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## MODELLING OF SHEARED BEHAVIOUR BOLTS ACROSS JOINTS

Hossein Jalalifar<sup>1</sup>, Naj Aziz<sup>1</sup>, and Mohammad Hadi<sup>1</sup>

**ABSTRACT:** A three dimensional numerical model was developed to simulate the shearing of reinforced joints. Reinforcement of the shearing surfaces is effected with pretensioned bolts installed perpendicular to the sheared joint surface. The influence of bolt pretension forces examined included 20 kN, 50 kN and 80 kN respectively and aimed to complement the experimental work on double shearing of bolts installed in two different strength concrete blocks. Post shear stresses were analysed for both linear and nonlinear regions of the load - deflection curve. Simulation of several models in varying conditions provided a better understanding of the role of bolt pretensioning in sheared joint and bedding plane reinforcement. There was a clear relationship between the level of bolt pretensioning and the shear load applied. It was shown that the strength of the sheared composite medium was influenced by the applied shear load. The modeling study is part of a comprehensive programme of research work aimed at providing a better understanding of load transfer mechanisms in bolt /resin /rock for effective strata reinforcement.

### INTRODUCTION

Significant research have, in the past, been undertaken to study the mechanical behaviour of bolted rock joints, Spang and Egger (1991), Pellet and Boulon(1993), Ferrero (1995). Bjurstrom (1974) was the first to report on the systematic research work on fully grouted rock bolts. His shear tests were conducted on fully cement grout bonded rock bolts embedded in blocks of granite. According to Bjurstrom, inclining the bolt resulted in stiffening the shearing surface by increasing the shear strength at small displacement. Dight (1982) carried out a series of laboratory tests, to evaluate the shear resistance of bolted joints using various materials and he found that the normal stress acting on the joint surface had no influence on the shear resistance and joints with inclined bolts were stiffer than the perpendicular ones. Dight (1982) proposed an expression to predict the maximum force mobilized in the bolt. He found that the failure of the bolt was caused by the combination of axial and shear forces. Ferrero (1995) proposed a shear strength model for reinforced rock joints based on numerical modeling and laboratory tests. The overall strength of the reinforced joint was considered to be the combination of both the dowel effect and the incremental axial force increase due to the bar deformation. Also, Ferrero proposed a modified analytical model for bolts installed perpendicular to the joint plane in stratified bedding plane. Aziz, Pratt and Williams (2003) conducted laboratory studies of double shearing of bolts in concrete and found that the medium strength and the axial tensional load influenced the level of shear load.

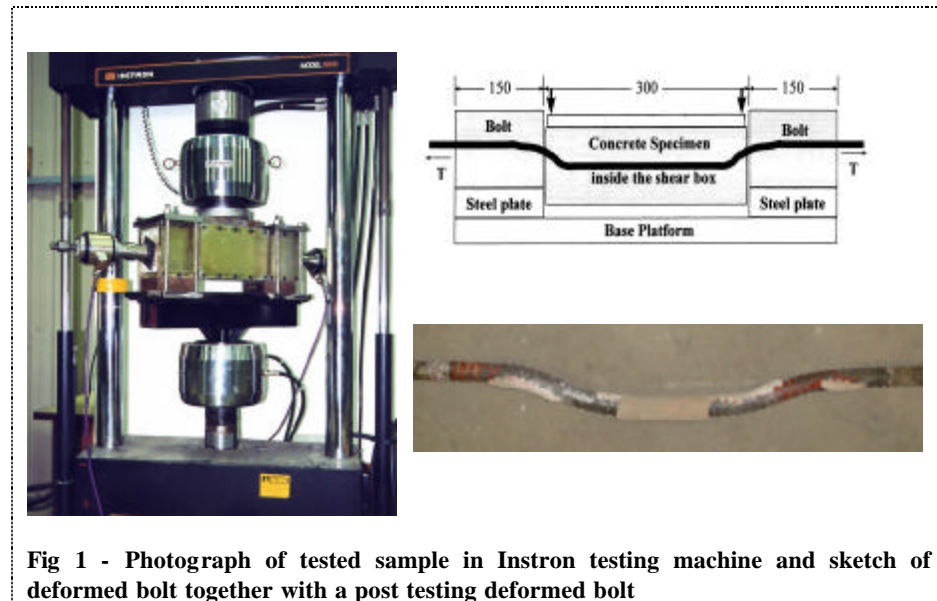
As a continuation of the research on bolting, a programme of numerical modeling is currently been undertaken to simulate the role of profile configuration on the load transfer mechanism bolt pull/push testing and the influence of bolt pretensioning across the sheared surfaces to complement the laboratory studies by Aziz, Pratt and Williams (2003). The 3D FEM modeling was carried out with ANSYS 3D.

