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Technical Note

New technology for measuring the *in situ* performance of rock bolts

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1. Introduction

The vast majority (> 90%) of the approximately 68 million roof bolts installed annually in underground coal mines in the USA are fully grouted using resin cartridges. Among the bolts that use resin, the most popular is the fully grouted rebar bolt, which is considered a passive (un-tensioned) support as it is un-tensioned on installation. In 2005 it was estimated that about 68% of all the bolts used in US were passive rebar bolts [1]. The other class of roof bolts is considered active systems, as they apply some amount of load to the roof at the time of installation.

Despite the safety consequences of ground instability, there remains a deficit of practical engineering understanding indicating how roof bolts provide reinforcement to the rock mass surrounding an excavation as mining proceeds. This applies particularly to “fully grouted” bolts which have gained popularity due to their superior performance compared to “end-anchored” bolts. Whereas for an end anchored bolt the tension in the bolt is constant over its length, for a fully grouted bolt the load distribution is more complex and varies depending on factors such as (i) the physical properties of the bolt (ii) the installation procedure, (iii) the polyester resin bond between the bolt and the rock borehole, and finally but importantly (iv) the distribution of movement in the rock mass surrounding the bolt.

Whereas all previous attempts at rock bolt instrumentation have used resistive strain gauges, the new instrumentation described in this paper is based on an array of small diameter displacement sensors that directly measure the induced displacement (*i.e.* stretch) of the bolt over its entire length. Upon dividing the measured displacement by the base-length of the sensor, an average strain is determined, which based on the material properties of the bolt, can be related to the load. The design and development of the instrumentation has been influenced by the

requirement to provide both (i) a tool that can validate existing experimental and theoretical research, and (ii) a commercial and cost effective rock bolt monitoring solution. Results are presented from two initial field trials in room-and-pillar coal mines operated by Peabody Energy.

2. Theoretical load distribution along rock bolts

Theoretical understanding provides the foundation for the design, implementation and interpretation of any successful instrumentation project, especially one relating to a problem as complex as the rock bolts/rock mass interaction. As mentioned above, for a fully grouted bolt the rock displacement profile has a major effect on the load distribution induced along the bolt.

2.1. Rock bolt behaviour in a continuous rock mass

The majority of recent theoretical and analytical studies have been focussed on the axial deformation of fully grouted rock bolts in response to continuous distribution of rock mass convergence [2–7].

Freeman [2] presented a conceptual model for the axial loading of a rock-bolt in a continuum that divided the bolt into two sections (Fig. 1). 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

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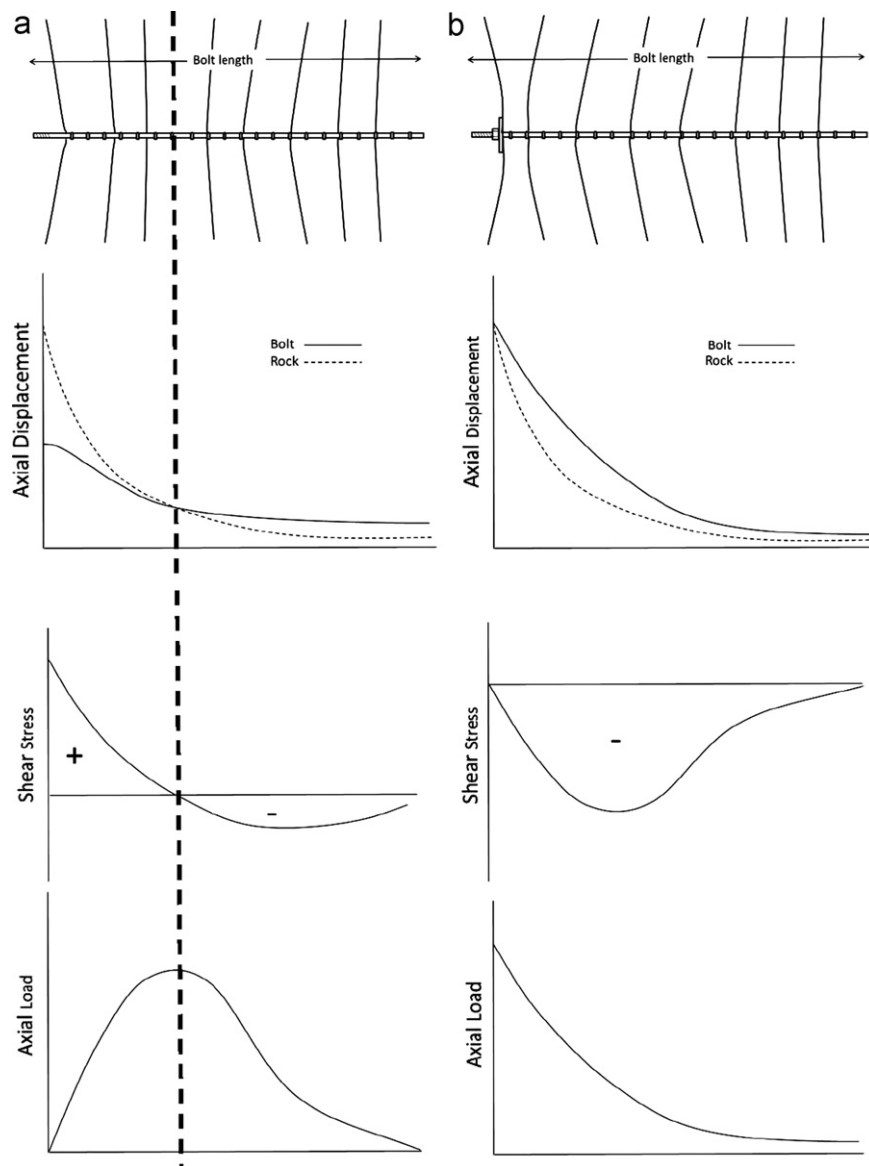


Fig. 1. The two different end conditions for the rock bolt problem: (a) no faceplate, and (b) faceplate attached at the excavation surface.

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 tight to the excavation surface with a rigid plate, the bolt head and the excavation surface will displace equally so that the neutral point occurs at the excavation surface (Fig. 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 behaviour intermediate between the two cases shown.

Analytical and theoretical studies [3–7] have established that axial 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 actively pre-tensioned, (iii) the bond properties between the bolt and the grout, (iv) the bond properties between the grout and the rock, and (v) the timing of bolt installation after excavation is created [5].

2.2. Rock bolt behaviour in a discontinuous rock mass

In a discontinuous rock mass the load distribution along the bolt will be dominated by a limited number of localized

discontinuity openings [8]. A closed form solution to this problem when a rock bolt transects a rock discontinuity undergoing dilatational behaviour was developed by Hyett et al. [9]. Numerical models demonstrated that, especially for longer bolts such as fully grouted cable bolts, several peaks in load may occur along the bolt length. Such an effect was observed experimentally by Bjornfot and Stephansson [10] for bolts in very hard, blocky rock at the Kiruna Mine in Sweden.

A localized "dowel" effect [11] occurs when a rock-bolt transects a rock discontinuity which is undergoing shear behaviour. If the rock bolt is aligned perpendicular to the discontinuity under shear then the deformation may be almost perfect shear, however in the general case a combination of shear and axial deformation will occur. Numerous instances have been reported by Li [8] for which failed bolts displaying permanent shearing have been exposed after a fall of ground, and McHugh and Signer [12] state that shear loading can significantly contribute to the failure of bolts used for rock reinforcement in coal mines. However, the literature on the shear loading of rock bolts is less extensive than that for that for axial loading, because this complex loading mechanism is difficult to monitor without *a priori* knowledge of the shearing location.

3. Previous experimental work

Over the preceding 30 years an extensive body of experimental research has been conducted by NIOSH [12–15], formerly the US Bureau of Mines in the US. The instrumented bolts developed involve populating two diametrically opposed slots with pairs of short base-length resistive strain gauges to compensate for any bending or shear deformation diametrically opposed grooves or slots are populated with pairs of strain gauges. Hence the axial strain is determined at discrete locations along the bolt length, each based on the equation:

$$e_{\text{axial}} = (e_A + e_B)/2 \quad (1)$$

where e_A and e_B are the strain gauge reading on Side A and Side B of the rock bolt at the same point along the bolt. The instrumented bolts were calibrated using a uniaxial tension machine in order to eliminate factors such as (i) the cross sectional area not being well defined for a slotted bolt, and (ii) inaccuracies due to strain gauge misalignment [13].

The corresponding bending strain is usually calculated from

$$e_{\text{shear}} = (e_A - e_B)/2 \quad (2)$$

although strictly this value only yields a minimum value, since strain gauges located in at least three orientations would be required to determine complete bending strain vector. If the direction of rock mass shear displacement is known *a priori*, and the orientation of the bolt can be accurately controlled during installation then, in theory, measurement of shear loading can be performed based on strain data from two slots.

