HIGH SPATIAL RESOLUTION MEASUREMENT OF TENDON REINFORCEMENT IN UNDERGROUND CONSTRUCTION WORKS

by

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Abstract

Distributed optical fiber strain sensing is investigated as a technology to enhance the assessment of tendon reinforcement elements that are commonly used in mines and tunnelling projects. Central to this research effort is the application of a particular high spatial resolution Rayleigh optical frequency domain reflectometer that is able to distinguish strain along a standard optical fiber at spatial increments as low as 0.65 millimeters. In applying this technology to reinforcement elements, the objective is to measure a nearly continuous strain distribution, such that complex and non-uniform in-situ reinforcement responses that are induced by ground deformations can be characterized and quantified.

This has been approached from an intrinsic sensing perspective, whereby several instrumenting procedures have been developed to directly couple fiber optic strain sensors with various reinforcement elements. Notably, a delta-shaped sensor arrangement and accompanying analysis have been developed in order to calculate the coaxial strain scalar and bending moment vector mobilized along a reinforcement element. Significantly, this allows the maximum strain distribution along a reinforcement element to be measured without prior knowledge of the location(s) or orientation(s) of strain inducing features.

Through a series of laboratory experiments and in-situ studies it was demonstrated that load experienced by a reinforcement element can be measured as a function of distance from the load inducing source. This critically improves common pull test assessment of reinforcement because bond stress and load development length can be quantified and related to bond strength models. Furthermore, several in-situ reinforcement mechanisms mobilized by excavation advancement were measured. These included stress redistributions facilitated by umbrella arch elements in the form of distributed bending moments as well as localized dowel reinforcement across shearing discontinuities. The latter was found to concentrate strain within several centimeters of an active discontinuity, necessitating the high spatial resolution measurements.

In view of use of this technology by practitioners, it has been established that the sensor arrangement can be installed in conformance with standard reinforcement handling and mechanized
installation procedures. The end benefit is the developed sensing technique can be used by ground control engineers to more confidently justify alterations and optimizations to support design.
Co-Authorship

The thesis High Spatial Resolution Measurement of Tendon Reinforcement in Underground Construction Works is the product of research conducted by the author, Bradley James Forbes. The manuscripts included in this thesis have several co-authors, specifically: Dr. Nicholas Vlachopoulos (Chapter 2-7), Dr. Mark Diederichs (Chapters 3-7), Dr. Andrew Hyett (Chapters 2,3,7), Mr. Jonathan Aubertin (Chapter 5), and Mr. Thomas Roper (Chapter 6), and Mr. Allan Punkkinen (Chapter 7). While scientific and editorial feedback was provided by the listed co-authors, the written content of this thesis is solely that of the author. Complete references for the journal publications resulting from this research are listed in Chapter 8.
Acknowledgements

Towards the end of writing this thesis, a friend remarked that I must be excited to end such a solitary time in my life. This caught me a bit off guard. I have to concede that there are many early mornings and late nights that are spent constructing specimens in an empty lab, restless sleeps from running through mental checklists of equipment for a field study, and, of course, days, that turn into weeks and, in my case, that turned into months of isolated writing. But I never felt that this journey was travelled alone. In fact, it was the time spent working alone that always seemed to drive home the vast extent of support and companionship that were unfailingly accessible to me for academic, professional, or personal themes. I cannot confirm that this is the experience of every doctoral student, but it was for me. I owe my deepest gratitude to each person that made this possible.

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Table of Contents

Abstract ........................................................................................................................................ ii
Co-Authorship ................................................................................................................................. iv
Acknowledgements ............................................................................................................................ v
List of Figures .................................................................................................................................... xii
List of Tables .................................................................................................................................... xxvii
List of Abbreviations .......................................................................................................................... xxix
List of Symbols .................................................................................................................................. xxxi
Chapter 1 Introduction ...................................................................................................................... 1
  1.1 Problem Statement ..................................................................................................................... 1
  1.2 Research Objectives and Methodology ................................................................................... 5
  1.3 Thesis Structure ........................................................................................................................ 9
  1.4 References ................................................................................................................................ 13
Chapter 2 The application of distributed optical strain sensing to measure the strain distribution of ground support members ................................................................................................. 14
  2.1 Introduction ............................................................................................................................ 14
  2.2 Monitoring Ground Support: Fiber Optic Strain Sensing ....................................................... 16
    2.2.1 Fiber Bragg Gratings ........................................................................................................... 18
    2.2.2 Distributed Optical Strain Sensing ................................................................................... 21
      2.2.2.1 Brillouin Optical Time Domain Reflectometry/Analysis ............................................... 22
      2.2.2.2 Rayleigh Optical Frequency Domain Reflectometry ................................................... 25
      2.2.2.3 Distributed Fiber Bragg Gratings ................................................................................. 26
    2.2.3 Applicability to Ground Support ....................................................................................... 28
  2.3 Application of DOS to Ground Support Members ..................................................................... 29
    2.3.1 Rock Bolt Support ............................................................................................................. 31
    2.3.2 Umbrella Arch Support .................................................................................................... 34
    2.3.3 Cable Bolt Support ........................................................................................................... 36
  2.4 Laboratory Results .................................................................................................................... 38
    2.4.1 Rock Bolt Experiments ..................................................................................................... 39
      2.4.1.1 Coaxial Loading ........................................................................................................... 40
      2.4.1.2 Double Shear Plane Loading ...................................................................................... 43
    2.4.2 Umbrella Arch Experiments ............................................................................................. 47
    2.4.3 Cable Bolt Experiments ................................................................................................... 49
Chapter 6 Measuring the in-situ response of tunnel support using high spatial resolution optical fiber strain sensing

6.1 Introduction .............................................................................................................................................. 168
6.2 Fiber Optic Strain Sensor Instrumentation Procedure .............................................................................. 169
6.3 Sensor Alterations and Improvements for In-Situ Application ................................................................. 171
   6.3.1 Sensor Connection Improvements .................................................................................................. 173
   6.3.2 Sensor Termination Improvements .................................................................................................. 175
   6.3.3 Lead Wire Protection and Management .......................................................................................... 177
6.4 In-Situ Results ........................................................................................................................................ 178
   6.4.1 Coaxial Pull Testing ......................................................................................................................... 178
      6.4.1.1 Installation and Pull Test Procedure ............................................................................................. 180
      6.4.1.2 Installation and Pull Test Results ................................................................................................. 181
   6.4.2 Umbrella Arch Response to Tunnelling .......................................................................................... 187
      6.4.2.1 FOS Instrumented Spile Installation .......................................................................................... 188
      6.4.2.2 OF Lead Management ................................................................................................................ 192
      6.4.2.3 FOS Measurements ................................................................................................................... 192
      6.4.2.4 Mobilized Spile Strain Observations ......................................................................................... 195
   6.4.3 Measuring Crown Reinforcement Effect in Thickly Bedded Ground .............................................. 198
      6.4.3.1 Mobilized CT-Bolt Strain Observations ..................................................................................... 200
6.5 Discussion ................................................................................................................................................. 208
   6.5.1 Application of the FOS Technique .................................................................................................. 208
   6.5.2 Measuring the Mechanistic Behaviour of Reinforcement Elements .................................................... 209
   6.5.3 Measuring Ground Deformations Surrounding the Excavation ....................................................... 211
6.6 Summary .................................................................................................................................................. 211
Appendix A - Additional In-Situ Development and Application Material

A1 FOS Construction and Protection (Bar Elements) .......................................................... 290
A2 Initial Handling and Acceptance Testing ...................................................................... 292
A3 Example Periodic Monitoring Protocol Used In-Situ ..................................................... 298
A4 Additional In-Situ Pull Test Components ................................................................... 307
## List of Figures

**Figure 1-1:** Primary functions required of a ground support system. Reinforcement elements (such as rock bolts and cable bolts) internally reinforce and hold back fractured and damaged ground while support elements (such as steel mesh and shotcrete) retain damaged ground at the excavation periphery and provide interconnectivity between individual reinforcement elements (after Hutchinson and Diederichs, 1996). ................................................................. 2

**Figure 1-2:** Discontinuous and localized loading of reinforcement elements in-situ. Left – A fully grouted rebar rock bolt exposed during face advance of a cut-and-fill stope. Differential shear movement along transecting discontinuities resulting in localized bending of the bolt, deviating it from its original borehole trace (after Li, 2010). Right – Rebar element visible after a fall of ground in a highly stressed underground mine. Permanent deformation is visible along the element in the form of localized bending and coaxial stretch (image courtesy of Brad Simser, Glencore). ........................................................................... 4

**Figure 1-3:** Overview of the interrelated components defining the experimental framework of this thesis. 6

**Figure 2-1:** Example tunnel support scheme including: tendon support (rock bolts / cable bolts), umbrella arch support (forepoles / spiles), steelsets / girders, and shotcrete lining. ......................................................... 15

**Figure 2-2:** Schematic depiction of discrete sensing techniques along an example support element. a) and b): Local, discrete measurement points provided by short gauge length sensors (e.g. electrical resistive strain gauge). c) Averaged, discrete measurement zone(s) provided by longer gauge length sensors (e.g. linear-variable-displacement-transformer). ..................................................................................... 17

**Figure 2-3:** Example Bragg grating structure fused into the core of a single mode optical fiber. Light travelling through the Bragg grating will be partially reflected according to the Bragg wavelength ($\lambda_{\text{Bragg}}$) and will experience a shift ($\Delta\lambda_{\text{Bragg}}$) associated with local strain and temperature change (after FBGS 2017). ................................................................................................................................. 19

**Figure 2-4:** Schematic operation of Brillouin optical time domain analysis (after Zhang & Wu 2012). .... 24

**Figure 2-5:** Example optical network with a sensor arm added to a Mach-Zehnder interferometer to interrogate strain along an optical fiber (i.e. DUT) (Soller et al. 2005). ................................................................. 25

**Figure 2-6:** Typical optical fiber cable profile. Note the core and cladding assembly is roughly the diameter of human hair ........................................................................................................... 30

**Figure 2-7:** Diametrically opposed 2.5 by 2.5 millimeter machined grooves running lengthwise along a rebar specimen. Right – Schematic representation of the optical sensor. ......................................................... 32
Figure 2-8: Rebar tensile experiment. Left – Testing apparatus (MTS 810 loading frame). Right – Comparison of rebar strain measured using ROFDR and electrical resistive strain gauges. Strain gauge positions and measurements are indicated by square symbols. Note: Tensile strain is taken positive. 33

Figure 2-9: Forepole symmetric bending experiment. Left – Testing apparatus (MTS 324 loading frame). Right – Comparison of the forepole strain measured using the ROFDR technique, electrical resistive strain gauges, and the forepole strain predicted by Euler-Bernoulli beam theory. Note: Compressive strain is taken negative. 35

Figure 2-10: FOS cable bolt construction using a 15.2 millimeter nominal diameter steel strand (140mm$^2$ steel cross-section). 38

Figure 2-11: Load (full cable bolt) versus percent strain (central wire/tube) comparison between a standard cable bolt member (Standard cable), FOS cable bolt (FOS cable), and strain measured using the FOS technique (DOS). 39

Figure 2-12: Coaxial rock bolt loading arrangement. The optically instrumented rebar is cement grouted within a confining concrete cylinder. Coaxial load is applied to the rebar while the concrete cylinder is restrained (MTS 810 loading frame). Electrical resistive strain gauges and linear-variable-displacement-transformers are used to monitor external apparatus displacements. 41

Figure 2-13: Rock bolt coaxial loading results. Upper – Experimental arrangement. Lower – Averaged strain profile taken along the rebar at various levels of applied load. 42

Figure 2-14: Detailed view of the strain distribution along the grouted segment of rebar at 70 kN of applied coaxial load. 43

Figure 2-15: Double shear plane experiment. Left – Loading apparatus (MTS 810 loading frame). Right – Strain measured along the entirety of the optical sensor (i.e. both opposing sides) at various levels of applied load (i.e. causing displacement of the central block). Note: The rebar has been orientated such that the optical sensing lengths are situated along the top and bottom alignment of the rebar. 44

Figure 2-16: Comparison of the strain profile measured along the top alignment of the rebar. Upper plot – Strain profile measured using ROFDR (i.e. optical) and the interpolated strain profile from electrical resistive strain gauges positioned at 250 millimeter increments at 50 kN and 200 kN of applied load (i.e. causing displacement of the central block). Lower plot – Strain profile measured using ROFDR at various spatial resolution samplings at 200 kN of applied load (i.e. causing displacement of the central block). 46

Figure 2-17: Forepole symmetric bending experiment. Top – Normalized strain profile along the top (i.e. compressed) alignment of the forepole member at 15 kN of applied load. Bottom – Absolute strain profiles measured along the top (i.e. compressive) and bottom (i.e. tensile) alignment of a 1250 millimeter long forepole member at 20 kN of applied load in comparison to Euler-Bernoulli beam theory and the absolute average of the opposing sensing alignments (i.e. top and bottom). 48
Figure 2-18: Cable bolt coaxial pull-test results: Strain distributions measured along the grouted length of the central tube of a seven strand steel cable at various applied loads. .......................................................... 50

Figure 2-19: Cable bolt load-displacement relationships measured from the actuator stroke (stroke), corrected actuator stroke obtained by subtracting the apparatus deflection (LVDT) from stroke (Cable stroke), and displacement obtained from the strain measured using the FOS technique (DOS). ............... 52

Figure 3-1: Failed rock bolt segment. Permanent lateral deformation is shown in the form of a singular bend at the bolt head and a double bend at the failure location (Image courtesy of Brad Simser, Glencore). .......................................................................................................................... 63

Figure 3-2: Schematic representation of strain gauge positioning along a bolt specimen within a pair of diametrical opposed machined out grooves (Modified after Serbousek & Signer, 1987). ...................... 65

Figure 3-3: Schematic representation of the optical sensor embedded into a pair of diametrically opposed grooves. The sensor is initialized at the head end of the rebar via an LC connector, looped at the opposing toe end of rebar, and terminated in proximity with the rebar head. ................................................................. 67

Figure 3-4: Top - Optical sensor configuration. Bottom – Strain profiles captured along the optical sensor at various levels of applied symmetric bending load. Two orientations of the rebar are considered: Sensing lengths orientated within proximity of the plane of applied loading (Left), and sensing lengths orientation within proximity of the neutral axis (Right). Tensile strain is considered positive while compressive strain is considered negative. ..................................................................................................... 68

Figure 3-5: Schematic representation of the optical sensor embedded into three grooves orientated in a delta configuration (i.e. 120 degree spacing). Left – Cross sectional view along the specimen where \( \theta \) is the angular distance of lateral loading relative to sensing length 1 and \( \phi \) is the angular distance of a given sensing length relative to sensing length 1. Right – The sensor is initialized at the head end of the bolt via an LC connector, looped at the opposing toe end of the bolt, subsequently looped in proximity to the head end of the bolt, and terminated in proximity with the bolt toe. .......................................................................................... 69

Figure 3-6: Schematic representation of the optical sensing length positioning in comparison to the rebar symmetric bending test apparatus. ................................................................................................................ 74

Figure 3-7: Symmetric bending experiment. Top – Optical sensor configuration and approximate orientation in section view. Bottom – Measured strain profiles at an applied bending load of 50kg along the entirety of the optical sensor (Left) and along the bending span of the first sensing length (Right). ......................... 75

Figure 3-8: Symmetric bending experiment. Left – Coaxial and bending induced components of strain measured along the bending span of the rebar at 50kg of applied load. Right – Clockwise orientation of the first sensing length relative to the direction of applied load. ..................................................................................................... 75

Figure 3-9: Combined axial and bending apparatus. A 20 tonne hollow plunger cylinder (a) is used to apply axial load to a 1500mm end anchored rebar (b). A scaled turnbuckle is used to apply a point
bending load (c) while spherical washers and domed plates minimize bending as a result of tensioning the rebar (d)(e).

Figure 3-10: Combined axial and bending experiment. Top – Optical sensor configuration. Bottom – Measured strain along the entirety of the optical sensor.

Figure 3-11: Combined axial and bending experiment. Left – Component of measured coaxial strain. Right – Component of measured bending induced strain.

Figure 3-12: Idealization of the symmetric bending experiment (left) and combined axial and bending experiment (right).

Figure 3-13: Double shear apparatus. Left – An example test specimen and optical interrogation unit during test preparation. Right – Schematic representation of test specimens.

Figure 3-14: Left – Depiction of optical sensing length orientation. Right – Top: Schematic depiction of the optical sensor configuration. Bottom: Strain profiles measured along the entirety of the optical sensor at various central block displacements. Tensile strain is considered positive while compressive strain is considered negative.

Figure 3-15: Left – Strain comparison of sensing lengths along the rebar at an applied displacement of 2mm. Middle – Resolved sensor orientation. Right – Orientation of the load vector along the rebar. Note: Zero is referenced as the running along the top axis of the rebar.

Figure 3-16: Left – A comparison of the total strain, coaxial strain, and being induced strain along the rebar at an applied displacement of 2mm. Right – Lateral deflection of the rebar at various central block displacements.

Figure 3-17: Post double shear test removal of the rebar and encapsulating grout from the concrete blocks.

Figure 4-1: Left - Rebar element exposed after a fall of ground in an underground mine. Permanent deformation is visible along the rebar in the form of coaxial elongation and transverse, bending moment induced deformation (Image courtesy of Brad Simser, Glencore). Right – Schematic illustration of coaxial and transverse loading along a reinforcement element (after Mark et al. 2002).

Figure 4-2: Schematic depiction of the reinforcement response of a fully grouted element. At the head of the element (i.e., at the excavation periphery), the element resists movement of the ground mass towards the excavation ($U_{ex}$). Towards the end of the bolt, more competent, deeper seated ground restrains the element from moving towards the excavation. Accordingly, there is a reversal in the sense of shear traction or relative movement between the element and the ground mass when comparing the pickup length and the anchoring length. The neutral point corresponds to the position where there is an inflection in the direction of the shear stress and corresponds to the position of maximum axial load along the
element. \( U_{dm} \) refers to a discrete movement (such as dilation across a discontinuity located along the reinforcement element).

Figure 4-3: Principle components of a reinforcement element (after Windsor 1997).

Figure 4-4: Schematic comparison between rebar, plain strand cable, CT-Bolt, and D-Bolt reinforcement elements. Further details on these elements are listed in Table 4-1.

Figure 4-5: Grouting procedure for the test specimens. A) View of 1500 mm embedment length cables post grouting. Note: the bottom end of the specimen (where grout is pumped from) is ultimately flipped upside down and is the end that load is applied to using the testing frame. B) View of a cable specimen bottom end prior to grouting. The element is centered in the confining pipe using a centering cap and is sealed to prevent water/cement leak during grouting and curing. The 19.05 mm threaded fitting used as the inlet for grout is also shown. C) View of the vinyl tube used for the flow of grout into the pipe. Note: After filling the entire pipe with grout the vinyl tube was pinched (as shown in the Figure) and left in place until cured. The centering caps, vinyl tube, and threaded fitting were removed prior to testing.

Figure 4-6: Schematic depiction of the coaxial testing apparatus. The metal pipe or concrete cylinder test specimen is constrained by a pair of 25.4 mm thick steel plates. The bottom steel plate (attachment plate) is fixed to the MTS workbench using four T-nuts and four 19.1 mm diameter, 75 mm long bolts. Six 19.1 mm diameter threaded bars are circumferentially spaced around the test specimen to restrain the top steel plate (bearing plate).

Figure 4-7: Coaxial testing apparatus and DOS interrogation unit. An example concrete cylinder is constrained to the MTS workbench. The DOS readout, showing strain along a FOS, is displayed.

Figure 4-8: Detail view of the coaxial testing apparatus. A) Constrained concrete specimen and the LVDT arrangement used to measure specimen displacement. B) Top view of the bearing plate. A 25.4 mm hole was drilled through the plate for the reinforcement element to extend through to the hydraulic grips. C) View of the LVDT arrangement underneath the MTS workbench that was used to measure slip of the reinforcement element. Penny and Giles SLS190 LVDTs were used in this study (200 mm capacity, repeatability of at least 0.01 mm).

Figure 4-9: Schematic drawing of the FOS. The active sensing length of the FOS refers to the length of the sensor that is along the encapsulated segment of the reinforcement element (i.e., grouted with the confining cylinder or pipe). The arrangement of the FOS varied depending on the support element type (refer to Table 4-1 and Table 4-2). For solid bar elements, a single optical fiber was embedded along a pair of diametrical opposed grooves machined along the bar or within three lengthwise grooves equally spaced at an angular distance of 120 degrees around the bar. This required the optical fiber to be looped into subsequent grooves. For cable elements, the optical fiber was encapsulated within the central wire of the strand, situating the optical fiber within proximity to the centroid of the element.
Figure 4-10: Top: Coaxial Load-displacement response curves for cable test specimens (refer to Table 4-2). Bottom: Coaxial Load-deformation response curves for cable reinforcement elements. Displacement has been determined from the measured actuator stroke and the LVDTs. Deformation has been determined from the coaxial strain measured along the cable element with the FOS. Note: The deformation scale is 10% of the displacement scale. Note: Positive load, displacement, and deformation are taken as tensile.

Figure 4-11: Coaxial load measured along the short (750 mm) embedment length cable specimens at 25 kN loading increments up to 100 kN.

Figure 4-12: Coaxial load measured along the long (1500 mm) embedment length cable specimens at 25 kN loading increments up to 100 kN.

Figure 4-13: Post-test inspection of test specimen CS49-1500. A) View of the loaded end of the test specimen (i.e., 0 m on the strain profiles). A conical failure surface is evident. B) View of the distal end of the test specimen (i.e., 1.5 m on the strain profile). The unscrewing failure mechanism is evident – the cable spun through the grout flouts.

Figure 4-14: Comparison between the shear stress distribution at the cable-grout interface and the coaxial load distribution corresponding to 100 kN of applied load to test specimen CS49-1500. Positive shear stress denotes differential movement of the cable relative to the grout. Note: A 50-point moving average (i.e., 32.5 mm interval) was applied to the shear stress distribution in order to reduce the amplification of measurement noise introduced through numerical differentiation.

Figure 4-15: Top: Coaxial Load-displacement response curves for bar (i.e., rebar, CT-Bolt, D-Bolt) test specimens (refer to Table 4-2). Bottom: Coaxial Load-deformation response curves for bar reinforcement elements. Displacement has been determined from the measured actuator stroke and the LVDTs. Deformation has been determined from the coaxial strain measured along the cable element with the FOS. Note: The deformation scale is 10% of the displacement scale.

Figure 4-16: Coaxial load measured along the rebar specimens at 25 kN loading increments up to 100 kN. Note: Load was not applied above 75 kN for test specimen RC31-500.

Figure 4-17: Comparison between the shear stress distribution at the rebar-grout interface and the coaxial load distribution corresponding to 100 kN of applied load to test specimen RS40-500. Positive shear stress denotes differential movement of the rebar relative to the grout. Note: A 50-point moving average (i.e., 32.5 mm interval) was applied to the shear stress distribution in order to reduce the amplification of measurement noise introduced through numerical differentiation.

Figure 4-18: Coaxial load measured along the CT-Bolt specimens at 25 kN loading increments up to 100 kN. Note: The load distribution of CTS49-1000(1) was nearly equal to CTS49-1000(2); although much
greater measurement noise was present. Therefore, for visualization purposes of comparing with the no
sheath sample, CTS49-1000(1) has not been shown.

Figure 4-19: Comparison between the shear stress distribution at the CT-Bolt-grout interface and the
coaxial load distribution corresponding to 100 kN of applied load to test specimen CTS49-1000(2).
Positive shear stress denotes differential movement of the CT-Bolt relative to the grout. Note: A 50-point
moving average (i.e., 32.5 mm interval) was applied to the shear stress distribution in order to reduce the
amplification of measurement noise introduced through numerical differentiation.

Figure 4-20: Schematic depiction of the D-Bolt smooth bar segments and anchor positions along the test
specimen (i.e., the grouted length of the element).

Figure 4-21: Coaxial load measured along the D-Bolt specimens at 25 kN loading increments up to 100
kN. The relative positions of the two anchor sections along the element are indicated by dashed lines in
the plot (and further described in Figure 4-20).

Figure 4-22: Comparison between the shear stress distribution at the D-Bolt-grout interface and the
coaixial load distribution corresponding to 100 kN of applied load to test specimen DS49-1500. Positive
shear stress denotes differential movement of the D-Bolt relative to the grout. Note: A 50-point moving
average (i.e., 32.5 mm interval) was applied to the shear stress distribution in order to reduce the
amplification of measurement noise introduced through numerical differentiation.

Figure 4-23: Top: Coaxial Load-displacement response curves for all reinforcement elements. Bottom:
Coaxial Load-deformation response curves for all reinforcement elements. Displacement has been
determined from the measured actuator stroke and the LVDTs. Deformation has been determined from
the coaxial strain measured along the given element with the FOS. Note: The deformation scale is 10% of
the displacement scale. For the cable specimens, short and long embedment length refers to the 750 mm
and 1500 mm samples, respectively.

Figure 4-24: Comparison of the load transfer response between the rebar, cable, CT-Bolt, and D-Bolt
reinforcement elements. Top: Comparison of the coaxial load distributions measured along the various
reinforcement element types at 100 kN of applied load. Bottom: Comparison of the interface shear stress
distributions of the various reinforcement element types at 100 kN of applied load. Note: The rebar, cable,
CT-Bolt, and D-Bolt load distributions and shear stress distribution correspond to test specimens RS40-
500, CS49-1500, CTS49-1000(2), and DS49-1500, respectively (refer to Table 4-2).

Figure 5-1: Example in-situ pull test unit in an underground mine (Modified after Nicholson, 2016).