### EXPERIMENTAL STUDY

Figure 1 shows the general set up of the assembled double shear box unit in a testing machine and sketch of a deformed bolt together with a post testing deformed bolt. Details of the experimental study on the shear behaviour of bolts in jointed concrete blocks were reported by Aziz, Pratt, and Williams (2003). Three bolt types were used for the study, they were known as Bolt Types T1, T2 and T3 Respectively. The properties of various bolts are reported by Aziz (2004).

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Tests were made with axial confining loads of 20, 50 and 80 kN respectively. The development of shearing loads was examined with respect to the surface profile configuration of the bolt used. Shearing tests were conducted in two different concrete blocks of 20 MPa and 40 MPa strength. Figure 2 shows a typical Shear load and displacement profiles of three different bolts cast in 40 MPa concrete.

The following points were noted from shear load and deflection graphs (Aziz, Pratt and Williams, 2003):

- 1) The shear load of the bolt increased with increasing bolt tension. This behaviour was obvious in bolts with low profiled and widely spaced profiled bolts.
- 2) 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 strength concrete medium in comparison to the shear load values of bolts tested in 40 MPa concrete.

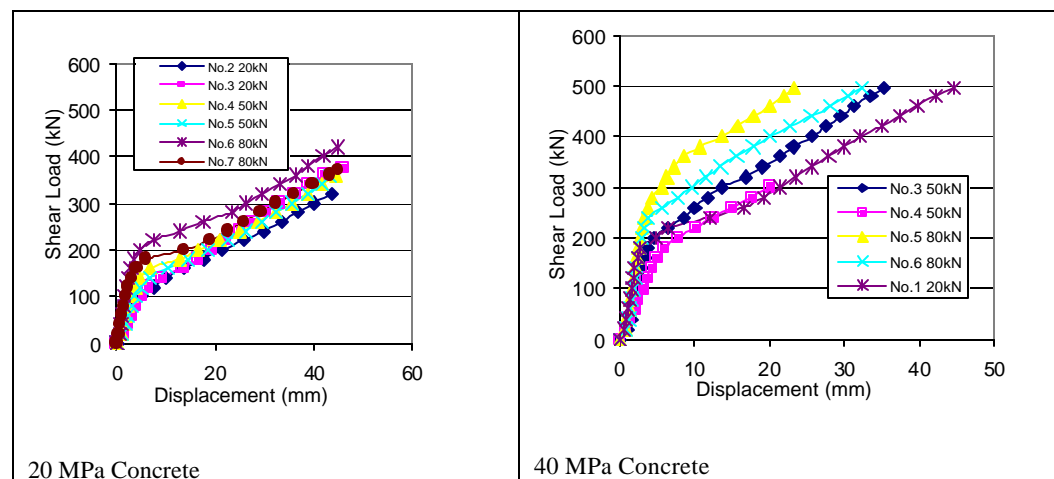


Fig 2 - Shear load and shear displacement of a bolt tested in both 20 and 40 MPa strength concrete and under different tensile loading conditions

- 3) The shear displacement at elastic yield point was not consistent irrespective of the concrete type and the axial load. This was the same for all three bolt types tested.
- 4) High profiled and closely spaced bolts such as Bolt Type T2 displayed constant shear load at all three levels of bolt tension loads in both 20 and 40 MPa concrete mediums. The consistency of shear loads at bolt type elastic yield point was more pronounced than the other bolt types.
- 5) Deflected bolt sections experienced regions of tension and compression. Resin columns remained adhered to the sides of the bolt region that experienced compression, but had broken off the sides that were in tension (Figure 1)

### 3D NUMERICAL ANALYSIS

A three dimensional finite element model of the reinforced structure subjected to the shear loading was used to examine the behavior of bolted rock joints and compare it with experimental results. Three governing materials (steel, grout, rock) with two interfaces (bolt-grout and grout-rocks) were considered for the 3D numerical simulation.

