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New Technology for Measuring the In Situ Performance of Rock Bolts

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ABSTRACT:

Understanding the interaction between rock bolts and underground rock movement is critical for safe and cost effective underground excavation design. Although early research on this subject involved a balance of theoretical analysis and field measurement, recent work has been heavily focused on analytical and numerical studies. This paper describes technology that has the potential to redress the balance through instrumentation of rock bolts deployed under routine operating conditions in underground mines

Almost all previous instrumented rock bolt studies have used core technologies based on resistive strain gauges according to two scenarios (*i*) load cells positioned at the head of the bolt, and (*ii*) resistance strain gauge arrays recessed into grooves along the length of the bolt. Both approaches are complimentary with the selection criterion for each depending on factors such as whether the rock-bolt is fully grouted or end-anchored.

A new product is introduced in this paper. The *d*-REBAR involves an array of small-diameter long-base-length displacement sensors recessed into grooves along the entire length of the bolt. The transducers are interfaced with on-board digital signal conditioning and telemetry.

Methodologies for the deployment of the new instrumentation are presented, and guidelines are presented for the interpretation of results obtained based on data obtained from field trials. The results demonstrate the viability of these new technologies and moreover provide important insight into rock bolt/rock mass interaction.

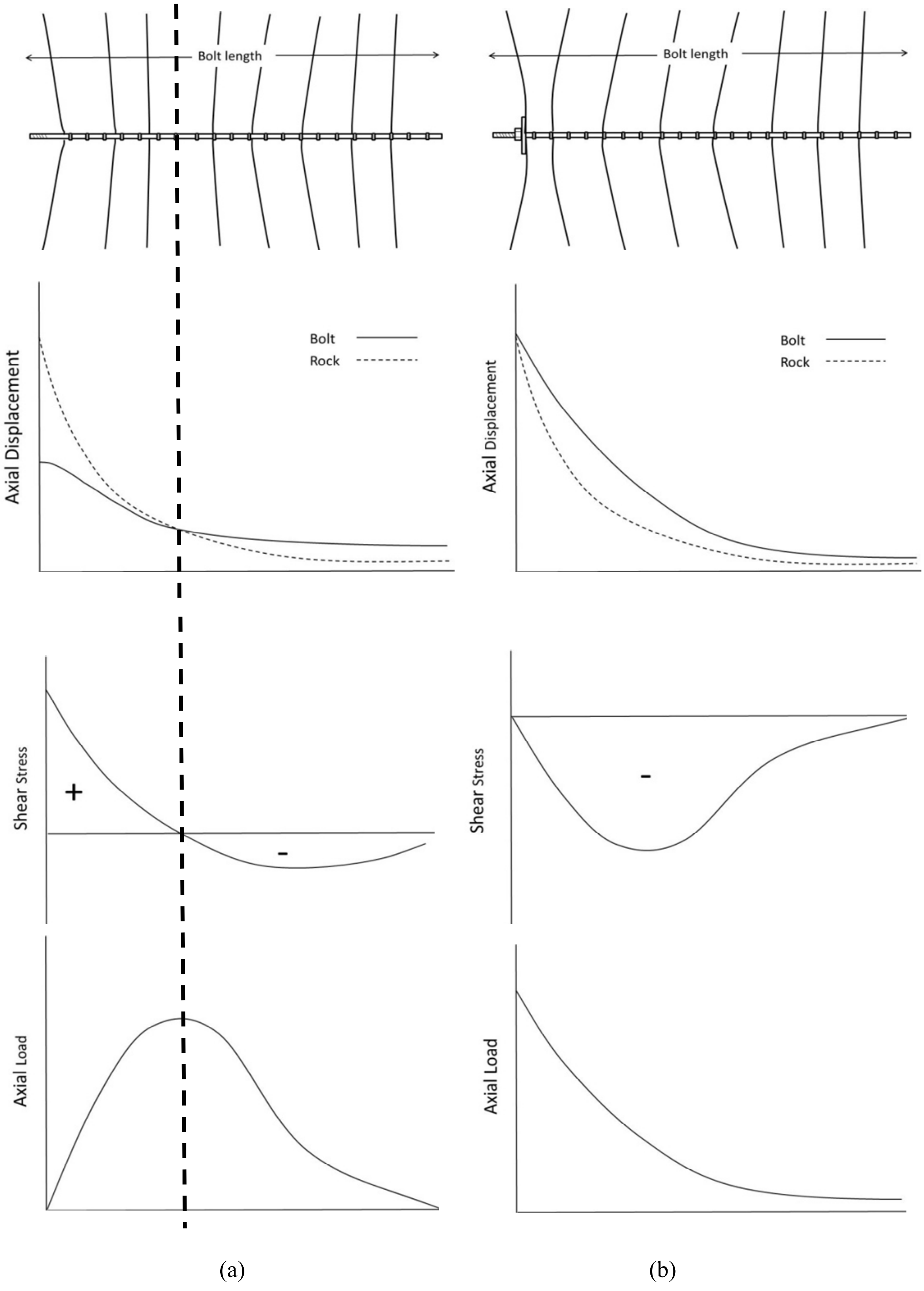
1. INTRODUCTION

Whereas early researchers such as Freeman[1] and Farmer[2] combined theoretical and experimental research, in recent years the research emphasis has become progressively biased towards analytical and numerical investigations. The advantage of combining experimental with theoretical research components is that a support design *feedback loop* can be established for which predictions from the theoretical and numerical models can be compared against experimental data to calibrate the modeling input parameters. Such a feedback loop is more likely to engage practicing engineers and operational decision-makers who may dismiss theoretical or numerical studies alone.