Figure 5-2: General layout of the FOS along a rebar. A single 155 µm diameter optical fiber is embedded
along the length of diametrical opposed grooves machined along the rebar. The FOS initiates with an FOS
connector at the head of the rebar, is looped within a milled semicircle 0.19 m from the toe end of the
rebar, and is terminated near the head end threads. The active sensing length is defined as the length of
the FOS that runs straight along the rebar and is the length analyzed further in this study. Not to scale. 144
Figure 5-3: A – Example FOS instrumented rebars and the DOS measurement unit. B – Example stainless
steel protective cap used to cover the OF connector during handling and installation. .............................. 145
Figure 5-4: FOS modified in-situ pull test unit. A – Installed FOS instrumented rebar with the protective
cap removed to allow the coupler piece to be threaded on. B – Threaded coupler piece used to connect the
rebar with a high strength threaded rod (25.4 mm diameter) used to extend through the hydraulic cylinder.
A coupler piece was manufactured for both the 19.05 mm diameter rebars and the 22.22 mm rebars. C –
Fully assembled pull test unit displaying the digital instrumentation used to measure load and stroke of
the hydraulic cylinder (326 kN capacity, 64 mm stroke). ........................................................................... 146
Figure 5-5: Modified pull test apparatus for operation with DOS. ................................................................. 148
Figure 5-6: Load-displacement response curve measured for FOS_02. Load was applied in five cycles. A
solid line indicates the loading domain of the load cycle, while a dashed line indicates the unloading
domain. Theoretical elastic deformation of the pull test assembly at a given load has been removed from
the displacement measurements ........................................... 149
Figure 5-7: Load-displacement response of load cycle 3 for FOS_02 (refer to Figure 5-5). The reloading
segment transitions to the loading segment at the maximum applied load of the previous load cycle..... 150
Figure 5-8: Strain distributions measured along the active sensing length of FOS_03 at selected applied
loads .................................................................................................................................................. 152
Figure 5-9: 100 µε scale view of strain distributions measured along the encapsulated length of FOS_03 at
selected applied loads. A Hampel filter has been used to reduce the low-strain measurement noise for
visualization purposes (solid distribution versus shaded distribution). ........................................... 153
Figure 5-10: Strain distribution comparison between FOS_01, FOS_02, and FOS_03. Top: Sketches
depicting the installation variations between each FOS instrumented rebar. Bottom: Strain measured
along the active sensing length of each FOS instrumented rebar at an applied load equivalent to 300 MPa
(85 kN for FOS_01 and FOS_03 and 115 kN for FOS_02). ................................................................. 155
Figure 5-11: FOS_03 shear stress distributions at the support element-resin interface for selected loads.
The dashed line corresponds to Equation 2 and the solid line corresponds to Equation 3 (from the FOS
strain profile). A 25-point moving average (i.e., 16.25 mm interval) was applied to the DOS
measurements in order to reduce the amplification of measurement noise introduced through numerical
differentiation ........................................................................................................................................ 157
Figure 5-12: Alteration of head connector for forger head support elements. A – Forged head rebar and
example protective cap with external threads. B – View of female OF connector (male OF connector also
applicable) recessed within a centered hole and internal threads for protective cap. ............................. 158
Figure 5-13: FOS_03 coaxial deformation distribution referenced from the toe end of the rebar at selected applied loads. The dashed line segment of each distribution provides an estimate of displacement at the head of the rebar. This has been calculated by determining the average stiffness along the free length of rebar.

Figure 5-14: Load-displacement and stiffness comparison of FOS_03. The anchor length profiles refer to displacement of the rebar at the transition point between the free length and anchor length. Anchor length displacement from the digital displacement sensor has been estimated by correcting for calculated deformation of the free length of rebar (elastic theory). Stiffness has been determined from the linear regression of all loading cycle measurements.

Figure 6-1: Schematic drawing of the FOS. Three grooves are machined along the length of a reinforcement element. The lengthwise machined grooves are approximately 2.5 mm deep by 2.5 mm wide and are equally spaced at an angular distance of 120 degrees (\( \phi \)). The active sensing length refers to the length between the pair of machined loops that connect the grooves in a clockwise direction from the head of the reinforcement member. A single connecting loop is located at the positions “a” and “b” from the head, defining the active sensing length. The FOS connector and end termination are both situated outside of the active sensing length at opposing ends.

Figure 6-2: FOS connector configuration at the head of a reinforcement element. A: Example stainless steel protective cap threaded onto the element’s head, covering the FOS connector. B: View of a FOS connector after installation of the reinforcement element (protective cap removed).

Figure 6-3: Schematic drawing of an example FOS head construction for in-situ application. A 6 mm diameter, 5 mm long centered hole at the head of the reinforcement element is used to recess a FOS connector. OF extending from the FOS connector is slackened within the centered hole and run through a 2.5 mm diameter angled hole into a stainless-steel tube (1.75 mm diameter, 0.25 mm wall thickness, 20 cm length) positioned at the bottom of a machined groove. The OF is unbonded to the reinforcement element between the FOS connector and the distal end of the stainless-steel tube.

Figure 6-4: Schematic drawing of the FOS end termination assembly, including a detail view of the fusion splice between the OF (that composes that active sensing length of the FOS) and the coreless OF. Section AA is displayed at a 5:1 diameter to length scale for visualization purposes. The length of the end termination is the length of the stainless-steel tube, which is 5 cm for the FOSs discussed in this research.

Figure 6-5: FOS instrumented CT-Bolt in-situ pull test. A: Pull test assembly mounted onto the instrumented bolt. B: View of the modified coupling nut used to exit the OF lead and connect the instrumented bolt with the extension rod (and hydraulic ram).
Figure 6-6: Mechanized installation acceptance test strain profiles. The baseline measurement was recorded prior to installing the instrumented CT-Bolt, with the bolt positioned flat on the tunnel floor. The install measurement was recorded immediately following the bolt being inserted into the borehole with the jumbo and spun in order to expand the end anchor and tension the bolt. The grout measurement was recorded after encapsulating the bolt with a cement grout. Note: Tensile strain is taken as positive.

Figure 6-7: In-situ pull tests results. Top: Coaxial strain measured along the length of the instrumented CT-Bolt at various applied loads. Bottom: Coaxial deformation (elongation) profiles referenced from the distal end of the active sensing length, determined from the measured coaxial strain profiles. Note: Tensile strain and elongation are taken as positive.

Figure 6-8: In-situ versus laboratory pull test comparison. Coaxial strain profiles are presented for a 2.4 m long in-situ installed CT-Bolt and a CT-Bolt grouted within a 1.00 m long steel pipe (60.5 mm outer diameter, 5.59 mm wall thickness) in the laboratory. Strain profiles are presented at 50 kN, 100 kN, 150 kN, and 200 kN of applied load. Note: Tensile strain is taken as positive.

Figure 6-9: Spile installation and lead cable protection. A: View of a lattice girder and spile round. A FOS spile is also positioned on the jumbo to be installed near center span of the crown. B: Connection between the OF lead cable and the FOS connector. A protective cap covers the connection at the head of the spile after completing the connection. C: Flexible plastic conduit positioned over top of the head of the FOS spile, secured using a hose clamp. The OF lead cable runs through the conduit for protection. D: Positioning of the OF lead cable and plastic conduit assembly around the perimeter of the excavation (along the lattice girder) for improved accessibility during monitoring.

Figure 6-10: FOS spile installation coupler. A 280 mm long steel pipe (42.6 mm outer diameter) acts as a coupling nut (80 mm thread length) for a FOS spile and an additional threadbar length necessary to connect with the jumbo. A 120 mm unthread length across the center span of the coupler (35.5 mm inner diameter) provides a cavity for water or grout to flow around the protective cap housing the FOS connector and into the 14.5 mm diameter core of the instrumented spile through 9.50 mm diameter inlets.

Figure 6-11: Grout inlet at the head of the FOS spiles. A pair of 9.50 mm diameter holes were drilled on diametrical opposed sides at approximately 60.0 mm from the head of the spile to allow water or grout to enter the hollow core. These holes are beyond the FOS connector construction (i.e., Figure 6-3), which was blocked off using a 10.0 mm layer of metal bonding adhesive. A slot, 10.0 mm wide and 4.50 mm deep (max depth from circumference), was milled from the head of the spile to the grout inlet holes. The milled slots were necessary to allow water or grout to reach the inlet holes, which were positioned along the threaded section of the installation coupler (refer to Figure 6-10) during installation.
Figure 6-12: Absolute maximum strain measured along the active sensing length of an instrumented spile at selected measurement dates. The baseline (i.e., surface) measurement was taken prior to installing the instrumented spile with it laying flat on the tunnel floor. The Install Reading was taken immediately after installing the spile and lead cable assembly. The 1st Reading and 2nd Reading correspond to the first and second excavation advances, respectively. The 1 Month Reading corresponds to a measurement approximately one month after installation. The approximate position of the tunnel face along the spile for the Install Reading, 1st Reading, and 2nd Reading are indicated by the dashed lines. Note: Tensile strain is taken as positive.

Figure 6-13: Moment induced strain and coaxial strain components determined along the active sensing length of the instrumented spile at selected measurement dates. Top: Bending moment induced strain corresponding to the top alignment of the spile. Bottom: Coaxial strain acting through the cross-section of the spile. The approximate position of the tunnel face along the spile for the Install Reading, 1st Reading, and 2nd Reading are indicated by the dashed lines. Note: Tensile strain is taken as positive.

Figure 6-14: Orientation of the bending moment inducing strain around the circumference of the spile with respect to the top alignment of the spile (i.e., 0° refers to load acting vertically downward on the top profile of the spile). The approximate position of the tunnel face along the spile for the Install Reading, 1st Reading, and 2nd Reading are indicated by the dashed lines. Note: Positive angles refer to a clockwise orientation from the top alignment.

Figure 6-15: Absolute maximum strain measured along the active sensing length of a FOS CT-Bolt. The location of sub-horizontal discontinuities as observed using a borehole camera in an adjacent borehole following the installation of the CT-Bolt are shown as black dashed lines. Note: Tensile strain is taken as positive.

Figure 6-16: Coaxial strain and moment induced strain components determined along the active sensing length of the FOS CT-Bolt. Top: Coaxial strain acting through the cross-section of the CT-Bolt. Bottom: Bending moment induced strain corresponding to one side of the element’s most eccentric external fiber (i.e., one of the pair of external fibers subjected to maximum moment induced strain, furthest from the neutral axis). The location of sub-horizontal discontinuities as observed using a borehole camera in an adjacent borehole following the installation of the CT-Bolt are shown as black dashed lines. Note: Tensile strain is taken as positive.

Figure 6-17: Shear response comparison between in-situ and laboratory measurements. The selected in-situ strain profile and load orientation correspond to the 4th Reading at the shear plane located at approximately 3.22 m along the CT-Bolt (refer to Figure 6-16). The laboratory results correspond to a cement grouted CT-Bolt subjected to approximately 1.75 mm of differential shear across a thin (3 mm thick), frictionless shear plane. Top: Bending moment induced strain measured along the top alignment of
the CT-Bolt. Bottom: Orientation of the bending moment inducing strain around the circumference of the CT-Bolt with respect to the top alignment of the element (i.e., 0° refers to load acting vertically downward on the top profile of the element). Note: Tensile strain is taken as positive and a positive orientation angle refers to a clockwise rotation from the top alignment.

Figure 6-18: Comparison between the shear stress distribution at the reinforcement element-grout interface and the coaxial strain corresponding to the 4th Reading. Positive shear stress denotes slip of the ground mass (towards the excavation) relative to the reinforcement element while negative shear stress denotes the opposite. Note: A 25-point moving average (i.e., 16.25 mm interval) was applied to the shear stress distribution in order to reduce the amplification of measurement noise introduced through numerical differentiation. Tensile strain is taken as positive.

Figure 6-19: Coaxial deformation of the CT-Bolt referenced to the end of the active sensing length. The location of sub-horizontal discontinuities as observed using a borehole camera in an adjacent borehole following the installation of the CT-Bolt are shown as black dashed lines. Note: Tensile deformation is taken as positive.

Figure 7-1: Plan view of the proposed 8010 level (2.4 km depth) as of January 2016. The pillar of study was located between the 6230 and 6245 sills. The instrumented segment of the pillar is highlighted in red. Stopes taken throughout the monitoring period are also shown in the ore body. FW refers to footwall (Modified after Punkkinen et al. 2018).

Figure 7-2: Plan view displaying the installation locations of six digital extensometers (blue), three BHPCs (red), and two FOSs (green) as of May 4th, 2016. The dates and advance lengths of mine-by-excitation rounds along the 6230 sill are also displayed. FW refers to footwall (Modified after Punkkinen et al. 2018).

Figure 7-3: Geological plan view of the 6245 sill, 6230 sill, and 0002 footwall (FW) drift within proximity of the instrumented pillar segment as of February 28th, 2017. The locations of FOS instrumented cable bolts are displayed with reference to the digital extensometers installed approximately nine months prior (Modified after Punkkinen et al. 2018).

Figure 7-4: Multi-point borehole extensometer anchor locations referenced to the head anchor (closest to the 6245 sill).

Figure 7-5: Core assembly of the optical fiber displacement sensors. Left - Coaxial Sensor: Tensioning of the optical fiber to be situated along the centroid of the assembly. Right – Shape Sensor: Tensioning of the optical fiber to be situated along three lengths of the assembly in a delta (i.e., 120-degree) configuration.

Figure 7-6: Example optical fiber sensor. The 25.4 mm outer diameter flexible plastic conduit houses the optical fiber core assembly which has been centered and fully cast using a urethan rubber compound.
Figure 7-7: Optical fiber instrumented plain strand cable bolt construction using 15.24 mm nominal diameter steel strand (Forbes et al. 2018). .............................................................. 235

Figure 7-8: Measured anchor displacement profiles referenced to the third anchor position (approximately the center position of the pillar) from May 19th, 2016 to December 21st, 2016 for each digital extensometer. Positive displacements are in the direction of the 6230 sill while negative displacements are in the direction of the 6245 sill. A dashed line displacement profile indicates an issue or limitation with the measured displacements (refer to Table 7-3) (Modified after Punkkinen et al. 2018). .......................... 239

Figure 7-9: Displacement-time series profiles (top) and corresponding velocity-time series profiles (bottom) for selected anchor positions of extensometer EXTO_08 from September 10th, 2016 to September 28th, 2016. ........................................................................................................ 242

Figure 7-10: Visual explanation of event time-duration determination from an example velocity-time series profile. IOLV refers to an interval of low velocity which is not indicative of displacement stabilization ................................................................. 243

Figure 7-11: Histogram plots displaying the frequency distribution (log-scale) of mining activity induced extensometer anchor displacements (for all extensometers). Left plot: Time until anchor displacement stabilizes following a mining activity. Right plot: Comparison between Immediate and extended anchor displacement. An increased bin-width resolution for displacements between 0 mm and 1 mm is presented within the right plot. Note: There are three events not shown in the left plot (for visualization purposes): individual instances with time durations of 40, 49, and 57 hours. ................................................................. 244

Figure 7-12: Histogram plots displaying the frequency distribution (log-scale) of mining activity induced displacement (cumulative) and displacement time-duration classified by extensometer anchor position. Anchor positions: Head, 1, and 2 are located approximately 0.40 m, 1.77 m, and 2.99 m from the 6245 sill. Anchor positions: 6, 5, and 4 anchor are located approximately 0.40 m, 1.16 m, 1.92 m from the 6230 sill ........................................................................................................................................ 245

Figure 7-13: Strain-time series profiles for extensometers EXTO_08 and EXTO_11. Strain profiles correspond to the strain at the midpoint between two adjacent anchor positions of the given extensometer. Positive strain indicates extension and negative strain indicates contraction. A dashed line indicates an issue with the measured displacements (refer to Table 7-3). ........................................................................................................ 247

Figure 7-14: FOS strain and displacement profiles compared against interpolated extensometer displacement profiles for selected monitoring periods. The FOS displacement profiles have been obtained by integrating the given strain profile (from the 6245 sill for FOS_01 and from the toe of the sensor near the pillar core for FOS_02). For comparison with the extensometer displacement measurements, the measured anchor displacements have been averaged between extensometer EXTO_08 and EXTO_15 (i.e., the immediately adjacent extensometers to the FOSs). Extensometer anchor displacements have been
referenced to the first anchor position for FOS_01 and the third anchor position for FOS_02. Note:
Negative strain is contractional and positive strain is extensile.

Figure 7-15: FOS cable strain profiles and their corresponding cable displacement profiles for the
monitoring period of March 1st, 2017 to March 6th, 2017. Negative strain is contractional and positive
strain is extensile.

Figure 7-16: Example un-plated cable bolt coaxial displacement and load distribution response to coaxial
rock mass displacements. Two conditions are considered (following the analytical solutions proposed by
Hyett et al. 1996 – refer to sketches at top): Left - A relatively distributed rock mass displacement profile
resulting in a displacement $U_{ex}$ at the excavation periphery; Right – Discrete rock mass displacements at
various distances from the excavation periphery ($U_{r1}$, $U_{r2}$, and $U_{m}$) summing to a displacement $U_{ex}$ at the
excavation periphery.

Figure 7-17: Shear traction (stress) profiles obtained from the March 6th, 2017 measurements of the FOS
cables. Negative tractions denote slip of the rock mass while positive tractions denote slip of the cable.

Figure 7-18: Pressure-time series plots for BHPC_62 (aligned to measure vertical stress change within the
pillar) and BHPC_64 (aligned to measures stress change along the pillar axis). Top: Rock mass stress
change throughout the entire monitoring duration. Bottom: Rock mass stress change during the mine-by
phase. Selected event dates are identified (refer to Table 7-2). A dashed line pressure profile indicates
missing measurement data.

Figure 7-19: Microstrain ($\mu$ε) contour profiles. Left: Strain induced from mining the 6327 crown on June
6th, 2016. At this stage of mining the 6230 sill was not developed. Right: Strain induced from the 6230 sill
development round on September 20th, 2016. This mine-by round corresponded with daylighting of the
PSZ, situated near EXTO_08 and EXTO_09, as depicted in the geological overlay. Anchor position 6
measurements have been omitted from the strain contours due to missing data. Tensile strain is taken
positive. Approximate distances of anchor positions from the 6245 sill: A – 1.09m, B – 2.38m, C –
3.52m, D – 4.51m, E – 5.35m.

Figure 7-20: Load-displacement profiles (from Blanco Martin et al. 2011; Li 2012; Li 2017;
Vlachopoulos et al. 2018) (top) and corresponding SM profiles (bottom) for three support element types:
20 mm cement grouted rebar, 46 mm friction set, and 22 mm resin grouted D-Bolt. In the SM plots the
solid line corresponds to the elongation SM per excavation displacement and the dashed line corresponds
to the load SM per excavation displacement. A depiction of the support element elongation and load
consumption is shown using measured displacements of the 6230 and 6245 sidewalls from EXTO_11 (as
per Table 7-5). The three selected dates coincide with: 1) September 11th, 2016 - Installation of support
from the 6230 sill, near-parallel with the selected extensometer. 2) November 6th, 2016 – Completion of the 6230 sill, and 3) December 22nd, 2016 – Final production blast of the 6347 stope.
List of Tables

Table 2-1: Influential summary: Operational features/capabilities and pricing for wavelength division multiplexing fiber Bragg gratings (FBG), quasi-distributed FBG (DFBG), Brillouin based distributed sensing (BOTDR/BOTDA), and Rayleigh based distributed sensing (ROFDR) techniques. .................................27

Table 4-1: Reinforcement element overview. The modified cross-sectional area was determined by accounting for the material removed through machining grooves along the solid bar elements and the replacement of the central wire of the cable elements. HDPE refers to high density polyethylene. Rebar rib descriptions were described following the criteria discussed by Jalalifar (2006). Rebar (2) and Rebar (3) refer to rebar modified with two lengthwise machined and three lengthwise machined grooves, respectively. ..........................................................................................................................................................................................98

Table 4-2: Overview of coaxial test specimens. Specimen CTS49-1000(ns) was grouted with no plastic sheath (“ns”). Confinement radial stiffness was determined according to the thick-walled cylinder equation described by Hyett et al. (1992) – Equation 4-1. Steel and aluminum were assumed to have Young’s modulus of 200 GPa and 72 GPa, respectively, and a Poisson’s ratio of 0.25. The Young’s modulus and Poisson’s ratio of the concrete was experimentally determined to be 15.6 GPa and 0.173 (refer to Cruz 2017). The Cable, CT-Bolt, and Rebar elements were obtained from DSI Underground Canada Ltd. The D-Bolt elements were obtained from Normet Canada Ltd. ......................................................................................................................103

Table 5-1: Rebar size and strength parameters. ........................................................................................................................................................................................................................................................................................................................................................................................................................................................................................................................................145

Table 5-2: FOS instrumented rebar variations. Expansion anchors were sourced from DSI Underground Canada LTD (73 mm length for 35 mm borehole) and were threaded onto the toe threads of specified rebar prior to installation. J-LOK resin cartridges were sourced from Jennmar USA (31.75 mm diameter, 762 mm length, 120 second gel time) and were inserted into the borehole prior to installing and spinning specified rebar. The calculated encapsulation fill position is referenced from the head of the rebar and is a volume calculation that assumes the resin completely fills the toe end of the borehole prior to filling the annulus between the rebar and borehole wall. .............................................................................................................................................................................................................................................................................................................................................................................................................................................................................................................................................................147

Table 5-3: Mean secant stiffness determined for the reloading and loading segment for all pull test load cycles. ................................................................................................................................................................................................................................................................................................................................................................................................................................................................................................................147

Table 6-1: Overview of the FOS instrumented reinforcement elements by in-situ application. The active sensing length, which is defined by Position a and Position b from the head of the reinforcement element, is as described in Figure 6-1. HDPE refers to high-density polyethylene. The modified cross-sectional area includes the removed groove material but does not account for loop positions..................................................................................................................179
Table 6-2: Summary of FOS CT-Bolt strain measurements detailing: the distance of the CT-Bolt from the tunnel face, the time duration from the installation date of the FOS CT-Bolt, and additional information associated with the measurement (including auxiliary excavation activity near the FOS CT-Bolt).

Table 7-1: Estimated in-situ stress field on the 8010 level of the Creighton Mine (from measurements taken by Bawden and Coulson 1993; Malek et al. 2008; Morissette et al. 2017b).

Table 7-2: Summary of 8010 level mining operations. Note: BIE refers to Blast Induced Event (After Punkkinen et al. 2018).

Table 7-3: Identified issues and limitations with extensometer anchor displacement measurements.

Table 7-4: Summary of measurements taken for FOS_01 and FOS_02.

Table 7-5: 6230 and 6245 sidewall displacement measurements from extensometer EXTO_11 at selected measurement dates. September 11th, 2016 is referenced as the installation date for the support system aligned with EXTO_11. November 6th, 2016 corresponds to the completion of the 6230 sill. December 22nd, 2016 corresponds to the final mining operation of the 6347 stope. For simplicity it has been assumed that all support typologies have been pretensioned to 50 kN or approximately 2.5mm of elongation.
<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>APC</td>
<td>Angled physical contact polish connection</td>
</tr>
<tr>
<td>BHPC</td>
<td>Borehole pressure cell</td>
</tr>
<tr>
<td>BOCDA</td>
<td>Brillouin optical correlation domain analysis</td>
</tr>
<tr>
<td>BOCDR</td>
<td>Brillouin optical correlation domain reflectometry</td>
</tr>
<tr>
<td>BOTDA</td>
<td>Brillouin optical time domain analysis</td>
</tr>
<tr>
<td>BOTDR</td>
<td>Brillouin optical time domain reflectometry</td>
</tr>
<tr>
<td>CMC</td>
<td>Continuously mechanically coupled</td>
</tr>
<tr>
<td>CW</td>
<td>Clockwise</td>
</tr>
<tr>
<td>DFBG</td>
<td>Distributed fiber Bragg gratings</td>
</tr>
<tr>
<td>DOS</td>
<td>Distributed optical fiber strain sensing</td>
</tr>
<tr>
<td>DUT</td>
<td>Distributed optical fiber length under test</td>
</tr>
<tr>
<td>ERS</td>
<td>Electrical resistive strain gauge</td>
</tr>
<tr>
<td>FBG</td>
<td>Fiber Bragg grating</td>
</tr>
<tr>
<td>FOS</td>
<td>Fiber optic strain sensor</td>
</tr>
<tr>
<td>FS</td>
<td>Factor of safety</td>
</tr>
<tr>
<td>HDPE</td>
<td>High density polyethylene</td>
</tr>
<tr>
<td>LC</td>
<td>Lucent connection</td>
</tr>
<tr>
<td>LVDT</td>
<td>Linear variable displacement transducer</td>
</tr>
<tr>
<td>ODiSI-B</td>
<td>Optical distributed sensor interrogator</td>
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<tr>
<td>OF</td>
<td>Optical fiber</td>
</tr>
<tr>
<td>OFDR</td>
<td>Optical frequency domain reflectometry</td>
</tr>
<tr>
<td>OTDR</td>
<td>Optical time domain reflectometry</td>
</tr>
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<td>PSZ</td>
<td>Plumb shear zone</td>
</tr>
<tr>
<td>ROFDR</td>
<td>Rayleigh optical frequency domain reflectometry</td>
</tr>
<tr>
<td>Abbreviation</td>
<td>Description</td>
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<tr>
<td>--------------</td>
<td>--------------------------------------</td>
</tr>
<tr>
<td>RQD</td>
<td>Rock quality designation</td>
</tr>
<tr>
<td>SM</td>
<td>Safety margin</td>
</tr>
<tr>
<td>SRF</td>
<td>Stress reduction factor</td>
</tr>
<tr>
<td>UA</td>
<td>Umbrella arch</td>
</tr>
<tr>
<td>UCS</td>
<td>Unconfined compressive strength</td>
</tr>
<tr>
<td>WDM</td>
<td>Wavelength division multiplexing</td>
</tr>
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</table>
List of Symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Cross-sectional area</td>
</tr>
<tr>
<td>c</td>
<td>Speed of light in a vacuum (approximately 3E8 m/s)</td>
</tr>
<tr>
<td>C_e</td>
<td>Strain to Brillouin scatter frequency shift constant</td>
</tr>
<tr>
<td>d</td>
<td>Diameter</td>
</tr>
<tr>
<td>E</td>
<td>Elastic modulus</td>
</tr>
<tr>
<td>F_{coaxial}</td>
<td>Coaxial load</td>
</tr>
<tr>
<td>F_g</td>
<td>Strain to Bragg wavelength shift gauge factor</td>
</tr>
<tr>
<td>l</td>
<td>Second moment area</td>
</tr>
<tr>
<td>K_r</td>
<td>Radial stiffness</td>
</tr>
<tr>
<td>K_e</td>
<td>Strain calibration constant</td>
</tr>
<tr>
<td>L</td>
<td>Encapsulation (grouted) length of element</td>
</tr>
<tr>
<td>l</td>
<td>Specimen length</td>
</tr>
<tr>
<td>M</td>
<td>Bending moment</td>
</tr>
<tr>
<td>N</td>
<td>Normal force</td>
</tr>
<tr>
<td>P</td>
<td>Applied pull test load</td>
</tr>
<tr>
<td>pw</td>
<td>Pulse width of the optical laser source</td>
</tr>
<tr>
<td>r</td>
<td>Radius</td>
</tr>
<tr>
<td>u</td>
<td>Displacement</td>
</tr>
<tr>
<td>v</td>
<td>Poisson’s ratio</td>
</tr>
<tr>
<td>Λ</td>
<td>Spacing of Bragg grating structures</td>
</tr>
<tr>
<td>V_a</td>
<td>Acoustic velocity within the optical fiber core</td>
</tr>
<tr>
<td>w</td>
<td>Lateral deflection of the element</td>
</tr>
<tr>
<td>z</td>
<td>Orthogonal distance from a given location on the element cross section to the neutral axis</td>
</tr>
</tbody>
</table>
\( \alpha \)  
Angular distance from orthogonality with the neutral axis

\( \Delta F \)  
Frequency scanning range of the optical laser source

\( \Delta \lambda \)  
Spatial resolution of strain measurement points

\( \Delta \lambda_{\text{Bragg}} \)  
Shifted Rayleigh scatter signal

\( \Delta \lambda \)  
Shifted Bragg wavelength

\( \varepsilon_{\text{bending moment}} \)  
Bending moment induced strain

\( \varepsilon_{\text{coaxial}} \)  
Coaxial strain

\( \varepsilon_{\text{Total}} \)  
Total strain at a given segment of the bolt at the outer fiber (maximum when orthogonal to neutral axis when bending is present)

\( \varepsilon \)  
Strain

\( \eta_{\text{eff}} \)  
Effective refractive index of the optical component (e.g. optical fiber core)

\( \theta \)  
Angular distance between the direction of lateral loading and first sensing length (i.e. the angular distance from orthogonality with the neutral axis)

\( \lambda \)  
Wavelength of the incident light

\( \lambda_{\text{Bragg}} \)  
Bragg wavelength

\( \lambda_{bw} \)  
Wavelength bandwidth of the incident light

\( \nu_b \)  
Brillouin scatter frequency

\( \sigma_1 \)  
Major in-situ principal stress

\( \sigma_3 \)  
Minor in-situ principal stress

\( \sigma_c \)  
Unconfined compressive strength

\( \tau \)  
Interfacial shear stress

\( \varphi \)  
Angular distance between the specified sensing length and first sensing length (i.e. 1 = 0°, 2 = 120°, and 3 = 240°)

\( \delta \)  
Coaxial deformation of the given element
Chapter 1 Introduction

1.1 Problem Statement

The development of underground space for civilian and mining purposes will almost always entail the installation of a ground support system. From an operational standpoint, the ground support system is responsible for ensuring safe working conditions and maintaining the profile of the excavation throughout its designed life span. Accordingly, the demand or requirements of a support system can vary significantly depending on the end use of the underground structure and the geomechanistic conditions that the structure is constructed (or excavated) within. This thesis is primarily concerned with temporary ground support measures, or the ground support system that is used as part of the construction process.

