A new approach in determining the load transfer mechanism in fully grouted bolts

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CHAPTER FIVE

DOUBLE SHEARING OF BOLTS ACROSS JOINTS
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5.1. INTRODUCTION

Bolts installed in jointed rock undergo axial and shear loading when sheared. Figure 5.1 shows a typical bolt bending due to bedding displacement. To gain a better understanding of the effectiveness of bolt reinforcements, a series of laboratory based double shear tests were carried out. Using different types of bolts and strengths of concrete the study examined the influence of various parameters on the load transfer characteristics of bolts in strata reinforcement installations.

Figure 5.1. Bolt bending behaviour (after Indraratna et al. 2000)
5.2. EXPERIMENTAL PROCEDURE

5.2.1. Block casting

Double jointed concrete blocks were cast for each double shearing test. Four concrete blocks were cast, 20MPa, 40MPa, 50MPa, and 100MPa to simulate four different rocks. The solid ingredients comprised mainly sand and cement. The mix for the low strength batch consisted of ordinary Portland cement mixed with Nepean River sand but aggregate was added for the 50 and 100 MPa concrete.

Once mixed the concrete was poured into 600mm x 150mm x 150mm wooden moulds which were divided into three sections. A 24mm diameter length of plastic was set through the centre of the mould lengthways to create a hole for the bolt. Figure 5.2 shows a general view giving actual dimensions of the concrete blocks used. The concrete was left for 24hrs to set and then removed and placed in a water bath for 30 days to cure.

Figure 5.2. Laboratory and numerical model
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The plastic conduit was removed and the hole was reamed to a larger diameter to produce a rifled hole surface for effective bolt anchorage. Rifling was achieved by a specially machined tip of a wing bit, shown in Figure 5.3.

![Figure 5.3. Hole reaming for hole rifling](image)

5.2.2. Bolt installation in concrete blocks

A 1400 mm long bolt with 100 mm of thread on both ends was fixed into the concrete specimen using Minova PB1 Mix and Pour resin grout. Prior to installation the blocks were clamped together with straight metal pieces place down the sides to ensure alignment, then placed upright. A series of rubber stoppers and steel plates were attached to the bottom of the hole to prevent resin escaping. A funnel was used
to pour the resin into the holes. In addition two thick steel rings were inserted at the top and bottom of the collars to keep the bolt centrally located.

Resin was poured into the hole and then the bolt was pushed through the stopper plates. More resin was used as required while the bar was rotated to reduce voids and fill the space between the bolt and the sides of the hole. The resin mix was 100 grams of resin and 2 grams of catalyst. Every bolt was installed into their respective concrete blocks with uniform profile, flash orientation. The blocks were left for at least half an hour to allow the resin to cure before moving them to a storage place. Most specimens were left for a minimum of seven days before tested.

5.3. DOUBLE SHEAR BOX

Figure 5.4 shows the steel frame shear box. It was made from 20 mm plates, assembled with 34 cap screws 300 mm long, and then cad coated to prevent corrosion. When assembled the internal dimension was such that the concrete specimen fitted neatly inside.

One of the unique features of this double shear system was that it was a symmetric system of load application and shearing of the bolt, which was particularly relevant when the bolt was subjected to axial loading.

Figure 5.4. An assembled bolt fitted with load cells on both ends of the bolt.
5.4. TESTING

Figure 5.5a shows the sketch of the double shear box and bolt bending. Figure 5.5b shows the assembled shear box in 5000 kN capacity Avery testing machine. A base platform that fitted into the bottom ram of the testing machine was used to hold the shear box between the loading plates. Steel blocks about 55mm thick were placed beneath the two outer concrete blocks to allow for centre block vertical displacement when sheared. The two outer ends of the shear box were then clamped to the base platform so the blocks would not move during shearing. A pre-determined tensile load was applied to the bolt prior to shear loading. This acted as a compressive confining pressure to simulate different forces on the joints within the concrete. The pre-determined tension loads were 0, 5 kN, 10 kN, 20kN, 50kN and 80kN. The maximum applied pre-tension load was nearly 40% of the maximum tensile strength of the bolt. Axial tensioning was accomplished by tightening the nuts on both ends of the bolt. Simultaneous tension was preferred so that bolt is equally loaded either side thus avoiding any possibility of differential loading application, which could influence the encapsulation integrity. The applied loads were monitored by two hollow load cells mounted on the bolt on either side of the block. During testing load cell readings were taken every 10 kN at 0.04 sec minute loading rate. The outer sections of the shear box remained fixed as the central block was pushed down.

Double shear testing was carried out using either 5000 kN capacity Avery Testing Machine (Figure 5.5 b) or 500 kN Instron Universal Testing Machine (Figure 5.6). The selection of the machine type was dependent on the type of bolt and extent of shearing range required.

Information gathered from the test included the applied load, vertical displacement, and axial load generated from shearing.
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Figure 5.5. Schematic of post failed assembled shear box (a), and a set up of the high strength capacity machine -Avery machine (b).