An example of measurements obtained using this technology (Signer et al. [11]) is shown in Fig. 2. Even though the bolts are plated, they exhibit the characteristic behaviour predicted by the axial continuum theoretical model without a faceplate (Fig. 1a).

Specifically, a distinct maximum in the load profile representing the neutral point is observed around 0.75 m into the roof. The load decreases near the roof-line indicating that the faceplate is relatively compliant and not perfectly rigid as shown in Fig. 1b. In practice this will usually be the case (even in hard rock) since the rigidity of the faceplate at the roof is compromised by its primary role which is to secure screen and strapping. Hence, in operational reality, the situation in Fig. 1b, which continues to be widely analyzed in the literature (e.g. Ref. [12]), may be relatively rare.

4. A new experimental approach

Instrumentation design 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 can be resolved. Instrument cost is important since a percentage of instrumented bolts will be lost to production related attrition, firstly because the installation process involves spinning the grouted bolt in the polyester resin, and secondly 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 accuracy and resolution of the gauges, and (iii) the base-length of the gauges. Concerning the latter, short base-length strain gauges 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 strain gauges will capture localized deformation due to any discrete points of loading, but due to an averaging effect may also be unrepresentative of the extreme values if the strain profile varies dramatically.

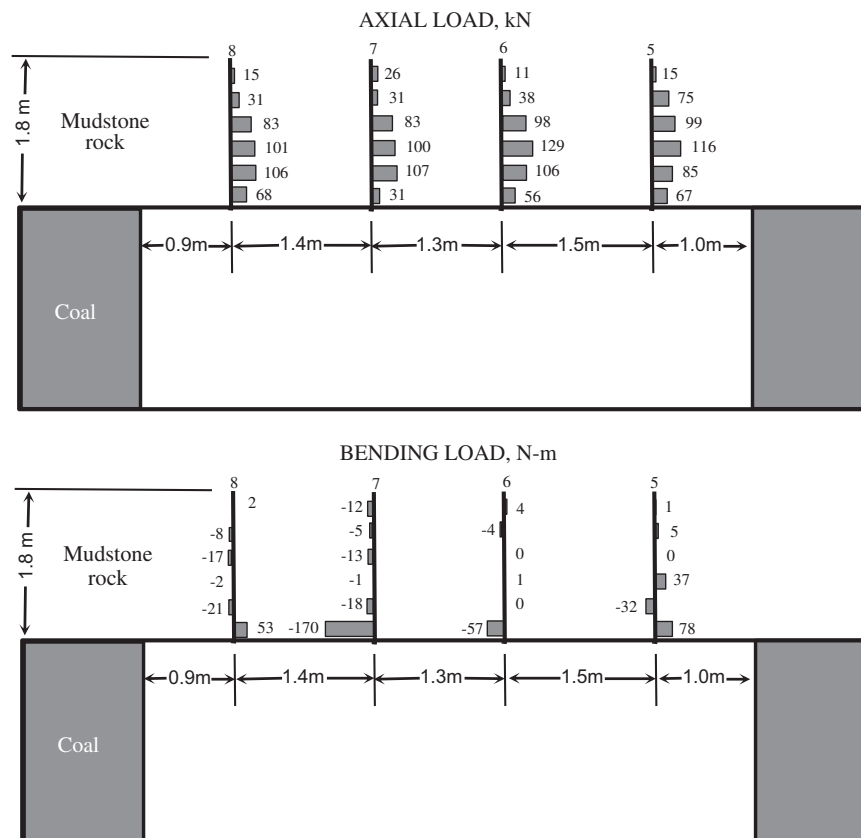


Fig. 2. Experimental load profiles determined using strain gauges [15].

A new rock bolt instrumentation strategy has been implemented based on an array of sub-micron resolution displacement sensors that measure the change in displacement or stretch (Δu^i) of the bolt. The end-points of the displacement sensor define a base-length (L), typically in the range 200–500 mm, which is more than an order of magnitude longer than for a resistive foil strain gauge.

The corresponding strain (e^i) for the i th gauge is determined from:

$$e^i = \Delta u^i / L \quad (3)$$

In order to control the unit cost the number of displacement sensors for each instrumented bolt was limited to six. The displacement sensors were arranged with three in each diametrically opposed slot (sides A and B of the rock bolt) in an end-to-end arrangement (Fig. 3). Sensors denoted by $i=1, 3$ and 5 are on side A and sensors 2, 4 and 6 are on side B. Two different configurations referred to as (i) stacked and (ii) staggered were investigated:

4.1. “Stacked” configuration

For the stacked configuration, the axial and bending strain were calculated according to Eqs. (4) and (5).

$$e_{\text{axial}}^i = (e^{2i-1} + e^{2i})/2 \quad (4)$$

$$F_{\text{axial}}^i = EAe_{\text{axial}}^i \quad (5)$$

with $I = 1$ to 3 corresponding to the three nodal points of load determination (Fig. 4a). Since, this configuration provides only three points of load determination located at the centre of each displacement sensor, the resolution of the load profile along the bolt is rather crude. The corresponding bending strain is

$$e_{\text{bending}}^i = (e^{2i-1} - e^{2i})/2 \quad (6)$$

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

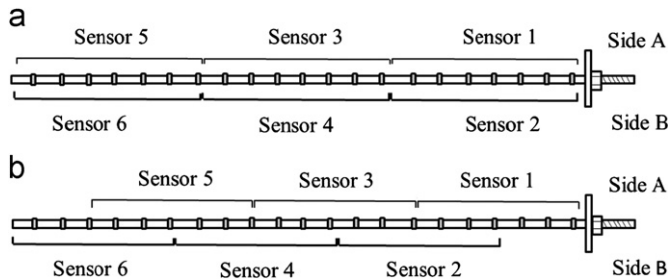


Fig. 3. The two strain gauge configurations used for the field trials. (a) Stacked configuration and (b) Staggered configuration.

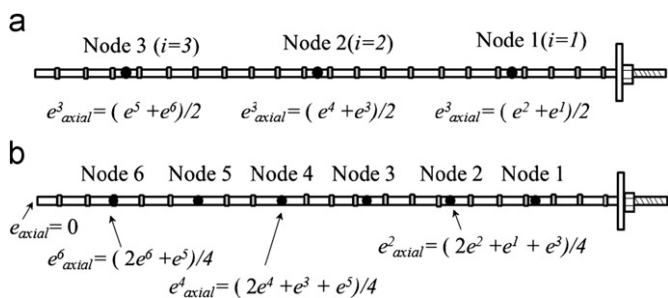


Fig. 4. The calculation of axial strain from the strain values measured by the six displacement sensors. (a) Stacked configuration and (b) Staggered configuration.

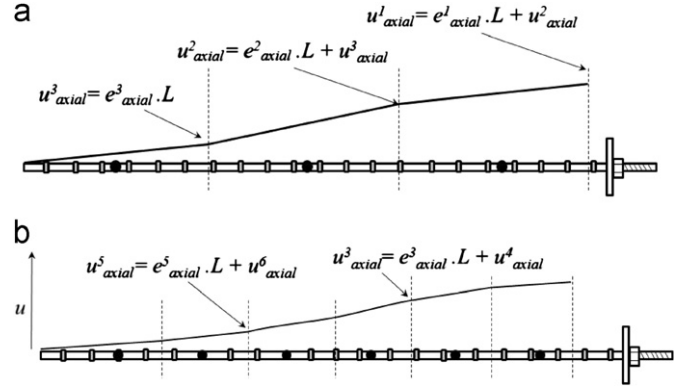


Fig. 5. The calculation scheme for axial displacement for stacked and staggered gauges. (a) Stacked configuration and (b) Staggered configuration.

be written as below, and represented in Fig. 5a:

$$\Delta u_{\text{axial}}^i = \sum_{n=3}^{n=i} (e_{\text{axial}}^n \times L) \quad (7)$$

It should be recognized that a distinction is made between the measured displacement (Δu^i) from each displacement sensor and the calculated axial displacement $\Delta u_{\text{axial}}^i$ which accounts for any bending.

4.2. “staggered” configuration

For the staggered configuration, the gauges on Side A or the bolt are offset from those on Side B by one half the base-length of the sensor ($L/2$). 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 [2]. The strains at the i th nodal point can be approximated as

$$e_{\text{axial}}^i = (e^{i-1} + 2e^i + e^{i+1})/4 \quad (8)$$

$$e_{\text{bending}}^i = (e^{i-1} - 2e^i + e^{i+1})/2 \quad (9)$$

where e^i is the strain measured by the displacement gauge centred at the i th nodal point and e^{i-1} and e^{i+1} are the two overlapping gauges on the opposite side of the bolt (Fig. 4b). At the ends of the bolt the following two relations are used. For node 1, the following relation is used:

$$e_{\text{axial}}^1 = (2e^1 + e^2)/3 \quad (10)$$

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

$$e_{\text{axial}}^6 = (2e^6 + e^5)/4 \quad (11)$$

Again, since the gauges are arranged 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:

$$\Delta u_{\text{axial}}^i = \sum_{n=6}^{n=i} (e_{\text{axial}}^n \times L/2) \quad (12)$$

relative to the toe end of the bolt which is assumed fixed ($\Delta u_{\text{axial}}^{\text{toe}} = 0$), as shown in Fig. 5b. In this manner an approximation of axial displacement, strain and load can be determined at six nodal points along the bolt (Figs. 4b and 5b). This provides additional axial resolution compared to the stacked configuration while sacrificing some degree of accuracy related to bending errors.