The term ground support generally describes the procedures and materials that are used in order to improve the stability and maintain the load bearing capacity of the ground surrounding the boundaries of an underground excavation. However, it must be recognized that a ground support system will often be composed of a combination of multiple support types that interact with the excavation by various means. Here a critical distinction is made between ground support types which provide a support function, where a passive, reactive force is provided at the excavation boundary, and those which provide a reinforcing function, where the ground mass is internally improved (e.g., Windsor, 1997). The latter is most often associated within the installation of tendon reinforcement elements (e.g., rock bolts, cable bolts, and ground anchors). Referring to Figure 1-1, the combination of reinforcement and support elements are required to perform three primary functions: 1) Reinforce or strengthen pre-existing structure of the ground mass, 2) Hold or tieback failed ground near the excavation boundary to more deeply seated ground further from the excavation, and 3) Retain failed ground at the excavation boundary (Kaiser et al., 1996).

From a structural perspective, an effective ground support system can have multiple meanings. It may imply that the support system is able to mitigate the geomechanical risk associated with excavation
instabilities (e.g., falling of loosened rock blocks into the excavation) or it may entail the control of excavation displacements within predefined design limits. Nevertheless, in addition to being effective, the ground support system must also be efficient (Cai and Kaiser, 2018). Depending on the prevalent ground conditions, ground support can form a substantial portion of an operation’s budget (e.g., cost of labour, equipment and material costs, supervision, etc.) and can dictate the pace of development (i.e., when a support cycle impedes the start of the following excavation cycle or production round). There are substantial cost-savings associated with implementing the most efficient support system at a given project. However, the process of optimizing a support system cannot come at the price of increased exposure to geomechanical risk.

Potvin et al. (2019) define an optimized ground support system as “the lowest cost system, including cost and productivity factors such as development mining rates, which can achieve a tolerable
probability of failure during the service life of an excavation.” From an empirical standpoint, it is very easy to distinguish an unsuitable ground support system (i.e., one that has resulted in failure). But in comparison, it may be challenging to accurately quantify how conservative a ground support system is since a successful support system will result in a non-event, or a stable excavation (Mercier-Langevin, 2019). This can result in a situation where it is difficult to justify alterations or optimizations to an effective support design as the consequences of trialing a more efficient (or reduced) system are often severe (e.g., exposure of workers to unsafe conditions, costly rehabilitation, and loss of production). While numerical and analytical analyses can be used to promote support design optimization, the strongest argument is often provided through direct measurement of support performance. This also has its own associated challenges.

The assessment of a support system’s response is a non-trivial undertaking as it requires the individual behaviour as well as the interconnectivity of composing elements to be measured. This has proven to be especially difficult to accomplish for tendon reinforcement elements as the majority of their length is not visible from within the excavation. Furthermore, reinforcement elements are routinely installed into fractured and jointed ground masses. Referring to Figure 1-2, this often results in a discontinuous and highly localized loading response along an element that is reflective of a number of discontinuity movements which may act coaxial and/or transverse to the element profile (Bjornfot and Stephansson, 1984). An extrinsic measurement procedure, whereby reinforcement behaviour is inferred from ground mass displacement measurements of the excavation periphery (e.g., Campbell et al., 2017) or within adjacent boreholes (e.g., with an extensometer or endoscopic camera), will not be able to accurately detail such a variable load response along an element. However, it is also very difficult to properly measure discontinuous behaviour with intrinsic measurement procedures (whereby sensors are directly coupled with the element) as it requires both discrete (i.e., short base length) and integrated measurements (i.e., spanning the element) (Windsor, 1992). Historically, this has not been practically feasible with conventional, discrete sensing techniques, such as an array of electrical resistive strain gauges (e.g., Spearing et al., 2013). As a
result, there are a limited number of available studies that have presented measured load distributions along reinforcement elements in-situ or even from laboratory experiments.

There is a fundamental need for an in-situ measurement technique that can capture a significantly increased number of strain measurement points along a reinforcement and also account for the three-dimensionality of ground mass induced loads (i.e. to account for both the potential unknown location(s) and orientation(s) of ground mass movements). Ideally, such a measurement technique would be able to monitor a continuous strain (or load) distribution along a reinforcement element, which would permit support design assumptions to be more accurately assessed and validated. This would also allow support optimization (specifically for tendon reinforcement) to be approached from a quantified perspective.

Within this context, distributed optical fiber strain sensing (DOS) is proposed within this thesis as possible technology that can be exploited in order to comprehensively measure the performance of reinforcement elements. DOS has been established in a limited number of controlled studies to be capable of measuring strain along a reinforcement element at an unprecedented, submillimeter spatial resolution.
(e.g., Hyett et al., 2013); however, extensive effort is required to further develop DOS for in-situ application. In particular, procedure(s) are required to couple fiber optic strain sensors (FOS) with various tendon reinforcement constructions in a manner that an instrumented reinforcement element can conform with standard operational support procedures (i.e., an instrumented reinforcement element must be able to be handled, installed, and function as a normal in-situ reinforcement element). Additionally, an FOS arrangement and accompanying analysis is required that can account for out-of-plane bending moments (as presented in Figure 1-2). Accordingly, the scope of this thesis lies within developing an FOS technique that can meet the discussed criteria and subsequently applying it as a solution for measuring and assessing reinforcement performance in active underground construction projects.

1.2 Research Objectives and Methodology
This thesis investigates the use of high spatial resolution DOS to measure the mechanistic response of various reinforcement elements that are commonly employed in underground excavations. The reinforcement elements considered in this study are typically those that are categorized to be continuously mechanically coupled (with the ground mass) and passive in nature (i.e., the reinforcement elements are mobilized by displacements of the surrounding ground mass). Accordingly, this study is inherently associated with measuring the distribution of excavation induced ground mass displacements and the manner in which a reinforcement element resists movements along pre-existing discontinuities as well as newly developed excavation damage.

This research effort has been approached through an experimental framework that initially identified the demand for an FOS technique, developed it through a series of controlled laboratory and in-situ experiments, and then applied it to multiple active mining and tunnelling operations. The primary components that form the experimental framework of this research effort are outlined in Figure 1-3 and the specific research objectives of this thesis are summarized as follows:
1) Investigate and develop a technique to measure the complete strain distribution along the length of various types of reinforcement elements using DOS – Chapter 2,3
Within the laboratory setting, it must be confirmed that the FOS technique measures strain magnitude that is comparable to other sensors that have been accepted within the mining and tunnelling industry (e.g., strain gauges, load cells) or that a calibration constant can be ascertained. However, in addressing the limitations of existing reinforcement element measurements techniques, it is necessary that the FOS technique can be used to measure the complete strain distribution along the length of the given element. This implies that the sensor arrangement must permit an accompanying analysis to distinguish between coaxial strain and moment induced strain (and its associated orientation) in order to derive the maximum magnitude of strain experienced by the element at a particular orientation (i.e., in an instance where maximum bending moment is not in direct alignment with the FOS). Furthermore, the operational sensing range of the FOS technique must be established.

2) Measure and identify the mechanistic behaviour of various reinforcement elements under controlled laboratory experiments – Chapters 2-4

Reinforcement elements are routinely assessed through laboratory configurations that simulate specific loading conditions of an in-situ ground mass (e.g., coaxial pull testing, differential shear plane testing). It is hypothesized that the FOS technique will be able to augment many laboratory configurations by measuring the manner in which load is mobilized along the length of tested elements. It is valuable to have a general understanding of a reinforcement element’s response under simplified loading arrangements in order to distinguish such responses from in-situ measured strain distribution, which are expected to be more complex and the result of a combination of load sources.
3) Advance the FOS instrumenting procedure for conformance with standard operational procedures – Chapters 4-6

Protection of the FOS is a necessary consideration for both laboratory and in-situ applications. The FOS must be coupled with the reinforcement element in a manner that promotes efficient strain transfer between the element and the sensor while simultaneously protecting the sensor from damage. For application of the FOS technique beyond controlled experiments, it is critical that the sensor design is rugged enough to survive typical handling and installation procedures of a reinforcement element in-situ (such as mechanized borehole installation). An FOS reinforcement element should function the same as a normal reinforcement element and provide little to no interruption to the standard operational procedures at the given project.

4) Measure and identify the mechanistic behaviour of various reinforcements installed in-situ – Chapters 4-7

There are a limited number of studies that have presented an in-situ measurement of the load distribution along a primary reinforcement element throughout its serviceability life. Accordingly, there is a great benefit to be gained from using the FOS technique to measure the mobilized strain response of reinforcement elements in-situ in order to compare and validate reinforcement element behaviour measured under simplified laboratory configurations and the behaviour predicted by established analytical models.

5) Incorporate the FOS technique within multi-sensor monitoring studies in-situ and ascertain its applicability and/or merit for application in underground excavations – Chapter 6,7
It is well established in the underground construction industry that there are trade-offs and/or compromises when selecting a particular sensing technology over another in order to monitor a reinforced ground mass (e.g., measurement range, measurement accuracy, cost per measurement point, post-processing requirements, etc.). Therefore, it is necessary to define the benefits and limitations of the FOS technique for monitoring reinforcement elements in-situ. This will allow identification of the most apt role for the FOS technique within a monitoring program that is focused on assessing the response of a reinforced ground mass during construction.

1.3 Thesis Structure

This thesis has been prepared in accordance with the manuscript format guidelines that have been outlined by the School of Graduate Studies at Queen’s University. Accordingly, the main body chapters of this thesis (Chapters 2-7) have been written as stand alone articles for publication in relevant journals. A brief description of each chapter is provided as follows.

Chapter 1 – Introduction:
This chapter provides a general description of the research scope and objectives addressed in the thesis. An overview of the organization of the individual chapters and a summary of how they contribute to the entire thesis is provided.

Chapter 2 – The application of distributed optical strain sensing to measure the strain distribution of ground support members:
This manuscript provides a background on tendon reinforcement elements and the conventional techniques used to monitor their performance in-situ. From this, a demand is identified for a sensing solution that can provide a significantly increased number of measurement locations (or spatial resolution) in order to more accurately assess reinforcement elements in the laboratory and in-situ. A rationale for selecting a particular high spatial resolution DOS technology over other commercially available solutions is provided. Initial
applicability tests are presented where the DOS technology was used in a series of controlled laboratory experiments on various ground reinforcement elements instrumented with FOSs.

A version of this manuscript has been published in *FACETS* (*Forbes et al. 2018* – doi: 10.1139/facets-2017-0093).

**Chapter 3 - A new optical sensing technique for monitoring shear of rock bolts:**

This manuscript is associated with the localized and complex reinforcement response of a tendon element that transects a differentially shearing discontinuity. An FOS arrangement and accompanying analysis is described that permits the true maximum strain along a given reinforcement element to be ascertained (demonstrated to be a product of coaxial and bending moment induced strain components).

A version of this manuscript has been published in *Tunnelling and Underground Space Technology* (*Forbes et al. 2017* – doi: 10.1016/j.tust.2017.03.007).

**Chapter 4 – Insight into the coaxial load distribution of fully grouted rebar, cable bolt, CT-Bolt, and D-Bolt reinforcement elements:**

The focus of this manuscript is on applying the FOS technique to measure the mechanistic response of fully grouted reinforcement elements subjected to laboratory coaxial pull tests. Unlike conventional pull tests, which solely measure the load-displacement response of the given element (external to the grouted length), the FOS technique is used to quantify the bond stress distribution at the element-grout interface as well as the load development length within elastic loading limits of the tested element.

This manuscript will be submitted to a journal with a major theme of underground construction and research.
Chapter 5 – Augmenting the in-situ rock bolt pull test with distributed optical fiber strain sensing:
This manuscript describes a procedure that was developed in order to use the FOS technique during in-situ coaxial pull tests. FOS instrumented rebar rock bolts were installed in an underground salt mine and were monitored through a series of pull tests using the FOS as well as conventional load and displacement sensors. The pull test is a convenient procedure to test the FOS technique in-situ because load is actively applied to the element in a controlled manner. Accordingly, monitoring is limited to a relatively short time span, allowing immediate assessment of the reinforcement element and the FOS technique.

A version of this manuscript has been published in the International Journal of Rock Mechanics and Mining Sciences (Forbes et al. 2019 – doi: 10.1016/j.ijrmms.2019.104202).

Chapter 6 – Measuring the in-situ response of tunnel support using high spatial resolution optical fiber strain sensing:
The focus of this manuscript is on the in-situ application of the FOS technique to monitor the strain distribution along reinforcement elements in two active tunnelling projects. The technique was used to investigate the mechanistic response of CT-Bolts during coaxial pull-out testing, the umbrella arch behaviour of spile elements, and CT-Bolt dowel behaviour induced by differential shear movements across transecting discontinuities. FOS design improvements and alterations as well as monitoring procedures that were necessary to accommodate project specific requirements are also discussed.

A version of this manuscript has been submitted to Tunnelling and Underground Space Technology (Forbes et al. 2019 – Manuscript reference: TUST_2019_1257).
Chapter 7 – An in-situ monitoring campaign of a hard rock pillar at great depth within a Canadian mine:

This manuscript discusses the use of the FOS technique in combination with multi-point rod extensometers and borehole pressure cells to monitor the response of a reinforced hard rock sill drift pillar under a high in-situ stress field. Throughout an approximately ten-month long monitoring duration, the pillar’s strain, displacement, and pressure were measured as mining development and production rounds were conducted. Particular attributes of each sensing technology that contributed to a comprehensive understanding of the pillar’s response are discussed.

A version of this manuscript has been accepted for publication in the Journal of Rock Mechanics and Geotechnical Engineering (Forbes et al. 2020 – Manuscript reference: JRMGE_2019_2).

Chapter 8 – Conclusion and Outlook:

This chapter provides a summary of the research effort, detailing major contributions and key findings of the thesis, as well as a discussion of future avenues to extend the research into.
1.4 References


Chapter 2 The application of distributed optical strain sensing to measure the strain distribution of ground support members

2.1 Introduction

A rising demand for subterranean transportation and resource management has led to the development of many more underground projects which are constructed at larger scales, over greater distances, increased depths, and within proximity to sensitive urban environments (i.e. reduced tolerances with respect to adjacent infrastructure). The result in any given project is a wide variety of ground mass and in-situ stress conditions that will often require some form of ground support or reinforcement. Ground support is a general term used to describe the procedures and materials used to improve the stability and to maintain the load bearing capacity of ground near the excavation boundaries (e.g. Brady and Brown 1993). This term can be subdivided into multiple distinct categories (e.g. Windsor & Thompson 1993) depending on conditions such as: the manner by which it strengthens the ground mass (i.e. internally, within the ground mass or externally, at the excavation boundary), whether it applies an active load to the ground (i.e. active or passive), and its expected serviceability life. An example underground support system that consists of multiple support components is shown in Figure 2-1.

During the construction stage of a project, the support system will act as the first line of defense for workers and equipment at the working face and it is pivotal in controlling excavation induced displacements in order to meet project related limitations and regulations. An incorrect evaluation of the support system can result in catastrophic consequences to both life and property when underestimated, but it can also result in excessive costs to the project if an overly conservative design is selected and implemented (Marr 2001). An observational construction approach (e.g. Austrian Society of Geomechanics 2010) provides a design rationale whereby the support system is systematically and continuously updated according to current excavation conditions, rather than designing for the worst-case scenario. This permits the installation of a less costly and time intensive support system (Kontogianni & Stiros 2002); however,
this also necessitates a comprehensive monitoring program in order to verify or falsify the assumptions made during the support design (Schubert 2008). A strong understanding of the geomechanical behaviour and response of the support system, and its constituent support elements, is therefore of critical significance to this design procedure (in addition to an understanding of the rock mass properties and expected behaviour). In terms of monitoring, this has conventionally been approached from an external perspective whereby measurements of displacement at the excavation periphery (e.g. surveying and other remote sensing techniques) and/or ground displacements surrounding the support elements (e.g. multi-point-borehole-extensometers) have been used to infer support behaviour rather than direct coupling with the support system itself. Possible explanations for this approach are the difficulties of operationally instrumenting ground support elements (i.e. to not impede construction operations) as well as the relatively coarse resolution an array of discrete, strain/load instrumentation techniques (e.g. electrical-resistance-strain-gauges) can provide along an individual support element or support system. Consequently, the

Figure 2-1: Example tunnel support scheme including: tendon support (rock bolts / cable bolts), umbrella arch support (forepoles / spiles), steelsets / girders, and shotcrete lining.
current practice of ground support sensing results in a partial understanding (and capturing) of related mechanisms and parameters that can be used in both the assessment and the predictive modelling of ground support, especially at the micro-scale.

Within this context, a consideration of innovative sensing techniques, such as fiber optic sensing (FOS), provides an opportunity to fill in the gap of knowledge with regards to the geomechanical response of support elements in isolation and as part of a multi-component support system. This research investigates several FOS techniques and provides a rationale for selecting a particular distributed optical strain sensing (DOS) technique with a view of measuring and capturing the performance of support elements and an unprecedented spatial resolution. The development of an instrumentation technique to couple the chosen DOS technology with rock bolt, umbrella arch, and cable bolt support elements is discussed and demonstrated through a series of laboratory experiments.

2.2 Monitoring Ground Support: Fiber Optic Strain Sensing

The geomechanistic response of a support system will depend on many of the physical and interface parameters associated with the host ground medium, the installation technique (e.g. the use of encapsulating grout, active loading), and the support member material/typology (e.g. Haas 1976; Azuar 1977; Spang & Egger 1990; Kilic et al. 2003). A sensing technique that is not inherent to the support system will, therefore, require significant assumptions regarding the transfer of ground mass displacements to the support member/system; although, there are numerous challenges to an intrinsic sensing solution, for example:

1) The chosen sensing technology must be coupled with support tendon;
2) The chosen sensor(s) and corresponding lead wires must be protected from environmental conditions; and,
3) The application of the sensor(s) must not interfere or alter the mechanistic behaviour of the support member/system.
The most common approach to an intrinsic sensing solution for support members has involved the use of discrete sensing techniques. Examples include: electrical-resistive-strain-gauges (e.g. Farmer 1975; Serbousek & Signer 1987), load cells (e.g. Rodger et al. 1996; Mitri 2011), long-base length induction gauges and displacement transducers (e.g. Choquet & Miller 1988; Spearing et al. 2013), and inclinometers (e.g. Volkmann 2003). Using these sensing techniques, the support member under study is effectively discretized into a number of discrete (or individual) measurement points or zones depending on the gauge length of the chosen sensor (see Figure 2-2). The spatial resolution, or density of measurements along the length of the support member will, therefore, be controlled by the number of discrete sensors that can be

![Figure 2-2: Schematic depiction of discrete sensing techniques along an example support element. a) and b): Local, discrete measurement points provided by short gauge length sensors (e.g. electrical resistive strain gauge). c) Averaged, discrete measurement zone(s) provided by longer gauge length sensors (e.g. linear-variable-displacement-transformer).](image)
applied to a given support member. This will ultimately be limited by the increased cost and manufacturing
difficulties of adding more measurement points. Consequently, the capability of a discrete monitoring
solution to measure the behaviour of a support member will be contingent upon load inducing events
occurring within the region of a discrete measurement point or uniformly across a discrete measurement
zone. This holds most significance when the ground behaviour is discontinuous in nature (Björnfot &
Stephansson 1983; Hyett et al. 1996; Li & Stillborg 1999). Realizing the spatial limitations of conventional
discrete sensing solutions, it is necessary to consider innovative strain sensing techniques such as FOS.

Fiber optics was originally proposed as an improved method for the communications industry in
the 1960’s (Koa & Hockham 1966) by replacing copper wire and electric current with a glass optical fiber
and light, respectively. The optical fiber is composed of a high quality, fused-silica core surrounded by a
lower refractive index silica cladding that acts as a dielectric waveguide, achieving much lower attenuation
rates (Keck and Schultz 1970). However, external perturbations that cause physical changes to the optical
fiber (e.g. temperature and strain) will also disturb the propagating signal within. In this manner, an optical
sensor can be realized by determining a relationship between the physical change of the optical fiber and
the spectral shift of the signal (i.e. amplitude, frequency, and phase). This provides an intrinsic solution
whereby the optical fiber itself acts as both a transmission medium and transducer. Current commercially
available solutions that can determine strain and temperature change locally along a micro-meter scale,
single mode optical fiber include fiber Bragg grating (FBG) and distributed optical sensing (DOS)
techniques. The fundamental working principles of these techniques are discussed in order to provide a
rationale for choosing the most applicable technology for the application of monitoring ground support
members.

2.2.1 Fiber Bragg Gratings

A Bragg grating is fundamentally a fixed refractive index modulation of an optical fiber core acting as a
dielectric mirror (Venghuas 2006). Bragg gratings are commonly produced by laterally exposing a fiber
core to a periodic pattern of ultraviolet light. The exposure results in the fabrication of phase structures
directly into the optical fiber core which results in a permanent change of the refraction index at exposed sections (Meltz et al. 1989, FBGS 2017). An example Bragg grating is displayed in Figure 2-3. The incident light spectrum (i.e. the input light) will be partially reflected along the Bragg grating structure. At a particular wavelength the back reflected signals will be combined in a coherent fashion. This wavelength is termed as the Bragg wavelength ($\lambda_{\text{Bragg}}$) and will have a centered wavelength position per the spacing of the grating structures ($\Lambda$) and the refractive index of the single mode optical fiber ($n_{\text{eff}}$), Equation 2-1.

$$\lambda_{\text{Bragg}} = 2n_{\text{eff}} \Lambda$$

All other wavelengths forming the incident light will pass through the Bragg grating structure unaffected.

Strain and temperature perturbations to a Bragg grating structure will alter both the refractive index and the periodicity of the grating structures. In this manner, the center position of the reflected Bragg wavelength will also be influenced by external disturbances resulting in change in length or temperature.

![Diagram of Bragg grating](image)

Figure 2-3: Example Bragg grating structure fused into the core of a single mode optical fiber. Light travelling through the Bragg grating will be partially reflected according to the Bragg wavelength ($\lambda_{\text{Bragg}}$) and will experience a shift ($\Delta\lambda_{\text{Bragg}}$) associated with local strain and temperature change (after FBGS 2017).
change of the optical fiber. The shift of the Bragg wavelength ($\Delta\lambda_{Bragg}$) as a result of strain being applied to the optical fiber can be determined according to Equation 2-2, where $l$ is the specimen length.

$$
\Delta\lambda_{Bragg} = 2\left[\Lambda \frac{dn_{eff}}{dl} + n_{eff} \frac{d\Lambda}{dl}\right] \Delta l
$$

In a single mode optical fiber the shift of the Bragg wavelength will respond linearly with applied strain ($\varepsilon$). A dimensionless gauge factor ($F_g$) for the strain to Bragg wavelength shift can be determined according to Equation 2-3 (Micron Optics Inc. 2012).

$$
\mu \varepsilon = \left[\frac{\Delta\lambda_{Bragg}/\lambda_{Bragg}}{F_g}\right] \times 10^6
$$

For conventional silica optical fibers, the gauge factor will be a value within the range of 0.75-1.3 (Haase 2007; Black et al. 2008; FBGS 2015). In this manner, the initial, unperturbed Bragg wavelength can be recorded as a reference measurement to compare against future conditions whereby a shift in the Bragg wavelength can be used to determine strain across the Bragg grating with microstrain accuracy. FBG are optically analogous to conventional electrical strain gauges as a single strain measurement is obtained per transducer. However, a distinguishing feature of the FBG technique is that the transducer (i.e. the optical fiber) is also the lead. Furthermore, multiple Bragg gratings can be multiplexed into a single fiber, such that one optical fiber is used to monitor an array of locations along the fiber (Davis and Kersey 1994). Yet, it is important to note that the FBG technique fundamentally remains a discrete solution, as a limited number of Bragg gratings can be inscribed into an individual sensor, and therefore, a limited number of strain measurements can be taken along the optical fiber.
The maximum number of Bragg gratings will be dictated by numerous factors, including: manufacturing limitations (e.g. the precision of the UV laser source), the sensor length in comparison to the Bragg grating length, and the demodulation technique used to interpret the reflected signal. Regarding the latter, the most common demodulation technique is wavelength division multiplexing, WDM (e.g. Zhang et al. 1995; Araújo et al. 1998). This requires each Bragg grating to be manufactured to reflect a different Bragg wavelength (or reflect a unique wavelength with respect to other Bragg gratings). FBG interrogation units (e.g. the sm125 produced by Micron Optics Inc.) will generally implement a swept wavelength technique where the incident light wavelength spectrum will be tuned through a set range of wavelengths (often in the range of 1510-1590 nanometers). This implies there is a limited wavelength spectrum that can be reflected by the Bragg gratings. For example, considering a gauge factor of 1 and an initial reflected Bragg wavelength of 1550 nanometers, a 2 nanometer Bragg wavelength shift would correspond to approximately 1300 microstrain according to Equation 2-3. In terms of monitoring ground support members, strains well over 10,000 microstrain (i.e. 1% strain) can be expected. Care must be taken that Bragg gratings are not manufactured within a spectral proximity such that shifted Bragg wavelength will be prone to overlapping or reflecting the same Bragg wavelength as subsequent Bragg gratings.