Figure 5.6. The set up of the Instron machine with load cell connection
It must be stressed that the axial load cells readings were manually read from GEOKON read out units strain indicator P-3500, supplied by the Vishay measurement group, and then processed.

**5.5. BOLT TYPES**

Six types of bolts were tested in various combinations with respect to concrete strength, and are shown in Figure 5.7. These bolts were of different diameters and profile configurations, as shown in Table 3.2 in Chapter 3.

![Figure 5.7. Different bolt types](image)

The range of double shearing tests carried out in this programme consisted of the following:

a) Testing of bolts in 20 MPa concrete, representing soft rocks

b) Testing of bolts in 40 MPa concrete representing medium strength rocks
c) Testing of bolts in 100 MPa concrete representing high strength rocks

d) Testing of bolts in different encapsulation annual thickness

e) Testing of bolts without resin thickness

f) Strain gauge installed along the bolt

g) Testing of bolts for complete failure

h) A comparative study of bolts of different diameters.

i) Bolt contribution in different characteristics

Tables 5.1 to 5.3 show various tests carried out in different strength concrete combinations and a number of tests for each bolt with different pre-tension loads. A total of 77 bolts were tested. It should be noted that five tests were carried out without using resin and they are shown in Appendix C.

Table 5.1. Experimental schedule indicating the number of samples tested per bolt type in 20 MPa concrete

<table>
<thead>
<tr>
<th>Bolt Type</th>
<th>Pretension load (kN)</th>
<th>Total</th>
<th>Remark</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>0</td>
<td>20</td>
<td>50</td>
</tr>
<tr>
<td>T1</td>
<td>2</td>
<td>9*</td>
<td>2</td>
</tr>
<tr>
<td>T2</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>T3</td>
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<td>Total</td>
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<td>13</td>
<td>6</td>
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Table 5.2. Experimental schedule indicating the number of samples tested per bolt type in 40 and 100 MPa concrete strength

<table>
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<th>Bolt Type</th>
<th>Strength (MPa)</th>
<th>Pretension load (kN)</th>
<th>Total</th>
<th>Comments</th>
</tr>
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<td>20</td>
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</tr>
<tr>
<td>T1</td>
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<td>2</td>
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<tr>
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<td>2</td>
<td>1</td>
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<td></td>
<td>100</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>T2</td>
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<td>2</td>
</tr>
<tr>
<td>T4</td>
<td>40</td>
<td>1</td>
<td>-</td>
<td>1</td>
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<tr>
<td>Total</td>
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<td>9</td>
<td>9</td>
<td>8</td>
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</table>

Table 5.3. Experimental schedule indicating the number of samples tested per bolt type T5 and T6 (low strength steel) in 40 MPa concrete strength

<table>
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<th>Bolt Type</th>
<th>Pretension load (kN)</th>
<th>Total</th>
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<td>0</td>
<td>5</td>
</tr>
<tr>
<td>T5</td>
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<td>T6</td>
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<tr>
<td>Total</td>
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</tbody>
</table>

5.6. RESULTS AND DISCUSSION

5.6.1. Shear load and shear displacement

5.6.1.1. Profile description

Figure 5.8 shows a profile of the general load displacement of a double shearing test. Three distinct stages are shown which are similar to the profile of the three-point load bending tests of a steel bar. The stages are elastic, non-linear, and plastic.
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Generally the profiles are of similar configurations irrespective of the test conditions, however the load build up and resultant displacement were influenced by diameter of bolt, strength of concrete, profile configuration, resin thickness and axial pre-tension.

i) Elastic Stage

This part of the graph is associated with the elastic behaviour of the sheared system. The surfaces start sliding against each other as the shear load is applied. This linear section of graph is characterised by a rapid increase in shear load at a relatively small displacement of less than 5 mm. In most cases the highest stiffness and elastic recovery of the system after the shearing load is withdrawn depends on the confining pre-tension load applied initially. There will be some minor fracturing of the grout and concrete, which will cause a loss of bonding. The displacement level at the elastic yield stage reduces as the pre-tension load increases.

![Figure 5.8. Typical shear load displacement profile stages of the sheared bolt](image-url)
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ii) Non-linear Stage

This is the transitional zone between the elastic and plastic zones and is called the elasto-plastic stage. There is a sharp drop in the rate of shear stiffness post the peak elastic yield load (P). Displacement and deflection at this stage can be the same rate or slightly greater than the linear stage section by about 6 mm, and it also depends on the strength of material, bolt profile type, and axial pre-tension load. Stiffness decreases towards the plastic range and the bolt undergoes irreversible bending, particularly at post peak yield point (P). Occasionally, a small drop in the shear load occurs beyond the elastic yield point due to axial fractures developing in the concrete and along the bolt longitudinally. The elastic peak yield point (P) is likely to occur at a reduced displacement with an increased pre-tension load.

iii) Plastic Stage

The plastic limit of the bolt is characterised by a low rate of shear loading at increased vertical displacement, in other words, low stiffness in the system. Hinge points are created in the bolt on both sides of the shear joint plane because of reduced shear stiffness. The concrete and grout are completely damaged at the compression zones with excessive fracturing along the axis of the bolts in all three blocks (Figure 5.19).

