 nodes and 20 nodes were used for the concrete, grout and steel, with each node having three translation degrees of freedom, which tolerated irregular shapes without significant loss in accuracy. The 3D surface-to-surface contact element (contact 174) was used to represent the contact between 3D target surfaces (steel-grout and rock-grout).

The results of numerical modeling in different rock strengths and different pretension effects are shown in the following Figures. The numerical simulations were found to be in good agreement with the experimental results.

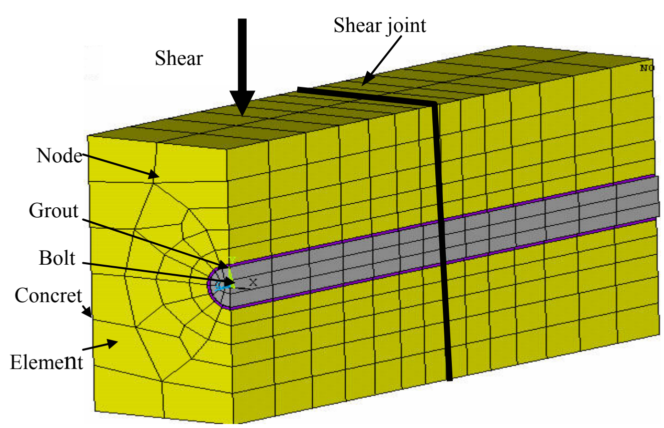


Figure 13. Three-dimensional meshed model of a quarter section of the composite concrete/resin and bolt

NUMERICAL RESULTS

By increasing bolt pretension, the tensile stresses in the axial direction of the bolt are expanded, while the compressive stresses reduced. This trend was clearly more evident in the linear stress-strain region of the bolt compared the post failure region of the bolt, where the hinge points are located. Clearly pretension load in post failure state has significantly affected bolt deflection, which was also demonstrated by both the numerical and experimental results. It shows that the stresses at these zones (tension and compression) are significant and the bolt appears to be in yield state. The shear displacement is decreased with increasing the strength of surrounding concrete. Increasing the confining pressure causes a reduction in bolt deflection, but this reduction, if occurred prior to elastic yield point, is not significant as demonstrated in both the experimental and numerical results.

From the results it was found that the maximum shear stress is concentrated in the vicinity of the joint plane, and these stresses slowly increase, beginning with the plastic deformation and ending with a stable situation as discussed previously. By increasing the bolt initial tensile load, the shear stress was decreased and this was also observed in different concrete strengths.

Figure 14 shows the trend of changes in stress profile with the shear stress tapering off to a stable state past the yield point. It means the shear stress at bolt joint intersection beyond the yield point is almost constant in spite of increasing bolt deformation.
With regard to bolt deflection, plastic strain is induced in critical locations in all three materials (bolt, resin and concrete). The strains and the rate of strain changes along the model in 20 MPa concrete are shown in Figures 15 and 16.

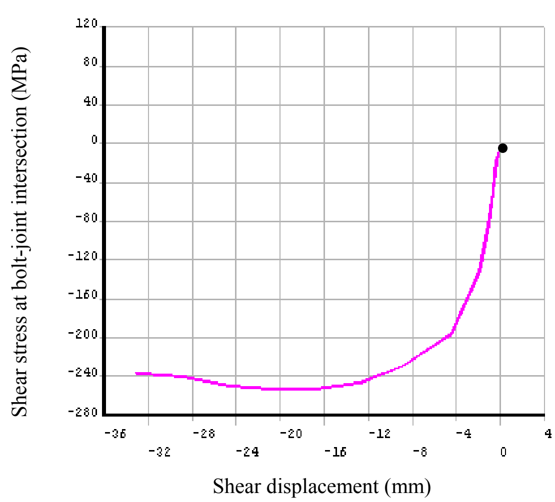


Figure 14. Shear stress trend in bolt –joint intersection in 20 MPa concrete in post failure region and 20 kN pretension load

From the numerical simulation it was found that the plastic strain was slightly decreased with increasing pretension load. As the Figure 15 shows the outer fiber of the bolt has yielded, whereas the middle part of the bolt cross section still has remained in the elastic state. Figure 17 shows the induced stresses in 20 MPa concrete in non-pretension load. As it shows concrete was completely crushed and overwhelmed at vicinity of shear joint.