A general purpose finite element program (ANSYS, Version 7), specifically designed for advanced structural analysis, was used for 3D simulation of elasto-plastic materials and contact interfaces behaviour. 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 3 shows the three -dimensional model, which consists of 9964 nodes and 6460 elements, and Figure 4 shows the model cross section.

The elastic behavior of the elements was defined by Young's Modulus and Poisson's ratio of various materials as per Table 1.

**Table 1 - Elastic material properties**

| Material          | E (GPa) | Poisson's Ratio |
|-------------------|---------|-----------------|
| Concrete (20 MPa) | 20      | 0.25            |
| Concrete (40 MPa) | 32      | 0.25            |
| Grout             | 12      | 0.2             |
| Steel             | 200     | 0.3             |

The interface behaviour of grout -concrete was considered as a perfect contact, and was determined from the test results. However, the low value of cohesion (150 KPa) was adopted for grout-steel contact.

3D solid elements (Solid 65 and solid95) that have 8 nodes and 20 nodes were used for concrete, grout and steel respectively, with each node having three translation degrees of freedom, that tolerated irregular shapes without significant loss in accuracy. 3D surface-to-surface contact elements (contact 174) were used to represent the contact between 3D target surfaces (steel-grout and rock-grout). This element is applicable to 3D structural contact analysis and is located on the surfaces of 3D solid elements with midside nodes. The numerical modeling was carried out at several sub steps and the middle block of the model was gradually loaded in the direction of shear. Figure 5 shows deformed shape of elements.

Simulation of several models in varying conditions (a range of bolt pretension load and concrete strength) were carried under a vertical load and results were analysed for both linear and non-linear regions of the load - deflection curve.

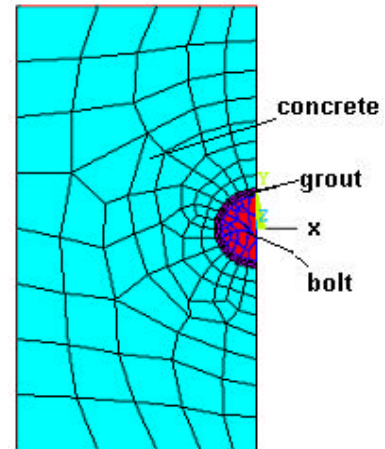
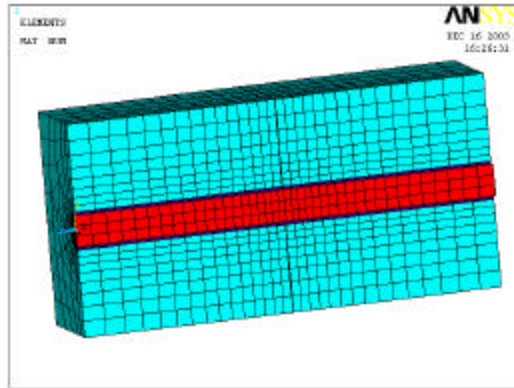


Fig 3 - Three-dimensional image of the numerical model

Fig 4 - Cross-section of the numerical model

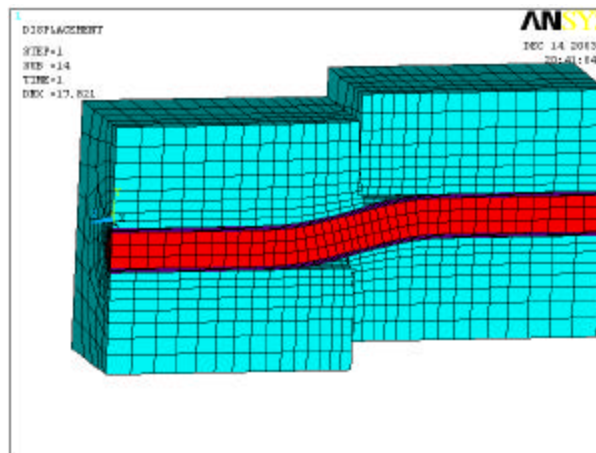
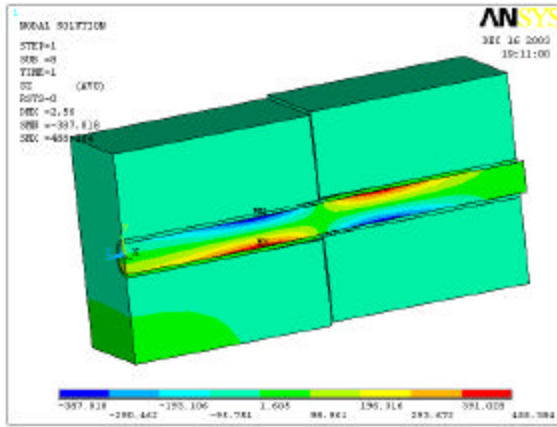