1.1 Axial behavior in a continuum rock mass

The concepts associated with end anchored and partially grouted rock bolts are generally well understood. Along the free length of the bolt the load is constant and equal to the resisting force developed by either a mechanical anchor or a partial section of resin grout.

The case for fully grouted rock bolts is more complex, which probably explains why the majority of recent theoretical studies have been focused on the axial deformation of fully grouted rock bolts in response to a continuous distribution of rock mass convergence. A conceptual model was presented by Freeman [1] that divided the bolt into two sections (Fig. 1).



(a) (b)

Fig. 1: The two different end conditions for the rock bolt problem: (a) no faceplate, and (b) faceplate attached at the excavation surface

The pick-up length, which is closest to excavation surface, is that length of the bolt for which the rock displaces more than the bolt and for which the shear forces developed in the grout annulus restrain rock movement. To maintain equilibrium along the bolt, there must exist also a section of bolt for which the shear forces are of the opposite sense. This section is referred to as the anchor length because the bolt displaces more than the rock and therefore the rock restrains the bolt from moving towards the excavation surface. The point of maximum axial load along the bolt is referred to as the *neutral* point and is defined as that point at which the rock mass and the bolt displace equally so that the shear stress within the resin annulus is zero. Freeman's conceptual model was in agreement with *in situ* data collected from instrumented bolts installed in soft mudstone as part of the Kielder (UK) experiment. If the bolt is installed with a rigid plate at the bolt head and the excavation surface displace equally and the neutral point occurs at the excavation surface (Figure 1b). However, in practice the flexural response of the plate and its bearing action will be somewhat compliant, so that even with a face plate the bolt will exhibit behavior intermediate between the two end-member cases shown in Figure 1.

Subsequently many variations of analytical and numerical models have been developed [3-6] providing progressively more sophisticated predictions of how the axial load distribution will change along the bolt, and more importantly, how the bolt will interact with the rock mass to restrain ground movement. For a specific rock mass behavior the rock bolt performance depends on such factors as: (*i*) the mechanical properties of the bolt and faceplate, (*ii*) installation characteristics such as whether the rock-bolt is pre-tensioned, (*iii*) the bond properties between the bolt and the resin grout, (*iv*) the bond properties between the resin grout and the rock, and (*v*) the timing of bolt installation after excavation [4].

1.2 Axial rock bolt behavior in a discontinuous rock mass

In a discontinuous rock mass the load distribution along the bolt will be dominated by discrete rock mass displacements on a limited number of discontinuities (Li [3]). A closed form solution to this problem was developed by Hyett *et al.* [6]. Numerical models demonstrated that, especially for longer bolts such a fully grouted cable bolts, several peaks in load may occur along the bolt length. Such an effect was observed experimentally by Bjornfot and Stephansson [7] for rock-bolts in very hard, blocky rock at the Kiruna Mine in Sweden.

1.3 Lateral rock bolt behavior in a discontinuous rock mass

A dowel effect occurs when a rock-bolt transects a rock discontinuity which is undergoing shear behavior. If the rock bolt is aligned perpendicular to the discontinuity then the deformation may be almost perfect shear, although in general, a combination of shear and axial deformation will occur. Experimental evidence shows that dowel effect is often localized within a 200mm section (Ferrero [8]). Numerous instances have been reported for failed bolts which display permanent shearing following exposure after a fall of ground (Li [4]). McHugh and Signer [9] stated that shear loading can significantly contribute to the failure of bolts used for rock reinforcement in coal mines. However, research on the shear deformation of rock bolts is far less extensive than that of axial deformation, because the complex loading mechanism is difficult to monitor and the uncertainties of determining just where loading is taking place make it difficult to accurately determine shear loads [9].

2. PREVIOUS *IN SITU* MONITORING OF ROCK BOLTS

Previously all attempts at monitoring rock bolt loads have concentrated on short base-length (typically < 20mm) strain gauges adhered into grooves machined along the length of the bolt. To compensate for any bending or shear deformation diametrically opposed grooves or slots are populated with pairs of strain gauges, from which the axial strain is given by:

$$\varepsilon_{axial} = (\varepsilon_A + \varepsilon_B)/2, \qquad (1)$$

where the ε_A and ε_B are the strain gauge reading on side A and side B of the rock bolt at the same distance along the bolt. The corresponding bending strain can be written as:

$$\varepsilon_{shear} \geq (\varepsilon_A - \varepsilon_B)/2, \qquad (2)$$

where the inequality acknowledges that strain gauges in three grooves would be required to definitively determine the maximum magnitude of the bending strain vector.