2.2.2 Distributed Optical Strain Sensing
DOS techniques utilize the back-reflected component of light scattering phenomenon which occurs continuously along the length of an optical fiber. Contrary to FBG techniques, DOS techniques do not require the spectral measurement to be induced (i.e. modification of the optical fiber in the form of Bragg gratings). Scattering is a spontaneous, diffuse reflection that is a result of Raman, Brillouin, and Rayleigh mechanisms. A change in local strain or temperature along the fiber will induce a modulation of the scattered signal (amplitude, phase, and frequency) which can be realized spatially along an optical fiber through optical time domain reflectometry (OTDR) and optical frequency domain reflectometry (OFDR) methods. The quintessential distributed sensor would, therefore, provide a continuous strain profile (i.e. infinite measurement points) along the length of an optical fiber. In reality, the spatial resolution will be
controlled by technological limitations of the selected technique. Ultimately, the pulse width of the laser source ($pw$) and the frequency scanning range of the laser source ($ΔF$) will dictate the spatial resolution ($Δx$) of OTDR and OFDR techniques, respectively. These relationships are described by the following equations (Kingsley & Davies 1985; Froggatt et al. 2004), where c is the speed of light in the optical fiber core and $ΔF$ is further dependent on the center wavelength ($λ$) and the wavelength bandwidth ($Δλ_{bw}$).

$$Δx = \frac{pw c}{2n_{eff}}$$  \hspace{1cm} (2-4)

$$Δx = \frac{c}{2n_{eff} ΔF}$$  \hspace{1cm} (2-5)

$$ΔF = \left(\frac{c}{λ^2}\right) Δλ_{bw}$$  \hspace{1cm} (2-6)

It is important to note that an inverse relationship exists between the spatial resolution, measurement repeatability, and maximum sensing length. For example, the pulse width using OTDR can be increased to improve the maximum length of the sensor, but this will in turn, coarsen the spatial resolution according to Equation 2-4. Similarly, the repeatability may be increased by taking an average of a series of local measurements, but this in effect makes the spatial resolution coarser.

The fundamental working principles of Brillouin and Rayleigh based DOS techniques are discussed in further detail within the following sections. Raman scattering based techniques have been excluded from the following discussion as the respective scattering is primarily temperature dependent (Dakin et al. 1985).

2.2.2.1 Brillouin Optical Time Domain Reflectometry/Analysis

Brillouin optical time domain reflectometry (BOTDR) and Brillouin optical time domain analysis (BOTDA) are two DOS techniques that measure the Brillouin scatter frequency shift along a low cost single mode optical fiber. The former monitors spontaneous Brillouin scatter, an inelastic phenomenon
corresponding to a frequency shifted component of the input light which is attributed to the effective refractive index of the optical fiber and the interaction of optical (i.e. photon) and acoustic (i.e. phonon) waves in the optical fiber (Agrawal 2001). The frequency of the Brillouin scattering ($v_b$) will be maximum according to Equation 2-7, where $V_a$ is the acoustic velocity in the optical fiber.

$$v_b = \frac{2n_{eff}V_a}{\lambda} \quad 2-7$$

Strain and temperature perturbations to the optical fiber will shift the frequency of the Brillouin scatter. This shift predominately arises from the change in acoustic velocity as a result of the density of the fiber core being modulated. The shift of the Brillouin frequency has a linear relationship with applied strain according to Equation 2-8 (Horiguchi et al. 1989).

$$v_b(\varepsilon) = v_b(0) + C_{\varepsilon}\Delta\varepsilon \quad 2-8$$

A strain constant ($C_{\varepsilon}$) of 4.4 is regularly quoted for silica optical fibers and has been found to not vary significantly for various material compositions (Shibata et al. 1988). As Brillouin scattering occurs continuously along the length of an interrogated optical fiber, a distributed sensor can be realized by measuring the unperturbed Brillouin frequency and comparing to the Brillouin response at a later time. The Brillouin frequency is resolved spatially along the fiber by monitoring the return time of the signal and knowing the speed of light in the optical fiber (i.e. OTDR).

Monitoring the spontaneous Brillouin response (i.e. BOTDR) allows strain to be measured over kilometer lengths of optical fiber (Kurashima et al. 1989; Shimizu et al. 1993); although, the low-level detected signal (i.e. the Brillouin frequency) limits the spatial resolution to over one meter, even though Brillouin scattering occurs continuously along the fiber. This also constrains the accuracy of the strain measurement and acquisition rate, especially when long sensing lengths are considered. However, the low-
level signal response may be overcome by stimulating the Brillouin scattering process, which in return, amplifies the signal. BOTDA (Horiguchi & Tateda 1989a; Niklès et al. 1996) stimulates acoustic waves in an optical fiber sensor by injecting two counter propagating waves. This requires access to both ends of the optical fiber in order to launch both a pulsed signal (i.e. a pump laser) and a tunable counter propagating continuous wave (i.e. a probe laser), Figure 2-4. When the frequency difference between the pump and probe signal is equal to the Brillouin frequency, Equation 2-7, a resonant condition will be established and Brillouin scattering will be stimulated at the respective position along the optical fiber. The amplified signal, from the probe, carries local strain information (i.e. Equation 2-8) back to a receiver at a comparatively stronger signal than the BOTDR technique while also carrying time domain information from the pulsed signal (Horiguchi & Tateda 1989b; Bao et al. 1994). The amplified signal allows the BOTDA technique to measure strain at comparatively better spatial resolution, accuracy, and length than the spontaneous counterpart (i.e. BOTDR). Two commercially available BOTDA systems are the Neubrexcope-6000 (Zhang & Wu 2007) and the DITEST STA-R (Omnisens 2014). The former is capable of measuring strain at a spatial resolution of ten centimeters over a maximum optical fiber length of one kilometer and accuracy of ±25 microstrain. The latter is capable of a strain accuracy of ±1 microstrain, although this in turn reduces the maximum sensing length to 50 meters and the spatial resolution to half a meter. Alternative technologies that incorporate Brillouin Optical Correlation Domain Analysis (BOCDA) or Brillouin Optical Correlation Domain Reflectometry (BOCDR) have demonstrated 70 millimeter spatial resolution at 1 kilometer sensing.

Figure 2-4: Schematic operation of Brillouin optical time domain analysis (after Zhang & Wu 2012).
length (Hotate et al. 2008), 1.6 millimeter spatial resolution at 10 meter sensing length (Song et al. 2006) and, sampling rates of 5 kHz (Hotate et al. 2012); however, these technologies are still under development, with a limited number of prototype units that have field tested (e.g. Saito et al. 2014).

2.2.2.2 Rayleigh Optical Frequency Domain Reflectometry

Rayleigh scattering is a spontaneous loss mechanism arising from random fluctuations of the refractive index fused into the silica core of an optical fiber from the manufacturing process. This is not to be confused with intentional index variations as discussed for the FBG technique. But similar to FBG, local strain and temperature change to the optical fiber will lead to an alteration of the local refractive index, and therefore, the Rayleigh scatter signature. Unlike Brillouin scattering, Rayleigh scattering is elastic, resulting in virtually no frequency change when comparing the incident light and light scattered via the Rayleigh mechanism. Techniques implementing OTDR to capture Rayleigh scattering therefore require high powered (costly) lasers and long acquisition times to obtain comparable sensing lengths, accuracies, and spatial resolutions as the before mentioned Brillouin techniques (Lu et al. 2010). However, Froggatt et al. (1998) have discussed the potential of using the Rayleigh scatter frequency response (i.e. OFDR) to measure strain with an interferometric technique which compares the path length difference between a measurement and reference arm (Figure 2-5).

Rayleigh optical frequency domain reflectometry (ROFDR) monitors the amplitude and phase of Rayleigh scatter as a laser source is spectrally tuned through a range of frequencies. This provides a

![Figure 2-5: Example optical network with a sensor arm added to a Mach-Zehnder interferometer to interrogate strain along an optical fiber (i.e. DUT) (Soller et al. 2005).]
description of the Rayleigh scatter profile in the frequency domain over many sub-millimeter sections along the given optical fiber. Using a Fourier transform, the discretized sections of optical data are converted to the time domain, which allows the physical location of local scatter to be determined according to the time of flight of light in the optical fiber. The ambient Rayleigh signature is stable and unique to a given optical fiber (Froggatt et al. 2004). This is stored as a reference measurement or state. The spectral shift associated with strain or temperature change at a later point is determined through a cross-correlation of the reference and perturbed state (Froggatt and Moore 1998). Local strain will be manifested as a shift in the cross-correlation peak (Δλ) according to Equation 2-9.

\[ \frac{\Delta \lambda}{\lambda} = K_e \varepsilon \]  

2-9

For silica optical fibers, a strain calibration constant of 0.78 can be used; although, this may vary by approximately ten percent depending on the optical fiber composition (Kreger et al. 2007). ROFDR can achieve spatial resolutions under one millimeter in addition to comparable strain accuracies as the FBG and Brillouin-based techniques. However, the maximum sensing length is limited to 40 meters due to system noise associated with the laser and the low reflected power of the Rayleigh signal.

2.2.2.3 Distributed Fiber Bragg Gratings

ROFDR utilizes a standard (i.e. unmodified) single mode optical fiber and measures low-level scatter associated with inhomogeneities in the index profile of the fiber. However, backscattered/reflected light that is intentionally induced by writing fiber Bragg gratings into the core of the optical fiber can also be measured using OFDR as the demodulation technique (e.g. Kreger et al. 2013), essentially providing a distributed fiber Bragg grating solution (DFBG). This results in a significantly more powerful measured signal which allows long lead lengths (km scale), ± 1 microstrain accuracy, and fast acquisition rates (250 Hz). But this also requires fiber Bragg gratings to be continuously written along the length of the optical fiber, compromising the spatial resolution (> 6.35mm) and the sensor cost.
Table 2-1: Influential summary: Operational features/capabilities and pricing for wavelength division multiplexing fiber Bragg gratings (FBG), quasi-distributed FBG (DFBG), Brillouin based distributed sensing (BOTDR/BOTDA), and Rayleigh based distributed sensing (ROFDR) techniques.

<table>
<thead>
<tr>
<th></th>
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</tr>
</thead>
<tbody>
<tr>
<td>Max. sensing length</td>
<td>&gt; 1000 m</td>
<td>&lt; 52 m</td>
<td>&gt; 1000 m</td>
<td>&lt; 40 m</td>
</tr>
<tr>
<td>Measurement repeatability*</td>
<td>± 0.1-10 µε</td>
<td>± 1 µε</td>
<td>± 1 µε</td>
<td>± 5 µε</td>
</tr>
<tr>
<td>Spacing of measurements (i.e. spatial resolution)</td>
<td>0.10 m (practically)</td>
<td>6.35 mm</td>
<td>0.10 – 1 m</td>
<td>0.65 mm</td>
</tr>
<tr>
<td>Max. number of measurement points</td>
<td>10 – 20 (practically)</td>
<td>&gt; 1000</td>
<td>&gt; 1000</td>
<td>&gt; 1000</td>
</tr>
<tr>
<td>Sensing range</td>
<td>± 17,500 µε</td>
<td>± 30,000 µε</td>
<td>± 30,000 µε</td>
<td>± 30,000 µε</td>
</tr>
<tr>
<td>Acquisition time</td>
<td>&lt; 1000 Hz</td>
<td>&lt; 250 Hz</td>
<td>&lt; 1 Hz</td>
<td>&lt; 60 Hz</td>
</tr>
<tr>
<td>Unit price (approximate USD)</td>
<td>$15,000 – $125,000</td>
<td>&gt; $70,000 - $125,000</td>
<td>$100,000 - $250,000</td>
<td>$60,000 – $150,000</td>
</tr>
<tr>
<td>Sensor price (approximate USD)</td>
<td>~ $300 - $1000 per sensor</td>
<td>~ $300 - $5000 per sensor</td>
<td>$0.10 per meter of fiber</td>
<td>$0.10 per meter of fiber</td>
</tr>
<tr>
<td>Max. number of connected sensors**</td>
<td>&gt; 10</td>
<td>8</td>
<td>2</td>
<td>1</td>
</tr>
</tbody>
</table>

* Repeatability will ultimately be related to the level of strain experienced by the optical sensor. At higher strain levels (> 10000 µε) repeatability will decrease.

** Maximum number of connected sensors without the purchase of an additional switch unit.
2.2.3 Applicability to Ground Support

The working principles of commercially available FBG, BOTDR/BOTDA, ROFDR, and DFBG units have been discussed in order to provide a rationale for selecting the most applicable FOS technique to measure strain of ground support members. Table 2-1 provides a summary of operational features for each FOS technique.

An immediate consideration is the optical fiber to be used as the sensor. All FOS techniques utilized a single mode optical fiber as the transducer and lead; however, the FBG and DFBG techniques require additional manufacturing complexities to permanently inscribe Bragg gratings into the optical fiber core. This brings forth two distinct issues: i) the sensor price will be significantly more expensive than a standard optical fiber and ii) the sensors must be individually addressed and designed per order (i.e. spacing of Bragg gratings). Yet, the most substantial consideration is the choice between FBG and the DOS techniques. FBG provides a discrete sensing solution with a limited number of practical sensors per optical fiber. Therefore, FBG will be susceptible to the same spatial resolution concerns as electrical, discrete sensing techniques. For this reason, a DOS solution is preferred.

An apparent compromise exists between spatial resolution, accuracy of strain measurements, and maximum sensing length. This compromise is also apparent when comparing the Brillouin-based and Rayleigh-based (i.e. DFBG, ROFDR) DOS techniques. Both sensing techniques can achieve relatively similar strain accuracies, but BOTDR/BOTDA can monitor significantly longer lengths while ROFDR and DFBG can monitor finer spatial resolutions. A decision between Brillouin-based and Rayleigh-based DOS must, therefore, be based upon the fundamental requirements of an ideal support member monitoring technique. Previous efforts have indicated that the most pertinent shortcomings of conventional instrumentation have been in regard to the increased costs and manufacturing difficulties associated with providing an adequate spatial resolution to capture the geomechanistic response of a given support member. The length of the rock bolt, umbrella arch, and cable bolt support members considered in this research are often between one to ten meters long. Accordingly, the sensing length limitations of the Rayleigh-based DOS techniques are nullified and the superior spatial resolution is of advantage to measure localized
behaviour. However, it should be noted that the Brillouin-based DOS techniques are certainly of intrigue to larger geo-monitoring projects, including: slope stability (e.g. Shi et al. 2006), tunnel deformations (e.g. Moffat et al. 2015), geotextile performance (e.g. Habel and Krebber 2011), and pipeline monitoring (e.g. Inaudi 2005).

Within the context of monitoring ground support members, the choice between DFBG and ROFDR is primarily dictated by measurement rate and lead requirements. The induced (i.e. amplified) Rayleigh signature provided by the DFBG technique allows significantly faster measurement rate and longer lead lengths to be used, but also requires a much more expensive optical fiber segment for the transducer. The initial verification experiments proposed in this research effort will consider controlled laboratory conditions and relatively static loading increments. In addition, it is anticipated that an abundance of optical sensors will be required to design such a technique. Cost per sensor is, therefore, perhaps the most burdening contrast between the two techniques. Accordingly, ROFDR has been selected. The sub-millimeter spatial resolution measured with the ROFDR technique also provides the best potential to both identify and capture local and micro-scale ground support response mechanisms. However, the ability to only measure a single optical sensor does provide a drawback to the selected technique. The assessment of ROFDR for monitoring ground support members must, therefore, identify if the 0.65 millimeter spatial resolution is of significant benefit as to outweigh limitations of the technique.

2.3 Application of DOS to Ground Support Members
The design and application of a technique that couples DOS with ground support members is a non-trivial undertaking. Conventionally, optical fiber design has been aimed at providing the best protection from external influences (i.e. strain and temperature) in order to optimize signal transmission. In contrast, FOS requires loading of the specimen of interest to be directly transferred to the optical fiber. Regarding ground support, this requires a strong bond between the active sensing length of optical fiber cable and the given support member. The general construction of a single mode optical fiber cable (Figure 2-6) consists of a high-quality fused silica core (9 micrometers) and cladding (125 micrometers), a protective coating (250
micrometers), buffer (900 micrometers), strengthening yarn, and cable jacket (>1 millimeter). The buffer, strengthening yarn, and cable jacket are not directly bonded to the optical fiber core-cladding, and therefore inhibit strain transfer. Conversely, the protective coating is often applied directly to the core-cladding during the optical fiber drawing process, physically bonding it.

![Diagram of optical fiber cable profile](image)

**Figure 2-6: Typical optical fiber cable profile. Note the core and cladding assembly is roughly the diameter of human hair**

Depending on the application, ground support members can be expected to experience strains well over 1% (i.e. in excess of 10,000 microstrain); however, the scope of this research is to validate the technique primarily within the elastic response of steel support members (i.e. < 2500 microstrain) under a controlled laboratory setting. For this reason, the active optical sensing length will only make use of the core/cladding/protective coating assembly, although dampening the strain transfer through additional protective layers may have potential for use with yielding support members (e.g. Li 2012). Within this context, the goal is to have the optical fiber sensor strain directly with the given support member. Regarding the selected ROFDR technique, an optical sensor constitutes an optical fiber length that has been terminated with a connector and a non-reflective termination at opposing ends. The discussions herein have considered a lucent connector (i.e. LC) with an angled polish finish (APC) and a bend insensitive optical fiber as
recommended by Luna Innovations Inc. (2017). An acrylate protective coating has also been selected for its ideal strain transfer properties, ease of stripping (for optical splicing of termination segments), and its low cost. It has been noted that a more costly polyimide protective coating is preferred for long period tests (> 1 year) and diverse temperature ranges (Inaudi et al. 1996) but this does not fall within the scope of the research presented herein. However, the technique developed using an acrylate protective coating can be directly transferred to a polyimide coated optical fiber, which has been identified to provide a slightly improved strain transfer from the coating to the core (e.g. Inaudi et al. 1996; Regier 2013).

Three support member typologies have been considered in developing the FOS technique: 1) Rock bolt support, 2) Umbrella arch support, and 3) Cable bolt support. The physical geometries, installation procedures, and assumed loading behaviour of these support members are believed to provide an ideal scheme for transferring the technique to other support members. The methods taken to bond, protect, and validate/calibrate the FOS technique for these support members are discussed within this section.

2.3.1 Rock Bolt Support
Rock bolting is a very common technique used to support excavations in rock in both mining and civil applications. The element itself normally constitutes a solid or tube formed steel member that is inserted into a borehole, coupled to the rock mass using either a mechanical expansion anchor or encapsulating cementitious/resin grout, and potentially fastened to the excavation surface with the use of nut and face plate assembly. The steel element may be installed untensioned or tensioned within the rock mass (Stillborg 1994). There are many rock bolting variations that can be deployed in a given project, but the support type is perhaps most associated with the use of fully grouted steel rebar (i.e. a solid steel rebar that is fully encapsulated within a rock borehole).

There are two general methods that have been considered to couple either conventional strain sensors or FOS with rock bolt support, these being: i) External coupling (e.g. Farmer 1975; Schroeck et al. 2000) and ii) Internal coupling (e.g. Serbousek and Signer, 1987; Iten and Puzrin 2010). The former approach involves surface mounting the sensor to the rock bolt (e.g. ASTM E1237-93) and, consequently,
leaving the sensor and leads exposed. Surface mounting could possibly withstand controlled laboratory testing, but the preferred solution should have the potential to be furthered for in-situ use. An internal solution, whereby the sensor is confined within the rock bolt, is preferred, but this solution must not significantly alter the capacity or behaviour of the rock bolt itself. Using a standard #6 grade 60 rebar (i.e. 19.05 millimeter diameter, 120 kN minimum yield) this has been approached by machining out a 2.5 by 2.5 millimeter lengthwise groove to embed and encapsulate an optical fiber with a proprietary adhesive. The groove dimension provides ample space for the optical fiber to be situated below the exterior profile of the rebar, but more importantly provides a protective barrier once encapsulated. However, care must be taken to run the optical fiber straight within the groove to avoid potential orientation difficulties if more than one groove is to be analyzed (Chapter 3 - Forbes et al. 2017). A comparison between several different bonding agents has been discussed in Forbes (2015).

The rebar members utilized in this research were modified with a pair of diametrically opposed grooves. A single optical fiber sensor was used to monitor both grooves (roughly four percent reduction in cross-section area) by looping the sensor within a machined groove that connected the diametrically opposed grooves at one end of the rebar (Figure 2-7). This provides both a redundancy measure and a method to compensate for bending induced strain (Hyett et al. 2013; Forbes et al. 2017). Validation of this FOS technique was accomplished by conducting an end loaded, elastic tensile test with an 800 millimeter testing span using a 500 kN servo-controlled loading frame. In addition to the ROFDR, electrical resistive strain gauge pairs (i.e. on opposing sides of the rebar) were surface mounted every 160 millimeters along

**Figure 2-7**: Diametrically opposed 2.5 by 2.5 millimeter machined grooves running lengthwise along a rebar specimen. Right – Schematic representation of the optical sensor.
the testing span. A comparison of the averaged strain at various levels of applied load is presented in Figure 2-8. This is the bending compensated strain, or coaxial strain ($\varepsilon_{coaxial}$), according to Equation 2-10.

$$\varepsilon_{coaxial,i} = \frac{\varepsilon_{i}^{\text{sensing length 1}} + \varepsilon_{i}^{\text{sensing length 2}}}{2}$$

The ROFDR provides a strain profile (i.e. strain distribution along the rebar) at each loading increment as strain is measured at a spatial resolution of 0.65 millimeters. Conversely, each electrical strain gauge provides a single measurement point over its three millimeter gauge length. The difference in measured strain between the two techniques did not exceed 5.0% in a total of three validation tests. In addition, the strain measured with ROFDR was within 3.5% of the theoretical rebar strain value at each load increment.

Figure 2-8: Rebar tensile experiment. Left – Testing apparatus (MTS 810 loading frame). Right – Comparison of rebar strain measured using ROFDR and electrical resistive strain gauges. Strain gauge positions and measurements are indicated by square symbols. Note: Tensile strain is taken positive.
2.3.2 Umbrella Arch Support

The umbrella arch (UA) is a temporary support system forming a structural umbrella around an excavation from the insertion of longitudinal support members installed from within the tunnel, above and around the crown of the tunnel face (Oke et al. 2014a). The UA is often considered to be a pre-support technique as the support members are installed prior to the first pass of the excavation. According to the nomenclature developed by Oke et al. (2014b), the longitudinal support members can be distinguished into three main support element categories:

1) Forepoles: element length greater than the height of the excavation, installed at shallow angles to the tunnel axis (commonly a 114 millimeter diameter steel pipe);

2) Spiles: element length smaller than the height of the excavation, primarily installed to control structurally driven failure (commonly a sub-30 millimeter diameter steel bar), and;

3) Grouting elements.

From these categories, the forepole element has been selected as the focus for the UA FOS technique. A standard 114.3 millimeter outer diameter steel (ASTM A53 Gr. B) pipe with a 6.02 millimeter wall thickness was chosen for the forepole element in this research (a commonly used pipe size in industry). The hollow cross section of this steel member adds an inherent difficulty to the FOS technique. Considering the aforementioned rock bolt technique, a 2.5 millimeter deep groove is approximately 41.5% the thickness of the pipe and would notably impact the strength capacity. Additionally, the hollow cross-section prohibits the optical fiber from being looped at one end while still residing below the exterior profile of the pipe. Therefore, to conduct an initial proof of concept it was chosen to surface mount (ASTM E1237-93) the optical sensor to exterior pipe surface along a single length. Surface mounting to the interior pipe profile was considered (e.g. O’Looney 2009; Vishay Precision Group 2010; Landers and Philips 2014), but the reduced accessibility was believed to hinder the reliability in bonding the optical sensor and, therefore, in assessing the merit of the ROFDR technique. Furthermore, an interior surface mounting technique is not
believed to provide a feasible solution for future in-situ development as forepole members are predominately installed as a self-drilled member (i.e. installed with a sacrificial drilling-bit and requiring the hollow interior for debris-flow, water, and grout). Interior surface mounting is, therefore, considered to be an unnecessary complication.

In conducting an initial validation of this optical technique, it was chosen to laterally load a forepole specimen under a symmetric bending configuration using a 500 kN servo-controlled loading frame. Bending provides the most controlled method for establishing the baseline accuracy of the optical technique with the forepole element as comparatively low loads are required to deflect, rather than coaxially stretch, the steel member. As displayed in Figure 2-9, load was applied to the forepole at the center point between two roller supports, spaced approximately 1900 millimeters apart. The optical sensor was aligned along the top axis of the forepole member in order to measure compressive strain (as the rock bolt validation experiment measured tension) in addition to six surface mounted electrical resistive strain gauges. The

Figure 2-9: Forepole symmetric bending experiment. Left – Testing apparatus (MTS 324 loading frame). Right – Comparison of the forepole strain measured using the ROFDR technique, electrical resistive strain gauges, and the forepole strain predicted by Euler-Bernoulli beam theory. Note: Compressive strain is taken negative.
sensor alignment necessitated a platen piece to apply load to the forepole member, as to avoid directly loading the optical sensor itself. The plot in Figure 2-9 displays a comparison between the strain measured using the ROFDR technique, electrical resistive strain gauges, and theoretical strain distribution according to Euler-Bernoulli beam theory. Likewise to the initial rock bolt test, the difference in strain measured (or predicted) differed by less than 5.0%. In addition, the influence of using a platen piece at the loading position was discernable in the ROFDR strain distribution and presented the largest source of discrepancy between the three methods.

2.3.3 Cable Bolt Support
A cable bolt support element normally consists of a seven-steel wire strand (i.e. six steel wires layered in a helical configuration around a central steel wire) that is installed and cement grouted within borehole and used to support a given excavation similar to the aforementioned rock bolting technique. The helical composition of the cable bolt gives the element a significantly lower torsional and flexural rigidity compared to a solid bar or tube element of the same nominal diameter and, therefore, allows the element to be placed on a reel after the manufacturing process. This allows for long lengths of this support member (i.e. in excess of 10 m) to be easily transported and installed even within small excavation confinements. However, this helical composition also provides an additional complexity in applying the optical sensor. Choquet and Miller (1988) have discussed an external tension measurement technique where spiral resistance wire is wound into a flute (i.e. the notch between adjacent strands at the exterior profile) of the cable bolt, but this would require extensive protection measures to replace with an optical sensor (which also dampens strain transfer). Additionally, the external profile of the cable bolt (i.e. the flute geometry) has been noted to be very influential on the support member’s behaviour and capacity (Hyett et al. 1992). Therefore, it is preferred to couple the optical sensor within the central, straight, steel wire in a similar technique as discussed for discrete displacement sensors by Hyett et al. (1997). This considers replacing the central wire of the cable bolt with a hollow tube of the same diameter to house the optical sensor. Load is transferred to the central tube via a frictional relationship with the outer six wires. Considering FOS, this
was approached by first straightening (by applying a slight tension) and centering an optical fiber within a stainless steel tube and then fully encapsulating the optical fiber using a two-part epoxy resin. Once cured, the given cable bolt is opened to remove the central wire and then re-wound with the optically instrumented tube (Figure 2-10). The tests discussed within this research have considered a 15.2 millimeter nominal diameter low relaxation steel strand (ASTM A416).