5.6.1.2. Shear loading for a limited displacement

Tables 5.4 and 5.5 show the results of three types of bolts tested in 20 and 40 MPa concrete under different pre-tension loads. Also included in the table are the results of bolts without tension (i.e., 0 kN pre-tension load). The load displacement profile bolts T1, T2, and T3 are shown in Figure 5.9 (a-f). Figure 5.10 (a-f) shows the comparative shear load and vertical displacement (deflection) profiles in both 20 and
40 MPa concrete for the given pre-tension loads indicated. Individual comparative results between 20 and 40 MPa in different profiles are presented in Appendix B. However Bar charts 5.11 shows the comparative results in different types of bolts and concrete medium. Additional tests on Bolt Type T4 are listed in Appendix B.
### Table 5.4. Yield point shear load values for different bolts under different environment

<table>
<thead>
<tr>
<th>Concrete Strength (MPa)</th>
<th>Pretension load (kN)</th>
<th>Shear load at yield point (kN)</th>
<th>Shear displacement at yield point (mm)</th>
<th>Shear stiffness (kN/mm)</th>
<th>Comments</th>
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<tr>
<td></td>
<td>Type T1</td>
<td>Type T2</td>
<td>Type T3</td>
<td>Type T1</td>
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Table 5.5. Yield point shear load values for bolt Type T1 under different environment

<table>
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<tr>
<th>Concrete Strength (MPa)</th>
<th>Pretension load (kN)</th>
<th>Hole diameter (mm)</th>
<th>Shear load at yield point (kN)</th>
<th>Shear load at failure point (kN)</th>
<th>Shear displacement at yield point (mm)</th>
<th>Shear displacement at failure (mm)</th>
<th>Shear stiffness (kN/mm)</th>
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<td>25</td>
<td>85</td>
<td>-</td>
<td>7.6</td>
<td>-</td>
<td>11.18</td>
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<td>0</td>
<td>25</td>
<td>110</td>
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<td>25</td>
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<td>-</td>
<td>12</td>
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<td>17.6</td>
<td>With end plate</td>
</tr>
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<td></td>
<td>50</td>
<td>25</td>
<td>209</td>
<td>-</td>
<td>8.86</td>
<td>-</td>
<td>23.6</td>
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<td></td>
<td>80</td>
<td>25</td>
<td>274</td>
<td>-</td>
<td>10.57</td>
<td>-</td>
<td>26</td>
<td></td>
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<tr>
<td>100</td>
<td>Tests carried out in 0, 20, 50 and 80 kN and are discussed in related section</td>
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Figure 5.9 (a-f). Shear load and vertical displacement profiles of Bolts Types T1, T2 and T3 in both 20 and 40 MPa concrete
Figure 5.10 (a-f). Shear load versus vertical shear displacement profiles of various bolts in 20 and 40 MPa concrete at different pretension load
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Figure 5.11. Shear yield load values in different concrete strength of various bolt types and various pretension loads

The following can be deduced from the load displacement data and graphs:

1. The elastic peak load \( P \) for non pre-tension loaded Bolt Type T1 did not change significantly with changes in the concrete strength (see Figure 5.9 a and d). However there was a difference in \( P \) value in Bolt Type T2 and no conclusions were drawn for Bolt Type T3, as only one test made in 40 MPa concrete. A possible explanation could be attributed to the profile configurations between them.

2. For an increase in pre-tension 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 \( P \) values were 55 \% increase in Bolt Type T1, and 9 \% in Bolt Type T2. No tests were made for Bolt Type T3 in 80 kN. However, \( P \) value in Bolt Type T3 increased 25 \% from 20 to 50 kN.
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pretension load. This means that the bolt in tension acted as an active support system and provided confining pressure to the surfaces of the sheared joint.

3. The peak elastic shear load displacement level for a given axial pre-tension load was dependent on the type of bolt. This displacement was more likely to decrease with increased pre-tension load.

4. The strength of the medium influenced the shear load level but not the trend. The shear load for all bolts were generally less in 20 MPa concrete medium in comparison to the shear load of bolts tested in 40 MPa concrete.

5. Bolt Type T2 displayed consistent shear load, displacement profiles at all three levels of pre-tension loads (20, 50, and 80 kN), particularly in 40 MPa strength. This consistency was relatively less in 20 MPa concrete and remained less scattered than Bolt Types T1 and T2.

6. Bolt Type T3 load displacement profiles were inconsistent at different pre-tension loads which was expected because of the large surface profile spacing configurations.