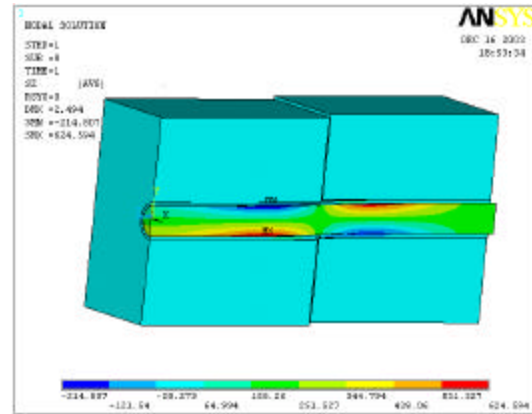
Fig 5 - Deformed shape of elements

### NUMERICAL RESULTS

Comparison of the stresses developed in both 20 and 40 MPa concrete, with the bolt pretension loads of 20 and 80 kN at both pre-failure and post failure were examined. In both concrete strengths (20 MPa and 40 MPa), the increase in bolt pretension led to the increase in the tensile stresses in the axial direction of the bolt (Figures 6 and 7), and the compressive stresses reduced, this trend is more dominant in the linear region than post failure region.

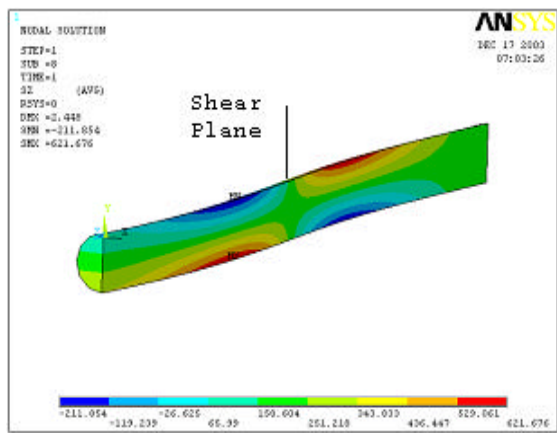


a - 20 KN initial tensile load

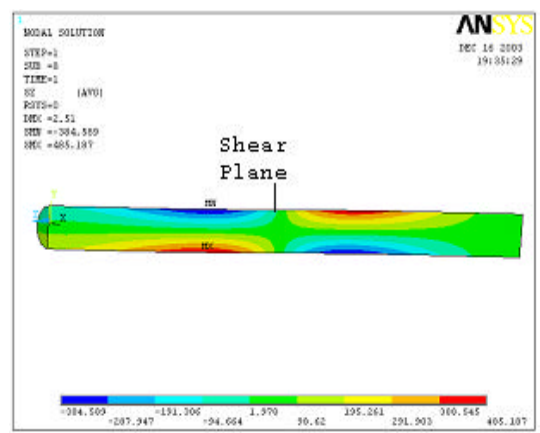


b - 80 KN initial tensile load

Fig 6 - Stress contours along the bolt length in 20 MPa concrete and in different pretension loads (a: 20 KN, b: 80 KN) in the pre-failure region.



a 20 KN



b 80 KN

Fig 7 - The stress contours along the bolt in 40 MPa concrete and in different pretension loads (a 20 KN and b 80 KN) in pre-failure region

