The Application of Distributed Optical Strain Sensing to Measure Rock Bolt Deformation Subject to Bedding Shear

Thomas P. Roper, Brad Forbes, Jurij Karlovšek

***Abstract*—**Shear displacement along bedding defects is a well-recognised behaviour when tunnelling and mining in stratified rock. This deformation can affect the durability and integrity of installed rock bolts. In-situ monitoring of rock bolt deformation under bedding shear cannot be accurately derived from traditional strain gauge bolts as sensors are too large and spaced too far apart to accurately assess concentrated displacement along discrete defects. A possible solution to this is the use of fiber optic technologies developed for precision monitoring. Distributed Optic Sensor (DOS) embedded rock bolts were installed in a tunnel project with the aim of measuring the bolt deformation profile under significant shear displacements. This technology successfully measured the 3D strain distribution along the bolts when subjected to bedding shear and resolved the axial and lateral strain constituents in order to determine the deformational geometry of the bolts. The results are compared well with the current visual method for monitoring shear displacement using borescope holes, considering this method as suitable.

***Keywords*—**Distributed optical strain sensing, geotechnical monitoring, rock bolt stain measurement, bedding shear displacement.

I. Introduction

FAILURE of hard rock in underground mines and tunnels can be broadly categorised into two forms; brittle failure of intact rock and shear failure along existing discontinuities. When tunnelling at depth through Sydney Basin's Hawkesbury Sandstone, both are observed. This paper focuses on the observation and measurement of the latter. The majority of tunneling in Hawkesbury Sandstone is undertaken through bedded, relatively stiff sandstone with sparse jointing relative to excavation size. This combined with a strongly anisotropic in-situ stress field, [10], [11], and, as a result, shear displacement along bedding planes can occur at relatively shallow depths (Fig. 1). 40-50 m is the typical depth where the problem is expected to start occurring [6], [11], [13]. This is a challenge for designers and tunnel builders wishing to install primary, and permanent, rock bolts close to the excavation face while using conventional mining methods. The bolt of choice in modern tunnelling projects is the Double Corrosion Protection (DCP) Bolt. When compared to other underground reinforcement systems subjected to corrosive environments, the DCP bolt ranks as the most corrosion resistant [14]. However, shear displacement along rock defects following rock bolt installation can reduce the longevity, and at higher strains, the integrity of installed DCP bolts. This was demonstrated in [1] where rupture of the protective sheath surrounding the bolt was observed at 15 mm and 18 mm shear displacement across a 5-mm lab model clay discontinuity, between two mass sandstone blocks. This damage compromises the DCP bolt's ability to meet its 100+ year design life required by tunnel owners. This problem is, by practical measures, unavoidable when installing primary bolts as permanent support at depths below 40-50 m without the adoption of one of four possible solutions. They include:

1) Accept the risk of damaged DCP bolts and the potential for reduced longevity
2) Systematic replacement of all DCP Rock Bolts after ground deformations have stabilised.
3) Utilisation of alterative ground support solutions that do not use primary, permanent DCP rock bolts.
4) Monitoring and replacement of DCP bolts that intersect known sheared defects exceeding a nominated trigger value.

Solution #1 is dangerous and contrary to standard engineering ethics code. #2-3 do not make use of the principles of the observational method in tunneling, [9], and can be deemed wasteful. #4 is the current method used in tunnelling projects and uses strain gauge bolts or, more commonly, a visual inspection by use of a borescope camera (Fig. 8) to monitor the extent and scale of bedding shear displacement. Borescope cameras can monitor shear displacement by inspecting holes drilled adjacent with primary bolted support and the displacements infer damage on the bolts. This can be a subjective task, as it is typically a visual estimate based on scaling and introduces problems associated with perspective and camera position. As a result, this method only works with very clean and simple shears on discrete discontinuities and does not work over a fractured zone where displacement could occur over multiple discontinues over a short section. Borescope inspection is also an indirect assessment of bolt strain as it estimates the shear displacement of the rock mass rather than the bolt's response. However, a direct correlation is a reasonable assumption within stiff rock.

Strain gauge bolts (making use of electrical resistance sensors) can be used, however without knowledge of where the displacement is going to take place beforehand they lack spatial resolution to accurately determine the true shear

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displacement. Long base-length strain gauges cover the entire length of the bolt, yet, unless the load is uniform over each gauge length, the loads can be underestimated [4]. This is a problem as 'guillotining' of the bolt cannot be differentiated by more gentle bending between any two sensors or along a gauge length. Previous studies have concluded that shear displacements result in a very localized load response along the bolt, within 2.5-6.25 bolt diameters from the shear plane [2], [5], [8].