7. As shown in Figure 5.12, shearing of the bolt without pre-tension can lead to an early loss of resin, bolt bonding and inward pulling and bending leading to an excessive gap. This situation became worse when the ends were not fitted with nuts and plates to hold against the concrete block. The presence of end plate provides better structure reinforcement (see Tadolini and Ulrich 1986).

8. As can be seen from Figure 5.13 the gap created by bending was different for different test environments. The height of the gap varied under different types of concrete, pre-tension loads, and types of bolts. The effective gap (Chen 1999) was determined from testing each type T5 and T6 and was 1.35 and 3
times the bent diameter ($D_b$) respectively. The formation of the gap is shown in Figure 5.34.

9. Figures 5.9 to 5.11 show the peak elastic yield load “P” in different types of bolt in 20 and 40 MPa concrete. Obviously, no definite conclusions can be made on different behaviour without the bolts being pre-tension loaded in 20 MPa concrete. However “P” values in Bolt Type T2 showed 38 % more than Bolt Type T1 in 40 MPa concrete and was almost the same as Bolt Type T3. Obviously pre-tension loading helped increase the elastic peak yield load, as shown in Figure 5.10 (a-f). The strength of the concrete was another factor.

10. Peak elastic yield point values changed with changes in resin annulus thickness. This is clearly evident when testing bolts installed in different diameter holes in 20 MPa concrete shown in Table 5.5 (Details in the next chapter).

![Figure 5.12](image)

Figure 5.12. Bolt slippage along the bolt-grout interface in case of non-pre-tension loading and non-plate
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5.6.1.3. Shear loading of bolt to ultimate failure

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

i. Shearing a small diameter bolt. The bolts were Bolt Types T5 and T6, tested in 40 MPa concrete.

ii. Shearing a 23 mm diameter bolt in 100 MPa high concrete. Only Bolt Type T1 was tested.

The above tests were undertaken at different confining pressures similar to tests carried out under limited displacement. The general descriptions of these bolts are shown in Table 3.2.

Tables 5.6 shows the test results on Bolt Types T5 and T6. Figure 5.14 shows the load displacement profiles under different axial loads. Maximum shear loads and displacements were different because of different pre-tension loads and types of bolts, as indicated in Figure 5.15. The relationship between shear yield load and pre-

Figure 5.13. Axial fracture along the concrete and grout breaking off in the tensile zone in Bolt Type T1 in 40 MPa concrete with 80 kN pre-tension loading
tension loading of Bolt Type T5 is shown in Figure 5.16a and sheared Bolt Type T6 is shown in Figure 5.16b.

Table 5.7 shows the results of the tests carried on Bolt Type T1 tested in 100 MPa concrete. Figure 5.17 shows the load displacement profiles of the bolt Type T1 in different pre-tension loading in 100 MPa concrete. Excessive necking in 100 MPa concrete is shown in Figure 5.18. Figure 5.19 shows the failed bolt across the joint planes and crushed zones near the sheared planes in Bolt Type T1 in 100 MPa concrete. Figure 5.20 shows sheared resin imprint separated from the bond section of the bolt.
Table 5.6. Test results in Bolt Types T5 and T6 surrounded by 40 MPa concrete

<table>
<thead>
<tr>
<th>Bolt Type</th>
<th>Concrete strength (MPa)</th>
<th>Preload (kN)</th>
<th>Yield point (kN)</th>
<th>Displ at yield (mm)</th>
<th>Failure load (kN)</th>
<th>Displ at failure (mm)</th>
<th>Gap (mm)</th>
<th>Hinge distance (mm) (***</th>
<th>Angle of bolt bending (°)</th>
<th>Stiffness (kN/mm)</th>
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</thead>
<tbody>
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<td>T5</td>
<td>40</td>
<td>0</td>
<td>48.7</td>
<td>6.3</td>
<td>76.5</td>
<td>23.8*</td>
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<td>5</td>
<td>70</td>
<td>4.23</td>
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<td></td>
<td>40</td>
<td>10</td>
<td>83</td>
<td>3.96</td>
<td>118.9</td>
<td>21.1*</td>
<td>18*</td>
<td>40</td>
<td>30</td>
<td>21</td>
</tr>
<tr>
<td>T6</td>
<td>40</td>
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<td>98</td>
<td>4.33</td>
<td>172</td>
<td>36**</td>
<td>35**</td>
<td>40</td>
<td>44</td>
<td>22.6</td>
</tr>
</tbody>
</table>

* Displacement at failure and gap in Bolt Type T5 is between 1.25 to 2 times diameter

** Displacement at failure and gap in Bolt Type T6 is around 3 times diameter

*** Hinge distance in two types of bolts is between 2.8 to 3.3 times diameter
Table 5.7. Bolt Type T1 in 100 MPa concrete