Table 1
Typical calibration test result.

Applied strain ($\mu\epsilon$)	Measured strain 1 ($\mu\epsilon$)	Measured strain 2 ($\mu\epsilon$)	Measured strain 3 ($\mu\epsilon$)	Measured strain 4 ($\mu\epsilon$)	Measured strain 5 ($\mu\epsilon$)	Measured strain 6 ($\mu\epsilon$)
0	0	0	0	–1	0	0
152.5	152	160	156	157	157	154
305	301	303	307	310	310	308
610	605	617	614	617	615	612
915	906	918	915	914	915	913
1220	1227	1233	1230	1226	1228	1226
1525	1531	1536	1532	1529	1530	1529
Calculated slope	1.0045	1.0069	1.0044	1.0013	1.0021	1.0021
Calculated offset	–3.8464	0.9573	0.8157	2.6536	2.5154	0.628

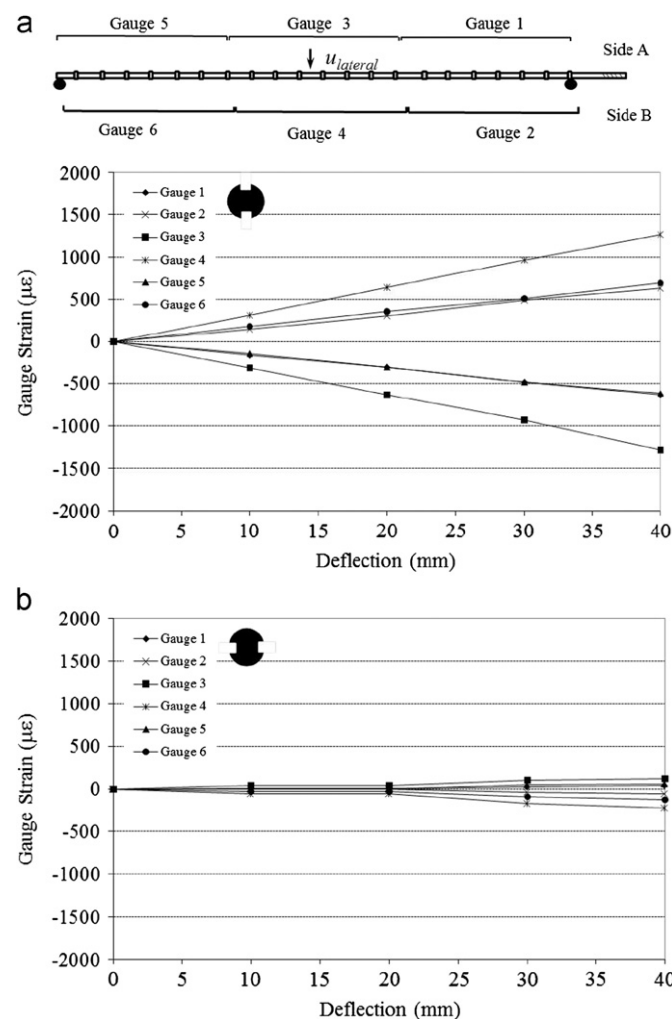


Fig. 6. Results of a simple 3-point bending test bending tests on the bolts. (a) with displacement sensors aligned with lateral deflection and (b) 90° to lateral deflection.

4.3. Laboratory validation and calibration

Instrumented rock-bolts need to be loaded to calibrate the relationship between load and deformation. In the current instrument the calibration coefficients are written into memory on the instrument's micro-controller. Results from a typical axial calibration experiment are shown in Table 1. Calibration data is typically collected on the third loading cycle. For the bolts used in the field trials presented below the stiffness of the bolt is 15.1 $\mu\epsilon$ /kN or 153 $\mu\epsilon$ /t.

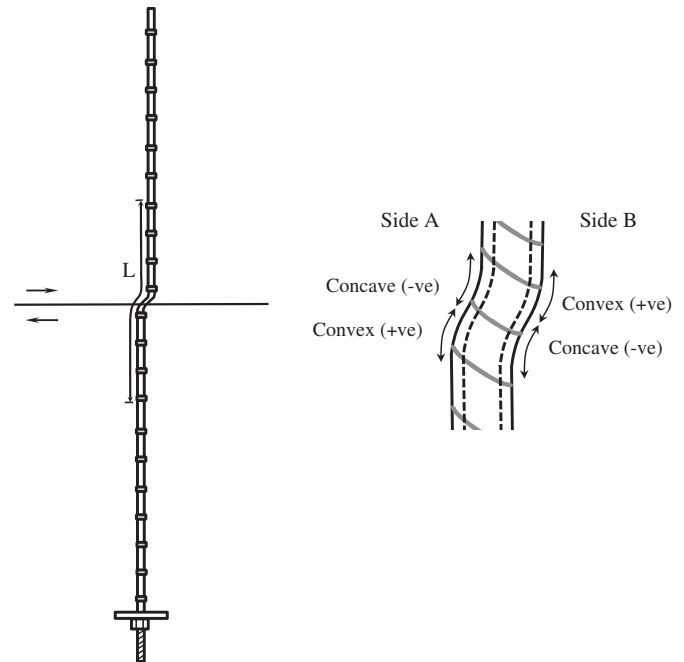


Fig. 7. Localized shearing on a bolt.

Table 2
Results of pull tests on different bolt types considered for the test programme.

Bolt type	Yield load	Ultimate load
#6 Grade 60 forged head	119.75 kN (minimum)	179.62 kN (minimum)
0.804 Grade 75 threaded	184.16 kN (actual)	257.31 kN (actual)
0.804 Grade 75 bar	183.25 kN (actual)	261.27 kN

Simple bending tests were conducted to establish the response of the displacement sensor array to bending. As would be expected from simple beam theory the gauges on the top of the rock-bolt display contraction and those on the bottom demonstrate extension (Fig. 6a). When the gauges are aligned at the neutral plane (Fig. 6b) they show minimal displacement. This situation referred to as “bending” should not be confused with that for localized shear when the zone of “shearing” is short compared to the base-length of the displacement sensor is shown in Fig. 7. On each side of the bolt the shear zone comprises a convex/concave pair which will exhibit corresponding zones of contraction and extension which should cancel out. Hence the long base-length sensors should be far less susceptible to perturbation related to localized shear.

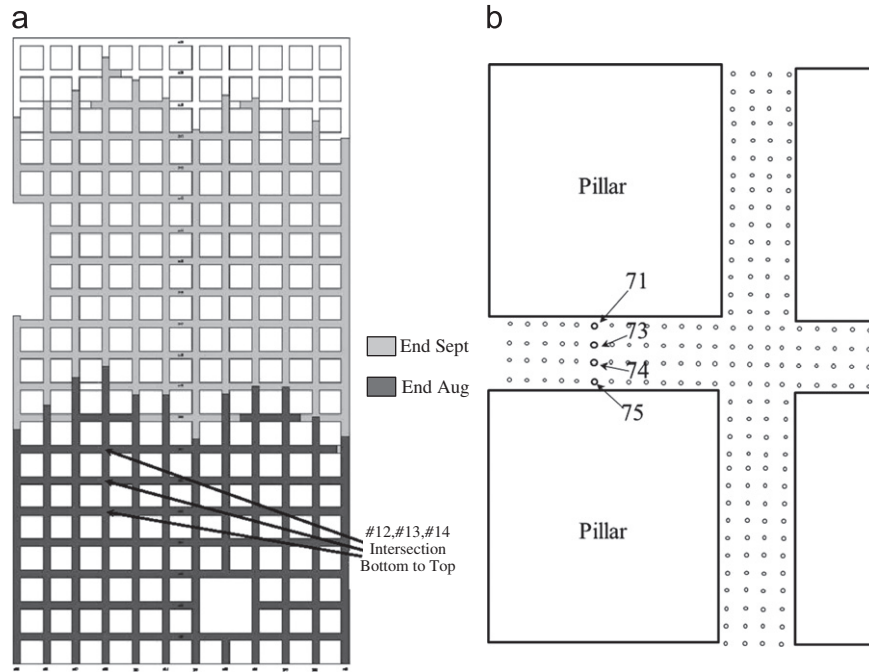


Fig. 8. Case Study 1 instrumented bolt site (a) The room and pillar mining sequence (b) the instrument bolt array across a heading at mid-pillar. The numbers refer to instrumented rock bolts.