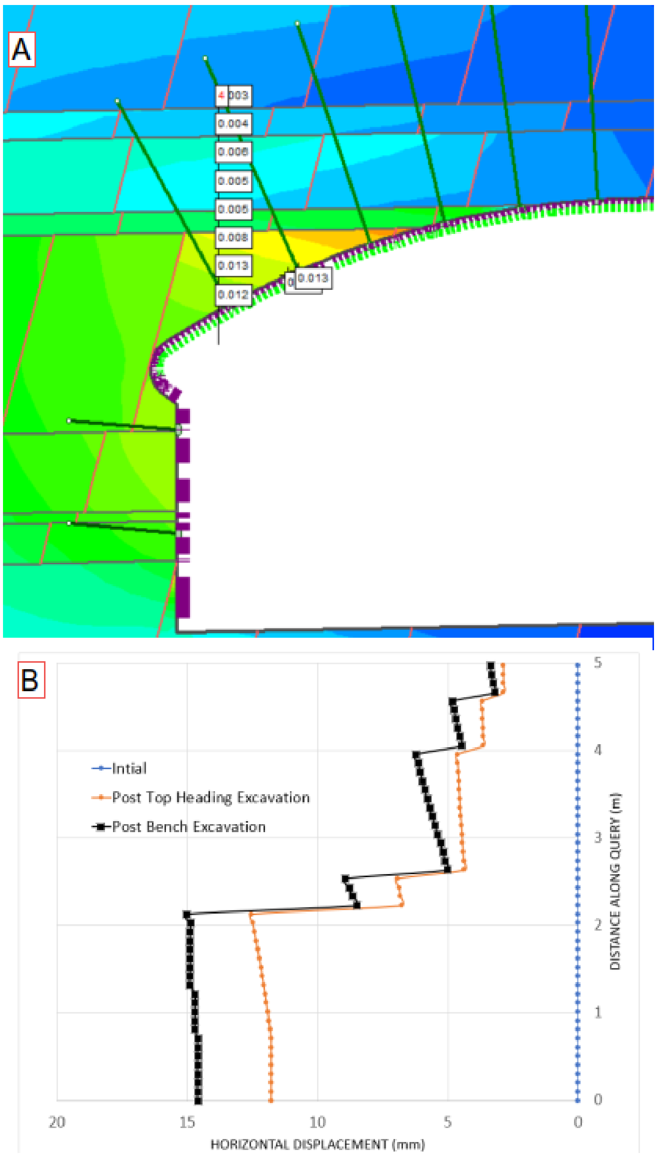


Fig. 1 (A) RS2 model of typical Hawkesbury Sandstone tunnel at 60 m depth after heading excavation and bench excavation (B) Horizontal displacement query above the tunnel shoulder showing displacement along the bedding planes (bedding shear) after heading and bench excavation

II. Fiber Optic Sensor (FOS) Instrumented Monitoring Bolts

Optical fibers have been used since the 1960s in communication lines, replacing copper wire due its lower attenuation. A weakness in fiber optic cabling, however, is that outside perturbations that produce physical changes to the cable (temperature and strain) disturb the propagating signal. Fiber optic sensing uses this property to determine strain. As described in [3] strain can be realised by determining a relationship between the physical change of the optical fiber and the spectral shift of the signal (amplitude, frequency and phase). A single fiber can act as both the transmission medium and the transducer. In recent years, commercially available analysers have been developed and are being applied in many different industries. This includes the use of Distributed optical strain sensing (DOS) to monitor ground support elements.

Fig. 2 Rupture of the DCP bolt plastic sheath after [1]

The optical fiber is essentially a glass; 9 µm fused silica core, surrounded by a cladding with a diameter of 125 µm. With the selected DOS measurement unit, the maximum strain that can be measured along an optical fiber is between 10,000-20,000 µε (1-2%). This is well beyond the yield of rebar rock bolts (arbitrarily 0.2%); however, this does present a limitation in many mining environments as strains can be far higher.