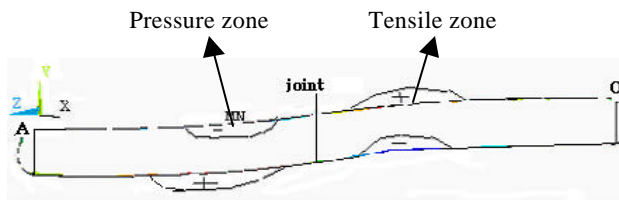


Fig 8 - Tensile and pressure stresses zones around the bolt

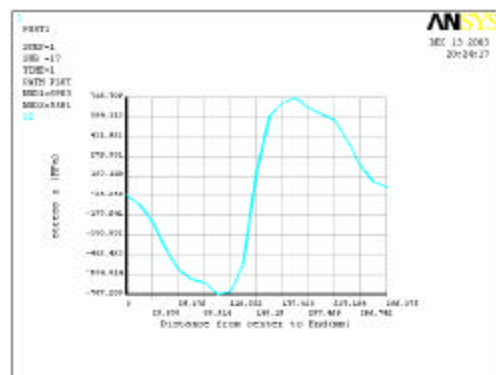
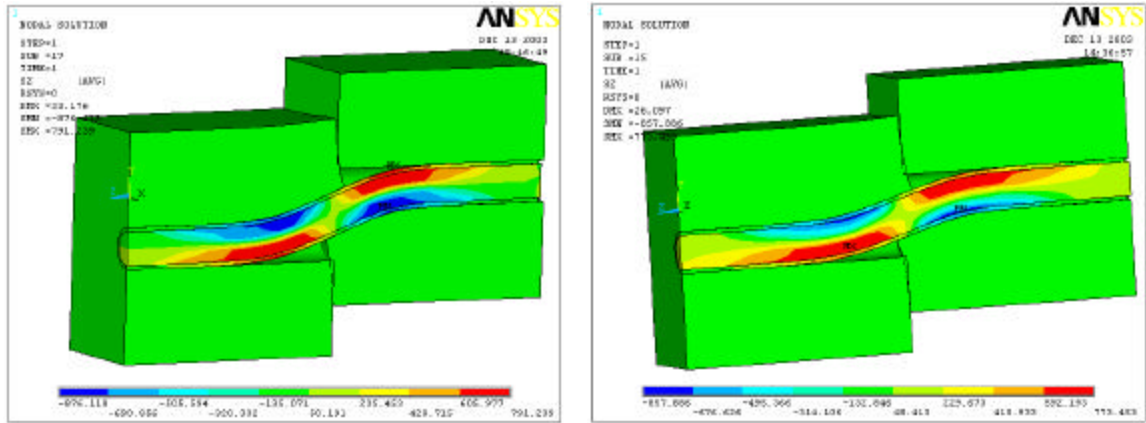
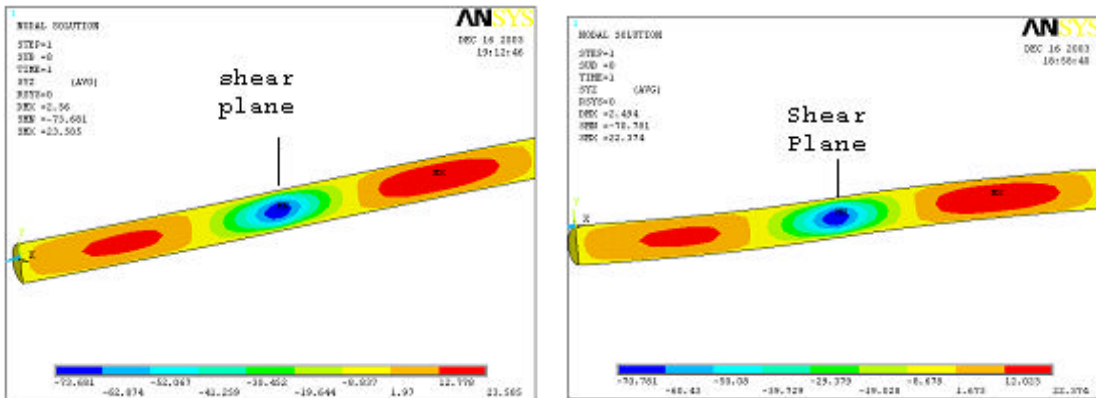