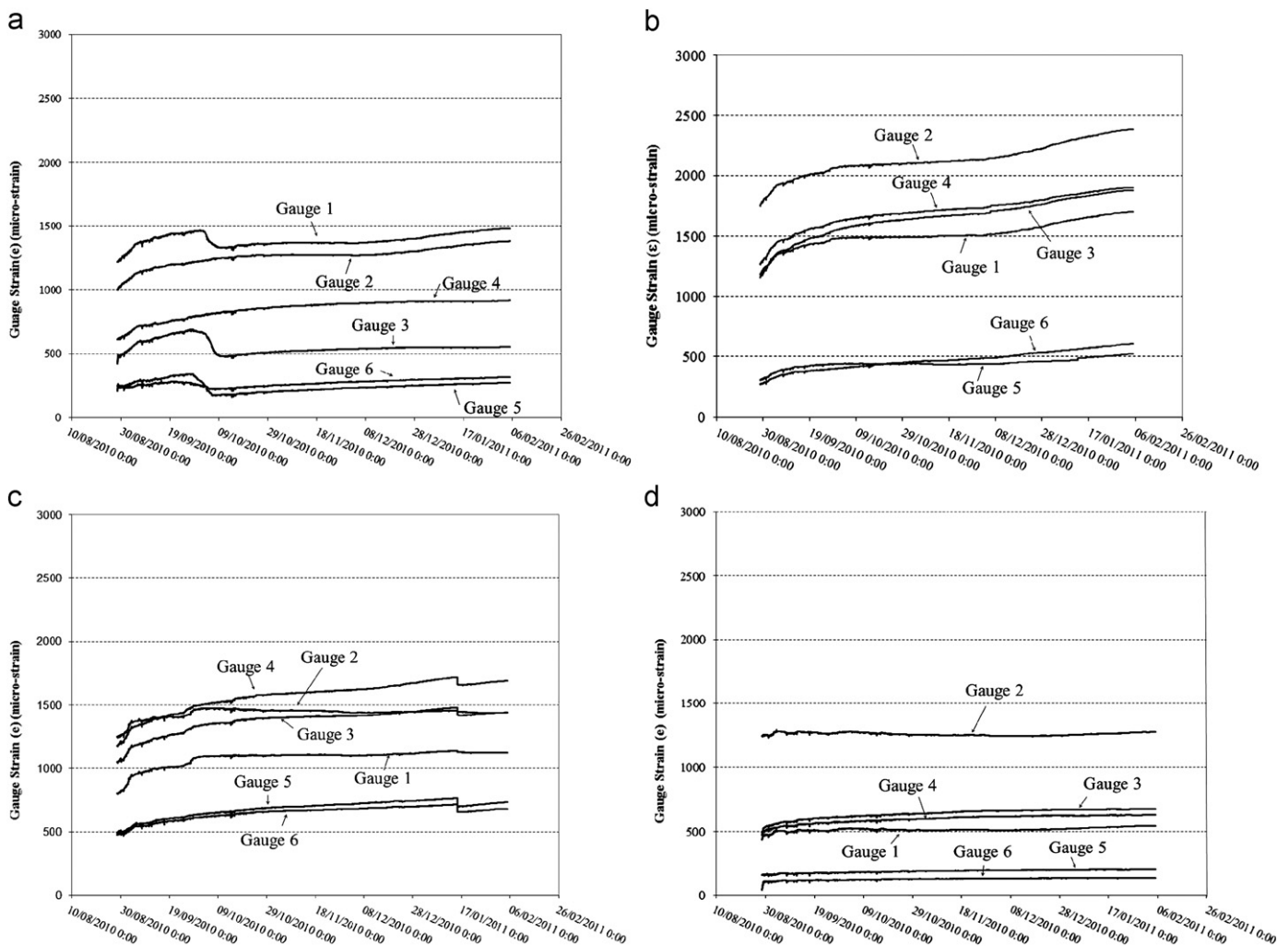


Fig. 9. Temporal strain gauge results a. Bolt 71, b. Bolt 73, c. Bolt 74, d. Bolt 75.

5. Field trials in two coal mines

Field trials were conducted in two room and pillar coal mines located in the Illinois Coal Basin. The results in this paper were a part of a larger research project funded by the National Institute of Occupational Safety and Health (NIOSH).

Both mines use #6 (19 mm diameter) Grade 60 fully grouted passive rebar for their primary support. In order to better capture the loads experienced by roof bolts, it was decided to install the instrumented bolts as primary support. Given that slotting for mounting displacement sensors would reduce the load carrying capacity of roof bolts, in order to achieve similar load capacity as

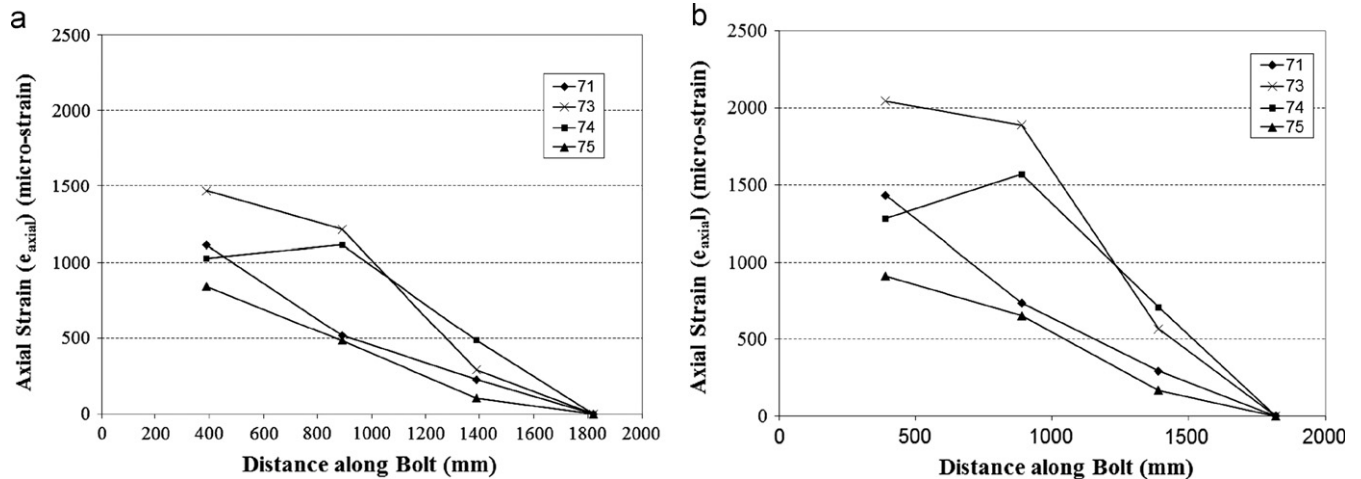


Fig. 10. The change in axial strain over the monitoring period (a) first logged reading 08/28/2010 (b) last reading 11/11/2010.

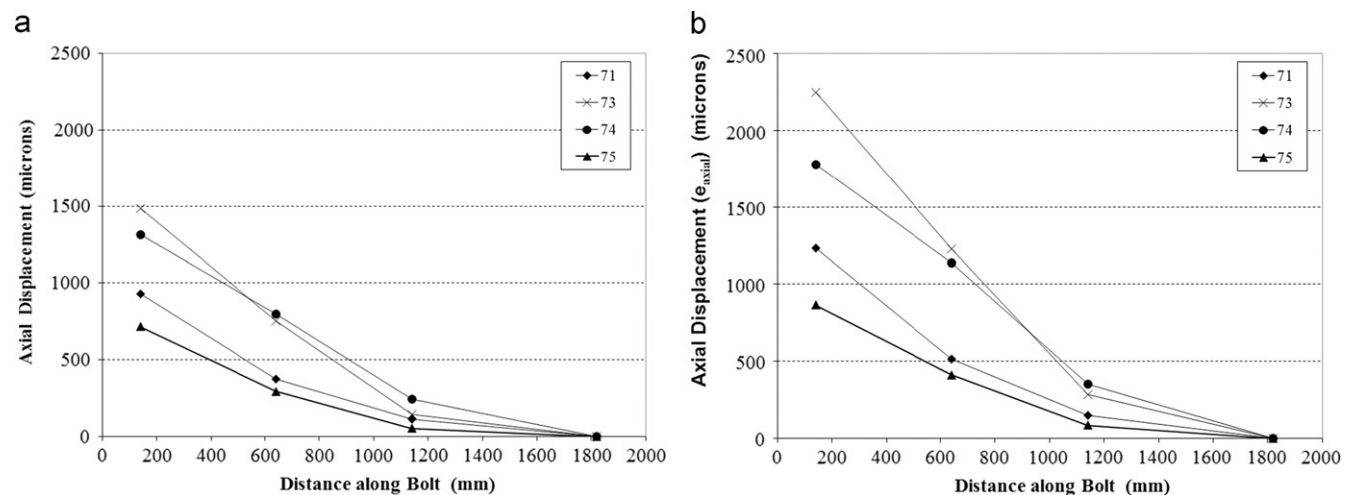


Fig. 11. The change in axial displacement over the monitoring period (a) first logged reading 08/28/2010 (b) last reading 11/11/2010.