<table>
<thead>
<tr>
<th>Pretension-load (kN)</th>
<th>Yield load (kN)</th>
<th>Displ. at yield (mm)</th>
<th>Peak load (kN)</th>
<th>Displ. at failure (mm)</th>
<th>Bolt deflection (mm)</th>
<th>Max displ. (mm)</th>
<th>Hinge distance (mm)</th>
<th>Angle of rotation (°)</th>
<th>Stiffness (kN/mm)</th>
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<td>272</td>
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<td>80</td>
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<td>53.5</td>
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<tr>
<td></td>
<td>20 kN pretension load carried out in 36 mm hole diameter</td>
<td>Failure occurred in 80 kN</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
CHAPTER 5: *Double shearing of bolts across joints*

Figure 5.14. Shear load versus shear displacement in 0, 5 and 10 kN pretension load in Bolt Types T5 and T6 in 40 MPa concrete

Figure 5.15. Bolt failure view in different pretensioning
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Figure 5.16. (a) Relationship between failure load and maximum tensile strength on one side of the shear joint on Bolt Type T5, (b) bolt failure angle

Figure 5.17. Shear load versus shear displacement in 100 MPa concrete and different pre-tension loading in Bolt Type T1
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Figure 5.18. Excessive bolt necking in 100 MPa concrete with 80 kN pretension load

Figure 5.19. Bolt/ joint concrete interaction at shear joint in 100 MPa concrete with 80 kN pre-tension load
Figure 5.20. Bolt imprint on resin in 100 MPa concrete at 50 and 80 kN pre-tension loads

The following were deduced from both sets of tests stated above:

A) Testing of Bolt Types T5 and T6 in 40 MPa concrete:

i. 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, as shown in Figure 5.16b.

ii. The peak elastic yield point “P” has gradually moved closer towards the bolt - joint intersection.

iii. Necking began around the peak elastic yield point “P” because of the predominantly tensile load at the bolt - joint intersection. It is possible that when necking commences the diameter decreases of the effective length between the hinge points near the shear joint.

iv. For a pre-tension load of 80 kN, shear displacement at failure for Bolt Type T6 was 40% higher than the corresponding shear displacement for Bolt Type T5. As Figure 5.16a shows there was a steady increase in the ratio of failure
load to max tensile strength with an increased initial bolt pre-tension load. The angle was 18°. This finding contradicted Ferrero’s result (1995), which stated that pre-tension load does not influence the maximum shear resistance of the system. Ferrero’s test was undertaken in a single shear test box where pre-tension loads were applied to one side of the bolt.

**B) Testing in 100 MPa concrete.**

As shown in Table 5.7 and Figures 5.17, the following were noted:

i. The displacement rate of the sheared block in 100 MPa strength concrete was lower than in both 20 and 40 MPa concrete, as expected.

ii. The failure load for Bolt Type T1 with a pre-tension load of 80kN was 799 kN. This was in excess of the axial tensile failure load of the bolt at 340 kN.

iii. 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.

iv. No failure occurred at 50 kN pre-tension load but it was achieved at 80 kN. The level of concrete crushing and shearing of the resin imprints are shown in Figure 5.20.

v. During shearing the bolt failed at around 66 % of its maximum tensile strength. It could not have failed purely on the axial load, which again demonstrates that failure was a combination of shear and axial loads at the bolt - joint intersection (see Figures 5.21).
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5.6.2. Influence of shearing load on pre-tension load

Figure 5.22 shows a typical shear load versus pre-tension load developed along a bolt installed in 20 MPa concrete. Point A is known as the Limit of Maximum Frictional Bonding Strength (LMFBS) which indicates shear load, whereas the pretension load, monitored by load cells mounted on either side of the bolt, began to increase from the initial load applied. This level of shear load is significantly higher than the peak elastic yield point (P) shown in Figures 5.9 and 5.10 respectively, and discussed in the previous section (5.6.1). This increase in shear load depended on the initial axial tensile load on the bolt, concrete type and profile pattern. Figure 5.23 (a-f) shows different shear load and load cell readings for various bolts. The graph profiles were different for different bolt types.
The following can be observed from the shear load versus axial load built up along the bolt in different pretension load and strength of concrete.

i. The level of initial confining axial load (pretension load) applied to the bolts had a profound influence on the position of LMFBS. The higher the initial tension, the greater was the positioning at the LMFBS.

ii. The shear load at the LMFBS were greater than P at the shear load shear displacement curve in all levels of pre-tension loading and strength of concrete.

iii. Back sloping of the load cell shear load graph prior to failure of the frictional bond in high pre-tension load (80 kN) was attributed to the concrete blocks being crushed, as indicated in Figure 5.24. Clearly the bolt appeared to have pulled through the concrete as the shear load increased. This phenomenon was more common in weaker concrete such as 20 MPa, which was too weak for testing 22 mm diameter core bolts.