DCP rock bolts with milled grooves at 120° spacing (Fig. 3) were made for the project. An optical fiber is looped around the bolt in these grooves and encapsulated with a resin. Knowing the length of each groove, elastic bending theory can be used to separate the coaxial strain and lateral strain using the method in [4]. A 3D strain profile is the result. Using DOS, strain can be measured every 0.65 mm along the bolt, which is equivalent to thousands of strain gauges. The maximum strain can be determined regardless of the installation orientation. This monitoring solution provides the accuracy and resolution required for monitoring of discrete deformation along the rock bolts.

III. Product Trial

A product test was conducted on site to determine whether the embedded fiber optic cable and the connections could survive the bolt installation, tensioning and grouting process. This involved feeding the DOS instrumented rock bolt into the bolt hole using a bolting rig, tensioning the bolt (Using an expansion shell anchor) and grouting it using the bolter's grouting cup. By connecting the bolt to the DOS analyser after each stage in the bolting process, it was confirmed that the FOS instrumented bolt was robust enough to survive the tunnel rock bolt installation process. The FOS monitoring bolts were tough. During the second trial, the bolts were shotcreted over and thereafter carefully exposed with a jack-hammer. The bolts survived this making used of a protective steel cap.

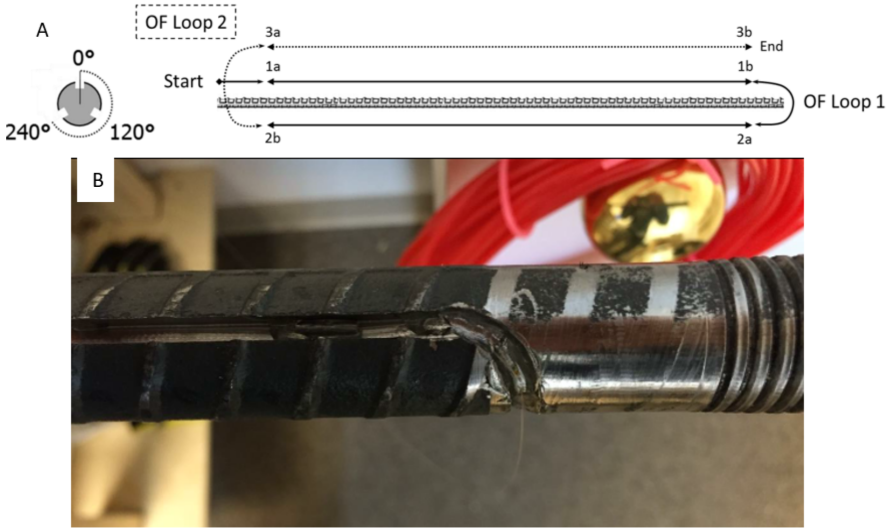


Fig. 3 A) Schematic representation of the optical sensor embedded in to the bolt in to the bolt at 120° spacing. Xa to Xb represents one sensing length of the bolt. These lengths are used allocate strain to the appropriate section of cable for accurate calculation B) photograph of the 2 mm x 2 mm milled groove in the bolt

One bolt was pull tested using a modified coupler (Fig. 4). The bolt was tensioned in 50 kN increments up to 250 kN. The intention of this pull test was to load up to failure of the bolt shank, however the custom coupler threads failed during the next increment. The results are shown in Fig. 5. This test proved that the bolt and equipment functioned well at loads well beyond the yield strength of the bolt. This test also provided some insight to the critical embedment length of a DCB bolt installed in a 45mm diameter hole within class SH-I/II Ashfield Shale [12]. The critical embedment depth is the length of bond required to mobilise the full tensile strength of the bolt [7]. At each increment of load, there was a strain response up to, but not beyond, the first 1.05 m of the bolt as shown in Fig. 5 (i.e. the bolt and bond length beyond this point did not resist any of the end load). A comparable trend, however with a more exponential decay in strain, was observed in [15]. This is a display of how the DOS instrumented bolts can be used to aid a tunnel designer in understanding the load, strain and system stiffness distribution in rock bolts and when the failure mode changed from bond strength to rebar strength.

IV. Monitoring Program

Following the success of the product trial, a working bolt trial was undertaken. The area chosen to install the bolts was based on where bedding shear was deemed likely to occur.

The tunnel area was comprised of competent class SS-I/II Hawkesbury Sandstone [12], with a bedding spacing 1-1.5 present in the tunnel shoulders and crown, no major geological structure, low frequency of joints, a multi-stage excavation, 75 m overburden (equivalent to tangential stress ~12 MPa in a simple linear model).