An extensive body of experimental and development research using strain gauged rebar has been conducted in the US over a 30 year period principally by NIOSH [10-12] and its predecessor, the United States Bureau of Mines. Signer and Jones [10] tested fully grouted roof bolts with strain gauge instrumentation. Signer *et al.* [12] continued the research using strain gauge instrumented rock bolts in coal mine gate roads, and determined that, even though the bolts were plated, they displayed the characteristic behavior predicted by the axial continuum theoretical model without a faceplate (Figure 1a). Specifically, a distinct maximum in the load profile representing the neutral point was observed at 0.75m into the roof. The load decreased near the roof-line indicating that the faceplate was relatively compliant and not perfectly rigid. In practice this is expected, even in hard rock, because the faceplate's primary role is to secure screen and strapping.

In all of the previous experimental studies using short base-length strain gauges, less than 10% of the total bolt is monitored and the short gauge lengths are likely to be strongly influenced by errors related to any localized shear deformation. Although important, this may overshadow the overall axial performance of the rock bolt that is primarily of interest to researchers and engineers interested in the optimization of support design.

3. A NOVEL APPROACH TO ROCK BOLT INSTRUMENTATION

A distributed instrumentation scheme for rock bolts involves a compromise between the number of gauges (i.e. cost) and the accuracy with which the strain profile along the bolt is resolved. Instrument cost is important since a percentage of instrumented bolts will be lost due to operational related attrition, firstly because the installation process involves spinning the grouted bolt in the resin, and thereafter because, at the production face, the proximity of mobile heavy machinery presents an on-going hazard. The accuracy with which the load distribution along the bolt is measured using a discrete number of gauges depends on (*i*) the number of gauges, (*ii*) the base-length of the gauges, and (*iii*) the intrinsic accuracy and resolution of the gauges. Short base-length measurements will very accurately measure the load at specific locations but may be unrepresentative of the intervening bolt length especially if load concentrations occur at discontinuities. Long base-length measurements will capture localized deformation due to any discrete points of loading, but due to an averaging effect will underestimate the extreme values especially if the strain profile varies dramatically.

3.1 Displacement sensor approach

An alternative approach has been proposed [13, 14] based on an array of sub-micron resolution displacement sensors that measure the change in displacement or stretch (u^i) of the bolt. The end-points of the displacement sensor are attached to the bolt to define a base-length (L), typically in the range 200-500*mm*; in other words more than an order of magnitude longer than for a resistive foil strain gauge. The corresponding strain (ε^i) for the *i*th sensor is determined from:

$$\varepsilon^i = u^i / L \; . \qquad (3)$$

The instrument has digital signal conditioning that transmits the strain values directly based on Equation 3. In order to control the units cost, for all the case studies conducted so far, the number of displacement sensors has been limited to six. The strain gauges were arranged with three in each diametrically opposed slot (sides A and B of the rock bolt) in an end-to-end arrangement (Figure 5) so that the entire length of the bolt was monitored. Sensors denoted by i = 1, 3 and 5 are located on side A and sensors 2, 4 and 6 are on side B. Two different configurations termed *stacked* and *staggered* have been used.

3.2 Stacked Configuration

In the stacked configuration (Figure 5a), the axial strain and load at the three nodal points can be determined according to:

$$\varepsilon^i_{axial} = (\varepsilon^{2i-1} + \varepsilon^{2i})/2 \qquad (4)$$

$$F^i_{axial} = EA \, \varepsilon^i_{axial} \qquad (5)$$

with i=1 to 3 (Figure 5a). This configuration provides only three points of lengthwise load determination, and therefore the load profile is somewhat crude. The corresponding bending strain is:

$$\varepsilon^i_{bending} \geq (\varepsilon^{2i-1} - \varepsilon^{2i})/2 \qquad (6)$$

Since the gauges are arranged end-to-end and monitor the whole length of the bolt the equivalent displacement profile can be written:

$$u^i_{axial} = \sum_{n=1}^{n=i} (\varepsilon^n{}_{axial} \times \mathrm{L}) \quad (7)$$

as graphically represented in Figure 6a. In other words the bolt displacement, or stretch, relative to the toe of the bolt is calculated from the summation of the measured axial strains (output from the instrument) multiplied by the gauge length. It should be recognized that a distinction is made between the measured displacement, u^i, from each displacement sensor and the calculated axial displacement, u^i_{axial}, which accounts for any bending.

3.3 Staggered Configuration

For the staggered configuration, the gauges on Side A of the bolt are offset from those on Side B by one half the base-length of the sensor. In this case a data reduction scheme is implemented based on the central difference approximation to the second order governing differential equation for displacement variation along a grouted bolt (Farmer [2]). The strains at the *i*th nodal point can be approximated as:

$$\varepsilon^i_{axial} = (\varepsilon^{i-1} + 2\varepsilon^i + \varepsilon^{i+1})/4 \qquad (8)$$

$$\varepsilon^i_{bending} = (\varepsilon^{i-1} - 2\varepsilon^i + \varepsilon^{i+1})/2 \qquad (9)$$

where ε^i is the strain measured by the displacement gauge centered at the *i*th nodal point and ε^{i-1} and ε^{i+1} are the two overlapping gauges on the opposite side of the bolt. At the ends of the bolt the following two approximations are used. For node 1,

$$\varepsilon^i_{axial} = (2\varepsilon^1 + \varepsilon^2)/3 \qquad (10)$$

and for node 6, knowing that ε_{axial} must be zero at the toe end of the bolt:

$$\varepsilon^i_{axial} = (2\varepsilon^6 + \varepsilon^5)/4 \qquad (11)$$

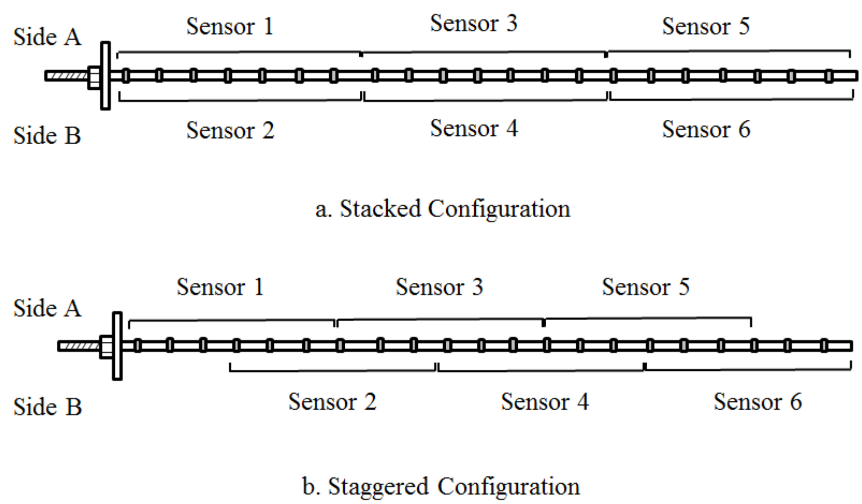


Fig. 5: The two different d^{6}REBAR configurations.

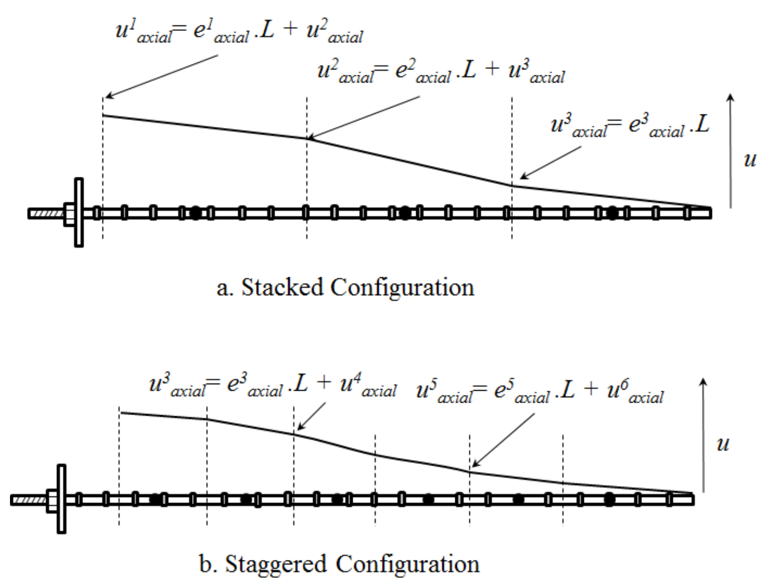


Fig. 6: Displacement determination based on summation along the bolt.

Again, since the gauges are configured end-to-end and monitor the whole length of the bolt the corresponding displacement profile at the *i*th nodal point can be calculated from the summation:

$$u^i_{axial} = \sum_{n=6}^{n=i} (\varepsilon^n_{axial} \times L) \qquad (12)$$

relative to the toe end of the bolt which is a reference. In this manner an approximation of axial displacement (Figure 6), strain and load can be determined at six nodal points along the bolt. This provides additional axial resolution compared to the stacked configuration while sacrificing some degree of accuracy related to bending strain errors.

Table 2: Axial calibration of the d[6]REBAR

APPLIED STRAIN (ue)	MEAS. STRAIN 1 (ue)	MEAS. STRAIN 2 (ue)	MEAS. STRAIN 3 (ue)	MEAS. STRAIN 4 (ue)	MEAS. STRAIN 5 (ue)	MEAS. STRAIN 6 (ue)
-0.3	+ 0	+ 0	+ 0	+ 0	+ 0	+ 0
165.8	+ 164	+ 166	+ 167	+ 166	+ 165	+ 163
328.6	+ 323	+ 327	+ 328	+ 326	+ 325	+ 324
668.6	+ 678	+ 678	+ 680	+ 677	+ 677	+ 673
992.0	+ 990	+ 994	+ 997	+ 993	+ 994	+ 990
1323.0	+ 1315	+ 1319	+ 1323	+ 1319	+ 1320	+ 1317
1652.6	+ 1649	+ 1653	+ 1654	+ 1651	+ 1653	+ 1649
CAL. SLOPE	0.9973	0.9992	1.0017	0.9985	1.0001	0.998
CAL. OFFSET	0.3247	1.5109	1.8226	1.3188	0.4321	-0.6371

3.4 Laboratory Validation and Calibration

Calibration of the instrumented rock bolts involves axial loading of the bolt to determine the relationship between applied and measured strain. Results from a typical axial calibration test are shown in Table 2. Calibration data is typically collected on the third loading cycle.