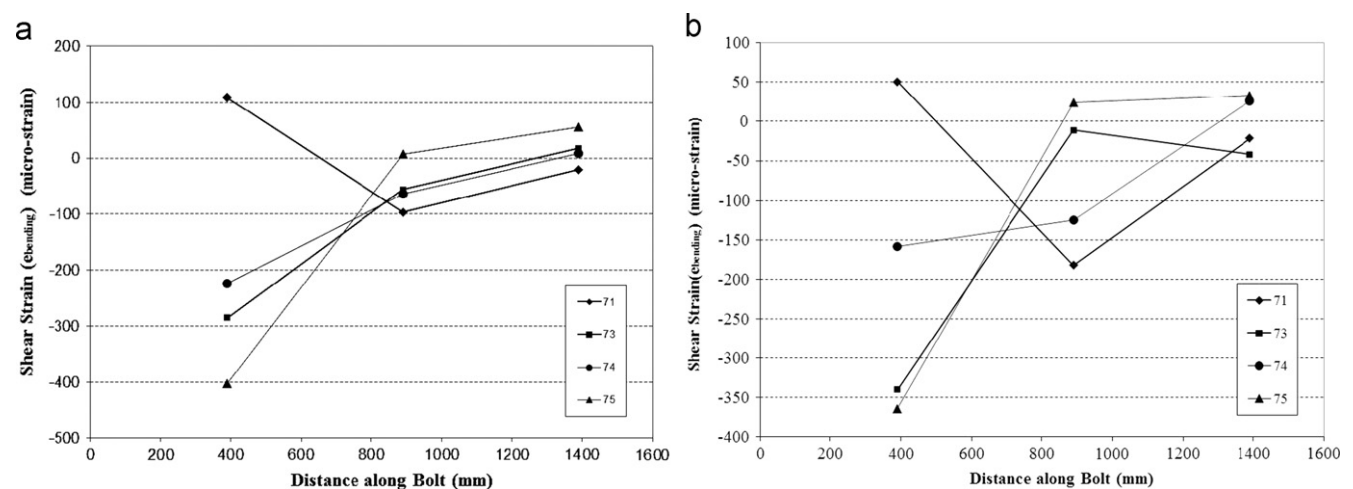


Fig. 12. The bending strain over the monitoring period (a) first logged reading 08/28/2010 (b) last reading 11/11/2010.

un-slotted #6 Grade 60 bar, initially it was proposed to use #6, Grade 75 rebar for the instrumented bolts. However, laboratory testing of slotted #6, Grade 75 bars produced slightly lower yield capacity than desired. Therefore to achieve a higher or at least comparable capacity to the pattern bolts, it was finally decided to use 20 mm diameter, Grade 75 bars for all instrumented bolts in these trials. Table 2 compares the tensile strengths at failure of these different steels. To minimize vendor related variability, all the instrumented roof bolts, plates and resin used were donated by the same manufacturer.

Each instrumented bolt was zeroed prior to installation in a 35 mm diameter drill hole that was drilled about 25 mm longer than the bolt. In order to ensure best practices were followed during bolt installations, a representative of a resin and bolt manufacturing firm was present during the instrumented bolt installations at both mines. The rock-bolts were installed as part of the routine rock-bolt installation cycle.

Unfortunately, the data-loggers could not be used until fresh air was established at both sites as they were not intrinsically safe as required by the Mine Safety and Health Administration (MSHA). Consequently, there was a delay between the time the instrumented bolts were installed as primary supports and when the first readings were taken (Figs. 9 and 10). Thereafter data collection from all the instrumented bolts was by automated data-logger at hourly intervals.

5.1. Case study 1: stacked displacement sensors

Four instrumented fully grouted passive rebar (FGPR) rock bolts were installed on 26th August 2010 at the mid pillar

location of a coal mine using a room-and-pillar mining geometry. Bolts #73 and #74 were installed at the mid-span of the heading and #71 and #75 were at the wall (Fig. 8b). Temporal plots for the four instruments are shown in Fig. 9, with the three displacement sensor pairs being arranged as follows: Gauges 1 and 2 are the stacked pair nearest the head, 3 and 4 in the centre of the bolt and 5 and 6 towards the toe. The length-wise distributions of axial strain (ϵ_{axial}), axial displacement (Δu_{axial}), and bending strain ($\epsilon_{bending}$) are plotted in Figs. 10–12, respectively. The axial stretch of the rock-bolts at the centre of the span (#73 and #74) is about twice as great as those nearer the wall (#71 and #75).

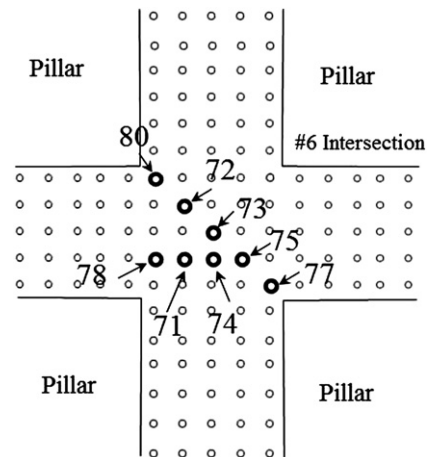


Fig. 15. The instrument layout for Case study 2.

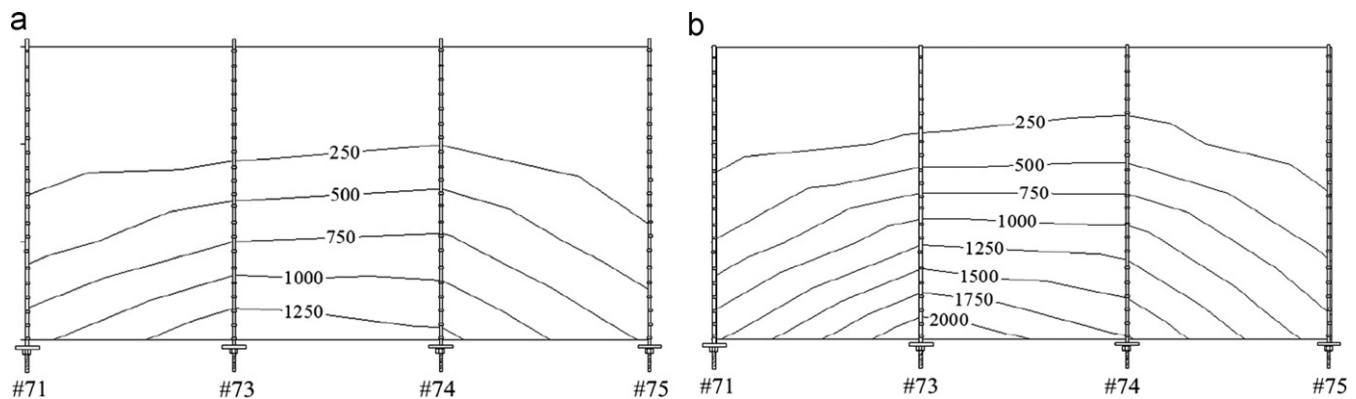


Fig. 13. A Contour diagram of the axial strain (u_{axial} in microns) in the roof of the heading (a) first logged reading 08/28/2011 (b) 11/11/2011.

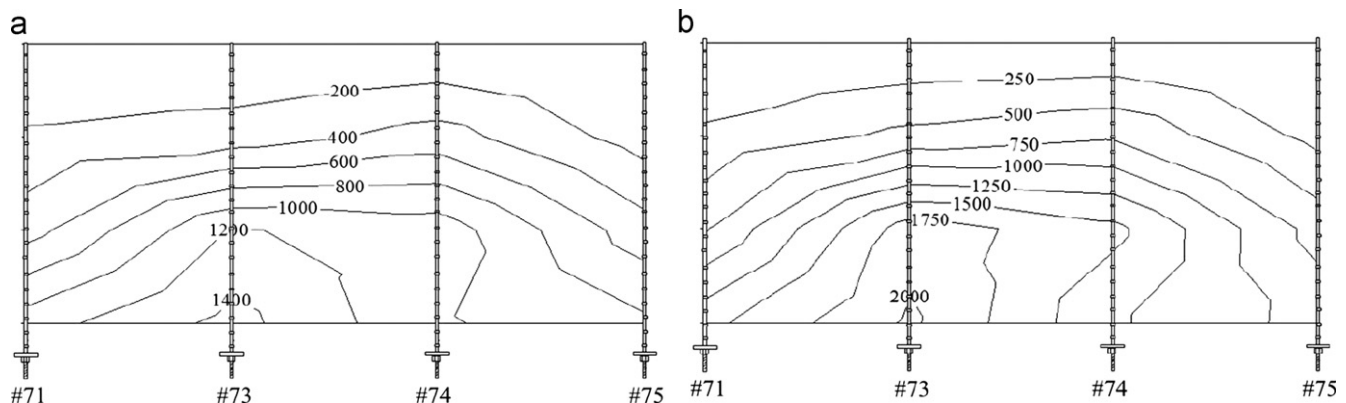


Fig. 14. A Contour diagram of the axial strain (ϵ_{axial} in microstrain) in the roof of the heading (a) first logged reading 08/28/2011 (b) 11/11/2011.

