An Assessment of Load Transfer Mechanism Using the Instrumented Bolts

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AN ASSESSMENT OF LOAD TRANSFER MECHANISM USING THE INSTRUMENTED BOLTS

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\textit{ABSTRACT:} Load transfer capacity and failure mechanism of a fully grouted bolt installed across joint plane in shear is evaluated experimentally and numerically, tests were made on un-instrumented and strained gauge instrumented bolts. Four types of bolts, with different properties and surface configurations were selected for the study. The changes in strength of the concrete, bolt mechanical properties and bolt pretension load were evaluated in different shear environments. Results from the instrumented bolts and numerical simulation showed that the tensile and compression stresses and strains are generated at early stage of loading and hinge points are created at both sides of the shear joints. Strains are in greater value in vicinity of the shear joint. The failure location moves towards the bolt joint intersection due to the increasing shear load, shear displacement and axial load developed along the bolt.

\textbf{INTRODUCTION}

Rocks bolting is the most common form of ground support in use in both civil and mining excavation engineering and are used for ground reinforcement as both temporary and permanent systems to support the ground. The efficiency of the reinforcement system depends on the load transfer mechanism and the shear stress sustained at the bolt - joint interface. Factors influencing the shear resistance are; bolt diameter, hole diameter, steel quality, confining pressure, and concrete strength (Bjurstrom 1974), (Azuar 1977), (Hibino and Motojima 1981), (Dight 1982), (Spang and Egger 1990), (Ferrero, 1995), (Grasseli 2004).

In the previous extensive series of experimental tests, in double shear method four different concrete strengths, 20, 40, 50, and 100 MPa and various pretension loads 0, 5, 10, 20, 50 and 80 kN were used. From the analyses it was found that;

- Bolt profiles plays a significant role in load transfer mechanism,
- Bolt pre-tension increases the level of shear resistance,
- The resistance of the bolt will depend on the strength of the concrete
- Increasing the strength of the concrete reduced joint shear displacement and increased shear stiffness.

To gain a clear understanding of the pattern of load and stress build up along the bolt, two tests from the extensive number of above tests were carried out on strain gauged instrumented bolts (both of Bolt Type T2), one test was made with a bolt not subjected to pretension load (zero pretension) and the other with a pretension load of 20 kN. During the shearing process, the bolt was deformed with joint displacement. The longitudinal axis of the sheared bolt is deformed into a curve producing a lateral shear load, an axial load, and two critical points: one in bolt-joint intersection and the other at the hinge point. These loads produce stress resultants in the form of bending moment, shear and axial forces throughout the beam at the bolt - joint intersection or at the hinge points. In addition, several tests were carried out until bolts reached to failure and necking and cutting were appeared.

\textbf{EXPERIMENTAL STUDY}

Laboratory tests were carried out in four types of bolt. These were bolt Types \textit{T_1}, \textit{T_2}, \textit{T_3} and \textit{T_4} (high strength steel and low strength steel). The tests were carried out in three-piece pre -cast concrete blocks, of strengths 20, 40, and 100 MPa respectively. Concrete was used to simulate different rocks, as it was easier to prepare and to simulate different strengths. Minova PB1 Mix and Pour resin was used to install bolts in precast and reamed holes in different strength concrete blocks. Bolt and hole diameters were proportioned to maintain resin thickness encapsulation constant. Figure 1 shows the profiles of various bolts used in the study. Bolt characteristics are shown in Table 1.

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To achieve a meaningful result, various tests were carried out including:

- Shear testing of the bolts in high capacity compression machine (5000 kN), thus allowing the bolts to snap at the sheared joint plane.
- Shear testing of the instrumented bolts for determining the hinge points position and strain built up along the bolt.
- Testing of the bolts in higher strength concrete.
- Shear testing of lower strength steel bolts, which allowed the bolt to fail at much lower shear loads.

Figure 2 shows the general set-up of the assembled double shear box in a 500 tonne capacity compression testing machine and the photographs of different deformed bolts. Tests were made with and without pretension loads of 5, 10, 20, 50 and 80 kN.