Four 4.0 m long x 22 mm diameter DCP instrumented bolts were installed across the tunnel profile, as shown in Fig. 6. A borescope hole was also drilled approximately 1.0m offset from each bolt (Normal to the profile in Fig. 6). Logging of the borescope holes allowed for a determination of the geological profile within very close vicinity of the bolt and importantly identification of where the bedding defects were. In the event of ground movement/bedding shear, the visual shearing could be compared to the bolt strain measurements.

Installed supplementary to the tunnel ground support, the bolts were monitored at different stages of excavation. After top heading excavation, strains of 3000-4000με were measured along some discrete locations on the bolts, in each case there was a component of lateral and axial strain. As expected, the strain locations occurred along discrete, concentrated sections of the bolt that correlated with observed bedding defects in the borescope holes (Fig. 6). Some bolts were in yield locally around each of the 2-3 the bedding partings they intersected. After inspecting the adjacent borescope holes, there was a very small indication of shear damage along beds, typically too small to measure. It is thought that this displacement is on the lower limit what can be identified visually through borescope holes.

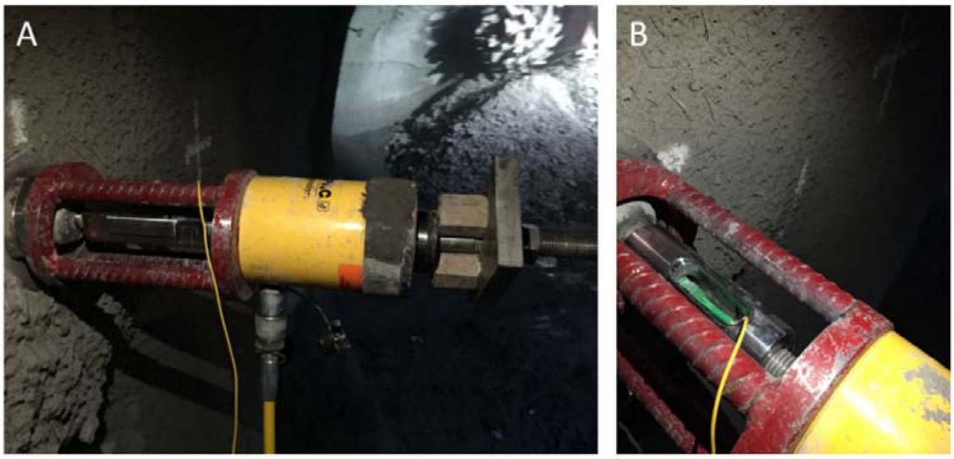
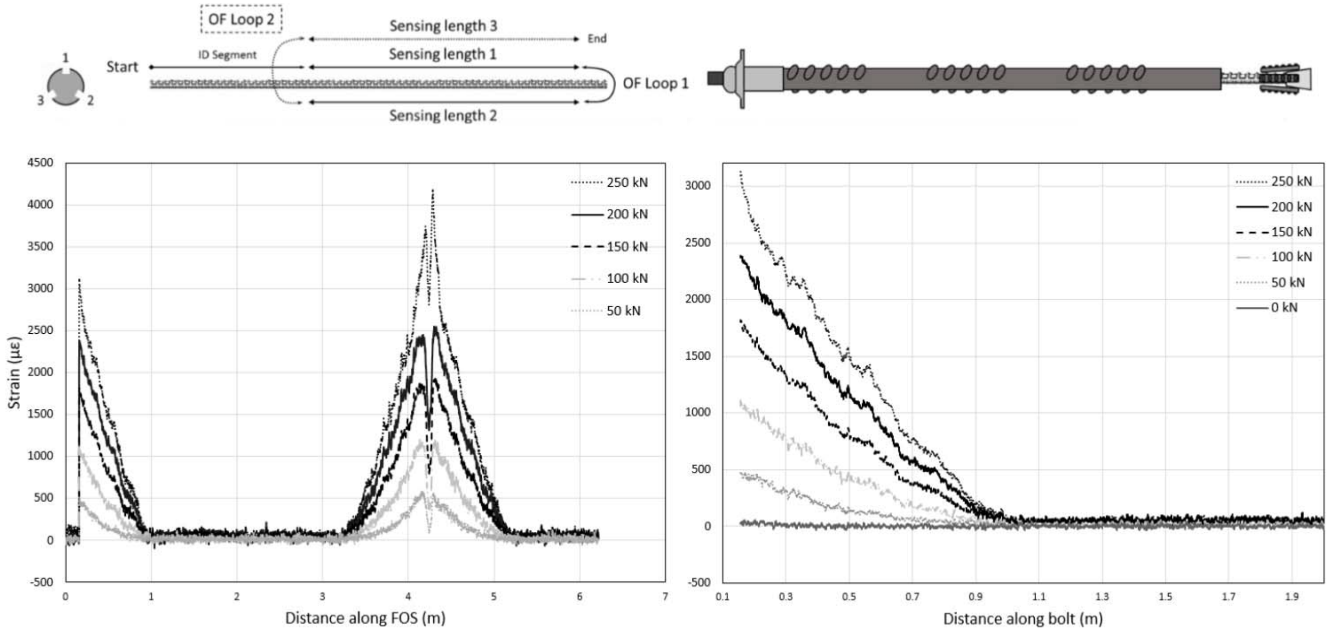