Figure 5.22. Shear load versus load cell readings on tensile load applied on a bolt installed in a 20 MPa concrete
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Figure 5.23 (a-f). Shear load and pretension loads (load cell readings) for various bolts with an initial pre-tension load of 20, 50 and 80 kN

(a) Bolt Type T1 in 20 MPa concrete
(d) Bolt Type T1 in 40 MPa concrete

(b) Bolt Type T2 in 20 MPa concrete
(e) Bolt Type T1 in 40 MPa concrete

(c) Bolt Type T3 in 20 MPa concrete
(f) Bolt Type T3 in 40 MPa concrete

Figure 5.23 (a-f). Shear load and pretension loads (load cell readings) for various bolts with an initial pre-tension load of 20, 50 and 80 kN
iv. Bolt Type T2 installed in the 40 MPa concrete had comparatively greater shear load at the LMFBS point than the other two bolts.

v. The level of shear displacement at the LMFBS point was dependent on the level of initial pre-tension load. As can be seen from Figure 5.25. The shear displacement was greater in 80 kN pre-tension load than the other two profiles with 20 and 50 kN pre-tension loads.

Figure 5.24. End crushing of the concrete in high pre-tension load

Figure 5.25. Axial load developed along the bolt versus shear displacement in Bolt Type T2 in 40 MPa concrete
5.6.3. Load transfer level in different profile

Figure 5.26 shows the comparison of the peak P values as a function of pre-tension in different profiles and strength concrete. From the graph it can be seen that the level of P has increased with increasing strength of concrete in different profiles. Bolt Types T3 and T2 had the lowest and highest (P) levels respectively in 20 MPa concrete. The graph also showed that in 20 MPa concrete, pre-tension in a lower pre-tension load was much more effective than a higher pre-tension load. In addition it shows that Bolt Type T2 in 40 MPa concrete has an almost constant trend. In all the laboratory tests it was noted that the bolt begins to yields at the plastic hinge point, which is 20 to 40 mm from the shear joint plane, and was dependent on the material properties and test conditions.

![Figure 5.26. Effect of pre-tension load, bolt profile and concrete strength on the bolt resistance](image_url)

What is also obvious is that the failed area in the concrete mass was two or three order of magnitude greater than the cross sectional area of the bolt. Once again this is
a clear indication that the bolt has failed under the combination of both shear and axial load.

5.6.4. Double shearing of instrumented bolt

To gain a clear understanding of the pattern of load and stress build up along the bolt, two tests were carried out on strain gauged instrumented bolts (both of Bolt Type T2). One test was made with a bolt not subjected to pre-tension load (zero pretension) and the other with a pre-tension load of 20 kN. Figures 5.27 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. The 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. Details of the strain gauges positions are clearly marked in Figure 5.27. 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 (Equation 5.1):

\[ \sigma_{ij} = E_b (\varepsilon_{ai} - \varepsilon_{aj}) \]  

(5.1)

and the shear stress distribution can be given by:

\[ \tau_{ij} = \frac{\sigma_{ij} A_p}{2 \pi l} = E_b (\varepsilon_{ai} - \varepsilon_{aj}) \frac{r}{2l} \]  

(5.2)

where:

\[ \sigma_{aij} = \text{ Change in axial stress between two adjacent gauges} \]
CHAPTER 5: Double shearing of bolts across joints

Figure 5.27. Schematic diagram of the strain gauges locations in the reinforcing element (a) without pretension load and (b) 20 kN pre-tension load

a) Without pretension load

b) 20 kN pretension load and the distance measurements
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\[ E_b = \text{Bolt modulus of elasticity (MPa)} \]

\[ \varepsilon_{ai} = \text{Axial strain at gauge 1 (}\mu s\text{)} \]

\[ \varepsilon_{aj} = \text{Axial strain at gauge 2 (}\mu s\text{)} \]

\[ l = \text{Distance between gauges (mm)} \]

\[ r = \text{Bolt radius (mm)} \]

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 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 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. Further analyses are presented in Appendix B. This situation occurred at the elastic region of the shear load, shear displacement curve, which was supported by experimental and numerical results. It should be noted that the distribution of shear stress prior to the yield point was in agreement with Farmer’s theory (1995). The load build up registered for the rest of the strain gauges are shown in Figure 5.28. Figure 5.29 shows a section of the bolt with strain gauges mounted on its outer surface. Figure 5.30 shows the variation of the strain changes along the bolt.

The following were observed:

i. 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 pre-tension load. This value of strain is in the range of the plastic region (higher than 0.3 % at the end of the
CHAPTER 5: *Double shearing of bolts across joints*

elastic region). The yield situation occurred around 20% of the maximum tensile strength of the bolt.

ii. 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 5.28 that both the tensile and the compression zones were initiated in the bolt during the early process of shearing.

iii. For the pre-tension case it was found that the hinge point was located 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 near 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 5.28. Shear load versus strain measurements in non-pretension load](image-url)
Figure 5.29. Bolt surface with strain gauges installed

Figure 5.30. Strain rate along the bolt, as measured on the bolt, in zero pretension load

Figures 5.31 and 5.32 show the relationship between the applied shear load and strains developed along the bolt in 20 kN pre-tension load.