The central wire of a cable bolt is shorter than the wrapped, outer six wires and, therefore, will take on more strain per displacement of the surrounding rock mass. This necessitates a calibration of the measured strain along the central optically instrumented tube with the load of the full cable bolt. A tensile test arrangement using the same loading frame discussed for the rock bolt validation test (Fig. 8) was selected for the calibration, but with minor adjustments to accommodate standard strand testing protocols (ASTM A1061). The test was conducted on both a standard cable bolt and a FOS cable bolt. Actuator load and displacement were measured for both cable bolt types in addition to strain being measured with the optical sensor for the FOS cable bolt. A comparison of the load-strain relationships for the standard cable bolt and the FOS cable bolt using both the actuator displacement and optically measured strain is displayed in Figure 2-11. Below half a percent strain (i.e. 0.5% strain) of the central wire/tube, the standard cable bolt and FOS cable bolt load-strain trend differs by approximately 1.05%. This implies that initial, elastic behaviour of the strand is not significantly impacted upon by the replacement of the central wire. Above half a percent strain the load-strain relationship between the two cable bolts begins to deviate, concurrent with a notable change in the slope of the profiles. This slope change corresponds with a torsional resistance limitation of the selected loading frame. At approximately half a percent strain for both cable bolt types, the hydraulic gripper assembly (see Fig. 8) began to rotate with increased applied load. This allowed the outer six wires to unwind, providing a less stiff response. For this reason, it was selected to calibrate the FOS technique using the initial, half a percent strain loading region. Comparing the three profiles, a load to measured strain relationship of approximately 0.020 kN/\(\mu\)e was obtained.
2.4 Laboratory Results

A comprehensive testing program has been performed to assess the merits of the FOS technique. The program consisted of subjecting optically instrumented rock bolts, forepoles, and cable bolts to expected...
in-situ loading behaviour through various laboratory arrangements. It should be noted that the intention of this experimental program is to demonstrate the capability of the FOS technique to capture expected support member behaviour, rather than conduct an intensive study on one particular mechanism. Within this context, the unprecedented insight provided by the FOS technique is discussed.

2.4.1 Rock Bolt Experiments

Fully grouted rock bolts (more specifically steel rebar) are predominately passive support elements. This implies that the majority of the bolt’s support capacity is mobilized as a result of subsequent rock mass movements. Therefore, the load/strain profile along the rock bolt will be controlled by the distribution of rock mass displacements once installed. In jointed and fractured rock masses, the rock bolt load distribution will most commonly be reflective of a number of localized discontinuity movements (Björn fot and Stephansson 1983; Hyett et al. 1996; Li and Stillborg 1999), which may act coaxial and/or transverse to the

Figure 2-11: Load (full cable bolt) versus percent strain (central wire/tube) comparison between a standard cable bolt member (Standard cable), FOS cable bolt (FOS cable), and strain measured using the FOS technique (DOS).
bolt axis. In simulating this loading behaviour in the laboratory, two experimental arrangements were conducted: i) Coaxial loading, and ii) Double shear plane loading. The behaviour of the latter has been particularly difficult to monitor with conventional strain techniques due to the locality of its effect on the bolt; often within 2.5-6.25 bolt diameters from the intersection of the bolt with the shearing plane (Ferrero 1995; McHugh and Signer, 1999; Aziz et al. 2005; Grasselli 2005).

2.4.1.1 Coaxial Loading

The most common in-situ assessment of fully grouted support is perhaps the coaxial pull-out test (ISRM 1974; ISRM 1984; ASTM D4435-13e1). This test consists of installing a rock bolt in agreement with the normal operational procedures at a given site and subsequently applying a coaxial load to the grouted support member from within the excavation. The pull-test test can be replicated under controlled laboratory conditions by replacing the host rock mass with a metal pipe, simulated rock mass (e.g. concrete), or a cored rock sample, providing a constant radial stiffness boundary condition. Accordingly, the assessment of the FOS rock bolt technique considered concrete (40 MPa UCS) block specimens (200 x 300 x 300 millimeters) and cylinder specimens (200 millimeter length by 300 millimeter diameter) with a preformed 31 millimeter diameter borehole. After allowing 28 days to for the concrete to cure, the preformed boreholes were roughened and cleaned using a rotary drill. The FOS rebar was then centered within the borehole and encapsulated with a polyester resin grout (i.e. a 200 millimeter grouted length). The grouted rebar specimen was restrained to a servo-controlled loading frame which was also used to apply coaxial load at a position 900 millimeters along the rebar from the concrete specimen. On the opposing side of the concrete block, the rebar was extended by approximately 100 millimeters to allow potential slip of the rebar to be measured during the experiment. The coaxial loading apparatus is displayed in Figure 2-12.

Figure 2-13 displays the bending compensated strain measured along the rebar at 20 kN load increments. As discussed for the tensile validation experiment, this strain is obtained by taking the average strain from opposing sides of the rebar. The free length of rebar between the position of applied load and concrete specimen (i.e. 0.00 to 0.90 meters) and the grouted segment of rebar (i.e. 0.90 to 1.10 meters) are
clearly distinguishable along the strain distribution. The former is fundamentally an end loaded coaxial arrangement and, therefore, depicts a uniform level of strain at the given loading increment. This agrees well the tensile validation experiment and the theoretical strain magnitude. This uniform strain is observed to decay in an exponential form within the grouted section towards the unloaded end as discussed by Farmer (1975), Serbousek and Signer (1987), and Li and Stillborg (1999). But more remarkably, a detailed examination of the grouted segment (Figure 2-14) appears to provide a correlation between periodic disturbances in the strain profile and the spacing of the rebar ribs (roughly spaced by 18.7 millimeters).

Figure 2-12: Coaxial rock bolt loading arrangement. The optically instrumented rebar is cement grouted within a confining concrete cylinder. Coaxial load is applied to the rebar while the concrete cylinder is restrained (MTS 810 loading frame). Electrical resistive strain gauges and linear-variable-displacement-transformers are used to monitor external apparatus displacements.
This would indicate that the FOS technique is capable of distinguishing the additional resistance provided by the individual rebar ribs within the grout (i.e. micro-scale geomechanisms). Such an effect has been discussed both analytically (e.g. Coa et al. 2014) and numerically (e.g. Jalalifar 2006), but never captured to such an extent.

Figure 2-13: Rock bolt coaxial loading results. Upper – Experimental arrangement. Lower – Averaged strain profile taken along the rebar at various levels of applied load.
Shear displacement of a rock bolted discontinuity will rarely result in a direct guillotining of the steel member (Stillborg 1994). Instead, the shear response of a grouted rock bolt will more commonly take on a combination of coaxial and bending induced loads local to the discontinuity. This is often referred to as the “dowel” reinforcement effect (Spang and Egger 1990; Ferrero 1995; Grasselli 2005; Li et al. 2016a). In testing the FOS technique under such conditions, a double shear plane experiment was constructed. This

Figure 2-14: Detailed view of the strain distribution along the grouted segment of rebars at 70 kN of applied coaxial load.
consisted of centering and resin grouting an optically instrumented rebar into a 31 millimeter diameter reamed borehole which was precast through three individual concrete blocks (240 x 300 x 300 millimeters block dimensioning, 40 MPa UCS). The outer two blocks of this apparatus were restrained while a vertical load was applied to the central block using a servo-controlled 500 kN loading frame. A five millimeter thick nylon sheet was also placed between the blocks during the casting stage in order to promote a frictionless and non-dilating surface for shear displacement. The result is two vertical shear planes acting along the bolt, providing a much less demanding apparatus to restrain in comparison to a single shear plane test (Aziz et al. 2003). The experimental apparatus as well the strain measured along the entirety of the optical sensor (i.e. both opposing sides of the rebar) are displayed in Figure 2-15.

Figure 2-15: Double shear plane experiment. Left – Loading apparatus (MTS 810 loading frame). Right – Strain measured along the entirety of the optical sensor (i.e. both opposing sides) at various levels of applied load (i.e. causing displacement of the central block). Note: The rebar has been orientated such that the optical sensing lengths are situated along the top and bottom alignment of the rebar.
At each discontinuity (i.e. shear plane) a distinct shear couplet is measured. This corresponds to a pair of extensile (positive) and compressive (negative) strain concentrations that mirror at the shear displacement plane. However, the absolute strain magnitude of the compressive arm is less in each case. A possible explanation for this is the additional resistance provided by the grout on the compressed side of the shear plane and the relatively negligible adhesional resistance provided by the grout on the tensile side. This agrees well with previous research efforts where the dowel-like response has been noted to result in “plastic-hinges” or an “S-bend” of the rock bolt at each discontinuity (e.g. Spang and Egger 1990; McHugh and Signer 1999; Aziz et al. 2005).

In this experimental setup, the strain appears to decay within 150 millimeters from the intersection with the discontinuity; although, the peak strain is concentrated within 50 millimeters. This implies that a discrete sensing solution would require a fine concentration of sensors within a close proximity of the discontinuity to detect the event. This is certainly obtainable under controlled laboratory conditions, but in reality, the precise location of such load inducing discontinuities will not be known beforehand. Therefore, it was chosen to compare the strain distribution measured with the FOS technique against an interpolated strain distribution obtained from electrical resistive strain gauges spaced at 250 millimeters increments (a very fine spatial resolution for such instrumentation in practice) along the same rebar specimen. It was also selected to compare the 0.65 millimeter spatial resolution measured with ROFDR against hypothetical strain profiles obtainable with the other discussed FOS techniques by sampling the measured data in three ways: i) Taking one measured value from the ROFDR profile every 100 millimeters (equivalent to the measurement that would be provided by an FBG solution), ii) Taking an averaged value over 100 millimeters from the measured ROFDR profile (equivalent to the measurement that would be provided by BOTDR/BOTDA) and, iii) Taking an averaged value over 10 millimeters from the measured ROFDR profile (3.6 millimeters coarser than that which would be provided by DFBG). The strain distributions measured along the top alignment of the rebar for both comparisons are displayed in Figure 2-16.
Figure 2-16: Comparison of the strain profile measured along the top alignment of the rebar. Upper plot – Strain profile measured using ROFDR (i.e. optical) and the interpolated strain profile from electrical resistive strain gauges positioned at 250 millimeter increments at 50 kN and 200 kN of applied load (i.e. causing displacement of the central block). Lower plot – Strain profile measured using ROFDR at various spatial resolution samplings at 200 kN of applied load (i.e. causing displacement of the central block).
Comparing both the FOS technique and the strain gauge measurements, it is apparent that the coarse spatial resolution provided by the discrete sensing technique does not sufficiently capture the shearing mechanism. Furthermore, it suggests that it is completely fortuitous for such a discrete sensing technique to capture localized, discontinuous behaviour in-situ. Drastically different strain profiles are also obtained for the two samplings at a 100 millimeter spatial resolution. At a 10 millimeter spatial resolution, the shearing mechanism is evident and length of rebar influenced by the shearing displacement compares well with the ROFDR measurement, although the peak strain magnitude along the rebar is slightly underestimated. This suggests that the DFBG technique would provide an adequate spatial resolution to measure localized support behaviour if the ROFDR technique’s limitations regarding the number of connected sensors and/or lead lengths prohibit its use.

2.4.2 Umbrella Arch Experiments

Forepole support members are passive elements activated by movements of the surrounding ground mass. Their primary support contribution involves the longitudinal transfer of load away from the region at and directly behind the active excavation face (i.e. the unsupported span). This is accomplished through bending of the forepole member, which will be founded on a stiff steel-set / concrete lining at both ends, or steel-set / concrete lining and ground at the active excavation face depending on the construction stage. In this regard, forepole members will act as a multi-span beam for a majority of their serviceability life (John and Mattle 2002). The geotechnical and engineering properties of the ground mass as well as the steel-set / concrete lining will play a major role in the support contribution of the forepole member (Volkmann and Schubert 2007); however, this multi-span bending can be idealized as a symmetric bending experiment, as discussed for the UA validation experiment. Within this context, the loading arrangement used in the validation test of the UA FOS technique was modified to accommodate bending spans ranging from 0.50 to 3.00 meters (i.e. potential excavation advance lengths). A normalized plot of the strain profiles measured along the top alignment of various testing spans under 15 kN of applied load is presented in Figure 2-17 (left).
Euler-Bernoulli beam theory predicts a linear “V-shaped” strain profile for a point loaded
symmetric bending span. This strain profile corresponds to a parabola-like deflection profile of the forepole member. For the tested support spans that were greater than 1.50 meters, the measured strain profile agreed well with this theorized linear strain distribution. However, for lesser length support spans the measured strain profile was found to deviate from the expected linear response and instead concentrate within the region of applied load. This non-linear trend intensifies with decreased testing span, and in given circumstances, resulted in measured strains nearly twice than that expected from linear theory. A possible explanation for this non-linear response of the steel pipe is the natural tendency towards flexural buckling or non-linear elliptical behaviour (Timoshenko and Goodier 1951). Referring to Figure 2-17 (right), this material response was further investigated through a 1250 millimeter span bending experiment with the optical sensor positioned along both the top (i.e. compressed) and bottom (i.e. tensile) alignment of the forepole member. Quite interestingly, the strain measured along the bottom alignment of the forepole member was both non-linear and less than that predicted by linear theory. Furthermore, the difference between the predicted linear and measured non-linear profile was most at the position of applied load, and correspondingly, the position of strain concentration along the top alignment of the forepole member. But, taking an absolute average of the strain measured along the top and bottom alignments of the forepole member, a strain profile nearly identical to that predicted by linear theory was obtained. It is also important to note that this non-linear response was responsible for premature yielding along the compressed alignment of the forepole member.

2.4.3 Cable Bolt Experiments
Similar to rock bolting techniques, the coaxial pull-test is commonly performed to assess cable bolt support capacity in both mining and civil excavations. This was approached under controlled laboratory settings by replacing the host ground mass with a 1500 millimeter long, 50 millimeter nominal diameter schedule 80 aluminum pipe (i.e. constant radial stiffness boundary condition). A FOS cable bolt was then centered and cement grouted (0.4 w:c ratio) within the aluminum pipe and allowed to cure for 21 days. Coaxial load was applied to the cable bolt using the same loading apparatus as discussed for the rock bolt coaxial test, shown
in Figure 2-12. The strain distributions measured using the FOS cable bolt technique for incremental applied loads of 25 kN are displayed in Figure 2-18. As discussed for the validation tests of the cable bolt FOS technique, this is the strain measured along the instrumented central tube, and should not be confused with strain of the outer wires. As discussed by both Hyett et al. (1996) and Li and Stillborg (1999), strain decays from the position of applied coaxial load towards the unloaded end of the cable bolt. However, the continuous strain distribution captured by the FOS techniques also allows the anchoring length (i.e. length of cable being strained) of the support member to be discerned. This is clearly shown to increase with increased applied coaxial load and corresponds with a non-simultaneous failure of the cable bolt to grout.

![Graph showing strain distribution](image)

**Figure 2-18**: Cable bolt coaxial pull-test results: Strain distributions measured along the grouted length of the central tube of a seven strand steel cable at various applied loads.
interface (e.g. Kaiser et al. 1992). While such a response is expected, it has been previously difficult to capture with the limited spatial resolution of conventional sensing techniques. Accordingly, many previous testing programs (e.g. Kaiser et al. 1992; Benmokrane et al. 1995) have focused on short grouted lengths (< 200 millimeters) to permit the assumption of a uniform shear stress distribution across the cable bolt to grout interface. However, short grouted length experiments may not be representative of the actual behaviour of cable bolts installed in reality (grouted lengths are rarely less than 3 meters).

The capability to measure the entire strain distribution along the cable bolt length is also very advantageous for improving the coaxial load-displacement relationship of the support member. Previous testing programs (e.g. Kaiser et al. 1992; Hyett et al., 1992) have monitored the cable bolt (or rock bolt) apparatus external to the cable bolt-grout-rock apparatus. This includes: i) Load applied to the cable bolt outside of the grouted length, ii) Displacement of the cable bolt outside of the grouted apparatus (loaded and unloaded end), and iii) Displacement of the apparatus (if present). However, the strain distribution measured with the FOS technique can readily be converted to displacement (δ) according to Equation 2-11 and Equation 2-12.

\[
\delta = \frac{\text{Start of FOS}}{\text{End of FOS}} \int (\varepsilon_{\text{coaxial}}) \Delta x
\]

\[
\delta = \frac{\Delta x}{2} \sum_{i=1}^{N} (\varepsilon_{\text{coaxial},i} + \varepsilon_{\text{coaxial},i+1})
\]

A comparison between the load-displacement relationships of the aforementioned cable bolt experiment for displacements measured from: i) the stroke of the actuator (includes stretch of the free length of strand and apparatus deflection), ii) the FOS technique (strain measured along central tube), iii) apparatus deflection, and the corrected stroke (obtained by subtracting the apparatus deflection and stretch of the free length of
Figure 2-19: Cable bolt load-displacement relationships measured from the actuator stroke (stroke), corrected actuator stroke obtained by subtracting the apparatus deflection (LVDT) from stroke (Cable stroke), and displacement obtained from the strain measured using the FOS technique (DOS). The displacement measured by the FOS technique is significantly less than that measured by the actuator stroke and even the corrected stroke, although the latter compared well with previous work (Hyett et al. 1992). This implies that previous research, where displacement has been monitored by an external sensing technique, may detail an incorrect and less stiff load-displacement relationship for cable bolt to grout interface, especially for experiments that test support elements lengths in excess of a meter. The FOS technique, therefore, has a substantial potential to improve upon current analytical and numerical cable bolt solutions through a more intensive mechanistic study.

2.5 In-situ Development

It is important to note that this research effort has exclusively focused on developing and validating the FOS technique’s capability to capture expected support member behaviour under controlled laboratory
conditions. As such, the research scope did not include protection of the lead or temperature compensation of measurements. However, these topics are not believed to require significant modification to the current rock bolt and cable bolt FOS techniques. The UA FOS technique will require a separate approach to make it suitable for the harsh installation procedures in-situ, but the current technique is believed to be necessary for calibration of a future solution. Protection for the ROFDR measurement unit will also be a necessary development. This will best be approached per project requirements (e.g. measurement duration, measurement rate, distance from sensors, site environment, power availability, etc.).

2.6 Summary
This line of research has demonstrated a novel distributed optical strain sensing technique for measuring the response of rock bolt support members, elements that are used within umbrella arches, and cable bolt support members using a selected Rayleigh optical frequency domain reflectometry technology. Highlighted within this article is the fact that not all fiber optic strain sensing technologies are similar and that the choice of technology will ultimately depend on the nature of its intended use as well as the geotechnical project/aspect that warrants such instrumentation. Within the context of improving the current understanding of ground support members, and the related micro-mechanisms, Rayleigh optical frequency domain reflectometry has been rationalized as the ideal technology, primarily as a result of the state-of-the-art spatial resolution (i.e. 0.65 millimeters) and the low-cost per optical fiber sensor. However, the limited lead length (i.e. < 50 meters) of the technology favors its use towards laboratory and more controlled in-situ studies.

Through a laboratory testing programme, the developed optical strain sensing technique has been verified to have the capability to capture expected in-situ loading mechanisms of the given support members in the form of coaxial, lateral, and shear loading arrangements. Additionally, the continuous strain distribution captured along a ground support member with the optical technique has exposed the inability of conventional, discrete sensing solutions to capture fine scale support complexities, especially under discontinuous loading conditions. In this regard, it has been found more instructive to capture the
distribution of strain along a given support member rather than a highly accurate, localized set of measurements. The encouraging results from the initial laboratory testing suggest the optical sensing technique can provide significant benefit to improving the mechanistic understanding of support response, especially within the context of a more rigorous experimental study and in-situ as a support design optimization tool.

2.7 References


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Chapter 3 A new optical sensing technique for monitoring shear of rock bolts

Nomenclature

\( \varepsilon_{\text{Total}} \) Total strain at a given segment of the bolt at the outer fiber (maximum when orthogonal to neutral axis when bending is present)

\( \varepsilon_{\text{coaxial}} \) Coaxial strain component at a given segment of the bolt

\( \varepsilon_{\text{bending moment}} \) Bending (lateral) strain component at a given segment of the bolt at the outer fiber

\( N \) Normal force

\( E \) Elastic modulus of the given bolt

\( A \) Cross-sectional area of the given bolt

\( M \) Bending moment

\( z \) Orthogonal distance from a given location on the bolt cross section to the neutral axis

\( I \) Second moment area of the given bolt

\( r \) Radius of the given bolt

\( \alpha \) Angular distance from orthogonality with the neutral axis

\( \varepsilon_i \) Total strain at a given sensing length, i

\( \theta \) Angular distance between the direction of lateral loading and first sensing length (i.e. the angular distance from orthogonality with the neutral axis)

\( \varphi \) Angular distance between the specified sensing length and first sensing length (i.e. 1 = 0°, 2 = 120°, and 3 = 240°)

\( w \) Lateral deflection of the given bolt

3.1 Introduction

Fully grouted rock bolts are a commonly used reinforcement technique in underground mining and civil projects. The bolting system consists of a steel rebar element which is inserted into a borehole, encapsulated with either a cementitious or a resin grout, and fastened to the excavation surface with the use of a nut and face plate assembly. In many cases, a low level of post-tension (~25% of the bolt’s yield capacity) is applied
coaxial with the rock bolt after installation. This tension acts to provide a level of immediate active support to the excavation surface by transferring load to more competent rock further within the rock mass. However, fully grouted rock bolts are primarily passive reinforcement elements. This implies that the majority of the bolt’s reinforcement capacity is mobilized as a result of the subsequent rock mass movements. Therefore the distribution of the rock mass displacements (continuous versus discontinuous), dictates the induced load profile along the rock bolt.

Rock bolts will routinely be installed into jointed and fractured rock masses where the bolt will not take on a continuous load distribution that has been discussed conceptually by Farmer (1975) and Freeman (1978). Instead, the load distribution will be reflective of a number of localized discontinuity movements (Björnfot & Stephansson, 1983, Hyett et al., 1996, Li & Stillborg, 1999), which may act coaxial and/or transverse to the bolt axis. In general, the rock bolt will be subjected to a combination of axial, bending, and shear loads depending on the relative orientation of the bolt and the loading. It is well established that the global strength of a jointed or blocky rock mass will be improved with the addition of fully grouted rock bolts (Fairhurst & Singh, 1974, Bjurström, 1974, Haas, 1976, Azuar, 1977, Dight, 1982, Spang & Egger, 1990), yet the support response to rock mass displacements that are non-coaxial to the bolt remains poorly understood, despite being of critical design significance. Various field investigations (e.g. Turner, 1987, Stillborg, 1994, Signer & Lewis, 1998, Li, 2010) have presented bolts subjected to in-situ bending and shearing after a failure of ground, Figure 3-1. This is often referred to as the “dowel” reinforcement effect (Spang & Egger, 1990, Ferrero, 1995, Grasselli, 2005, Li et al., 2016a) and is typically observed as an S-like or double bend of the bolt.

Hyett et al. (2013) noted that monitoring the dowel effect of a rock bolt subjected to shearing between planes is a particular challenge due the locality of its effect on the bolt; often within 2.5 – 6.25 bolt diameters from the intersection with the discontinuity (Ferrero, 1995, McHugh & Signer, 1999, Aziz et al., 2005, Grasselli, 2005). In this regard, it may not be realistic to assume that an array of conventional discrete stress and/or strain measuring techniques can be used to capture localized shearing mechanisms, especially
without a prior knowledge of discontinuity locations and corresponding movement vectors. The ability to capture the potentially complex and highly variable load distribution of a fully grouted rock bolt will, therefore, require a sensing technique with the capability to: (i) measure the reinforcement response at a fine spatial resolution, (ii) distinguish between coaxial and lateral components of reinforcement, and (iii) derive the orientation of the reinforcement vector, which in turn allows the true magnitude of reinforcement to be resolved.

This paper describes the development of an innovative rock bolt strain sensing technique using a Rayleigh based optical frequency domain reflectometer (OFDR) to measure a distributed strain profile across the entirety of a reinforcement member at a millimeter scale. The technique has been developed within the framework of providing a research and industry solution for capturing the three-dimensional response of loaded rock bolts, and most notably, the response to shear. The potential of this optical technique to accurately capture the behaviour of a fully grouted rock bolt under generalized in-situ loading conditions is demonstrated through a series of laboratory tests.
3.2 Rock Bolt Measurement Techniques

Currently, a diverse set of sensing techniques exist which can be employed for the assessment of rock mass support systems. In general, there are two manners in which a support member can be measured or monitored: 1) Extrinsically using methods such as extensometers, inclinometers, and surveying to infer support member behaviour, and, 2) Intrinsically, whereby instrumentation is coupled directly with the support member of study. In regards to a fully grouted rock bolt, the response to shear movements of the rock mass will be influenced by the many physical and interface variables of the host rock medium, the encapsulating grout, and the bolt material and typology (Haas, 1976, Azuar, 1977, Spang & Egger, 1990, Ferrero, 1995, Aziz et al., 2005, Grasselli, 2005, Jalalifar & Aziz, 2010, Chen & Li, 2014, Li et al., 2016b). Three-dimensional considerations of the bolt inclination with respect to the movements of pre-existing joints and newly formed fractures will also dictate the bolt response. A sensing technique that is not inherent to the bolt will, therefore, not be a suitable solution due to the uncertainties introduced by inferring bolt behaviour. However, there are significant challenges to an intrinsic monitoring solution as the chosen sensing technique must be coupled with the bolt, be protected, and it cannot substantially modify the bolt’s physical dimensions in order for the measurements to be meaningful. This has been approached both in controlled laboratory experiments and in-situ most commonly using discrete sensing techniques such as foil-resistance strain gauges.

Early efforts to study the mechanisms of fully grouted rock bolts considered surface mounting an array of foil-resistive strain gauges to a machined surface along the length of rebar specimens (Farmer, 1975, Karabin & Debevec 1976, Freeman, 1978). The distribution of coaxial strain along the encapsulated member could, therefore, be determined through the interpolation of discrete measurements provided by each strain gauge. Two major improvements to this sensing technique are demonstrated in Figure 3-2. This includes the consideration of strain gauge pairs on opposing sides of the rebar (Radcliffe & Stateham, 1980) and the positioning of such pairs into diametrically opposed grooves machined along the length of rebar specimens (Serbousek & Signer, 1987, Johnston & Cox, 1993). This provides additional protection for the
sensors and lead wires and also allows the interpolated strain distribution to be separated into coaxial and lateral components through a comparison of the measured strain along opposing sides of the bolt. However, the short base-length of each strain gauge results in large sections of the bolt being left unmonitored. Consequently, localized loading features (such as shear) along the bolt are prone to being underestimated and possibly omitted. This impediment has been overcome to an extent in a number of laboratory experiments studying shear by concentrating the position of strain gauges within the region of shearing (Ferrero, 1995, Mchugh & Signer, 1999, Grasselli, 2005, Jalalifar, 2006, Chen and Li, 2014). Yet, the maximum number of measurement points along the entirety of the bolt will ultimately be limited by the economic and spatial requirements of adding additional sensors. The success of such a technique to capture shear in-situ will, therefore, be contingent on the loading mechanism occurring within the location of one of these discrete measurement points on the bolt. This will similarly be true for comparable load cell techniques (e.g. Rodger et al., 1996) and methods which consider monitoring the exposed head of bolt at the excavation periphery (e.g. Mitri, 2011).

Regarding sections of the bolt remaining unmonitored, a similar technique where the short base-length resistive strain gauges have been replaced with long base-length (commonly 200mm-600mm) inductive strain gauges has been presented (Spearing et al., 2013, YieldPoint Inc., 2016). This allows for

Figure 3-2: Schematic representation of strain gauge positioning along a bolt specimen within a pair of diametrical opposed machined out grooves (Modified after Serbousek & Signer, 1987).
the entire length of a given bolt element to be monitored by discretizing it into measurement “zones” as opposed to measurement points. Yet, similar to the traditional strain gauges, the long base-length gauges may result in peak loads being underestimated or misinterpreted if loading is not uniform across the gauge length. This implies that localized shearing will in most cases be averaged out along the discrete measurement zone. Therefore, while the technique is capable of monitoring the entire bolt profile, it will do so with a poor spatial resolution.