Fig. 4 Pull test apparatus set-up with modified coupler to allow the sensor connection with the analyser



(a) (b)

Fig. 5 Result of a pull test on 2m FOS instrumented monitoring bolt; (a) the fiber length strain. (b) is the equivalent strain along the bolt profile. Schematic of the of the looped fiber and DCP bolt above the graphs as a visual aid

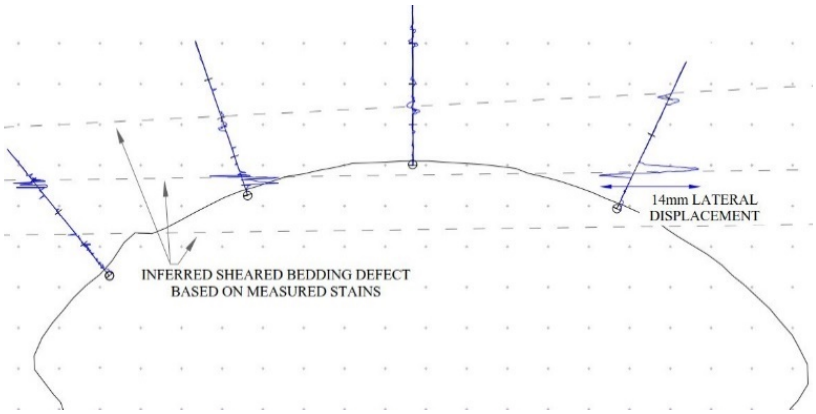


Fig. 6 Lateral strain of the FOS bolts superimposed on the excavation of the trial area; Sheared bedding planes could be identified across the tunnel profile and crown following the displacement event. Strain exaggerated by 170

During the excavation of an adjacent cross-passage a ground displacement 'event' occurred. Using the local industry-standard method of monitoring (and inferring) shear damage on bolts, through the borescope holes, bedding shear displacement of ~14mm was recorded (Fig. 8) along a very 'clean' and simple bedding shear plane. Several geologists and engineers independently scaled the same borescope hole and all estimated this value with +/-2 mm. The adjacent DOS monitoring bolt measured similar lateral displacement as shown in Fig. 7 of 14 mm. The bolt's 'S' shaped reaction occurred over a-290 mm section of the bolt, giving insight to the elongation geometry occurring around a bedding shear. Coaxial strain of 8720 με was measured corresponding to a load of 200-225 kN through the rebar at this bedding shear location. This triggered the reinstallation of rock bolts. The two bolts on the left-hand side of Fig. 6 were damage by equipment after the displacement event following shotcrete repairs and re-installation of bolts so unfortunately the post deformation event measurement could not be taken on these bolts. A visualisation of the behaviour measured in Figs. 6 and 7 can be seen in Fig. 9. In similar ground conditions, a temporary DCP rock bolt can be seen in cross section above the tunnel, after secondary excavation above a tunnel in creating a cavern and exposing the previously installed ground support. A DCP bolt is seen to intersect a bedding plane (between a sandstone bed and a siltstone bed) and dilated concentric fracturing. The bolt is resisting both the dilation of the concentric fracturing and (likely) shear along the bedding plane keeping the blocks secure. A 'real world example' of the behaviour models, such as that in Fig. 1, is represented. Having the opportunity to visually capture this behaviour is rare; however, the use of FOS monitoring bolts allows the capture and measurement of it.