Fig 9 - Stress changes along the bolt

Increasing the confining pressure causes a reduction in bolt deflection but this reduction, if it occurs prior to yield point, is not significant as demonstrated in both the experimental and numerical results. However, the effect of pretension in post failure has significantly affected bolt deflection and that is demonstrated in the numerical and experimental results. As can be seen from Figure 8 the stresses in the upper half of the bolt and towards the perimeter are tensile while it is compressive at the centre. However the stress conditions at the lower half section of the bolt is reversed. This can also be observed from the experimental results as shown in Figure 1. The degree of the changes in the post failure region is plotted in Figure 9. It can be seen that stresses in these zones are high and the bolt appears to be in a yield situation. The location of these stresses is shown in Figure 10. Figures 11 and 12 show the shear stress contours along the length of the bolt at different pretensions and different concrete strengths. The maximum shear stress is concentrated in the vicinity of the joint plane. These stresses slowly increased after beginning at the plastic deformation level and finally ending at a stable situation. By increasing bolt pretension load, the shear stresses were decreased in both concrete types (20 MPa and 40 MPa), causing an increase in the bolt resistance to shear. With increasing shear loads, deflection in the bolt increases and plastic strain is created in critical locations in all materials. The situation of these strains and the rate of strain changes along the model are shown in Figures 13a and 13b respectively. Figure 13 b shows the rate of changes in plastic strain along the bolt. Contact pressure contours were found to decrease with increased the confining pressures and visa versa.



a 20 KN b 80 KN  
**Fig 10 - The stress contours along the bolt in 20 MPa concrete strength with different pretension 1**



a 20 KN b 80 KN  
**Fig 11 - Shear stress contours along the bolt in different pre-tension (a: 20 KN and b: 80 KN) with 20 MPa concrete strength in linear region.oads (a: 20 KN and b: 80 KN) in post-failure region**

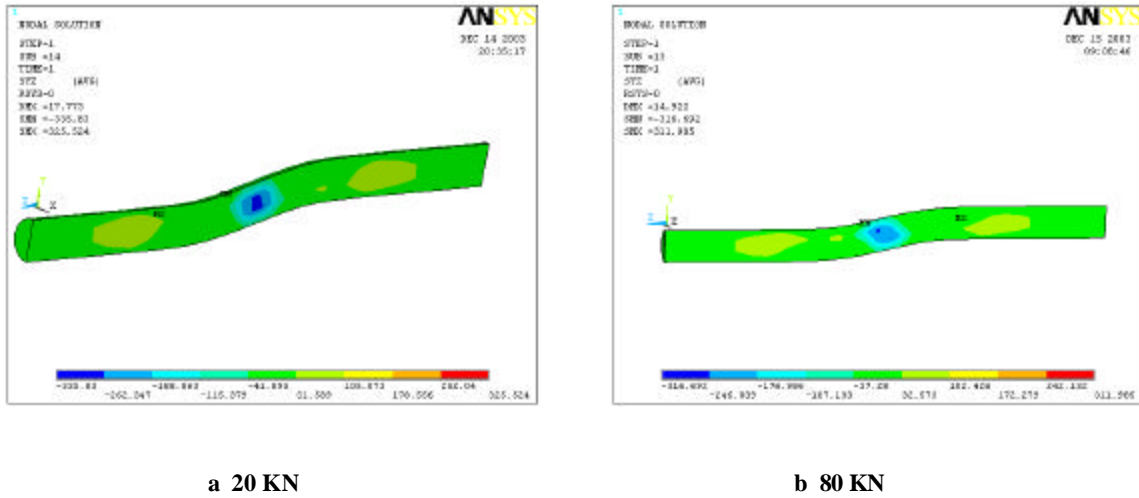


Figure 12 - Shear stress contours along the bolt in different pre-tension (a: 20 KN and b: 80 KN) with 40 MPa concrete strength in the post failure region