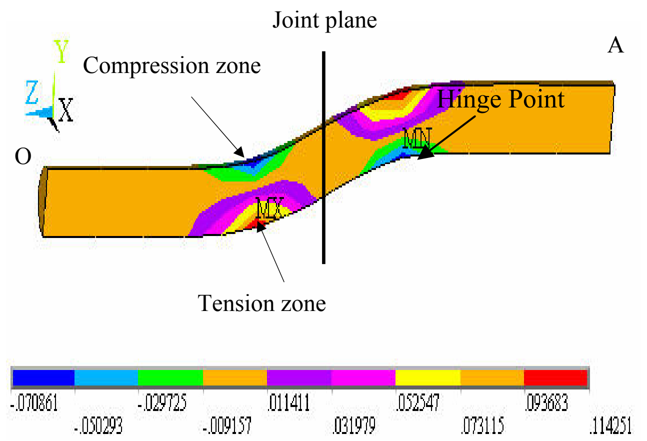


Figure 15. Plastic strain contour along the bolt axis in 20 MPa concrete, without pretension load

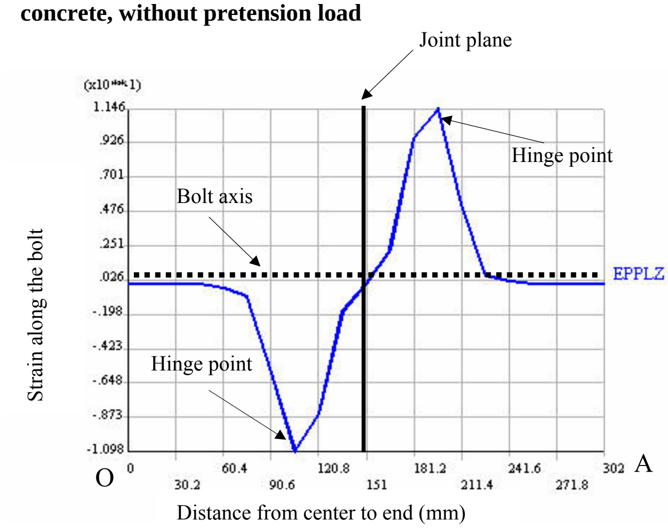


Figure 16. Plastic strain trend in 20 MPa concrete in upper fiber of the bolt without pretension load

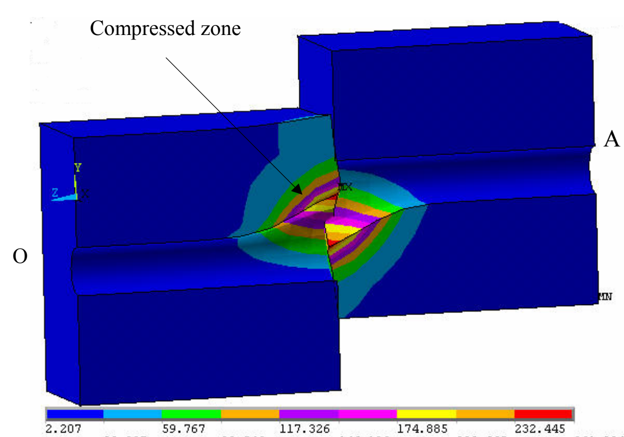


Figure 17. Yield stress induced in 20 MPa concrete without pre-tension

CONCLUSION

The study on the interaction between bolt, grout and rock during shearing process was carried out both in the laboratory and by the numerical modelling simulation. Research has lead to the following conclusions:

1. The design of bolt profiles has significant influence on the load transfer capacity between bolt, resin and rock.
2. The strength of concrete has influenced the shear load level and shear deformation. Shear load values for all bolts were generally less in 20 MPa concrete in comparison to that of 40 MPa and 100 MPa concrete.
3. The strength of the concrete and the level of bolt pretension influenced both the elastic limits and stiffness of the bolt.
4. The level of initial confining axial load applied to bolts had profound influence on the applied shear load
5. The numerical simulation of double shearing process provided a better understanding of the interaction between bolt, resin and concrete. The development of stress and strain profiles by the shearing process enabled the appreciation of the level and role of the bolt pretensioning played in joint plane strengthening during shearing process.

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