4.0 FIELD CASE STUDIES

4.1 Case Study 1: Rock bolt behavior in a continuous rock mass continuum.

Four instrumented (stacked gauges) rock bolts were installed across a heading in a coal mine. Bolts #73 and #74 were installed at the center of the excavation and #71 and #75 at the wall. The response of the individual strain gauges for #73 is shown in Figure 7. Lengthwise profiles of axial strain (ε_{axial}) and axial displacement (u_{axial}) are plotted in Figure 8 for all four bolts. 2000μm of axial displacement was measured by the rock-bolts at the center of the span (#73 and #74), which is approximately twice that measured by the bolts adjacent to the wall (#71 and #75). Figure 9 shows contour plots of bolt displacement (in microns) and bolt strain (in με – 153με being equivalent to 1tonne) superimposed on the location of the 4 rock bolts.

Case Study 2: Rock bolt behavior in a discontinuous rock mass.

Case study 2 presents measurements from a cluster of fully grouted passive rock bolts instrumented with staggered gauges that were installed at an intersection in a room-and-pillar coal mine. Initially all of the bolts exhibited low magnitudes of bolt loading, typically in the 2tonne range (153με/tonne). However, simultaneously the majority of the bolts in the intersection began to load up rapidly (Figure 10). Based on the complete dataset [14], it became apparent that bolts were being loaded at different heights above the roofline. Analysis based on spatial contouring revealed that the strain is asymmetric, with one interpretation being that it is concentrated along an inclined structure (Figure 11 bottom).The corresponding displacement contours (Figure 11 top) clearly seem to define a wedge. The weight of this wedge was not sufficient to overload the rock bolt array and consequently the roof of the intersection stabilized without remediation.

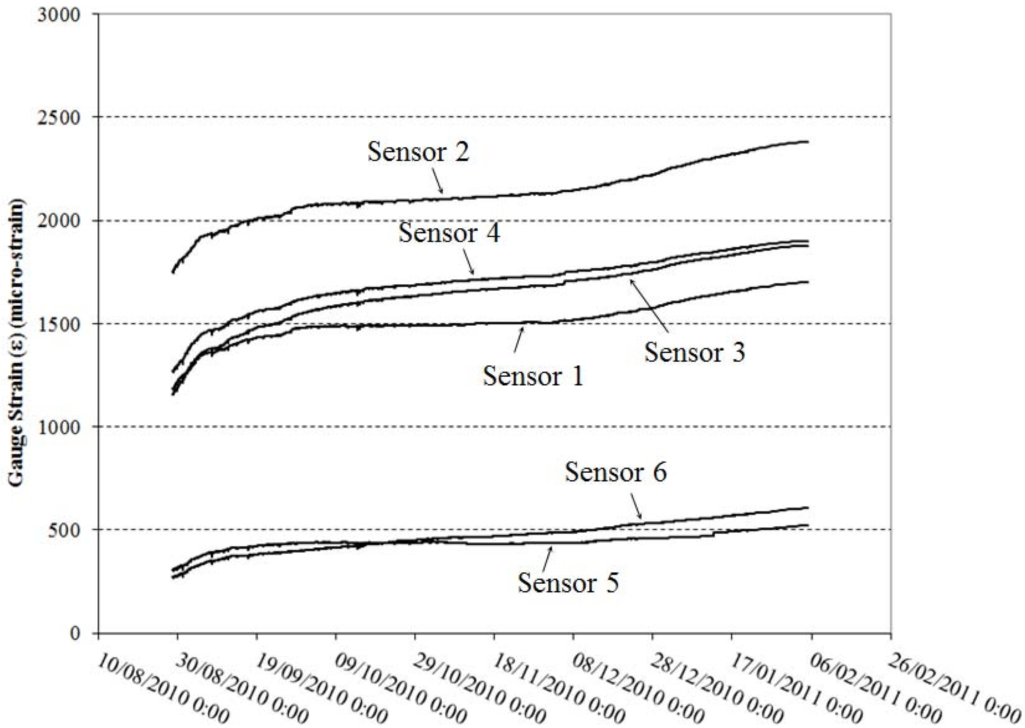


Fig. 7: Typical strain gauge results for a single rock bolt (#73) with stacked gauges. The three displacement sensor pairs are arranged as follows: 1 and 2 are the stacked pair nearest the head, 3 and 4 in the center of the bolt and 5 and 6 at the toe end.

5.0 CONCLUSIONS

An innovative instrumented rock bolt technology, the d-REBAR, has been described that offers significant potential for the accurate monitoring of rock bolt performance. The sensing elements for the d-REBAR have been interfaced with digital signal processing technology to provide actionable output data at the immediate underground location.

Data from case studies using the d-REBAR, presented in this paper and elsewhere [12, 13], indicate that this new instrument can provide a low-cost solution for the determination of axial loading of rock bolts. The design, which uses long base length strain gauges to monitor separate sections of the bolt, has two distinct advantages:

(i) it is able to accurately monitor both the strain and displacement of the bolt which can be compared with numerical and analytical models, and

(ii) the long base length tends to cancel out any local perturbations especially those related to localized lateral shearing movements which might otherwise have complicated the analysis.