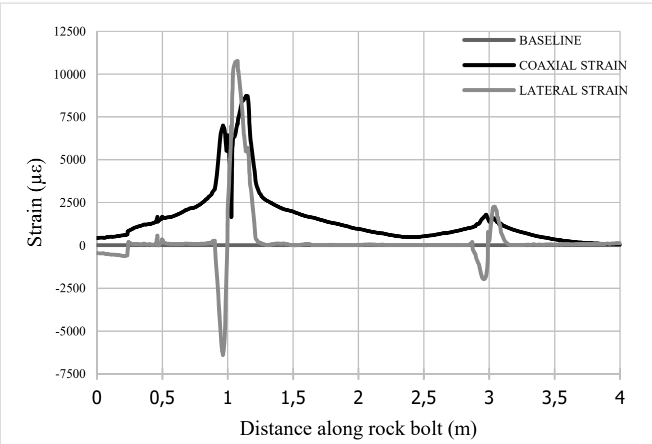


Fig. 7 Strain along the bolt length after a bedding shear event; The double peak of the axial response shows S-shaped bending over the sheared defect. Equivalent lateral displacement of 14 mm

The monitoring bolt installed in the centre of the tunnel performed as expected, with the axial component of the strain being far higher than the lateral strain. Fig. 10 shows the coaxial and lateral strain profile along the bolt. This behaviour is consistent with bolted beam analogies used to describe the behaviour of bedded rock where, bed dilation is expected at the centre of an excavation with increased shear occur towards the walls. Noting that because of an adjacent cross passage excavation and a multistage heading excavation, some asymmetry in the crown displacement was also expected.

V. Discussion

Some limitations and challenges were experienced during the trial. In what ended up being harsher ground/stress conditions, the monitoring bolts were subject strains that exceeded the sensing range of the selected DOS technology (i.e., 10,000-20,000ue). This does not correspond to failure of the optical fiber itself, but, nevertheless, is a limitation of DOS. The number of measurements that could be taken for each bolt was also severely limited by the selected lead wire assembly. This study required an elevated working platform to connect the DOS analyser directly with the instrumented bolts for each reading, which limited the frequency of measurements due to construction traffic and plant availability. In the future, it recommended to set up a more permanent lead wire assembly which brings the connect to an accessible level within the tunnel. Such an assembly has been discussed in [16].

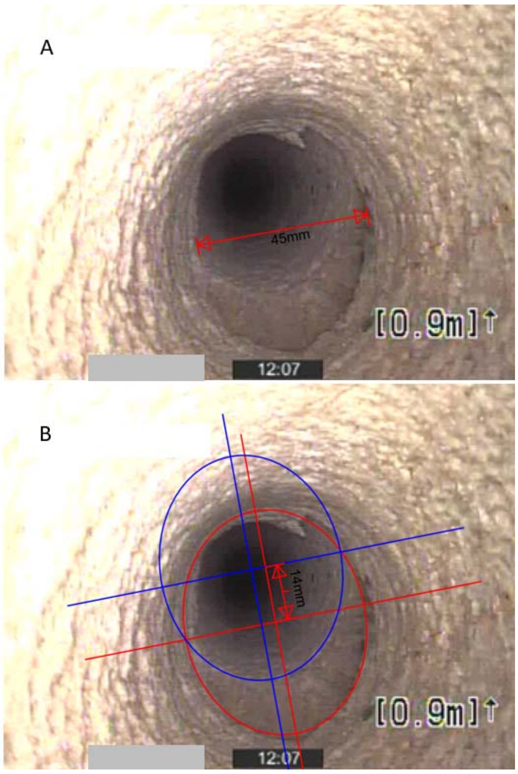


Fig. 8 Image captured in a borescope hole adjacent to the FOS bolt in Fig. 7; the 45-mm hole is off-set by ~14 mm following bedding shear