Figure 5.31. Shear load versus strain gauge measurements along the bolt in 20 kN of pre-tension.
Figure 5.32. The variation of the strain gauge measurements along the bolt at 20 kN pre-tension load.

It is clear from the strain gauge measurements that higher values of strain occurred within 30 mm of either side of the shear joint plane. Thus it is reasonable to assume that the location of the hinge points are likely to be in these zones, which agrees with the numerical studies discussed later in Chapter 7. Further analysis of the strain variations along the bolt are shown in Appendix B.

5.6.5. Medium (concrete and resin) reaction

When a bolted joint is sheared the surrounding materials (concrete and grout) deform and induce support reaction against the shear load longitudinally. This reaction depends on the mechanical properties of rock and grout. It is noted that at early stages of shearing, around 10-20% of loading time, the surrounding materials behaved elastically. This was determined from the numerical analysis, which will be discussed in Chapter 7. The severity of these changes depend upon several parameters, including the mechanical and physical properties of the bolt, rock and
grout strength and bolt pre-tension load. Yield in the surrounding materials will start in the vicinity of the shear joint and propagates outwards with increasing bolt deformation. The grout annulus yields when the shear load (lateral bolt pressure) at the bolt-joint intersection equals the grout yield strength. Grout separation will start from the hinge point towards the shear joint and completely separates from the bolt in the tension zone. Due to the axial bolt load, the yield in the grout can be determined when the actual bond stress, $\tau$, between bolt and grout is equal to the grout yield strength $\tau_y$, as:

$$\frac{\tau}{\tau_y} = 1$$  \hspace{1cm} (5.3)

When the yield occurs in the surrounding material, the axial bond strength between the bolt-grout will change and on the yielding section of the bolt, the residual bond strength is considered to be frictional and a function of the lateral pressure. This relationship can be written as:

$$\tau_{res} = \mu p_{(x)}$$  \hspace{1cm} (5.4)

where:

$\tau$ = Bond shear stress (MPa)

$\tau_y$ = Grout shear strength (MPa)

$\tau_{res}$ = Residual bond strength (MPa)

$\mu$ = Friction coefficient between bolt-grout interface

$p_{(x)}$ = Support reaction (MPa)

For lateral deformation the rock is affected by pressure from the bolt and the yield in concrete is initiated from this pressure zone situated near the bolt-joint intersection.
In other words yield appears in the concrete when the maximum elastic deformation of the concrete is exceeded. Yield is gradually developed and expanded through the rock as the bolt increasingly deforms.

Figure 5.33 shows the concrete block being split axially along the bolt due to the high stresses induced along the shear direction through the concrete block. These fractures originate from the compression zone (critical zone in the vicinity of the shear joint) and propagate into the upper side of the concrete block. By splitting the concrete, the reaction pressure reduces and then the bolt deformation increases with increasing the shear load. It is noted that the block fracturing was observed in all of the double shear tests performed. Figure 5.34 displays the gap created between the bolt and the grout in the plastic stage, which was around 0.8-1.0 D_b (D_b= bolt diameter). It is thus reasonable to conclude that the contact surface area from the shear joint plane along the bolt between bolt – grout - concrete interfaces gradually decreases to form the gap in the vicinity of the shear joint.

![Axial fracture](image)

Figure 5.33. Axial fracture developed along the bolt through the 20 MPa concrete
5.6.6. Bolt contribution

Bolt contribution to the shear strength of the reinforced joint plane depends upon the strength of the rock, concrete, grout, and bond between the interfaces, mechanical and physical properties of the steel bolt, joint specification and bolt pre-tension loads. Each of these parameters, as discussed in previous chapters, affects the shear resistance and failure mechanism. Some of the affected parameters are inherent specification for the shear joint which were found by direct shear tests in 20 and 40 MPa concrete joint planes discussed in Chapter 3. Based on the laboratory studies and shown in Table 3.7, the value of the friction angle for 20 and 40 MPa concrete were measured as 31° and 38° respectively. Thus the confining effect can be calculated as;

\[ N_c = c + n \tan \varphi \]  \hspace{1cm} (5.5)

where;

\[ N_c = \text{Confining load (kN)} \]
When a bolt is being sheared the total resistance is a combination of the joint without the reinforcement element and contribution by the bolt. According to the Mohr Coulomb Criterion, the sheared joint plane contribution under the confining pressure can be expressed as in Equation 5.6.

\[
T_i = \frac{T_v}{2} - N_c \tan \varphi
\]  

(5.6)

\[
T_b = \frac{T_v - 2N_c \tan \varphi}{2F_{\text{max}}}
\]

Also the bolt contribution can be expressed as in Equation 5.7.

\[
T_b = \frac{T_v - 2N_c \tan \varphi}{2F_{\text{max}}} = f(t)
\]  

(5.7)

where:

\[
f(t) = \text{Dimensionless factor in terms of bolt contribution}
\]

\[
T_v = \text{Shear load}
\]
\[ T_i = \text{Joint contribution} \]

\[ F_{max} = \text{Maximum tensile strength of the bolt} \]

\[ f(u) = \frac{u_b}{D_b} \]

\[ f(u) = \text{Dimensionless factor in terms of shear displacement,} \]

\[ u_b = \text{Shear displacement and} \]

\[ D_b = \text{Bolt diameter} \]

Table 5.8 shows various confining forces applied to bolts in different strength concrete levels of pre-tension loading. By using the above equations the bolt contribution can be determined.