The case studies conducted to this point have yielded promising results which have enabled techniques such as “spatial contouring” to be applied. This analysis technique provides a powerful visual rendition of the data which enhances data presentation at the operational level. Furthermore the plots can be directly overlaid on results from numerical models, so facilitating back-analysis, and completing a *feedback loop* for support design

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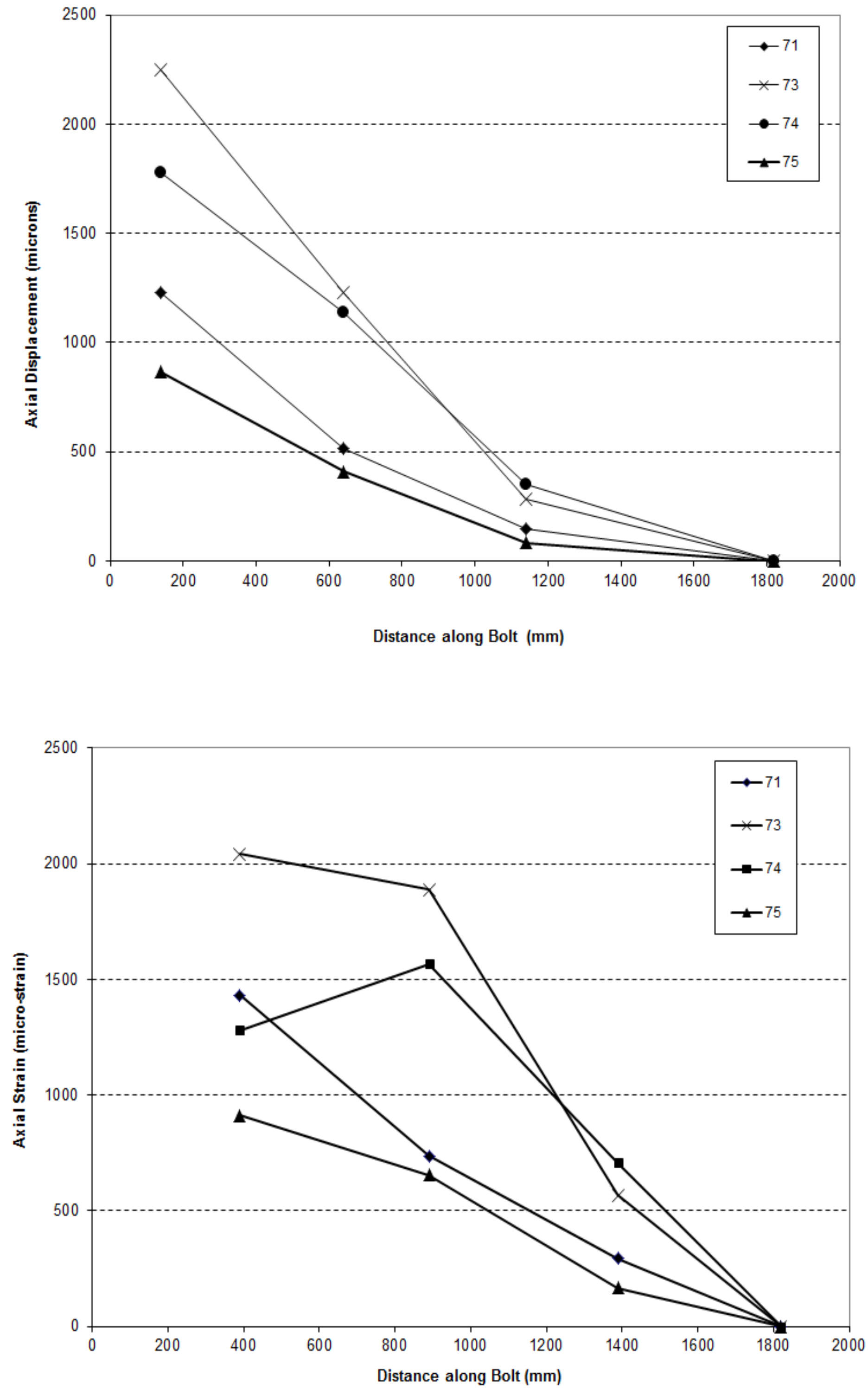

Fig. 8: (Top) Axial stretch or displacement along the bolt length and (Bottom) strain distribution along the bolt length (153με/tonne).