<table>
<thead>
<tr>
<th>Bolt Type</th>
<th>Rib Spacing (mm)</th>
<th>Core diameter (mm)</th>
<th>Rib height (mm)</th>
<th>Max. Tensile load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T₁</td>
<td>11.5</td>
<td>21.7</td>
<td>1.0</td>
<td>328</td>
</tr>
<tr>
<td>T₂</td>
<td>11.5</td>
<td>21.7</td>
<td>1.5</td>
<td>342</td>
</tr>
<tr>
<td>T₃</td>
<td>1.4</td>
<td>10.3</td>
<td>0.6</td>
<td>44</td>
</tr>
<tr>
<td>T₄</td>
<td>7.74</td>
<td>11.7</td>
<td>0.8</td>
<td>67</td>
</tr>
</tbody>
</table>

**DOUBLE SHEARING OF INSTRUMENTED BOLT**

Figure 3 shows the location of the strain gauges in Bolt Type T₂. In each location, designated 1 to 6, strain gauges were mounted on opposite sides of the bolt and single strain gauges were installed at locations 7 and 8, situated beneath the bolt. The spot where each strain gauge was located had the bolt profile ground flat and smooth.
21.7 mm core diameter bolts were installed in 27 mm holes as per previous tests. Both tests were carried out in 40 MPa concrete. The strain gauge measurements revealed that both the tensile and compression stresses were generated longitudinally during shearing.

By comparing the axial strain at each location along the bolt, axial stress could be determined by the following equation:
\[
\sigma_{ij} = E_b (\varepsilon_{ai} - \varepsilon_{aj})
\]

and the shear stress distribution can be given by:
\[
\tau_g = \frac{\sigma_{ij} A_b}{2\pi l} = E_b (\varepsilon_{ai} - \varepsilon_{aj}) \frac{r}{2l}
\]

Where:
- \(\sigma_{ij}\) = Change in axial stress between two adjacent gauges
- \(E_b\) = Bolt modulus of elasticity (MPa)
- \(\varepsilon_{ai}\) = Axial strain at gauge 1 (\(\mu\) s)
- \(\varepsilon_{aj}\) = Axial strain at gauge 2 (\(\mu\) s)
- \(l\) = Distance between gauges (mm)
- \(r\) = Bolt radius (mm)

Fig. 3 - Schematic diagram of the strain gauges locations in the reinforcing element (a) without pretension load and (b) 20 kN pretension load
Using the above equations in zero pretension conditions, it was found that, for a 30 kN shear load, the maximum tensile and shear stresses, between the strain gauges 3 and 4 at the bolt / grout interface were 196 MPa and 35 MPa respectively. Beyond this load, the stresses were reduced, indicating the bond failure between bolt and grout. The minimum axial and shear stresses were recorded at 50 kN shear load, which are approximately 18 and 3.25 MPa respectively. This situation occurred at the elastic region of the shear load-shear displacement curve, which was supported by both experimental and numerical results. Figure 4 shows the variation of the strain changes along the bolt.

The following were observed:

- Strain gauge No 3, located in the compression zone and placed 60 mm away from the shear joint, produced 2.5 % strain at 60 kN shear load (one half of the total shear load acting of two joint planes) at zero pretension load. This value of strain is in the range of the plastic region (higher than 0.3 % at the end of the elastic region). The yield situation occurred around 20 % of the maximum tensile strength of the bolt.

- The formation of two plastic hinge points in the bolt located symmetrically opposite either side of the sheared joint plane was determined by the strain measurements. Beyond the hinge point and towards the end of the bolts there was a gradual decline in the rate of strain. This was in line with the findings obtained from the numerical simulation. For the strain gauges located near the hinge points it was found that very small shear load (12 kN at strain gauge no 5) was required to strain the outer profiles of the bolt. Thus it was clearly evident from Figure 4 that both the tensile and the compression zones were initiated in the bolt during the early process of shearing.

- For the pretension case, it was found that the hinge point was located around 30 mm from the shear joint. The location of the hinge points was dependant upon the strength of the concrete. In weak concrete, there will be excessive crushing of the concrete in the vicinity of the sheared joint faces leading to greater distance between the hinge point and joint spacing. However, the hinge point location will be closer in high strength concrete. Figure 5 shows the strains developed along the bolt in 20 kN pretension load. Thus it is reasonable to assume that the location of the hinge points are likely to be in these zones and this finding is in agreement with the numerical studies.

![Fig. 4 - Strain rate along the bolt, as measured on the bolt, in zero pretension load](image)

![Fig. 5 - The variation of the strain gauge measurements along the bolt at 20 kN pretension load](image)
SHEAR LOAD BUILT UP AT BOLT FAILURE STAGE

Next, a series of tests were carried out to examine increased shear displacement until the bolt completely sheared (failed). Two approaches were adopted:

- Shearing a small diameter bolt. The bolts were Bolt Types \( T_3 \) and \( T_4 \), tested in 40 MPa concrete.
- Shearing a 23 mm diameter bolt in 100 MPa high concrete. Only Bolt Type \( T_1 \) was tested.