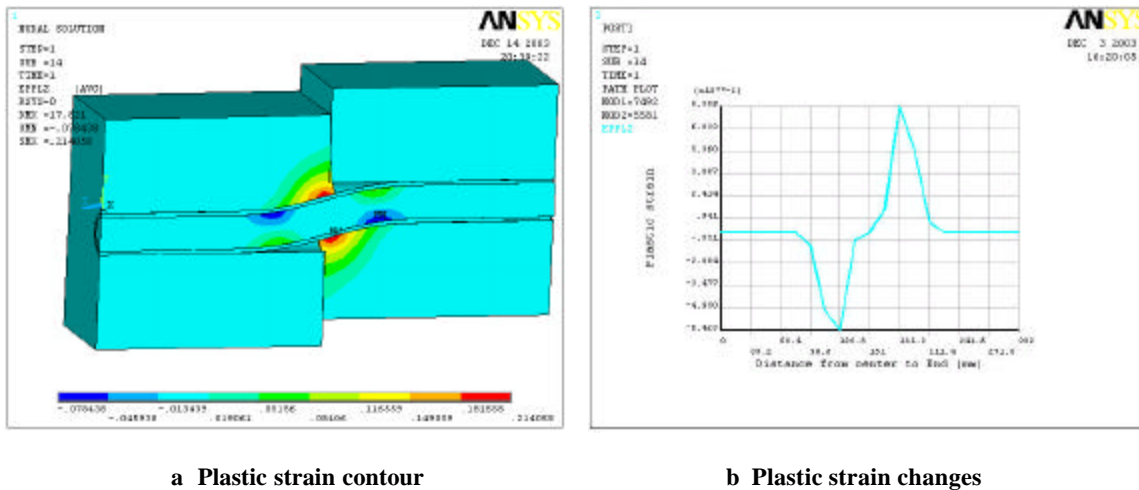
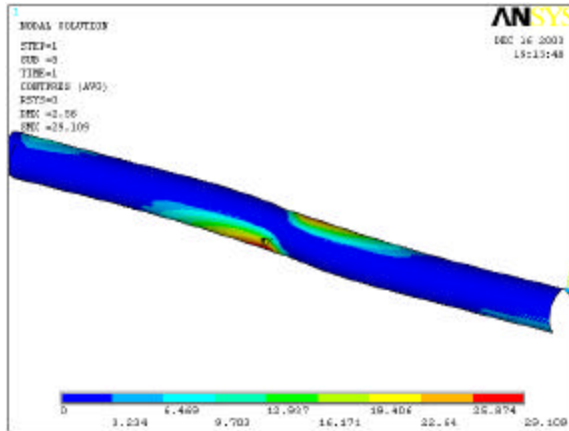


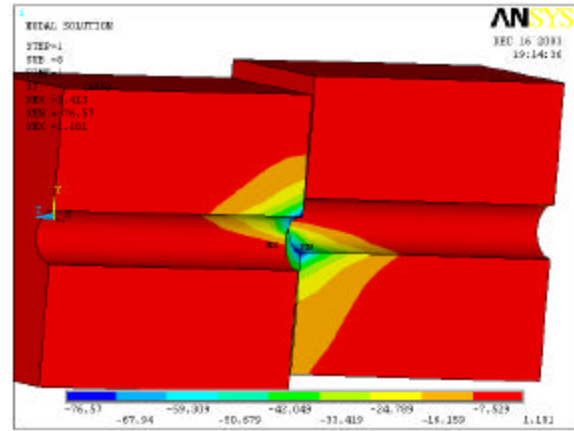
Figure 13 - Plastic strain contours and the rate of changes in concrete 40 Mpa strength with 20 KN pretension load

Fig 14 displays the contact pressure contour between the bolt-grout interfaces. With increasing shear load contact pressure was found to increase. However, the increase of pretension has reduced contact pressure. In addition induced stresses are produced in concrete blocks causing it to fracture and fail. Minimum main stresses in the corners of concrete blocks that are affected by the steel pressure are shown in Fig 15. As it can be recognized, the stresses around the edges in this area are high and higher pressure can induce longitudinal fractures in concrete blocks. This was also recorded in the experimental results.





**Fig 14 - Contact pressure contours in concrete 20 MPa strength with 20 KN tensile load in the pre-failure region**



**Fig 15 - Min stress contours in pretension load in the pre-failure region in concrete of 20 MPa strength and 20 KN**

## CONCLUSIONS

The numerical simulation of shearing associated with the reinforced joints and bedding planes provides a unique insight into the build up and distribution of stress when shearing occurred. The build up of stresses at both the bolt's elastic and plastic ranges were clearly demonstrated. Stresses generated as the shearing occurred on the reinforced block was quantified, which permitted a better understanding of the reinforcement applications and the role of bolt pretensioning. A number of conclusions drawn which were based on the numerical solutions were found to be in agreement with the experimental results and in particular:

- The maximum deflection at a particular applied vertical load was observed in softer medium and lesser initial tensile load.
- The tensile and compressive stresses were successfully simulated on each side of the shear plane. The level of stresses increased with increasing shearing load. However, post yield point there was no significant changes in their locations (Figures 1, 8, 10).
- Higher values of shear stresses were concentrated at the bolt sections near the shear plane (Figures 11,12) and by increasing pretension load causes led to reduction in shear load.

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