<table>
<thead>
<tr>
<th>Concrete strength (MPa)</th>
<th>Pretension load (kN)</th>
<th>Joint angle of friction ((\phi))</th>
<th>Confining load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
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<td>31</td>
<td>12</td>
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</tr>
<tr>
<td></td>
<td>80</td>
<td>38</td>
<td>62.5</td>
</tr>
</tbody>
</table>

The bolt contribution in bolt Types T1, T2, T3, T4, T5 and T6 in different concrete strength and bolt pre-tension loads are presented in Appendix B.

Based on the laboratory results the following relationships were established:

\[ T_b = 120.5 \ln(\sigma_c) + 0.014(f_y)^2 + 0.058(f_y) - 239 \]  \hspace{1cm} (5.8)
\[
\frac{u_y}{D_b} = -1.06 \ln(\sigma_c) + 4.96 \tag{5.9}
\]

where;

\( T_h \) = Yield point at shear load- displacement curve

\( \sigma_c \) = Uniaxial compressive strength of the rock (MPa)

\( f_{ov} \) = Pretension load (kN)

\( u_y \) = Joint movement (mm), which is usually twice bolt deflection

\( D_b \) = Bolt diameter (mm)

From the above it can be inferred that an increase in the rate of bolt contribution to joint shear resistance reduces when concrete increases in strength. Figure 5.35 shows the effect of concrete strength on the shear displacement factor, \( f(u) \), in numerical, experimental and predicted analyses. The numerical results were obtained without pretension load. Clearly, as the strength of the concrete increased \( f(u) \) drops gradually to a steady level.

Figure 5.35. Effect of concrete strength on the factor of movement
With the inclusion of resin thickness and strength properties of the steel while maintaining the other parameters constant, the following relationship was established using the statistical method SPSS software with 77% correlation factor;

\[
\frac{T_b}{\sigma_y} = -0.36\left(\frac{D_b}{D_h}\right) + 0.004(Pr) + 0.005(\sigma_c) + 0.53
\]

(5.10)

where;

- \(T_b\) = Shear yield load (kN)
- \(\sigma_y\) = Maximum tensile strength of bolt (kN)
- \(D_b\) = Bolt diameter (mm)
- \(D_h\) = Hole diameter (mm)
- \(Pr\) = Pretension load (kN)
- \(\sigma_c\) = Uniaxial compressive strength of the rock (MPa)

Figure 5.36 shows the relationship between the expected and observed results of bolt contribution under different conditions indicates acceptable agreement between predicted and observed results.
The following were deduced from the bolt contribution for all types of bolts.

- For all bolt types considered, concrete strength and bolt pre-tension load affects the level of bolt contribution to shearing.
- For a Bolt Type T1 installed in 40 MPa concrete, without pretensioning, there was no noticeable change in the level of bolt contribution to joint shear resistance. However, bolt contribution level was increased which bolt installed in 20 MPa concrete pre-tension load.
- Bolt Type T3 has showed lower level of contribution to shear resistance in comparison with Bolt Types T1 and T2 in 20 MPa concrete.
- Bolt contribution was increased by 15% with resin grout as opposed to none.
- Bolt contribution depends on the maximum tensile strength of the bolt. Accordingly, 135% bolt contribution in Bolt Type T5 against 125% in Bolt Type T6. (See Figure 5.37). More bolt contribution figures are presented in Appendix B.
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- The axial and shear loads are maximum at the bolt-joint intersection. For those cases where shearing occurred at zero pre-tension, the shear resistance factor will be one half of the maximum tensile strength of the bolt. In these cases the bolt fails in pure shearing.

- For the Bolt Type T1 installed in 100 MPa concrete, it was found that the maximum bolt-joint contribution at failure was about 120% of the maximum tensile strength of the bolt.

- The values of bolt contribution in Bolt Type T1 at yield point in 20, 40 and 100 MPa concrete was about 24%, 30% and 52% respectively.

Figure 5.37. Bolt contribution in Bolt Type T5 and T6

5.7. SUMMARY

The double shearing study has demonstrated its importance in better understanding the role a bolt would play in real ground reinforcements, particularly in sheared zones. The double shear system represented a better method because it enabled a
symmetrical study of bolt shearing analysis, which is not possible with the systems available now.

Accordingly the following were deduced from the study:

- Bolt profiles play 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.
- The study demonstrated that the current size of the double shearing apparatus is insufficient to conduct tests with larger diameter bolts.