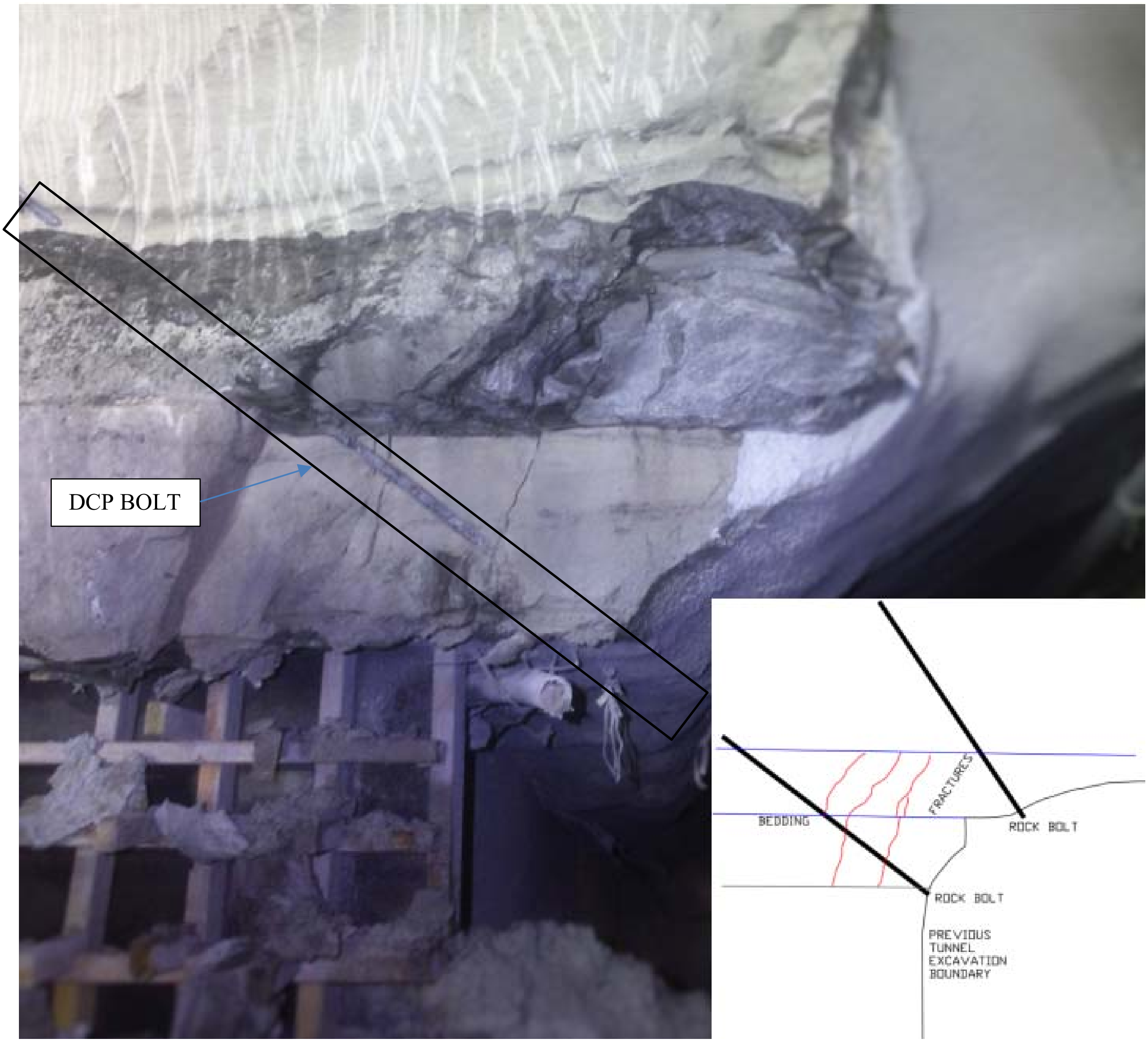


Fig. 9 A photo and explanatory sketch of an exposed DCP bolt above a tunnel shoulder following secondary excavation above the crown

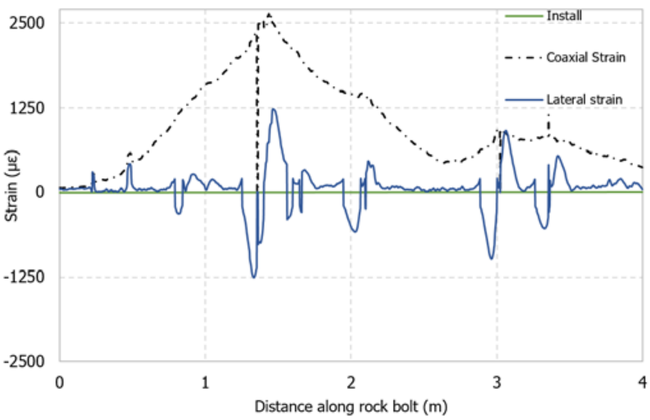


Fig. 10 Monitoring Bolt installed near the middle of the tunnel crown; largest strain component is axial, 2550 με, compared to lateral at 1250 με at a bedding parting