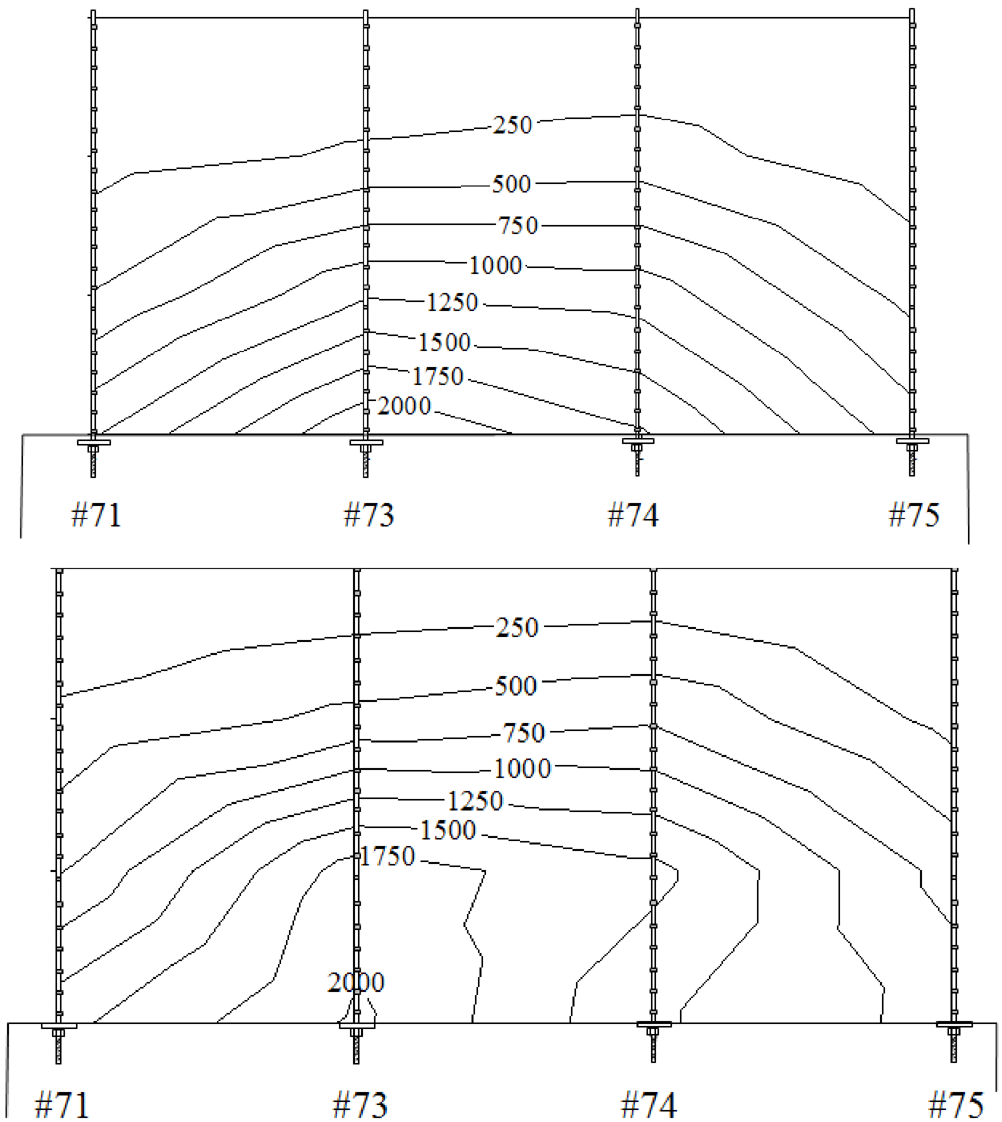

Fig. 9: Spatial contoured plots of the axial displacement in μm (upper plot) and axial strain in $\mu\varepsilon$ (lower plot). In the lower plot $153\mu\varepsilon$ is equivalent to 1tonne, so that the maximum load on bolt #73 is around 13tonnes.

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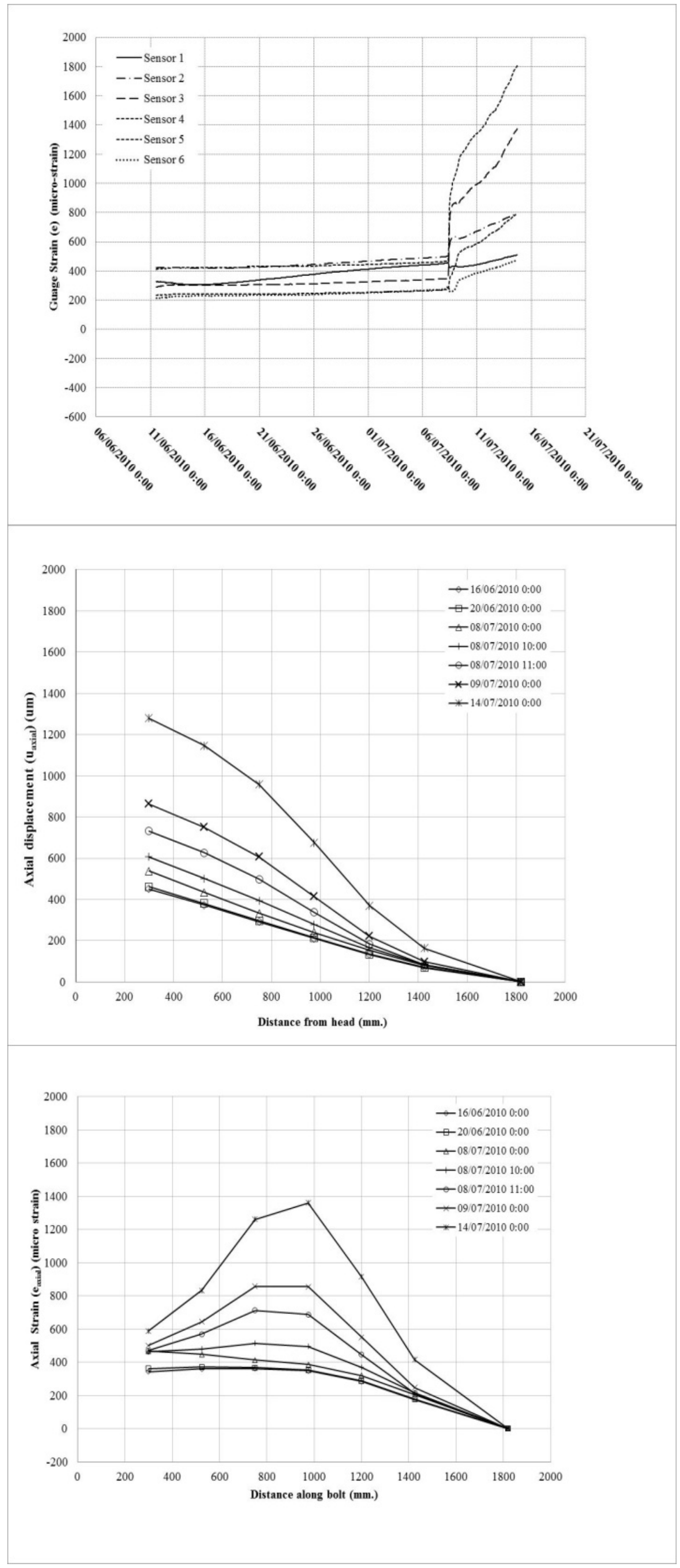