The above tests were undertaken at different confining pressures similar to tests carried out under limited displacement. During the shearing process the shear displacement is increased, lateral and axial loads are developed along the bolt and surrounding materials. The factor of shear resistance may be the resultant of both lateral and axial loads due to the bolt deflection.

Hinge points were created in the bolt at both sides of the shear joint plane, with the gap being increased between bolt-grout and grout-concrete. Grout was completely damaged at compression zones and concrete was fractured along the bolt axis in all three blocks. The failure process of the system appears to be influenced by concrete strength. Bolt failure in strong concrete occurred in shear at the shear joint plane between the hinge points. In weak concrete, no bolt failure occurred as the bolt cuts through the concrete.

Figures 6 shows the load displacement profiles of the bolts tested under different axial load conditions in bolt Types \( T_3 \) and \( T_4 \). The level of maximum shear loads and displacement were different because of different pretension loads. Failed sheared Bolt Types \( T_3 \) and \( T_4 \) are shown in Figures 7 and 8 respectively.

Figure 9 shows the load displacement profiles of the bolt Type \( T_1 \) in different pretensions in 100 MPa concrete. Figure 10 shows the failed bolt across the joint planes in Bolt Type \( T_1 \) in 100 MPa concrete. Figure 11 shows the longitudinal view of concrete blocks after failure in 100 MPa concrete. The bolt failure is located in the vicinity of the bolt-joint intersection, between the hinge points. Moreover, the bolt imprint on bent resin grout shows the grout being overwhelmed due to the compression stresses. The reaction forces are distributed about 60 mm in the bolt and away from the shear joint plane in the outer blocks. In this zone the concrete is extensively fractured.

Fig. 6 - Shear load versus shear displacement in 0, 5 and 10 kN pretension load in Bolt Types \( T_3 \) and \( T_4 \) in 40 MPa concrete
Fig. 7 - Bolt failure view in different pretensioning

Fig. 8 - Bolt failure angle surrounded in concrete 40 MPa and 18 mm hole diameter in Bolt Type T

Fig. 9 - Shear load versus shear displacement in 100 MPa concrete and various pretensions in Bolt Type T₁
Fig. 10 - Failure zone in bolt type T₁ in concrete 100 MPa and 80 kN pretension load

Confining effect on joint
Created gap between bolt grout interface
Overwhelmed grout under high pressure
High reaction zones in concrete
Grout and concrete

Fig. 11 - Failure location in Bolt Type T₁ surrounded in concrete 100 MPa and 27 mm hole diameter in full details of failure process
SHEAR LIP GENERATION

Figure 12 shows the side profile of the failed rock bolt embedded in 36 mm hole diameter and 20 MPa surrounding concrete in 20 kN pretension load. Inspection of the failed bolt showed that the failed surface was caused by the axial and shearing failures, which appears to be initiated with small cracks originating from the center. However, it is suggested that after the yield point, the shear stress generation in the vicinity of the shear joint, through the reinforced bar is almost constant and the bolt fails with the increase of the axial load along the bolt due to excessive bending with combination of shear load developed. It shows the shear lip in the failed bolt has created an ellipsoid shape.

The ratio of axial load developed along the bolt over ultimate tensile strength of the bolt versus shear displacement in concrete 100 MPa with 80 kN pretension load is shown in Figure 13.

Fig. 12 - Side profile of failed Bolt Type T₁ surrounded by concrete 20 MPa and 36 mm hole diameter at 20 kN pretension load

Fig. 13 - Ratio of axial load developed along the bolt over ultimate tensile strength of the bolt versus shear displacement in concrete 100 MPa with 80 kN pretension load