The spatial limitations of the previously discussed strain gauge techniques has led many researchers (e.g. Steblay, 1987, Schroeck et al., 2000, Hyett et al., 2013) to consider innovative sensing techniques such as guided ultrasonic waves and fiber optic sensing. Concerning the former, this has primarily found success as an encapsulation or bolt length inspection device whereby a low frequency signal (< 100 kHz) emitted from the exposed end of an encapsulated bolt will attenuate according to grout inconsistencies and reflect at the opposing end of the bolt (Steblay, 1987, Zou & Cui, 2011, Stepinski et al., 2014). Beard & Lowe (2003) have demonstrated that higher order frequencies (MHz range) are less susceptible to conditions at the surface profile of the bolt and that the signal attenuation is influenced by bolt curvature. Accordingly, this can be realized as a non-destructive testing technique to monitor bolt deflection. However, ultrasonic wave based techniques cannot presently provide a load or strain measurement accuracy that would make them a feasible application to inspect the shear response of bolts. Fiber optic strain sensing (FOS), on the other hand, provides a very encouraging solution for capturing shear.

Perhaps the most intriguing aspect of FOS is the potential to use a single optical fiber as both the lead and transducer for an array of measurements (i.e. a continuous or sub-continuous measurement sensor). Several researchers (e.g. Schroeck et al., 2000, Zhao et al., 2015, Weng et al., 2015) have discussed the use of multiplexed fiber Bragg gratings (FBG) to monitor strain along bolts. This technique is optical analogous to the short base-length resistive strain gauges in regard to providing a single measurement point per Bragg grating. Ultimately, the FBG solution will be limited by the number of Bragg gratings that can be inscribed per optical sensor (effectively 10) and will experience similar spatial resolution issues as described for the
electrical techniques. A promising solution to spatial resolution limitations can be realized through the application of distributed optical strain sensing (DOS), whereby strain is monitored continuously along the length of a standard single mode optical fiber.

Brillouin optical time domain analysis (BOTDA) and Rayleigh optical frequency domain reflectometry (ROFDR) are two DOS techniques that have been implemented with bolt specimens (Iten & Puzrin, 2010, Hyett et al., 2013, Forbes, 2015). Similar to strain gauge techniques, a single optical fiber was embedded and encapsulated within a pair of diametrically opposed grooves machined out along the length of a rebar specimen. As depicted in Figure 3-3, the optical sensor was looped at one end of the rebar in order to monitor both sides of the specimen (or two sensing lengths) using a single sensor.

![Figure 3-3: Schematic representation of the optical sensor embedded into a pair of diametrically opposed grooves. The sensor is initialized at the head end of the rebar via an LC connector, looped at the opposing toe end of rebar, and terminated in proximity with the rebar head.](image)

In comparison of the two DOS techniques, Hyett et al. (2013) and Forbes (2015) have discussed that the ROFDR technique is a particularly attractive solution for monitoring support members as it is capable of capturing strain along the optical sensor at a spatial resolution as low as 0.65 mm. This implies that a single optical fiber can be used to measure potentially thousands of strain readings along the length of a rock bolt, and therefore, capture a continuous strain profile of the bolt. The low cost of each optical sensor, which is composed of standard telecom optical fiber, also suggests the price per instrumented rock bolt can be minimized. This is a critical factor in convincing industry to adopt such a monitoring technique for the improvement of excavation design in-situ. A demonstration of this strain profile monitoring technique is presented for a symmetric three-point bending experiment on a 21 mm diameter rebar specimen with 0.80 m roller support span in Figure 3-4. The left and right plots present the measured strain along the
Figure 3-4: Top - Optical sensor configuration. Bottom – Strain profiles captured along the optical sensor at various levels of applied symmetric bending load. Two orientations of the rebar are considered: Sensing lengths orientated within proximity of the plane of applied loading (Left), and sensing lengths orientation within proximity of the neutral axis (Right). Tensile strain is considered positive while compressive strain is considered negative.

optical sensor for two orientations of the bolt with respect to loading: sensing lengths orientated in proximity with the direction of applied bending and sensing lengths orientated in proximity with the plane of the neutral axis. The sub-millimeter spacing of strain measurements can readily resolve the position of applied load and roller supports along the rebar; however, a comparison of the two plots exhibits an inherent deficiency of a two sensing length approach. Strain or load induced as a result of bending in the member section is dependent on the orthogonal distance away from the neutral axis. Therefore, in the situation of a circular rebar section, the true (or maximum) extent of bending induced (or lateral) strain will only be captured when the sensing lengths are orientated orthogonally with the plane of the neutral axis. At all other orientations of the sensing lengths the extent of strain will be underestimated and potentially unmeasured in the specific circumstance of alignment with the neutral axis. This is a noteworthy constraint for in-situ
application where the direction of laterally induced strains will most often be unknown and may similarly provide difficulty even under controlled loading conditions.

### 3.3 Three-Dimensional Sensor Design

The determination of the three-dimensional state of strain along a support member will almost always require a minimum of three individual measurement points. When considering the two sensing length technique using ROFDR (Hyett et al., 2013, Forbes, 2015) the most obvious improvement is therefore to implement an additional sensing length, and correspondingly, an additional loop of the optical fiber sensor. For the proposed technique demonstrated in this paper, it has been chosen to embed a single optical fiber sensor within three 2mm x 2mm machined out grooves which extend the entirety of the bolt’s length and are orientated at 120 degrees from each other in a clockwise manner around the bolt (Figure 3-5). The optical sensor is encapsulated in the grooves using a metal bonding adhesive (LORD Fusor 108B) which both bonds the sensor to the bolt and provides a protective barrier. Additionally, a milled loop provides a protected enclosure for the optical sensor to connect to succeeding sensing lengths. The 120 degree orientation, commonly referred to as a delta arrangement, provides the maximal spacing between sensing lengths which in turn minimizes measurement noise associated with sensor positioning and handling.

![Figure 3-5: Schematic representation of the optical sensor embedded into three grooves orientated in a delta configuration (i.e. 120 degree spacing). Left – Cross sectional view along the specimen where θ is the angular distance of lateral loading relative to sensing length 1 and φ is the angular distance of a given sensing length relative to sensing length 1. Right – The sensor is initialized at the head end of the bolt via an LC connector, looped at the opposing toe end of the bolt, subsequently looped in proximity to the head end of the bolt, and terminated in proximity with the bolt toe.](image-url)
uncertainties. A repercussion of the grooving process is a decrease in the cross-sectional area of the bolt. For bolt diameters of 19mm and greater, this process will decrease both the yield strength and flexural rigidity by a maximum of ten percent.

Similar to conventional strain gauge rosettes, the optical technique monitors only longitudinal strain along the three sensing lengths. However, a comparative analysis of these three sensing lengths allows for the determination of the principal strain and principal strain direction of the bolt. Regarding measurements with the optical fiber sensor, the maximum value of strain at a particular cross section of the bolt will be equal to the sum of the strain induced from coaxial forces and the strain induced from bending moments:

\[
\varepsilon_{Total} = \varepsilon_{coaxial} \pm \varepsilon_{bending\ moment}
\]

which can also be expressed as:

\[
\varepsilon_{Total} = \frac{N}{EA} \pm \frac{Mz}{EI}
\]

From elastic bending theory we assume that the coaxial force will result in a uniform strain distribution normal to a given cross section of the bolt. Conversely, bending moment will induce a linearly varying strain distribution normal to a given cross section of the bolt which is directly proportional to the curvature of the bar,

\[
\varepsilon_{bending\ moment} = z \frac{d^2w}{dx^2}
\]

Considering an elastic circular cross section of a typical rock bolt, the strain induced from bending can be rewritten as:
\[ \varepsilon_{\text{bending moment}} = r \cos(\alpha) \frac{d^2 w}{dx^2} \]  

3-4

The previous expression details the measurement dependency of sensor orientation as a maximum value will only be obtained when the sensor is at an orthogonal positioning to the neutral axis. The strain that is measured in any of the particular sensing lengths at a given cross section of the bolt can therefore be expressed as:

\[ \varepsilon_i = \varepsilon_{\text{coaxial}} + \varepsilon_{\text{bending moment}} \cos (\theta + \varphi_i) \]  

3-5

which in turn provides a system of three equations for the strain measured in each sensing length,

\[ \varepsilon_1 = \varepsilon_{\text{coaxial}} + \varepsilon_{\text{bending moment}} \cos (\theta + \varphi_1) \]  

3-6

\[ \varepsilon_2 = \varepsilon_{\text{coaxial}} + \varepsilon_{\text{bending moment}} \cos (\theta + \varphi_2) \]  

3-7

\[ \varepsilon_3 = \varepsilon_{\text{coaxial}} + \varepsilon_{\text{bending moment}} \cos (\theta + \varphi_3) \]  

3-8

The notation used for the proposed technique will always consider \( \varphi_1 \) to be equal to zero. Conversely, \( \varphi_2 \) and \( \varphi_3 \) are measured during the sensor construction process. The angular distance between the first sensing length and direction of applied bending, \( \theta \), can therefore be determined by solving the system of three equations as:

\[ \theta = \tan^{-1}\left\{ \frac{1 - \cos(\varphi_2) - \beta + \beta \cos(\varphi_3)}{\sin(\varphi_2) - \beta \sin(\varphi_3)} \right\} \]  

3-9

where,
The angular distance from Equation 3-9 provides the clockwise orientation of sensing length 1 from orthogonality with the neutral axis of the bolt; however, the π period of the tangent function requires a consideration of the following two conditions in order to ascertain which quadrant the orientation lies within

If, \[ \epsilon_1 < \frac{\epsilon_2 + \epsilon_3}{2}, \quad \theta = \theta \] 3-11

If, \[ \epsilon_1 > \frac{\epsilon_2 + \epsilon_3}{2}, \quad \theta = \theta + 180 \] 3-12

Axial strain and bending strain can be determined subsequently by rearranging the set of three equations describing the measured strain in each sensing length. The expressions given in Equations 3-1 to 3-12 provide a generalized solution for a variety of sensing length arrangements for a scenario where a delta configuration is not achievable. However, for the 120 degrees separation of sensing lengths described in this paper, the axial strain, the bending strain, and the clockwise orientation of the first sensing length can be determined more conveniently as:

\[ \epsilon_{coaxial} = \frac{\epsilon_1 + \epsilon_2 + \epsilon_3}{3} \] 3-13

\[ \epsilon_{bending\,moment} = \frac{\sqrt{3}}{2} \sqrt{(\epsilon_1 - \epsilon_2)^2 + (\epsilon_2 - \epsilon_3)^2 + (\epsilon_3 - \epsilon_1)^2} \] 3-14

\[ \theta = tan^{-1} \left( \frac{\sqrt{3}(\epsilon_3 - \epsilon_2)}{2\epsilon_1 - \epsilon_2 - \epsilon_3} \right) \] 3-15
Similar expressions have also been described by ASTM (2014) and Micro-Measurements (2014).

Applying Equation 3-13 and Equation 3-14 the total strain or “principal” strain can be determined according to Equation 3-1. In addition, the strain at any position surrounding the bolt can be determined according to Equation 3-5. However, the noteworthy distinction of the optical technique is the spatial resolution of this analysis. In comparison to conventional strain gauge rosettes, the optical solution allows the determination of principal strain and principal strain direction every 0.65 mm along the entirety of the rock bolt. This three optical sensing length approach can therefore be analogized as a distributed strain rosette. In this regard, both the loading orientation and loading location(s) along the fully grouted rock bolt do not need to be known prior to bolt installation, making it an ideal solution for resolving complex ground conditions in-situ.

3.4 Initial Validation Experiments

Two validation experiments were initially conceived in order to verify the competency of the proposed rock bolt optical sensing technique and accompanying analysis of the principal strain and principal strain direction. The experiments considered a symmetric bending as well as combined axial and bending loading apparatus. The bolt specimens used in these experiments consisted of a commercially available 22 mm diameter, 413 MPa minimum yield strength rebar from JENNMAR Canada Ltd. The rebar specimens were modified in accordance to the three sensing lengths discussed in Section 3.3. Tests were performed within the elastic limits of the rebar elements, which in turn, allowed multiple tests to be conducted with a single instrumented specimen. This was chosen to establish the repeatability and accuracy of the analysis.

3.4.1 Symmetric Bending

The relatively low loads required to conduct a three-point symmetric bending experiment makes it a very controlled method to assess the baseline performance of the optical technique. The setup of the bending apparatus is shown in Figure 3-6. Also provided is the lengthwise positioning of the optical sensor for each sensing length in comparison to the 800 mm support span and applied bending load. In conducting the symmetric bending experiments, a 50 kg level of load was applied vertically to the optical fiber.
instrumented rebar. Pertaining to the results presented in this paper, four tests are presented that can be described with regards to the orientation of the first sensing length during the experiment: Test 1 – situated along the top surface of the rebar (reference as 0 deg.), Test 2 – situated in proximity to the neutral axis of the rebar (90 deg. CW rotation), Test 3 – situated along the bottom surface of the rebar (180 deg. CW rotation), and Test 4 – situated at a location between the bottom surface and neutral axis of the rebar (220 deg. CW rotation). Figure 3-7 presents the strain profiles measured along the entirety of the optical sensor as well as the measured strain profiles along the bending span of the first sensing length for these tests. Positive strain values denote extension while negative strain values denote contraction.

The inherent measurement dependency of bending induced strain on sensor orientation is apparent when comparing the strain profile for each test. Along the individual sensing lengths a V-shaped profile that is maximum at the position of applied load and that returns to zero at the end of the bending span is measured. The maximum magnitude and the slope of these strain profiles varies drastically depending on the test orientation, despite the same level of load being applied in each instance. However, by applying Equation 3-14, the amount of bending induced strain along the rebar is determined to be equal for each test orientation. The bending induced strain profile along the rebar is presented in Figure 3-8, along with a plot of the coaxial strain and orientation of first sensing length determined from Equation 3-13 and Equation 3-15, respectively.

The coaxial strain measured in the bolt is negligible throughout the symmetric bending experiment. This was expected as the V-shaped strain profiles presented in Figure 3-7 are comparable in shape to a
moment diagram for a symmetric bending apparatus. These are indicative of a parabola-like deflection profile of the rebar. The measured strain profiles can accordingly be considered to be wholly constituted of

Figure 3-7: Symmetric bending experiment. Top – Optical sensor configuration and approximate orientation in section view. Bottom – Measured strain profiles at an applied bending load of 50kg along the entirety of the optical sensor (Left) and along the bending span of the first sensing length (Right).

Figure 3-8: Symmetric bending experiment. Left – Coaxial and bending induced components of strain measured along the bending span of the rebar at 50kg of applied load. Right – Clockwise orientation of the first sensing length relative to the direction of applied load.
bending induced strain, and therefore, the bending (or lateral) strain presented in Figure 3-8 also represents the total strain in the rebar.

The orientation of the first sensing length displayed in Figure 3-8 correlates well with the apparatus description for each test. Along a majority of the bending span the orientation remains largely constant; however, measurement noise (associated with repeatability of measurement technique) begins to dominate towards the end of the span. This can be expected as towards the end of the bending span (and outside of the bending span) a relatively low or even non-existent bending moment will be present along the rebar, even if a coaxial load is present. Minor fluctuations of the strain measured between sensing lengths (e.g. low level measurement noise or minor geometrical inconsistencies of the bolt) will correspondingly result in a major shift of the derived orientation. Therefore, the orientation that is derived in this analysis is only compatible when bending induced strain is evident.

3.4.2 Combined Axial and Bending Experiment

Shearing of fully grouted rock bolts will in many cases result in a combination of coaxial and lateral load being developed in the bolt. It is therefore crucial that the proposed technique and analysis is capable of distinguishing components of coaxial and bending induced strain. In conducting a simple, controllable experiment, a combined axial and bending load apparatus was constructed by building upon the symmetric bending experiment (which had negligible coaxial load). The procedure for this experiment firstly involved end anchoring a 1500 mm rebar span using spherical washers and domed plates. An incremental load up to two tonnes in extension was then applied to this free span. This was considered the axial load. Thereafter, while holding this level of axial load, an incremental point bending load up to 50 kg was introduced at an asymmetric position along the rebar (generalized case). The apparatus used in this experiment is presented in Figure 3-9.

The strain profiles measured along the entirety of the optical sensor during the combined axial and bending experiment are presented in Figure 3-10. The three optical sensing lengths are easily distinguished by abrupt decreases along the strain profile occurring at approximately 1.70 m and 3.20 m. These locations
correspond to the looping segments of the optical sensor which were situated outside of the anchored span. Accordingly, these two segments of the sensor take on negligible strain throughout the first stage of axial loading and secondary stage bending loading during the test. Referring to the former stage, both Hyett et al. (2013) and Forbes et al. (2015) have noted that it is extraordinarily difficult, if not impossible, to apply a purely coaxial load. Minor misalignments of the loading apparatus, initial washer seating, as well as initial straightening of the specimen will result in a component of inadvertent and non-negligible bending. This bending occurrence is examinable during the initial axial loading of the rebar. The strain profiles measured along each length display a moderate convex or concave like shape. This is indicative of a component of bending, as theoretically an end loaded member will exhibit a uniform level of strain across its span. This is most discernable along the third sensing length from 3.30 m to 4.80 m. The majority of strain during this first stage of loading does, however, appear to be constituted of coaxial strain. The succeeding application of bending load is then observed to either increase or decrease the magnitude of strain along each sensing length.

Figure 3-9: Combined axial and bending apparatus. A 20 tonne hollow plunger cylinder (a) is used to apply axial load to a 1500mm end anchored rebar (b). A scaled turnbuckle is used to apply a point bending load (c) while spherical washers and domed plates minimize bending as a result of tensioning the rebar (d)(e).
length. This again shows the measurement dependence of sensing length orientation relative to the bending inducing feature.

Applying Equation 3-13 and Equation 3-14, the coaxial and bending induced components of strain are separated along the length of the rebar. These are presented in Figure 3-11. The left plot of Figure 3-11 indicates, as expected, that the magnitude of coaxial strain remains consistent across the end anchored span of the rebar. The uniform level of strain is observed to increase from 0 με to 135 με at one tonne and to 275 με at two tonnes of applied axial load. But more importantly, thereafter, the measured coaxial strain profile

Figure 3-10: Combined axial and bending experiment. Top – Optical sensor configuration. Bottom – Measured strain along the entirety of the optical sensor.

Asymmetric Bending Load

End plate location

Axial Load

Sensor
Loop 1

Distance along optical transducer (m)

Strain (με)
appears to be unaffected by the subsequent application of incremental bending load. This demonstrates that the sensing technique can successfully separate the coaxial component of strain along the rebar in addition to determining the maximum bending induced component of strain as displayed in the right plot. Thereon the orientation of loading can be derived using Equation 3-15 and the total strain along the rebar can be determined according to Equation 3-1.

The resolved bending induced strain profile displayed in the right plot of Figure 3-11 provided an initially unanticipated result. In comparison to the symmetric bending experiment results displayed in the left plot of Figure 3-8, non-zero strains are prevalent towards both ends of the anchored rebar span. The contrast between these two plots can be attributed to the unique end conditions of both experiments. The symmetric bending experiment, which is ultimately a three-point-bending test, does not constrain the ends of the rebar specimen. Therefore, while the rebar does not deflect at the end of the span, it can rotate to accommodate a deflection slope. In contrast, the combined axial and bending experiment constrains both ends of the rebar with the holding devices used to apply axial load. Therefore, both deflection and slope are prevented at the end of the rebar span. A corresponding end moment will therefore be induced to maintain zero slope. In this regard, the bending induced strain profile determined for the combined axial and bending...
experiment correlates well with the solution to what is commonly referred to as a “built-in-beam” (Figure 3-12). Note the measured bending strain has been shown as an absolute profile.

![Diagram showing idealization of the symmetric bending experiment (left) and combined axial and bending experiment (right).](image)

**Figure 3-12:** Idealization of the symmetric bending experiment (left) and combined axial and bending experiment (right).

### 3.5 Double Shear Experiments

Shear displacement of a bolted discontinuity will rarely result in a direct guillotining of the bolt (Stillborg, 1994). Instead, the shear response of a grouted rock bolt will more commonly take on a combination of coaxial and bending induced loads (Bjurström, 1974) that will be localized within the region of the discontinuity. This suggests that the previously demonstrated symmetric bending and combined axial and bending experiments (with a minimum tested support span of 800 mm) may not definitively assess the optical sensing technique’s capability to resolve principal strain and direction across highly concentrated loading regions. In this regard, double shear loading experiments were conducted using a similar apparatus as that presented by Hyett et al. (2013). These tests considered centering and grouting (0.40 w:c Portland cement) an optically instrumented rebar into a 50 mm reamed borehole which was precast through three individual concrete blocks (400 x 400 x 400 mm block dimensioning, 46.8 MPa UCS, 4.2 MPa splitting tensile strength). The outer two blocks of this experiment were fixed while a vertical displacement was applied to the central block using a servo-controlled 500 kN loading frame. This results in two vertical shear planes acting along the bolt, which provides a much less demanding apparatus to control in...
comparison to a single shear plane test (Aziz et al., 2003). A 3 mm thick nylon sheet was also placed between the individual concrete blocks during the casting stage in order to promote a frictionless and non-dilating surface for shear displacement between blocks. This testing apparatus is visualized in Figure 3-13.

The double shear experiments were conducted at a constant displacement rate of 0.5mm/min. Likewise to previous research efforts, the actuator load and stroke were monitored at all times during the tests. This provided measurement of the shear load and corresponding shear displacement of the apparatus. A spring-loaded linear variable displacement transducer (LVDT) was also attached to the central concrete block in order to provide redundancy in shear displacement measurements. However, this displacement value should not be confused with actual deflection of the bolt (Li et al., 2016a), which is often of lesser magnitude as the bolt will routinely crush and cut through the surrounding grout and host mass (Schubert, 1984). No coaxial pretension was applied to the rebar, nor was normal displacement or load of the rebar monitored at the outer concrete blocks (other than that which can be inferred from the optical sensor). Tests

Figure 3-13: Double shear apparatus. Left – An example test specimen and optical interrogation unit during test preparation. Right – Schematic representation of test specimens.
were conducted with the first optical sensing length best aligned in plane with the direction of applied shear displacement. A depiction of the sensor orientation as well as the strain profiles captured along the entirety of the optical sensor at several applied vertical displacements of the central concrete block are presented in Figure 3-14.

![Direction of shear loading](image)

**Figure 3-14:** Left – Depiction of optical sensing length orientation. Right – Top: Schematic depiction of the optical sensor configuration. Bottom: Strain profiles measured along the entirety of the optical sensor at various central block displacements. Tensile strain is considered positive while compressive strain is considered negative.

At each discontinuity (i.e. shear plane) a distinct shear couplet is measured. This corresponds to a pair of extensile and contractive strain developments which mirror at the shear displacement plane; although, the absolute strain measured for the contracting side of the rebar is of a lesser magnitude in each case. This is explained by the compressive resistance provided by the grout on the contractive side of the discontinuity and by the relatively negligible adhesion resistance provided by the grout on the extensile side of the discontinuity. Respectively, shear displacement of the rebar is resisted by the grout in compression, but in extension the rebar will separate away from the grout and thus, deform more readily. This agrees well with the S-bend or plastic hinge behaviour that has been both simulated (Spang & Egger, 1990, Aziz
et al., 2005) and experimentally interpreted using discrete technologies (Ferrero, 1995, McHugh & Signer, 1999, Grasselli, 2005, Jalalifar, 2006, Chen and Li, 2014). The locality of the shear couplet to the discontinuity plane is more readily visualized in Figure 3-15, which shows a comparison of the three sensing lengths along the rebar at 2mm of central block displacement as well as the orientation of load along the rebar determined using Equation 3-15. It should be noted that the optical sensor failed at approximately 8mm of central block displacement for which strain values well over 20,000 µε were captured.

Figure 3-15: Left – Strain comparison of sensing lengths along the rebar at an applied displacement of 2mm. Middle – Resolved sensor orientation. Right – Orientation of the load vector along the rebar. Note: Zero is referenced as the running along the top axis of the rebar.

The measurement dependency of sensor orientation is again evident when referring to the left plot of Figure 3-15. Each optical sensing length captures a shear couplet, but the magnitude of these profiles varies significantly. The inequality of the second and third sensing length also reveals that the rebar was not installed with the first sensing length in plane with direction of shear displacement as previously discussed (i.e. along the top axis of the rebar). Instead, the orientation of load along the rebar, displayed in the right plot of Figure 3-15, describes the rebar to have been installed with an approximate 10 degrees clockwise rotation discrepancy. The orientation of load along the rebar also provides a further rationalization of the shear couplet mechanism. At both shear discontinuities the orientation flips by 180
degrees. This reversal coincides with the position of strain profile mirroring. The orientation plot also confirms that the contractive side of the rebar (compressing against the grout) is mobilized by the applied shear displacement. Furthermore, the orientation plot suggests that bending is present across the central block and also towards the ends of the specimen. However, applying Equation 3-14, the bending induced strain is found to be negligible past approximately 120mm from either discontinuity. A comparison of the bending induced strain profile, the coaxial strain profile calculated using Equation 3-13, and the total strain profile calculated from the sum of former two is presented in the left plot of Figure 3-16.

![Image](image.png)

**Figure 3-16**: Left – A comparison of the total strain, coaxial strain, and bending induced strain along the rebar at an applied displacement of 2mm. Right – Lateral deflection of the rebar at various central block displacements.

The total strain profile of the sheared rebar is observed to be composed of both coaxial and bending induced strain components. At the locations of maximum strain in the rebar, the shape of the total strain profile is primarily dictated by bending induced strain. At these locations the coaxial component was never found to constitute more than 25 percent of the total strain. However, the location of maximum coaxial strain was continually found to be situated within close proximity to the inflection position of the total strain profile, which coincides with the location of ultimate failure in previous experimental studies (Spang &
Egger, 1990, Ferrero, 1995, Aziz et al., 2005, Grasselli, 2005, Jalalifar, 2006, Li et al., 2016a, Li et al., 2016b). Additionally, the coaxial strain is observed to propagate to the end of the two outer blocks and correspondingly to overlap and form a zone of extension within the central block. In contradiction, the bending induced strain rarely propagated past 120 mm from either side of the discontinuity. This is an interesting outcome as under non-idealized shear plane conditions a more prominent dilation component would be expected to induce further coaxial loading of the rebar (Egger & Zabuski, 1991). A consideration of alternative rebar inclinations (i.e. non-orthogonality with shear planes) would also be expected to influence the extent of coaxial and bending induced strain constituents and their locality to the discontinuity (Haas, 1976, Azuar, 1977, Pellet & Egger, 1996). However, less notable apparatus considerations will also have a pronounced influence on the straining of the rebar.

A unique limitation noted for the double shear apparatus used in these experiments was the lack of vertical restraint for the specimens. The outer two concrete blocks were fixed against vertically downward (i.e. the direction of applied load) and horizontal displacements, but were free to vertically rise. While running the experiments this resulted in the entire specimen bending or bowing to a degree, which ultimately allowed one discontinuity to take on more displacement. This asymmetric tendency of double shear testing has similarly been discussed by Hartman & Hebblewhite (2003). Referring to the left plot in Figure 3-16, the magnitude of strain mobilized in the rebar at the left discontinuity is notably more than that of the right discontinuity. This effect is considerably more pronounced when examining the deflection profile of the rebar, displayed in the right plot of Figure 3-16, which was determined according to Equation 3-3. A post inspection of the rebar and encapsulating grout further confirmed this asymmetrical deflection of the rebar, as shown in Figure 3-17. It is therefore imperative that a study aimed at a more rigorous investigation of the shear mechanisms of fully grouted rock bolts utilize an apparatus that can both restrain and confine the surrounding host material. However, the application of the presented optical sensing technique, allowing both the strain and deflection profile of the support specimen to be observed, provides an unprecedented opportunity to ascertain the true bolt behaviour. The technique augments exterior load
and displacement measurements which have previously been the standard in assessing the strength and displacement of bolted discontinuities. This also suggests that the optical technique could be used to infer rock mass movement vectors in-situ.