Fig.10: The response of a rock bolt #75 in case study 2 (upper) response of the 6 strain gauges with time (middle)Axial displacement versus length and (lower) bolt strain versus length (153µe/tonne).

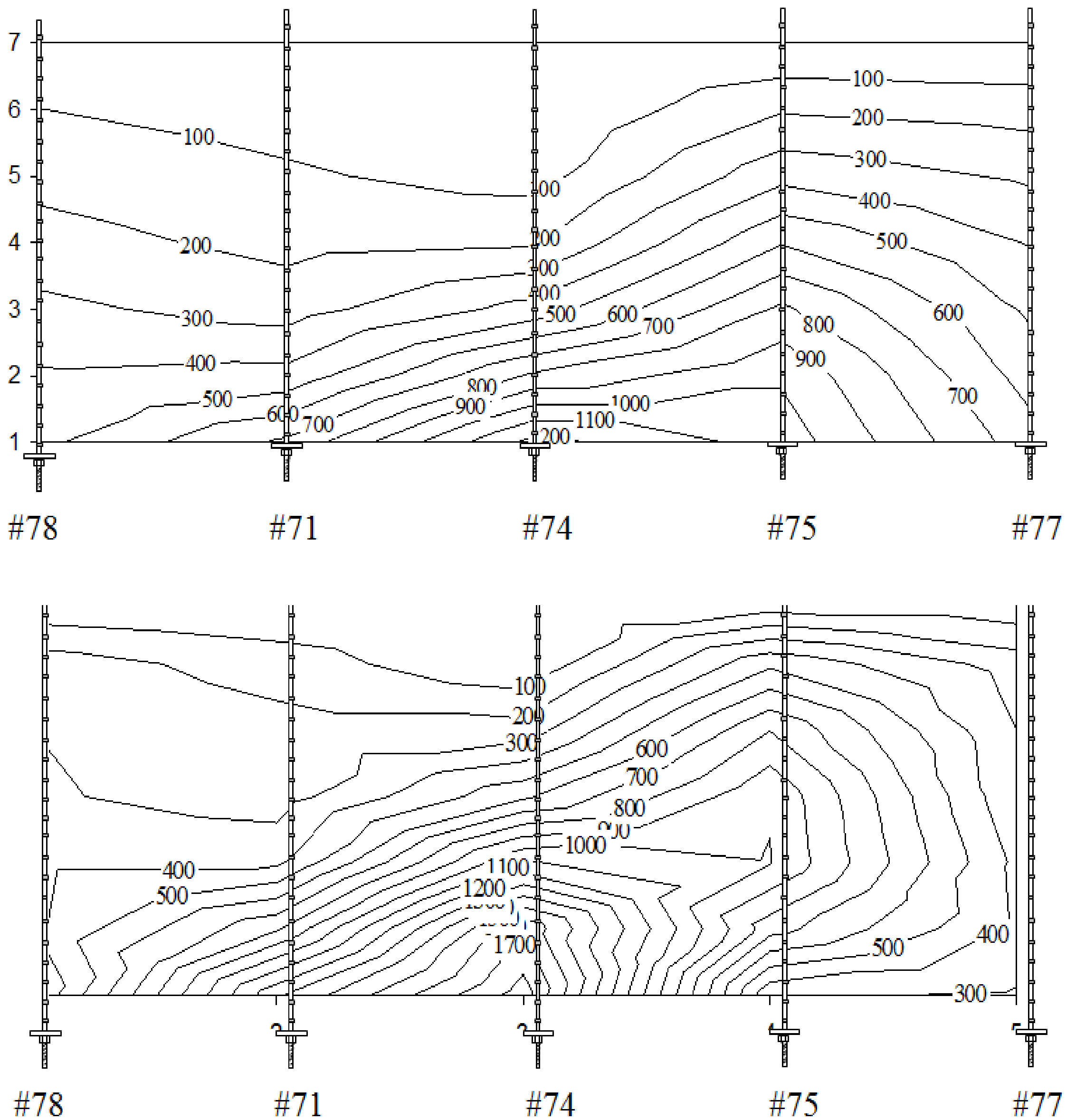

Fig. 11: Spatial contour plots of rock bolt displacement in μm (upper) and strain in με (lower). 153με=1tonne. The maximum load on #74 is approximately 11tonnes.