Fig. 13 represents a contour map of bolt stretch (in microns) based on the 4 instrumented bolts. An almost symmetrical “arch” pattern is defined, with a maximum stretch of 2 mm observed

at centre-span as mining progressed. The corresponding strain values in the bolt are contoured in Fig. 14, where 2000 microstrains correspond to 13.1 t of load. The following conclusions were drawn:

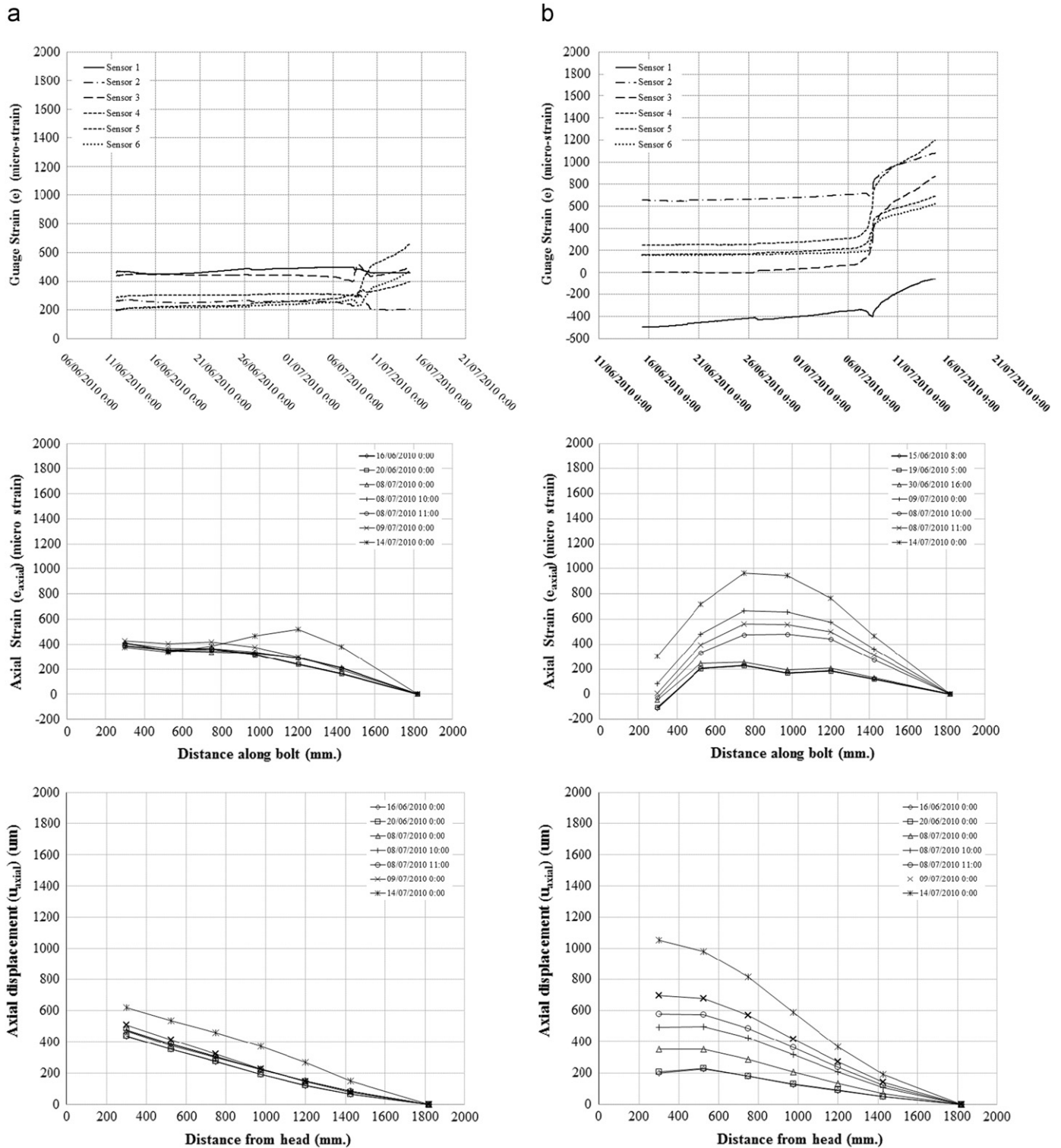


Fig. 16. (a): Gauge response (top), axial strain versus bolt length (middle) and axial displacement (stretch) versus bolt length (bottom) for FG bolt 100575078. (b): Gauge response (top), axial strain versus bolt length (middle) and axial displacement (stretch) versus bolt length (bottom) for FG bolt 100575071. (c): Gauge response (top), axial strain versus bolt length (middle) and axial displacement (stretch) versus bolt length (bottom) for FG bolt 100575074. (d): Gauge response (top), axial strain versus bolt length (middle) and axial displacement (stretch) versus bolt length (bottom) for FG bolt 100575075. (e): Gauge response (top), axial strain versus bolt length (middle) and axial displacement (stretch) versus bolt length (bottom) for FG bolt 100575077. (f): Gauge response (top), axial strain versus bolt length (middle) and axial displacement (stretch) versus bolt length (bottom) for FG bolt 100575073. (g): Gauge response (top), axial strain versus bolt length (middle) and axial displacement (stretch) versus bolt length (bottom) for FG bolt 100575072. (h): Gauge response (top), axial strain versus bolt length (middle) and axial displacement (stretch) versus bolt length (bottom) for FG bolt 100575080.

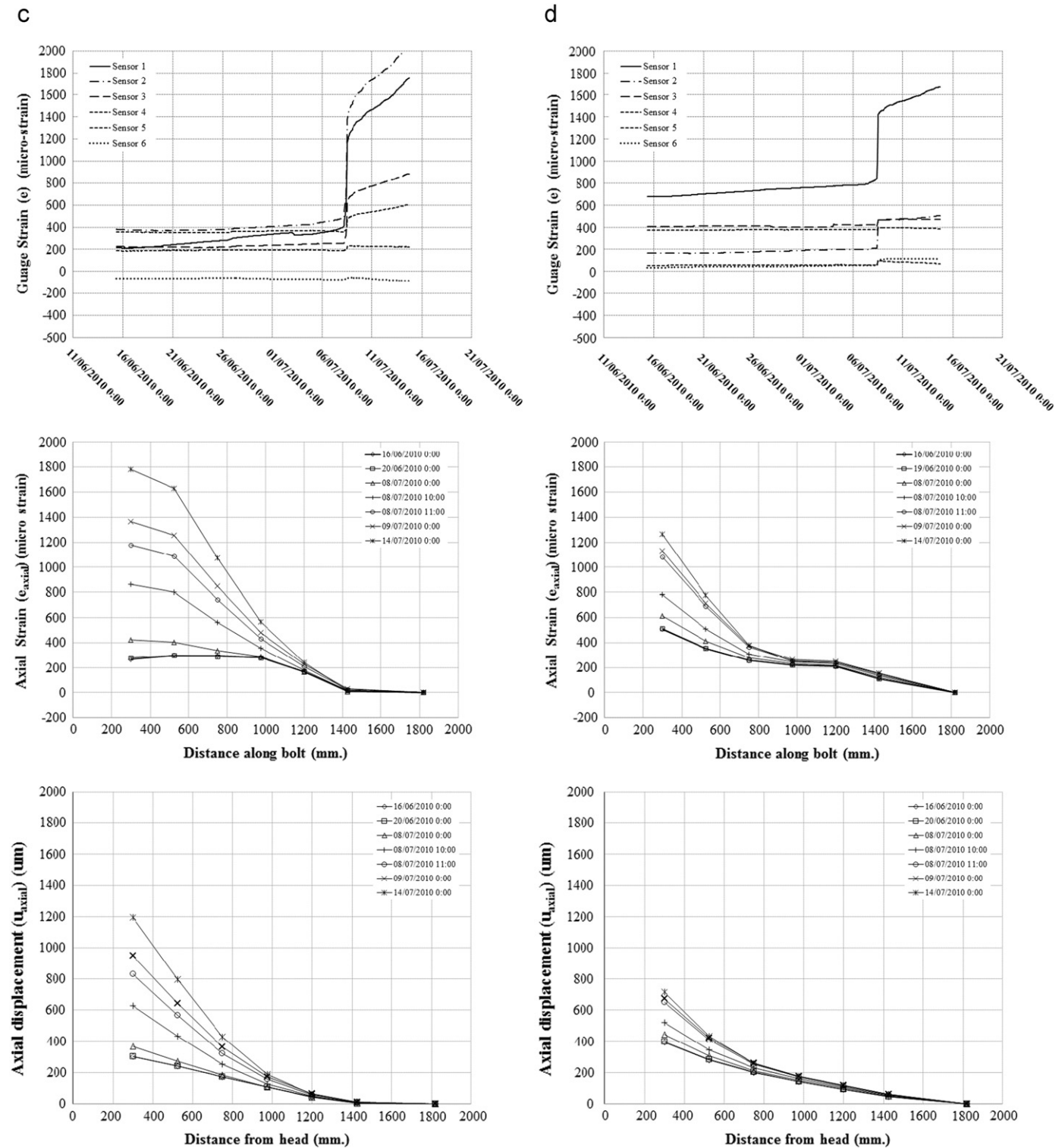


Fig. 16. Continued.