Figure 3-17: Post double shear test removal of the rebar and encapsulating grout from the concrete blocks.

3.6 Conclusion

A novel optical strain sensing technique for monitoring the strain profile of fully grouted rock bolts at a spatial resolution of 0.65 mm has been presented. The technique considers a three sensing length approach, for which both a generalized and a delta-configuration analysis of the principal strain and principal strain direction are provided. The technique can be analogized as a distributed strain rosette that acts along the entirety of a rock bolt specimen. This implies that both the location(s) and the direction(s) of load inducing rock mass movements do not need to be known prior to sensor installation in order to capture the maximum state of loading in the bolt. This has previously been the major impediment of discrete rock bolt sensing techniques.

A series of controlled laboratory experiments demonstrated the capability of the proposed technique to capture generalized in-situ loading behaviour(s) of a rock bolt, which included: axial loading, bending, and shearing. The latter was established to induce a strain profile that is constituent of both coaxial and bending induced strain components. The magnitude and propagation behaviour of these constituents could be distinguished along the bolt using the accompanying analysis, which included the sections of
concentrated loading at shear planes. In fact, the vast majority of shear induced strain along the rock bolt (~90% of total strain) was found to never extend past 120 mm from the discontinuity plane. This demonstrates the necessity of a sensitive spatial resolution monitoring approach in order to capture such discontinuous behaviour in-situ.

In addition to strain profiling, the optical sensing technique also allows the three-dimensional deflection profile of the rock bolt to be resolved. At the laboratory scale, this can be used to augment existing external measurement techniques of encapsulated bolt tests, which have previously been relied upon to infer bolt behaviour. However, when applied in-situ, the optical technique can be realized as an unprecedented tool to infer surrounding rock mass movement vectors along with bolt deflection. Both unquestionably can aid in the design and safety practices of underground excavations whenever rock bolts are being installed.
3.7 References


Chapter 4 Insight into the coaxial load distribution of fully grouted rebar, cable bolt, CT-Bolt, and D-Bolt reinforcement elements

4.1 Introduction

Rock bolting has been widely used over the past century to restrict ground mass displacements and improve the stability of underground excavations. While the term “rock bolt” perhaps originally described the use of borehole installed, mechanically anchored, tensioned steel bars (e.g., Stillborg, 1994), it often now synonymous with the application of some form of tendon reinforcement. In this study, the emphasis is placed on rock bolts or reinforcement elements that can be categorized as being continuously mechanically coupled (CMC) to the ground mass (e.g., Thompson et al., 2012; Windsor, 1997) by means of a cementitious grout or resin.

Over the past several decades, a variety of reinforcement elements have been introduced and made commercially available to the mining and tunnelling industry (e.g., Li, 2017). The manufacturing design and the distinguishing aspects of different elements has been brought about by tailoring the element’s material and profile to withstand imposed reinforcement demands from specific excavation instabilities and ground mass failure modes. These can include but are not limited to: 1) Gravity-driven and structural controlled unravelling and block loosening, 2) Stress-driven shear failure, squeezing, and extensile failure, and 3) Dynamic strain release in the form of rock bursting. Such mechanisms are inherently dependent on the excavation geometry, ground integrity, the presence of geologic structures, and the in-situ stress field (e.g., Kaiser, 2017; Kaiser et al., 2000).

In general, reinforcement elements will primarily perform a combination of reinforcement and holding functions (Hutchinson and Diederichs, 1996). These functions can be thought of in terms of classical reinforcement design concepts, such as: beam building and arching bolting, suspension bolting, keyblock and wedge bolting, as well as dynamic control (e.g., Hoek and Brown, 1980; Kaiser et al., 1996; Lang, 1961; Li, 2017b; Stillborg, 1994). Within the context of safety factor calculations during design, there
are three primary considerations of a reinforcement element or system: 1) The static load capacity, 2) The static displacement/deformation capacity, and 3) The energy absorption capacity. Referring to Figure 4-1, many field studies have indicated that in-situ reinforcement elements are often subjected to combination of coaxial and bending moment induced loads (e.g., De Ambrosis and Kotze, 2004; Li, 2010; Peng, 2007); however, the vast majority of reinforcement design is based solely on coaxial loading of the reinforcement. Accordingly, a large emphasis has been placed on quantifying the static and dynamic load transfer efficiency and capacities of reinforcement elements using pull test apparatuses in the laboratory and in-situ (e.g., ASTM, 2013; ASTM International, 2010; Lardner and Littlejohn, 1985).

There are a vast number of parametric studies using pull test apparatuses that have been conducted on various types of reinforcement elements. These tests often consider two measures: the applied coaxial load and the coaxial displacement of the element (at the position of applied load). It has been indicated that the element’s construction and external profile can greatly influence the peak and residual load-displacement behavior (e.g., Cao et al., 2016; Kilic et al., 2003; Moosavi et al., 2002) and its ability to withstand dynamic impacts (e.g., Cai and Champaigne, 2009; Li and Doucet, 2012). However, while there are a significant number of pull test studies that have presented results in the form of load-displacement

Figure 4-1: Left - Rebar element exposed after of a fall of ground in an underground mine. Permanent deformation is visible along the rebar in the form of coaxial elongation and transverse, bending moment induced deformation (Image courtesy of Brad Simser, Glencore). Right – Schematic illustration of coaxial and transverse loading along a reinforcement element (after Mark et al. 2002).
response curves, there are relatively few examples where the load distribution along the reinforcement element has been experimentally measured (e.g., Chen, 2014; Freeman, 1978; Jalalifar, 2006; Li et al., 2016; Li and Liu, 2019; Serbousek and Signer, 1987). Since the load-displacement response has been determined to vary depending on the reinforcement element type, the load mobilization behaviour would inherently be expected to differ as well.

The limited number of pull test experiments that have been devoted to measuring load mobilization length can be attributed to the high sensing demand that is involved in measuring an accurate load distribution along elements with realistic encapsulation. As discussed by Windsor (1992), monitoring the load profile along a reinforcement element requires both discrete (i.e., short base length ‘cells’) and integrated measurements (i.e., long base length ‘gauges’). This has not been practically achievable using conventional sensing technologies such as electrical resistive strain gauges and load cells (e.g., Radcliffe and Stateham, 1980; Rodger et al., 1996). However, one emerging technology that potentially conforms to the criteria of closely spaced, distributed measurement points is distributed optical fiber strain sensing (DOS). As discussed in Chapter 2 (Forbes et al., 2018), this technology, when coupled along the length of a reinforcement element, can measure strain at increments as low as every 0.65 mm along the element’s axis.

In this study, DOS is used to measure the strain distribution along instrumented reinforcement elements during laboratory pull tests. Several reinforcement elements are considered, including: rebars, plain strand cables, CT-Bolts, and D-Bolts (Li, 2010b). The focus of this study was to investigate the load mobilization characteristics of each element within the context of comparing the load development length. This is primarily discussed at pull load magnitudes that are below what would be considered the working capacity of the element (i.e., pre-yield – ASTM, 2016). This was required because the given DOS technology had a strain sensing capacity that was less than the ultimate strain of the steel elements (as will be discussed in this Chapter). While certain testing aspects, such as: borehole diameter, encapsulation
length, and confinement stiffness were varied in the presented results, this study is not intended to be a rigorous sensitivity investigation of the given reinforcement elements.

4.2 Background: Coaxial Performance and Testing of Reinforcement Elements

An in-situ installed reinforcement element may be subjected to coaxial load as a result of relatively continuous and/or discontinuous ground mass movements (e.g., Bjornfot and Stephansson, 1984; Hyett et al., 1996; Li and Stillborg, 1999). The general reinforcement response of a coaxial loaded rock bolt can be distinguished into three sections (Freeman, 1978): 1) The pickup length, 2) The neutral point, and 3) The anchoring length. Referring to Figure 4-2, the pickup length corresponds to the segment of the element that resists movement of the ground mass towards the excavation. This is counter-balanced by an anchoring length, where the element anchors into deeper seated ground (i.e., a reversal in the sense of the relative movement between the element and the ground mass). The neutral point is located in between the pickup

![Diagram](https://via.placeholder.com/150)

Figure 4-2: Schematic depiction of the reinforcement response of a fully grouted element. At the head of the element (i.e., at the excavation periphery), the element resists movement of the ground mass towards the excavation ($U_{ex}$). Towards the end of the bolt, more competent, deeper seated ground restrains the element from moving towards the excavation. Accordingly, there is a reversal in the sense of shear traction or relative movement between the element and the ground mass when comparing the pickup length and the anchoring length. The neutral point corresponds to the position where there is an inflection in the direction of the shear stress and corresponds to the position of maximum axial load along the element. $U_{dm}$ refers to a discrete movement (such as dilation across a discontinuity located along the reinforcement element).
length and the anchor length. This corresponds to the position of no relative displacement between the element and the ground mass and, correspondingly, the position of maximum coaxial load along the element. Coaxial pull tests (e.g., ASTM International, 2010; Lardner and Littlejohn, 1985) seek to replicate this loading response by reproducing the ground mass induced load along the pickup length with a point load along the element (which would correspond to the position of the neutral point). Accordingly, coaxial pull tests will only result in the anchoring length of an element being tested.

Referring to Figure 4-3, a reinforcement element can be categorized into four primary components (Windsor, 1997): 1) The host medium (i.e., the ground mass), 2) The reinforcement element, 3) The internal fixture (i.e., the encapsulating grout/resin), and 4) The external fixture (i.e., the face plate assembly). When coaxial pull tests are conducted under laboratory settings, the confinement provided by the ground mass is replaced by a passive, constant radial stiffness boundary (e.g., a steel pipe, concrete specimen, or cored sample) or an active constant radial pressure boundary (e.g., by means of a modified triaxial or biaxial cell) (e.g., Blanco Martín et al., 2013; Hyett et al., 1995). Nevertheless, a coaxial pull test will produce failure

| 1) Host medium | 2) Reinforcement element | 3) Internal fixture | 4) External fixture |

Figure 4-3: Principle components of a reinforcement element (after Windsor 1997).
in a manner that is consistent to what has been observed in-situ, which includes (e.g., Hutchinson and Diederichs, 1996; Jeremic and Delaire, 1983):

1) Rupture of the reinforcement element;
2) Failure within the grout column;
3) Failure within the surrounding ground mass (or ground mass replacement);
4) Bond failure at the element-grout interface;
5) Bond failure at the grout-ground mass interface, or;
6) A combination of the failure modes listed.

The failure mechanism(s) which ultimately define(s) the coaxial capacity of a given reinforcement element (in a particular set of conditions) will be dictated by the ability of load to transfer through the grout, between the element and the ground mass. The bond strength capacity and stiffness at the reinforcement element-grout interface are, therefore, critical to gaining an understanding of the full capacity of the element, as they directly control the manner in which load is able radiate between the element and the ground mass. At the interface between the reinforcement element-grout, load transfer efficiency is the results of three mechanisms (e.g., Benmokrane et al., 1995):

1) Chemical adhesion;
2) Mechanical interlock, and;
3) Friction.

These mechanisms are lost in succession in the form of a decoupling front that propagates from the position of maximum load to the distal end of the reinforcement element (Hyett et al., 1996; Li and Stillborg, 1999). However, many studies have indicated that the adhesion component of bond strength is negligible (e.g., Aziz and Webb, 2003; Hyett et al., 1995; Signer, 1990).

Within the context of the rebars, plain strand cables, CT-Bolts, and D-Bolts that were tested in this study, the unique construction of each reinforcement element type (Table 4-1) will result in distinct bond strength characteristics. The irregular surface of solid bar rebars, CT-Bolts, and D-Bolts (i.e., deformed ribs
Table 4-1: Reinforcement element overview. The modified cross-sectional area was determined by accounting for the material removed through machining grooves along the solid bar elements and the replacement of the central wire of the cable elements. HDPE refers to high density polyethylene. Rebar rib descriptions were described following the criteria discussed by Jalalifar (2006). Rebar (2) and Rebar (3) refer to rebar modified with two lengthwise machined and three lengthwise machined grooves, respectively.

<table>
<thead>
<tr>
<th>Element</th>
<th>Nominal Diameter (mm)</th>
<th>Modified Cross-Sectional Area (mm²)</th>
<th>Elastic Modulus (GPa)</th>
<th>Yield Strength (kN)</th>
<th>Ultimate Strength (kN)</th>
<th>Additional Comments</th>
</tr>
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</table>
| Cable      | 15.24                 | 125                                 | 0.023 (kN/µε)*        | 205                 | 260                    | - 7-wire steel cable  
|            |                       |                                     |                       |                     |                        | - Plain strand      
|            |                       |                                     |                       |                     |                        | - Lay length of 210 mm |
| CT-Bolt    | 21.5                  | 365                                 |                       | 230                 | 295                    | - Black finish steel rebar with HDPE corrugate sheath (32.5 mm diameter, 2 mm thick)  
|            |                       |                                     |                       |                     |                        | - Rebar rib height = 1 mm, rid width = 6 mm, and rid spacing = 8 mm |
| Rebar (2)  | 19.5                  | 285                                 | 200                   | 120                 | 180                    | - Black finish steel rebar  
|            |                       |                                     |                       |                     |                        | - Rebar rib height = 2 mm, rid width = 8mm, and rid spacing = 4 mm |
| Rebar (3)  | 19.5                  | 275                                 |                       | 140                 | 190                    |                                                                 |
| D-Bolt     | 20                    | 315                                 | 140                   | 190                 |                        | - 20 mm diameter smooth bar with 125 mm long anchor sections  
|            |                       |                                     |                       |                     |                        | - Anchor section are spaced (center-center) by 875 mm  
|            |                       |                                     |                       |                     |                        | - Anchor section has a maximum diameter of 27 mm |

* A cable load to central strand strain relationship of 0.020-0.0205 kN/µε was used as experimentally determined by Hyett et al. (1997) and Forbes et al. (2018)
and anchors) will firmly fix into the grout annulus and provide a strong mechanical interlock. Referring to Figure 4-4, the spacing between adjacent ribs along a rebar element are much closer than the spacing between anchor position along a D-Bolt. Correspondingly, the D-Bolt has a much less stiff response. The comparatively long smooth bar sections between anchor positions along the D-Bolt have a significantly lower bond transfer efficiency than the anchors (e.g., Aydan, 1989; Li, 2010b). This permits the smooth bar sections to freely deform between the anchor positions when subject to axial load, giving the D-Bolt a higher deformation capacity than grouted rebar (Li, 2012; Li and Doucet, 2012). The geometric mismatch between the confining grout and the irregular surface of a solid bar element has been observed to fail by two main mechanisms (e.g., Cao, 2012; Kilic et al., 2003; Tepfers, 1979): 1) By parallel shear failure (i.e., crushing of the grout parallel to element’s axis) or 2) dilational slip failure (i.e., conical grout failure surface

![Figure 4-4: Schematic comparison between rebar, plain strand cable, CT-Bolt, and D-Bolt reinforcement elements. Further details on these elements are listed in Table 4-1](image)

99
through the annulus). Following loss of mechanical interlock, the residual bond strength will be mobilized by friction (Benmokrane et al., 1995; Blanco Martín et al., 2013; Ren et al., 2010).

The interaction of a coaxial loaded cable bolt at the grout interface will differ from the irregular surface bar elements due to the helical structure of the strand (i.e., six outer wires wrapped around a central wire). In comparison to solid or hollow bar element, a cable bolt with an equal nominal diameter will have significantly reduced torsional rigidity. This promotes a tendency for the cable bolt to twist when coaxially loaded (e.g., Bawden et al., 1992). Accordingly, the cable bolt load transfer mechanism at the element-grout interface primarily arises in the form of a frictional-dilational relationship (Fuller and Cox, 1975; Yazici and Kaiser, 1992), which is highly influenced by radial confinement (Kaiser et al., 1992) and the pressure that can be generated by the element-grout interface (Hyett et al., 1995). Failure at the grout interface along the length of the cable bolt has been noted to take form by (e.g., Hyett et al., 1992):

1) Dilational slip between the cable bolt and the grout caused by radial splitting of the grout (favourable when the radial stiffness of the confining material is low);

2) Shearing of the grout flutes between individual strands (favourable when the radial stiffness of the confining material is high), and;

3) Unscrewing of the cable through the grout flutes.

The latter requires additional consideration when conducting pull tests on cable bolts, as this failure mechanism provides the most efficient path for the element (i.e. will require the least amount of work by the element).

Coaxial pull tests concerning the bond strength capacity of a reinforcement element have primarily been conducted on short embedment length apparatuses (i.e., where the element diameter to grouted length ratio is less than ten). This promotes shear failure at the element-grout interface and permits the assumption that the shear stress distribution will be uniform over the encapsulated length of the element (Benmokrane et al., 1995; Blanco-Martín, 2012; Indraratna and Kaiser, 1990). Therefore, the peak load sustained by the element corresponds to the bond strength per element segment. While this testing procedure is an efficient
method to quantify a parameter for bond strength, it is partly necessitated by the external monitoring nature of coaxial pull tests, for example:

1) The applied coaxial load;
2) The displacement at the free ends of the reinforcement element (i.e., outside of the encapsulated length);
3) The radial displacement, and;
4) The radial confining pressure (if applicable).

Realistic encapsulation lengths would not be expected to result in a uniform bond stress distribution (Hyett et al., 1996; Li and Stillborg, 1999). However, the listed external measurements are not able to discern coaxial load as a function of distance along the element, which is inherently necessary to obtain for the calibration and the verification of analytical solutions for reinforcement element behaviour (Blanco Martín et al., 2011). As will be shown in the following sections, DOS is a fitting solution to measure the non-uniform load distribution along a realistic length reinforcement element tested under coaxial load.

4.3 Testing Methodology

The laboratory tests discussed in this chapter include the results from 15 coaxial pull tests that were conducted on four reinforcement element variations: 1) cable bolts, 2) rebars, 3) CT-Bolts, and 4) D-Bolts. A description of each reinforcement element is provided in Table 4-1. All of the reinforcement elements were fully encapsulated into either a concrete cylinder or metal pipe (simulating the confinement provided by a ground mass) using a cement grout and all of the grouted specimens were loaded using the same testing apparatus. An overview of the test specimens, the coaxial testing apparatus, and the monitoring techniques used to quantify the mechanistic response of the reinforcement elements under coaxial load are described in this section.

4.3.1 Test Specimens

This research effort considered the testing of grouted reinforcement elements that were unconstrained, confined by a constant radial stiffness boundary condition, and had a constant embedment length during
loading (i.e., the reinforcement extends beyond the encapsulated length). The constant radial stiffness boundary condition was achieved by cement grouting the given reinforcement element within a metal pipe or concrete cylinder (limited to cracking strengths of the concrete). Table 4-2 provides a summary of the 15 test specimens considered in this research. In addition to varying the type of reinforcement element, other specimen variations included: the embedment length, the borehole size, and the confining material. As discussed by Hyett et al. (1992), the dimensions (i.e., the inner diameter, \(d_i\), and the outer diameter, \(d_o\)) and the elastic properties (i.e., the Young’s modulus, \(E\), and the Poisson’s ratio, \(v\)) of the confining cylinder or pipe can be used to ascertain the passive radial stiffness \((K_r)\) provided by the confining material, which can also be compared to the radial stiffness provided by a ground mass (Hutchinson and Diederichs, 1996), according to Equation 4-1.

\[
K_r = \frac{2E}{(1 + v)} \left( \frac{d_o^2 - d_i^2}{d_i \left[ (1 - 2v)d_i^2 + d_o^2 \right]} \right)
\]

All the reinforcement elements were encapsulated within a confining cylinder or pipe using a cement grout with a 0.4 water to cement ratio by mass. The grout was experimentally determined to have an unconfined compressive strength of 39 MPa (the reader is referred to Cruz, 2017), which is consistent with past studies (e.g., Benmokrane et al., 1995; Hyett et al., 1992). As shown in Figure 4-5, all the specimens were encapsulated vertically and were grouted from the bottom end of the specimen. It was considered pragmatic to vertically pump the grout in order to mitigate air entrapment and variability of the grout consistency along the encapsulated length. As detailed by O’Connor et al. (2019), post-test opening up of grouted test specimens that used this procedure found the grout quality to be consistent throughout the encapsulated length, with little air entrapment.

The grouting procedure for each specimen initially involved degreasing, abrading, and cleaning the inner surface of the given pipe/cylinder. The abrading procedure involved high speed rotation of a crimped wire wheel along the inner surface of the pipe/cylinder, which removed a weaker corroded layer (if present)
Table 4-2: Overview of coaxial test specimens. Specimen CTS49-1000(ns) was grouted with no plastic sheath (“ns”). Confinement radial stiffness was determined according to the thick-walled cylinder equation described by Hyett et al. (1992) – Equation 4-1. Steel and aluminum were assumed to have Young’s modulus of 200 GPa and 72 GPa, respectively, and a Poisson’s ratio of 0.25. The Young’s modulus and Poisson’s ratio of the concrete was experimentally determined to be 15.6 GPa and 0.173 (refer to Cruz 2017). The Cable, CT-Bolt, and Rebar elements were obtained from DSI Underground Canada Ltd. The D-Bolt elements were obtained from Normet Canada Ltd.

<table>
<thead>
<tr>
<th>Test Specimen</th>
<th>Element</th>
<th>Confining Material</th>
<th>Confinement Outer Diameter (mm)</th>
<th>Confinement Inner Diameter (mm)</th>
<th>Confinement Radial Stiffness (MPa/mm)</th>
<th>Embedment Length (mm)</th>
<th>Grout</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS40-750</td>
<td>Cable</td>
<td>Steel</td>
<td>48.3</td>
<td>40.9</td>
<td>1630</td>
<td>750</td>
<td></td>
</tr>
<tr>
<td>CS49-750</td>
<td></td>
<td></td>
<td>60.1</td>
<td>49.3</td>
<td>1590</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CS59-750</td>
<td></td>
<td></td>
<td>73.0</td>
<td>59.0</td>
<td>1420</td>
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<td></td>
<td></td>
<td>48.3</td>
<td>40.9</td>
<td>1630</td>
<td>1500</td>
<td></td>
</tr>
<tr>
<td>CS49-1500</td>
<td></td>
<td></td>
<td>60.1</td>
<td>49.3</td>
<td>1590</td>
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<tr>
<td>CS59-1500</td>
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<td>73.0</td>
<td>59.0</td>
<td>1420</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CA49-1500</td>
<td></td>
<td>Aluminum</td>
<td>60.1</td>
<td>49.3</td>
<td>570</td>
<td></td>
<td>Cement (0.4 water to cement by mass)</td>
</tr>
<tr>
<td>CTS49-1000(1)</td>
<td>CT-Bolt</td>
<td>Steel</td>
<td>60.1</td>
<td>49.3</td>
<td>1590</td>
<td>1000</td>
<td></td>
</tr>
<tr>
<td>CTS49-1000(2)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CTS49-1000(ns)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RC31-1000</td>
<td>Rebar (2)</td>
<td>Concrete</td>
<td>200</td>
<td>31</td>
<td>825</td>
<td>1000</td>
<td></td>
</tr>
<tr>
<td>RC31-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>500</td>
<td></td>
</tr>
<tr>
<td>RS40-500</td>
<td>Rebar (3)</td>
<td>Steel</td>
<td>48.3</td>
<td>40.9</td>
<td>1630</td>
<td></td>
<td></td>
</tr>
<tr>
<td>DS49-1500</td>
<td>D-Bolt</td>
<td>Steel</td>
<td>60.1</td>
<td>49.3</td>
<td>1590</td>
<td>1500</td>
<td></td>
</tr>
<tr>
<td>DA49-1500</td>
<td></td>
<td>Aluminum</td>
<td>60.1</td>
<td>49.3</td>
<td></td>
<td>570</td>
<td></td>
</tr>
</tbody>
</table>
and slightly roughed the surface. A pair of centering pieces were manufactured and were placed at opposing ends of the cylinder or pipe in order to position the given reinforcement element. The centering pieces fixed
the reinforcement element prior to grouting and also centered the element (refer to Figure 4-5B). At the bottom end of the specimens, the reinforcement element-centering piece boundaries were completely sealed by a combination of epoxy putty and silicon sealant. For the cable specimens, an additional sealing procedure was followed to mitigate capillary movement of water and cement fines in the void between external wires and the central wire. This included opening up the strand to the position of the centering piece, applying silicon sealant to the central wire, and then closing the strand while the sealant was still within its working life. After all sealants had cured, the given specimen was vertically erected and cement grout was pumped from the bottom until completely filled. For metal pipe specimens, the grout was pumped through a 19.05 mm threaded fitting positioned at the bottom of the specimen (Figure 4-5C). For concrete cylinder specimens, grout was pumped through a 10 mm vinyl tube. The tube was fed from the top of the borehole down to the bottom of the specimen, initially positioning the end of the tube at the bottom of the borehole. The tube was then raised with the grout fill position as the grout filled the annulus between the borehole wall and reinforcement element. All samples were given, at minimum, 28 days for the cement grout to cure within a temperature and humidity controlled room (ASTM, 2018). Prior to testing, the centering pieces and the grouting tubes and fittings were removed from the specimens.

4.3.2 Coaxial Testing Apparatus
A 500 kN capacity, servo-controlled 322.41 Material Testing System (MTS) machine was used to load the reinforcement elements in the study. As shown in Figure 4-6, a testing apparatus was constructed to constrain the test specimens between two 25.4 mm thick steel plates as pull load was applied to the reinforcement element. The bottom plate (considered the attachment plate) was fixated to the MTS workbench using four T-nuts and four 19.1 mm diameter, 75 mm long bolts. Six 19.1 mm diameter steel threaded bars connected the top plate (considered the bearing plate) to the bottom plate. For each test specimen, the reinforcement element extended 100-150 mm from both ends of specimen. The element length extending from the top of the specimen was used to grasp onto with V-cut hydraulic grips and,
accordingly, was the end of the element that was loaded. All test specimens were loaded at a displacement-controlled rate of 1 mm/min. An example test specimen is shown in Figure 4-7.