EXPERIMENTAL OBSERVATIONS FROM FAILED BOLTS

- The snapping or failure of a bolt across joint planes was the result of shearing and tensile loading because the shear surface was not vertical and parallel to the vertical joint planes. The surface angle was 12° from the joint plane in bolt Type T₃ as shown in Figure 8.
- The peak elastic yield point “P” has gradually moved closer towards the bolt-joint intersection.
- For a pre-tension load of 80 kN, shear displacement at failure for Bolt Type T4 was 40% higher than the corresponding shear displacement for Bolt Type T3 (see Figure 6).
- The displacement rate of the sheared block in 100 MPa strength concrete was lower than in both 20 and 40 MPa concrete, as expected.
- The failure load for Bolt Type T1 with a pre-tension load of 80 kN was 799 kN. This was in excess of the axial tensile failure load of the bolt at 340 kN.
- Crushed zones in 100 MPa concrete were less than in 40 MPa concrete. The length of the crushed zone was 60 mm either side of the joint plane which demonstrated significant resistance from the concrete and less vertical displacement during shearing.
- During shearing the bolt failed at around 66% of its maximum tensile strength. This failure could not have occurred purely due to axial loading, which again demonstrates that failure was a combination of shear and axial loads at the bolt-joint intersection (see Figures 13).

3D FEM SIMULATION

Using ANSYS, version 9 (ANSYS, 2003), three dimensional simulation of the bolt shearing process was carried out to examine the behaviour of bolted rock joints in relation to the experimental results. Simulation of several models in varying conditions (a range of bolt tensile load and concrete strength) was carried out under a progressive vertical load and results were analyzed for both linear and nonlinear regions of the load-deflection behaviour. From the analyses it was found that, the stresses in the upper convex section of the bolt are in tension, while the lower opposite side are in compression. This situation will occur in reverse on the concave section of the bolt in the other side of the shear joint plane. In addition, the numerical simulation showed that the tensile stress in the bolt was increased and expanded towards the shear joint with increasing the pretension load and bolt deformation.

The distribution of the axial strain and shear stress along the bolt in the vicinity of the sheared joint is drawn in Figure 14a and b respectively. It shows that higher strain is generated from the hinge points and propagates towards the bolt-joint intersection. The maximum shear stress is concentrated at the bolt joint plane intersection. Pretension caused a reduction in bolt shear stress. With increasing shear load and bolt deflection, axial stresses are expanded and moved towards the shear joint location. The combination of shear and axial stresses, the bolt will fail at the joint plane area, as it was discussed in the experimental section. At the post-elastic yield point, the shear stress was almost constant and unaffected by the increase in both the shear and pretension loads. This behaviour occurred at approximately 4 mm of bolt deflection in 20 MPa concrete. The shear stress diagram for all concrete strength, had the same trend, but the value of shear stress was found to decrease with increasing the pretension load.

Softer concrete has experienced higher strain along the bolt and the value of induced tensile strain was higher than the compressive strain. Induced strain is increased with increasing the shear load. With increasing pretension load in post failure behaviour there was no significant changes in strains along the bolt. However, the area of tensile strain has expanded and compression strain reduced. The high level of stress produced in the concrete caused fractures and failures in the vicinity of shear joint region. The zone of high stress concentration in weaker concrete is significantly broader, at 90 mm from the shear joint plane, in comparison to 60 mm length, obtained for 40 MPa concrete.

When the shear load increases, grout will break at the tensile zone and overwhelmed at compression zones. This situation usually will start at the bolt elastic behaviour region and progress beyond the yield point, and the grout will separate from the bolt at tension zone in the vicinity of the shear joint. Due to the axial bolt load, yield in the grout can be determined when the actual bond stress, between bolt and grout, is equal to the grout yield strength. The plastic strain is generated along the grout layer, in particular between the hinge points, when the induced stresses exceeds the grout strength. The values of strain in the grout layer, in plastic state, is ten times greater than at the linear region, particularly at the critical zones in the vicinity of shear joint. Obviously the grout under such severe condition will be fragmented.
The double shear testing represents a useful method of assessing the bolt behaviour in a stratified reinforcement. The evaluation of the shear strength of rock joints reinforced with fully grouted bolt was analysed both experimentally and numerically. The main conclusions drawn were as follows.

- Bolt will fail in vicinity of shear joint plane, between hinge points, due to the combination of both the shear and the axial loads.
- Tensile and compressive stress zones along the bolt are located on each side of the shear plane. The nature of stress concentration is dependent upon the deflection direction of the sliding blocks relative to each other.
- Higher value of shear stress contours occurs along the shear plane.
- The amount of bolt shear displacement due to differing pretension loads is insignificant at the elastic range. However, this is more significant after the yield point, as demonstrated by both experimentally and numerically.
- In all type of bolts tested experimentally, the shear load of the bolt, in general, increased with increasing bolt tension.
- The distance between hinge points was reduced with increasing the strength of material. However, there are no significant changes in hinge point distance with increasing pretension effect.

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REFERENCES