- (i). Higher bolt loads were recorded at the mid-span of the heading.
- (ii). For three of the four bolts the highest strain values were measured for the gauges nearest the head of the bolt.
- (iii). Near the head of the bolt, a pre-existing bending effect is indicated by the offset in magnitude for gauges 1 and 2. This is suspected to be related to bending close to the head of the bolt induced during the installation process.

- (iv). Over the monitoring period, the loading remained below the yield capacity of the bolts.

5.2. Case study 2: staggered displacement sensors

At this field site an array of instrumented rock bolts was installed in an intersection between two headings (Fig. 15).

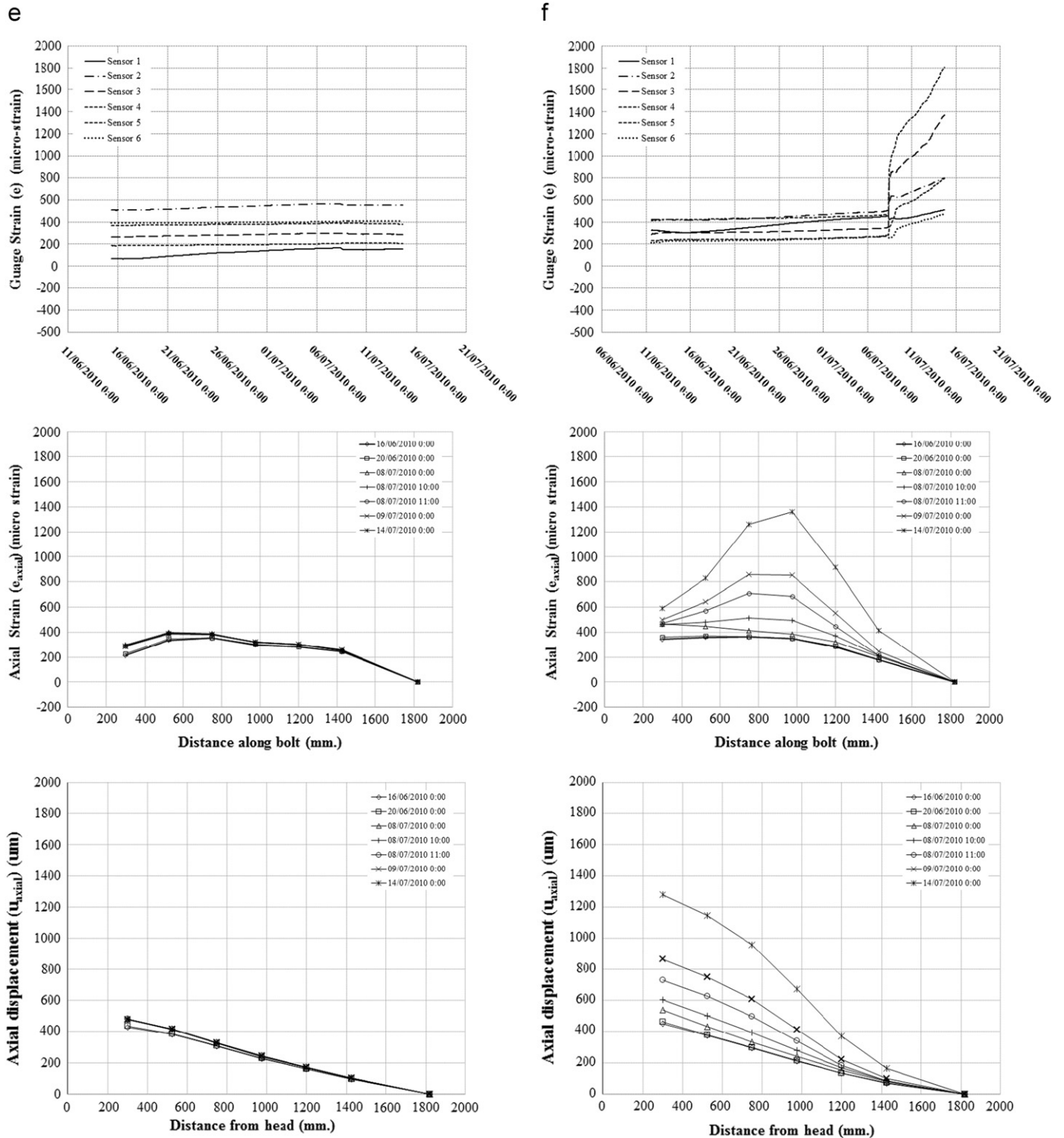


Fig. 16. Continued.

The displacement sensors for the instrumented bolts were arranged in a “staggered” configuration (Fig. 4b). As for case study 1, the bolts were fully grouted passive (FGPR). The bolts were installed on 5th June 2010 using standard operational procedures.

Initially all of the bolts exhibited low magnitudes of bolt loading. The typical loads are in the 2–3 t range. However it was noted that there were several small jumps in the bolt load with some anchors

increasing around 5–10 μm while other decreased. These events were synchronously detected by several bolts suggesting that they were related to definite changes in rock mass condition or altered mining geometry induced by asymmetrical progressive failure of the ribs.

However, on July 8th 2010 the bolts suddenly began to load up much more rapidly. Fig. 16a,b,c,d and f display a precipitous increase in bolt strain. Undoubtedly, all of these bolts are

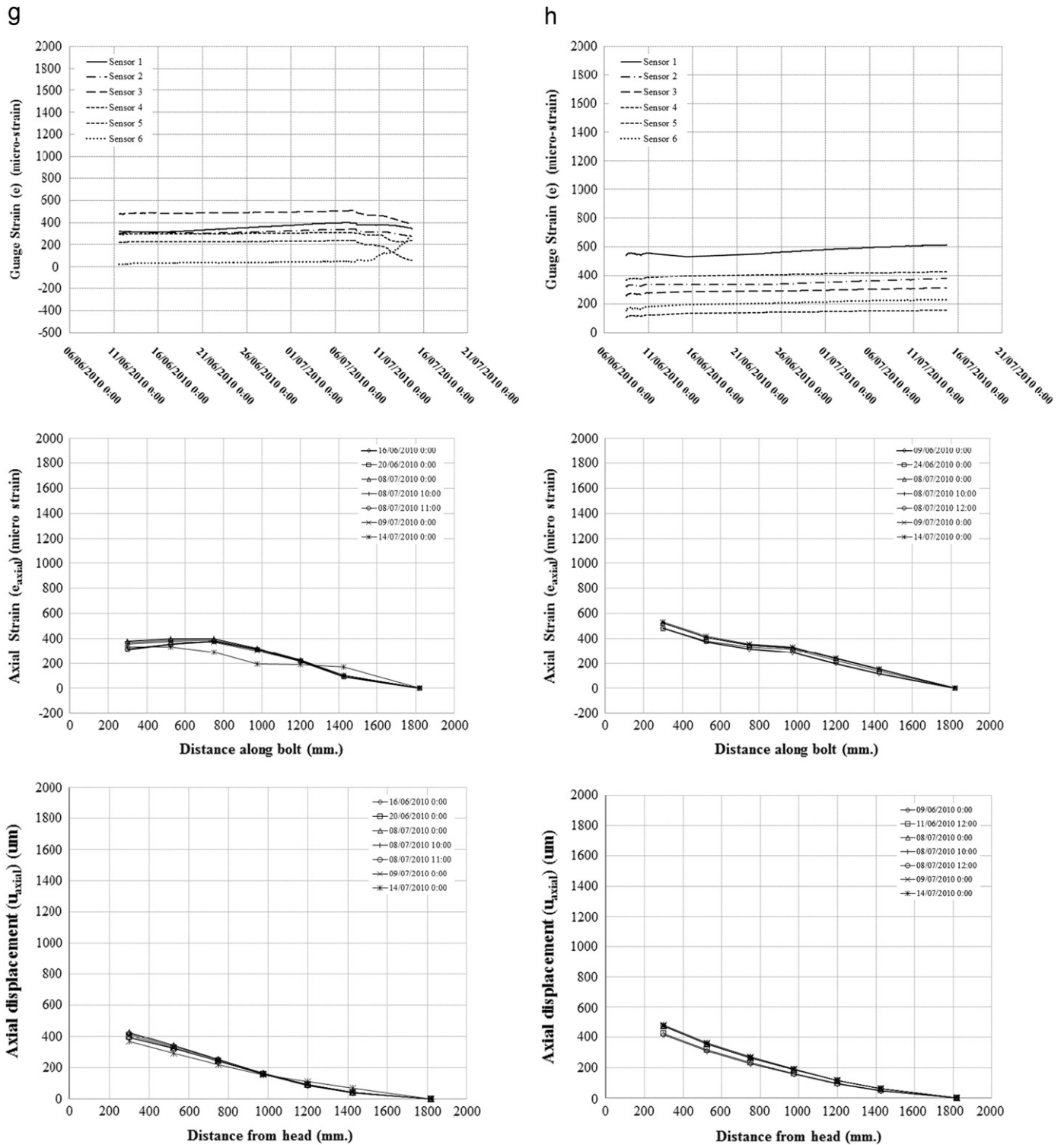


Fig. 16. Continued.