VI. Summary

The FOS instrumented DCP bolt experiment was a success. It provided unprecedented verification of bolt/bedding defect behaviour. It also has provided an opportunity for forensic back analysis and verification of analytical bolt and rock beam analogy models at a level of accuracy that has never been captured before. The project also demonstrated that with limited sample size, that borescope boreholes are an accurate means of shear displacement monitoring for displacement greater than 5 mm. Providing agreement that this is well below the point at which sheathing rupture occurs, 15 mm [1], the observational approach through the use of borescopes or DOS instrumented bolts, is a suitable means of monitoring damage to DCP bolts. Applying DOS to rock bolts has provided an unparalleled insight into the micro-scale complexities of rock bolt and rock mass interaction.

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REFERENCES

[1] R. Bertuzzi. 100- Year design life of rock bolts and shotcrete. Ground Support in Mining and underground construction- Villaescusa and Potvin (eds) 2004 246-249

[2] A. M Ferrero.. The shear strength of reinforced rock joints. International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts, 1995. 32: 595–605.

[3] B. Forbes, N, Vlachopoulos and A. J. Hyett. The application of distributed optical strain sensing to measure the strain distribution of ground support members. 2018. FACETS. 3: 195–226. DOI: 10.1139/facets-2017-0093

[4] B. Forbes, N, Vlachopoulos, A. J. Hyett and M. S. Diederichs. A new optical sensing technique for monitoring shear of rock bolts. Tunnelling and Underground Space Technology, 2017. 66: 34–46. DOI: 10.1016/j.tust.2017.03.007

[5] G. Grasselli. 3D behaviour of bolted rock joints: experimental and numerical study Int. J. Rock Mech. Min. Sci. 2005. 42, 13–24.

[6] U. Hesterman, L-P. Calbrix, C Bodner. NorthConnex – A New Perspective for Tunnelling in Sydney. 16th Australasian Tunnelling conference 2017.

[7] CC. Li, G. Kristjansson and A.H. Høien. Critical embedment length and bond strength of fully encapsulated rebar rockbolts. Tunneling and Underground Space Technology 2016;59:16e23.

[8] E. McHugh and S. Signer. Roof bolt response to shear stress: laboratory analysis. In Proceedings of the 18th International Conference on Ground Control in Mining, Morgantown, West Virginia. 1999; pp. 232–238.

[9] D. Nicholson, C-M. Tse and E. I. C. Penny. CIRIA Report 185. The Observational Method in ground engineering: principles and applications. 1999.

[10] L. B. McQueen. In situ rock stress and its effect in tunnels and deep excavations in Sydney. Australian Geomechanics, 2004; 39(3), pp.43-57

[11] D. Oliveira and M.S. Diederichs. Tunnel support for stress induced failures in Hawkesbury Sandstone. Tunnelling and Underground Space Technology. 2007; 64, pp.10-23.

[12] P.J.N. Pells, G. Mostyn and B.F. Walker. Foundations on sandstone and shale in the Sydney region. Australian Geomechanics; 1998; Vol 33(3):17–29.

[13] D. Tepavac, P. Mok, D. Oliveria, H. Asche, Y. Sun and S. Simmonds. High In-Situ Stress and its Effects in Tunnel Design: An Update Based on Recent Project Experience from WestConnex M4 East & New M5 Tunnel, Sydney Australia. 16th Australasian Tunnelling conference 2017, 375-386

[14] E. Villaescusa, E. Hassel and R. A. Thompson. Corrosion of rock reinforcement in underground excavations. Minerals and Energy research institute of Western Australia. Report No. 263. 2007.

[15] N. Vlachopoulos, D. Cruz and B. Forbes. Utilizing a novel fiber optic technology to capture the axial responses of fully grouted rock bolts. Journal of Rock Mechanics and Geotechnical Engineering 10 (2018) 222-235

[16] B. Forbes, N. Vlachopoulos, M.S. Diederichs, A. J. Hyett. 2018. Spile support performance monitored in a shallow urban tunnel using distributed optical strain sensing, in: 52nd U.S. Rock Mechanics/ Geomechanics Symposium. Seattle, Washington, p. 8.