responding to the same rock mass deformation event occurring at this intersection, however very different responses were measured from bolt to bolt. If, as predicted by the numerical solutions for loading of a bolt in a discontinuous rock [9], the maximum load develops where the bolt intersects a discontinuity, then it is conceivable that markedly different load profiles may be measured within an array of bolts if that discontinuity is inclined. Such numerical models also display

sharp peaks in axial load around the discontinuity. As shown in the axial load profiles in Fig. 16a to d, even though there was a sharp jump in the axial loads of five intersection bolts when viewed temporally, the axial load distribution along bolt length did not display any sharp peaks that are expected at a discrete discontinuity.

To further understand the axial load trends in the intersection, the bolt strain distribution was contoured for a row

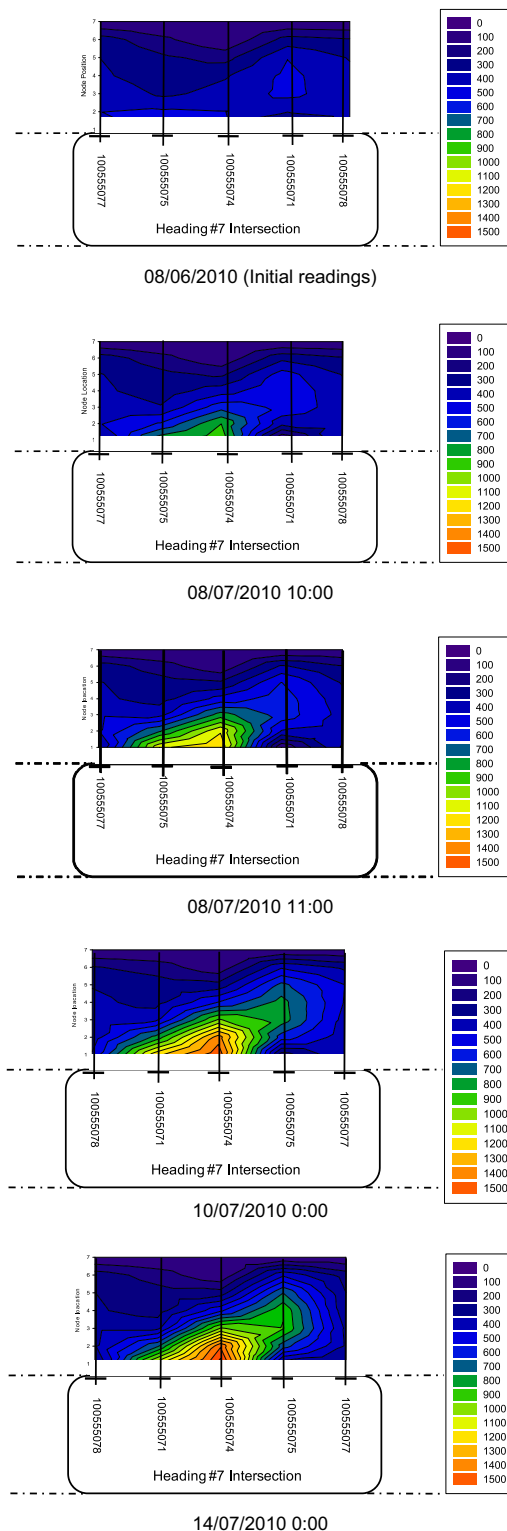


Fig. 17. A series of contoured plots of strain (in $\mu\epsilon$) the bolts for the array of five FG bolts in intersection #6. A very definite trend at 30° to the roof is displayed. The deformations become more “distributed” with deep into the roof. All views looking outby.

of five bolts (Fig. 17). The intent was to “image” the spatial distribution of bolt loading to identify if any strain localization occurred at the time of the sharp load increase noticed on July 8th. In effect, Fig. 17 represents a series of temporal ‘snapshots’

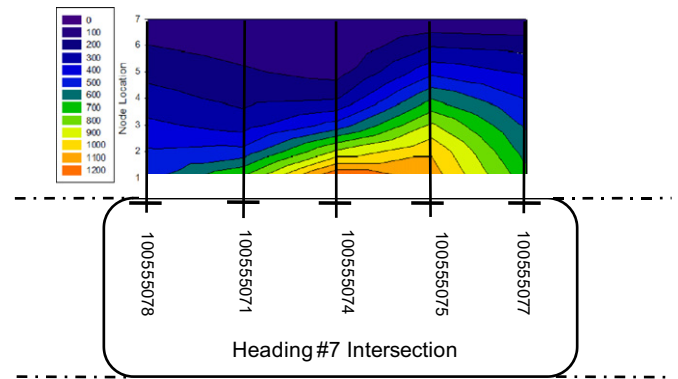


Fig. 18. Corresponding bolt stretch contours (in μm) for the array of FG bolts.

of the strain induced in the bolts, which clearly shows that the highest axial strain on the bolts is concentrated towards the centre of the intersection. Furthermore, the contour plots define a zone of intensified bolt loading inclined at 30° to the roof. Fig. 18 is a corresponding plot indicating stretch (in μm) for the bolt array. The maximum end-to-end stretch is 1.2 mm. No matter what the true mechanism is, the displacement contours in Fig. 18 delineate a wedge shaped block of ground that has become mobilized, the weight of which was suddenly applied to the bolts. A geotechnical sketch of one possible explanation is shown in Fig. 19. This wedge was not sufficiently heavy to overload the bolts by deadweight, and therefore the rock-bolts did their job and maintained stability with only minimal rock movement ($< 1.5 \text{ mm}$).

6. Conclusions

A novel rock-bolt instrumentation strategy using long base-length strain gauges has been introduced and based on two case studies the following conclusions can be drawn.

The long strain-gauge based roof bolt instrumentation appears to provide a satisfactory performance while covering almost the entire length of the roof bolt. Peak loads could however be underestimated.

The stretch along the bolt is accurately measured and contour patterns can indicate well known geotechnical characteristics such as (a) arch formation, and (b) wedge formation.

A staggered long gauge configuration provides an enhanced axial distribution profile which may more than compensate for errors introduced in magnitude. In other words from an engineering perspective it may be more informative to measure where the bolt is being strained, rather than by how much.

When subjected to continuum behaviour classical symmetrical rock bolt loading patterns were measured and the results were easily interpreted (Case Study 1). In contrast, when rock bolts become loaded by a discontinuum rock mass behaviour (Case Study 2), then radically different results can be reported by bolts in close proximity.

For the latter, it was essential to have a critical spatial density of instrumented bolts to capture the geomechanical behaviour in three dimensions. Thus in general it will be more informative to deploy an array of low cost instruments rather than fewer, more expensive units.

From a geomechanical perspective, the derivation of failure mechanisms is a major justification for the installation of instrumented rock bolts.

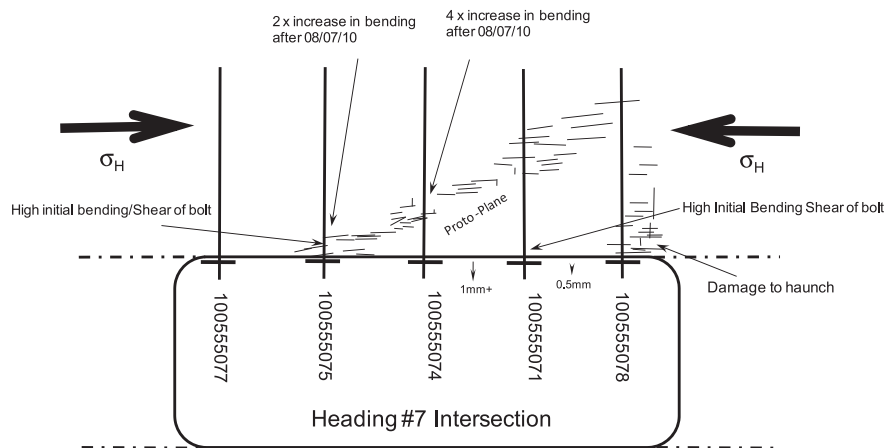


Fig. 19. A proposed mechanism for instability in the roof of intersection based on the data presented above.

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