It should be noted that the loaded end of the reinforcement element was initially the bottom end of the specimen during grouting. This was selected to ensure that the cement grout anulus was completely filled up to the end of the confining cylinder or pipe. This was not guaranteed at the opposing end of the test specimen as leakage and shrinkage of the grout often lowered the grout anulus several millimeters from
being flush with the top end of cylinder or pipe after fully curing. Accordingly, by using the bottom end (during grouting) of the specimen, both the grout annulus and the confining pipe/cylinder would bear off of the top plate when the reinforcement element was loaded. While this is not necessarily a replication of in-situ behaviour, since the grout annulus would not be restricted at a dilating discontinuity (e.g., Thomas, 2012), it ensured a consistent boundary condition across all of the test specimens. As shown in Figure 4-8, a 25.4 mm hole in the bearing plate was drilled to allow the reinforcement elements to extend through to
the MTS hydraulic grips. Inherently, this promoted shear failure at the element-grout interface near the bearing plate; however, this is not believed to have substantially influenced or dictated the failure behaviour of the specimens as a whole in this research effort, as the shortest encapsulated length tested was 500 mm. In comparison, a 500 mm grouted length is greater than or equal to the longest encapsulated length tested by many previous research efforts (e.g., Benmokrane et al., 1995; Blanco-Martín, 2012; Li et al., 2016; Li, 2018; Thomas, 2012), which would be more susceptible to influence from the load bearing plate.

4.3.3 Measurement Techniques

Referring to Figure 4-6, the monitoring apparatus for the pull test specimens consisted of the MTS actuator and load transducer, two linear variable differential transformers (LVDT), and a fiber optic sensor (FOS) situated along the entire encapsulated length of the given reinforcement element. The MTS actuator and the load transducer measured the applied load to the given reinforcement element and the coaxial displacement of the element at the position of the hydraulic grips. Accordingly, the displacement value measured by the actuator is the sum of the deformation of the entire reinforcement element, the deformation and the displacement of the constraining apparatus, and, if present, the differential displacement between the reinforcement element and grout and/or the grout and the confining material. The two LVDTs, therefore, were positioned in an arrangement to distinguish and compensate for constituent components of the measured actuator displacement. Referring to Figure 4-8A, one LVDT was connected either directly to the test specimen or to the top steel plate adjacent to the specimen. This measured the displacement of the specimen, which was a result of displacement and deformation of the constraining apparatus. Referring to Figure 4-8B, a second LVDT was arranged to be independent of the MTS workbench in order to measure the displacement of the reinforcement element segment extending from the unloaded end of the test specimen (which went through a hole in the workbench). The displacement of the support element at the unloaded end was the result of displacement of the specimen (measured by the previously discussed LVDT) and slip at either of the grout interfaces (if present). Therefore, the MTS and LVDT arrangement permitted the load-displacement response of the embedded reinforcement element length to be quantified during
testing (as deformation of the free length of the reinforcement element between the top steel plate and the hydraulic grippers was readily estimated from elastic theory or tensile testing results). However, a major limitation of the discussed load and displacement measurements is that they are external to the encapsulated length of the reinforcement element. As such, they provide little insight into the manner in which load is
mobilized along the length of the reinforcement element during the pull test. To capture this behaviour, each reinforcement element was instrumented with a FOS.

The fiber optic sensing technique considered in this research is centered around the application of a commercially available DOS technology from Luna Innovations (2017). This technology measures the shift of the Rayleigh backscatter spectra along the length of an optical fiber in order to resolve strain at spatial increments as low as 0.65 mm (Soller et al., 2005). Rayleigh scattering is a spontaneous loss mechanism arising from random fluctuations of the refractive index along the core of an optical fiber. This is not to be confused with fiber Bragg grating based technologies, which permanently inscribe a modulation of the refractive index within the core of an optical fiber to compose a sensor (e.g., Meltz et al., 1989). Accordingly, standard, low-cost, telecommunication optical fiber can be used as both the transducer and the lead of a FOS. The FOSs in this research were constructed from a 242 µm diameter, acrylate coated, single-mode optical fiber. This allowed strain to be measured every 0.65 mm along the length of a reinforcement element, or essentially permitted a continuous strain profile to be measured.

The procedure to instrument a reinforcement element varied depending on the element type, but always entailed the entire grouted length of the element being measured (refer to Figure 4-9). For cable elements, the procedure consisted of unwinding (or opening) the cable strand and replacing the central wire with a stainless-steel tube with a matching diameter that contained a FOS. The FOS was centered and cast within the stainless-steel tube using an epoxy resin. Being situated in close proximity to the centroid of the cable, the strain measured along the FOS was considered to be the coaxial stretch of the element. More information on this procedure and its calibration are discussed in Chapter 2 (Forbes et al., 2018). For solid bar reinforcement elements (i.e., rebars, CT-Bolts, and D-Bolts), the FOS was embedded within lengthwise, square grooves (3.0 mm by 3.0 mm) that were machined along the element’s length. Two configurations were considered: 1) A pair of diametrically opposed grooves or 2) Three grooves spaced equally at 120 degrees from each other around the circumference of the element (or a delta arrangement). In either case, the optical fiber was tensioned along the bottom of the groove (to keep the optical fiber at a constant distance...
from the centroid) and encapsulated with a LORD Fusor 108b metal bonding adhesive (both bonding the optical fiber to element and providing a protective barrier). This procedure necessitated the optical fiber to be looped within a heat shrink tube when transitioning the optical fiber into a separate groove (i.e., a single optical fiber was used to measure strain along all grooves). The loop was always situated at a position along the element that was outside of the grouted length. Further discussion on this instrumenting procedure can be found in Chapter 3 (Forbes et al., 2017), which also demonstrates that the coaxial strain at a position along the element can be determined by taking the average of the strain measured at each groove at the centroid of the element.
given position along the element. The two-groove and three-groove arrangements were both employed in order to compare the ability of each to derive coaxial strain during testing.

The FOS measurements, the applied load to the element, and the displacement measurements were recorded at 1 Hz across all tests and were triggered simultaneously (i.e., synchronized) at the start of a pull test.

4.4 Results

The following section presents a comparison of the anchoring behaviour of the cable bolt, rebar, CT-Bolt, and D-Bolt reinforcement elements under coaxial loading. For each test specimen, the load-displacement and load-deformation response curves are plotted. Displacement \((u)\) was determined according to Equation 4-2.

\[
u = u_{actuator} - u_{free\ length} - u_{LVDT\ specimen} - u_{slip}\]

where

\[
u_{slip} = u_{LVDT\ end} - u_{LVDT\ specimen}\]

It was determined that the LVDT positioned at the distal end of the element was prone to misalignment and, in select instances, detaching from the element at an arbitrary load during a given test. This was readily discernable in the LVDT measurements and was used as the cut-off load for the plotted load-displacement response curves.

Deformation \((\delta)\) was determined by numerically integrating the FOS coaxial strain distribution \((\varepsilon_{coaxial})\) that was measured along the grouted length of the test specimen from the toe end, Equation 4-4.
\[
\delta = \int_{\text{Start of FOS}}^{\text{End of FOS}} (\varepsilon_{\text{coaxial}}) \Delta x = \frac{\Delta x}{2} \sum_{i=1}^{N} (\varepsilon_{\text{coaxial},i} + \varepsilon_{\text{coaxial},i+1})
\]

Where \( \Delta x \) is the spatial resolution of the FOS, 0.65 mm. For a representative comparison of the various reinforcement element types, it was selected to convert the FOS measured coaxial strain distributions into coaxial load distributions \( F_{\text{coaxial}} \), Equation 4-5.

\[
F_{\text{coaxial}} = (\varepsilon_{\text{coaxial}}) AE
\]

Where \( A \) and \( E \) correspond to the modified cross-sectional area and the Young’s modulus of the given element, as listed in Table 4-1. The FOS measurements were also used to gain insight into how load attenuates along the reinforcement element in terms of the bond transfer efficiency at the reinforcement-grout interface. This was approached by determining the interfacial shear stress \( (\tau) \) according to Equation 4-6 (Farmer, 1975).

\[
\bar{\tau}_i = \frac{r_e E}{2\Delta x} (\varepsilon_{\text{coaxial},i+1} - \varepsilon_{\text{coaxial},i-1})
\]

Where \( r_e \) is the radius of the reinforcement element, as listed in Table 4-1.

It should be noted that the strain sensing range of the DOS interrogation unit is approximately 1.0-1.5% strain (i.e., 10,000-15,000 με). This is not the strain capacity of the FOS (i.e., the optical fiber), but, inherently, is the limit to which strain could be measured along a FOS with the selected DOS technology without multiple reference scans. The limited strain range of the FOS is less than the strain capacity of the steel reinforcement elements that were tested. Accordingly, the load-deformation response curves are typically restricted to lower loads than the corresponding load-displacement response curves. Furthermore, when the given steel element began to exhibit a plastic response (i.e., yielding), deformation was quickly
localized at the free length of the element (between the hydraulic grips and the start of the grouted length). This was often found to result in the FOS measurements becoming compromised prior to the steel element beginning to exhibit a hardening response. Therefore, the focus of the FOS measurements was limited to pre-yield loads of the steel elements. For consistency across all the test specimens, the reporting of the FOS measurements was restricted to 100 kN of applied coaxial load. As will be discussed in this section, this predominantly comprised the loading range that defined load development along the reinforcement elements.

4.4.1 Cable Bolt Specimens

The load-displacement response curves and the load-deformation response curves for the cable bolt specimens are presented in Figure 4-10. The corresponding coaxial load distributions at selected pull loads are presented in Figure 4-11 and Figure 4-12, which separates the cable specimens by the encapsulated lengths (i.e., 750 mm and 1500 mm) for visualization purposes. The FOS for test specimen CS59-1500 was damaged during the assembly of the testing apparatus. Accordingly, only the load-displacement response curve is presented for this specimen.

The initial discussion of the cable results pertains only to the cable test specimens that used a steel pipe for confinement (having a similar confining radial stiffness, Table 4-2). Referring to the load-displacement response curves, the 1500 mm encapsulated length specimens were measured to respond in a stiffer manner than the 750 mm encapsulated length specimens. The load-displacement slopes of all the cable test specimens were initially relatively equal; however, between 50-75 kN, the displacement response curves began to deviate between the two different embedment lengths. This behaviour can be attributed to the limited length of cable available to resist coaxial load at the cable-grout interface. Referring to Figure 4-11, the coaxial load was measured to have been mobilized along the entire length of the 750 mm test specimens at approximately 50 kN of applied load. In comparison, the mobilized length along the 1500 mm length test specimens (Figure 4-12) was relatively equal to the 750 mm specimens at 25 kN; however, above
25 kN, coaxial load was measured to develop continually towards the end of the encapsulated length (exceeding the limited length of the 750 mm specimens). Accordingly, the anchor performance of the cable...
A bolt was determined to have been improved through an increased encapsulation length. The increased length of grouted cable was able to collectively resist differential displacement at the cable-grout interface. At coaxial loads between 100-125 kN, load mobilized along the entire 1500 mm grouted length of the cable specimens. Therefore, the load capacity could have been further increased through additional encapsulation length. However, this statement is limited to the unconstrained end condition of the cable. Accordingly, this finding is restricted to certain in-situ conditions that are representative of the coaxial test apparatus. This could include a cable element that is installed as a tie-back (i.e., toe grouted in stable ground) or the distal ends of a fully grouted cable bolt.

Figure 4-11: Coaxial load measured along the short (750 mm) embedment length cable specimens at 25 kN loading increments up to 100 kN.
As discussed by Bawden et al. (1992), the most efficient failure mechanism, resulting in the lowest load-transfer efficiency at the cable-grout interface, is through unscrewing of the cable through the grout flutes. At the unconstrained end of the specimen (i.e., the furthest point along the grouted length), the cable is free to rotate. However, at the loaded end of the specimen, the cable is rotationally constrained by the hydraulic grippers. The cable element can be considered to have been subjected to decreasing torsional resistance when moving from the loaded to unloaded end of the specimen. Accordingly, the load transfer mechanism at the cable-grout interface transitioned from dilational slip and/or shearing of the grout flutes to non-dilational unscrewing towards the free end of the specimen. This was observable from visual post-

Figure 4-12: Coaxial load measured along the long (1500 mm) embedment length cable specimens at 25 kN loading increments up to 100 kN.
test inspection of the samples (Figure 4-13), which showed a conical failure surface at the loaded end of the cable and evidential unscrewing at the unloaded end. Referring to Figure 4-14, this resulted in the highest interfacial shear stress (or resistance to displacement) being generated over the first 0.25 m of the cable, which then dropped to lower and relatively uniform resistance further along the cable. This failure behaviour was evident for all cable specimens and provides a rationale for why the load-deformation response curves exhibited an inverse behaviour to the previously discussed load-displacement response curves (i.e., the deformation response was stiffer for 750 mm samples than the 1500 mm samples post 50 kN – Figure 4-10). For the 750 mm samples, where load was mobilized over the entire encapsulated length between 25-50 kN, displacement of the specimen became primarily associated with slip and non-dilational unscrewing of the element at a lower load than for the 1500 mm sample.

Referring to Figure 4-11 and Figure 4-12, the diameter of the borehole, defining the thickness of the grout annulus, was determined to have a distinguishable impact on the load transfer efficiency of the cable. For both the 750 mm and 1500 mm long specimens, the slope of the coaxial load distributions was

Figure 4-13: Post-test inspection of test specimen CS49-1500. A) View of the loaded end of the test specimen (i.e., 0 m on the strain profiles). A conical failure surface is evident. B) View of the distal end of the test specimen (i.e., 1.50 m on the strain profile). The unscrewing failure mechanism is evident – the cable spun through the grout flouts.
measured to become less steep with increased borehole diameter. This can be attributed to the thicker grout annulus promoting a less efficient transfer of radial confinement by the steel pipe, ultimately resulting in less pressure being generated at the cable-grout interface. While the larger borehole diameters may have resulted in only a slight reduction in the interface pressure that was generated, plain strand cables are known to be very sensitive to radial confinement (Hyett et al., 1995; Yazici and Kaiser, 1992). There was not a clearly identifiable trend regarding the size of the borehole diameter on the load-displacement response;

Figure 4-14: Comparison between the shear stress distribution at the cable-grout interface and the coaxial load distribution corresponding to 100 kN of applied load to test specimen CS49-1500. Positive shear stress denotes differential movement of the cable relative to the grout. Note: A 50-point moving average (i.e., 32.5 mm interval) was applied to the shear stress distribution in order to reduce the amplification of measurement noise introduced through numerical differentiation.
however, as discussed, the slight performance impact was discernable with the FOS measurements. Quite interestingly, the aluminum confining pipe resulted in the shortest coaxial load mobilization length along the cable for the presented applied loads. This result is unanticipated since the aluminum pipe provides comparatively less radial confinement than the steel pipe (refer to test specimens CS49-1500 and CA49-1500). Accordingly, less pressure would be expected to have been generate at the cable-grout interface, resulting in an increased load mobilization length per applied load. However, referring to Figure 4-10, the aluminum pipe specimen resulted in a much lower load capacity than the steel pipe specimen (also found by Hyett et al., (1992)) and also a reduced residual load carrying capacity. This is attributed to radial splitting of the grout annulus as the tensile strength of the grout was more readily exceeded from the reduced, passive confinement provided by the aluminum pipe in comparison to the steel pipe. Accordingly, with increased applied load, radial cracks in the grout would propagate along the length of the test specimen. While this may have initially promoted a greater geometric mismatch between the cable and the grout (promoting reduced load development), it ultimately resulted in a lower load carrying capacity.

In addition to the discussed findings for the cable bolt specimens, it should be noted that the cable strand did not fail during any of the coaxial pull tests. Under the unconstrained testing apparatus, this indicates that a longer encapsulation length would have been required in order to generate enough non-dilation, frictional resistance at the cable-grout interface to fail the strand. The shear stress distribution indicated an average shear resistance of 2.25 MPa acted over a majority of the specimen. Using this measured value, the critical encapsulation length required to fail the strand would have been approximately 2.5 m (for the steel samples). The relatively uniform shear stress distribution suggests that an average shear stress capacity that could be quantified from a short encapsulation length specimen is relevant to longer cable samples.

More testing is required to definitively state the influence of different radial stiffness conditions, but it can be inferred that a lesser radial stiffness would necessitate even longer encapsulation lengths. Therefore, in weaker and softer ground masses, or projects that require resisting loads near the capacity of
the cable stand, it is recommended that modified cable bolt geometries be considered (e.g., Windsor, 1992). Modified cable bolt geometries have been determined to produce radial dilations that are, at minimum, an order of magnitude greater than those produced by plain strand cables (e.g., Moosavi et al., 2002) and, therefore, are much less sensitive to radial confinement. Nevertheless, it must be recognized that the improved load transfer efficiency of a modified cable will reduce the displacement capacity of the element.

4.4.2 Bar Specimens

The load-displacement response curves and the load-deformation response curves for the rebar, CT-Bolt, and D-Bolt specimens are presented in Figure 4-15. The displacement response was measured to be very similar for the elastic loading range of all the elements; although, the CT-Bolts had a significantly higher load capacity as they were manufactured from a higher-grade of steel (Table 4-1). In terms of the deformation response to applied load, the stiffest to most ductile reinforcement element followed in the order of the rebars, the CT-Bolts, and then the D-Bolts. This was an expected outcome considering the difference in the extent of the irregularity of the external profile of the given elements. As discussed in Section 4.2, the rebar (including the rebar used for the CT-Bolts) had a much closer spacing between the adjacent deformed ribs along the bars than the D-Bolt had for the spacing between adjacent anchor positions along the bar. Accordingly, the D-Bolt was expected to experience the most deformation of the bar elements. The CT-Bolt was also expected to have deformed more than the rebar as the plastic sheath was anticipated to inhibit passive confinement feedback from the steel pipe. However, the extent to which load transfer was reduced was only discernable from the FOS measurements.

4.4.2.1 Rebar Specimens

Figure 4-16 presents the coaxial load distributions measured along the rebar specimens. Depending on the confining material and embedment length, coaxial load was measured to have been mobilized along 0.40-0.45 m of the element. This load development length was measured to be consistent up to failure of the element and therefore represents the critical encapsulation length or grouted length required to fail the element shank (a similar length was also reported by Li et al., 2016). This indicates that a much greater load
transfer efficiency was generated by the mechanical interlock mechanism of the rebar-grout interface in comparison to the frictional-dilation interface mechanism reported for the cable specimens. Referring to 4-17, the shear stress distribution at the rebar-grout interface was measured to generate a much higher shear...
stress concentration near the position of applied load that then attenuated in an exponential manner along the element (as proposed by Farmer, 1975). An additional insight into the mechanical interlocking mechanism that was obtained by the FOS measurements is that the full load development length of the rebar (i.e., 0.40-0.45 m) was mobilized at very low applied loads (under 25 kN). Therefore, load development length through mechanical interlock is semi-independent of applied load. Only a small amount of load and relative displacement at the rebar-grout was required to activate the entire critical encapsulation length. The mobilized length of the rebar will only extend if lack of confinement or the strength of the grout compromises the load transfer interface (e.g., through radial splitting or parallel shearing of the grout – Cao

Figure 4-16: Coaxial load measured along the rebar specimens at 25 kN loading increments up to 100 kN. Note: Load was not applied above 75 kN for test specimen RC31-500.
et al., 2013), in which case a decoupling front will emerge (Li and Stillborg, 1999). In comparison, load development length was measured to have a strong dependence on applied load for the cable specimens, with progressive lengths of the cable being mobilized by increased applied load. It is very important that an analytical or numerical model of such reinforcement elements is able to account for this difference in mechanistic behaviour in order for coaxial load to accurately be predicted along the element.

4.4.2.2 CT-Bolt Specimens

Figure 4-18 presents selected coaxial load distributions measured along the CT-Bolt specimens. The construction of the CT-Bolt includes a 2mm thick high-density-polyethylene (HDPE) sheath that effectively separates the encapsulating grout into two separate grout annuli: 1) A grout annulus between the rebar and the sheath, and, 2) A grout annulus between the sheath and the confining pipe (i.e., the borehole wall). The sheath provides an efficient procedure to cement grout the bolt in-situ as well as an additional protective barrier to corrosion; however, the sheath is also anticipated to inherently inhibit the load transfer efficiency of the bolt. The relatively soft plastic sheath creates a preferential coaxial shear failure plane (parallel to bolt’s axis), softens the confinement feedback between the confining borehole and the rebar, and reduces the tensile capacity of grout annulus (i.e., the inner annulus will be more prone to radial splitting). Accordingly, coaxial load was measured to have been mobilized over the full 1.00 m encapsulated length of the CT-Bolt. Referring to Figure 4-19, the shear stress distribution at the CT-bolt-grout interface was determined to attenuate over an extended length and at a lower resistance than the rebar elements discussed previously (Figure 4-17). Although similar to the rebar specimens, the full load development length mobilized below 25 kN of applied load (albeit more encapsulating length being required).

The rebar used for the CT-Bolt specimens had a less geometrically varying external profile than the rebar used for the rebar specimens (Table 4-1). Therefore, in order to further investigate the impact of the HDPE sheath, it was selected to conduct a coaxial pull test on a CT-Bolt rebar without the sheath (test specimen CTS49-1000(ns)). The load transfer efficiency was slightly improved by removing the sheath (i.e., a steeper load gradient was measured in comparison to the sheathed samples – Figure 4-18), but not
nearly to the extent that it was comparable with the rebar specimen (Figure 4-16). Therefore, while the sheath did inhibit the load transfer to a measurable capacity, the predominant reduction in the load transfer efficiency is attributed to the difference in the deformed rib profiles of the rebar specimens and the CT-Bolt specimens; the CT-Bolt specimens having a shallow rib profile. The deformed rib profile (including rib height, spacing, and face angle) has been experimentally determined to impact the bond capacity by several other research efforts (e.g., Cao, 2012; Jalalifar et al., 2006; Kilic et al., 2003; Tao et al., 2017), but there is

Figure 4-17: Comparison between the shear stress distribution at the rebar-grout interface and the coaxial load distribution corresponding to 100 kN of applied load to test specimen RS40-500. Positive shear stress denotes differential movement of the rebar relative to the grout. Note: A 50-point moving average (i.e., 32.5 mm interval) was applied to the shear stress distribution in order to reduce the amplification of measurement noise introduced through numerical differentiation.
a massive benefit to be gained by conducting a series of parametric experiments on the deformed rib’s influence on load distribution through further application of the FOS technique presented in this study.

4.4.2.3 D-Bolt Specimens

As discussed in Section 4.2, the D-Bolt construction consists of a smooth steel bar with periodically spaced deformed anchors. The smooth bar segments along the D-Bolt provide little to no load transfer resistance at the bolt-grout interface and, therefore, are free to deform between adjacent anchor positions.

Figure 4-18: Coaxial load measured along the CT-Bolt specimens at 25 kN loading increments up to 100 kN. Note: The load distribution of CTS49-1000(1) was nearly equal to CTS49-1000(2); although much greater measurement noise was present. Therefore, for visualization purposes of comparing with the no sheath sample, CTS49-1000(1) has not been shown.
Accordingly, the position of the D-Bolt anchors within a coaxial test specimen can potentially have a significant influence on the load-displacement response. Figure 4-20 displays the anchor arrangement considered for the D-Bolt specimens. The first anchor position was situated approximately 0.25 m from the start of the encapsulated length (i.e., the position of applied load) and the anchors had a center-center spacing of 0.875 m.

Figure 4-19: Comparison between the shear stress distribution at the CT-Bolt-grout interface and the coaxial load distribution corresponding to 100 kN of applied load to test specimen CTS49-1000(2). Positive shear stress denotes differential movement of the CT-Bolt relative to the grout. Note: A 50-point moving average (i.e., 32.5 mm interval) was applied to the shear stress distribution in order to reduce the amplification of measurement noise introduced through numerical differentiation.
Figure 4-21 presents selected coaxial load distribution measured along the D-Bolts and Figure 4-22 presents the corresponding interface shear stress distribution for test specimen DS49-1500. Like the rebar element, the D-Bolt primarily resisted load through mechanical interlock at the D-Bolt-grout interface; however, where the rebar has many, closely spaced deformed ribs, the D-Bolt has a limited number of deformed anchors. Accordingly, interface shear stress is predominantly generated at the anchor positions; however, a low-level of frictional induced shear stress was also measured along the smooth bar sections. When coaxial loading is induced by a single source, as in the case of a coaxial pull test, the anchor position closest to the loading source will be mobilized to a much larger extent than the subsequent anchor position(s) along the bar. In this regard, the D-Bolt is a very fitting reinforcement element for blocky ground mass conditions that may produce multiple, discrete coaxial loading sources along the reinforcement element since a single anchor location would be anticipated to be able to withstand loads past the yield capacity of the smooth bar segment.

Referring to Figure 4-21, similar to the results discussed for the cable bolt elements, the use of an aluminum pipe for radial confinement was found to reduce the load development length of the element. Again, this is a counter-intuitive finding. The constant radial stiffness boundary conditions considered in the coaxial pull tests resulted in confinement being applied in a passive manner. Since the aluminum pipe has a lower radial stiffness than the steel pipe (Table 4-2), it would be expected to result in less radial pressure being generated at the element-grout interface, resulting a in lower load transfer efficiency and,
therefore, an increased load development length. Nevertheless, both the cable and D-Bolt specimens tested with an aluminum pipe indicated that at pre-yield loads, load development length was reduced for lower radial stiffness. It is recommended that more coaxial pull tests be conducted with confining pipes with various radial stiffnesses.

4.5 Discussion and Overview of Findings

This research has discussed a coaxial pull test apparatus and a measurement technique that was used in order to measure the load-displacement response of a reinforcement element under coaxial load as well as
to measure the coaxial load distribution along the element with DOS. Four reinforcement element variations, including: rebars, plain strand cables, CT-Bolts, and D-Bolts were instrumented with FOSs and cement grouted into either a confining metal pipe or concrete cylinder that replicated the confinement of a host ground mass. Grout encapsulation lengths were tested between 0.5-1.5 m. These lengths are much longer than what traditionally has been considered in many laboratory pull tests because of the inherent inability of discrete, external measurement techniques to accurately quantify non-uniform coaxial load.

Figure 4-22: Comparison between the shear stress distribution at the D-Bolt-grout interface and the coaxial load distribution corresponding to 100 kN of applied load to test specimen DS49-1500. Positive shear stress denotes differential movement of the D-Bolt relative to the grout. Note: A 50-point moving average (i.e., 32.5 mm interval) was applied to the shear stress distribution in order to reduce the amplification of measurement noise introduced through numerical differentiation.
distributions along CMC elements with bonded length to element diameter ratios in excess of 10. Using DOS, such behaviour was readily measured for loads at and below the elastic threshold of the given reinforcement element, with coaxial strain being measured at 0.65 mm spatial increments along the test specimens. This allowed the load transfer efficiency of the various reinforcement elements to be experimentally investigated and evaluated in an unprecedented manner in comparison to existing coaxial load studies.

Figure 4-23 presents a comparison of the load-displacement response curves and the load-deformation response curves categorized by the reinforcement element type. Similarly, Figure 4-24 presents a comparison of the coaxial load distribution and the interface shear stress for a selected test specimen from each reinforcement element type at 100 kN of applied coaxial load. The cable elements were determined to have softest displacement response to coaxial load and were also measured to generate the lowest magnitude of shear stress resistance at the element-grout interface. This was attributed to the frictional-dilational mechanism that governed the bond strength of the cable elements as well as the tendency of the cable to unscrew through the grout due to the element’s relatively low torsional rigidity. In comparison, the solid bar elements (i.e., the rebars, the CT-Bolts, and the D-Bolts) generated bond stress at the element-grout interface primarily through mechanical interlock, which was attributed to the irregular surface of the given element. This was determined to produce a stiffer load transfer response than the that for the cable elements and, accordingly, the coaxial load development length was less for bar elements. However, the spacing and profile of the deformed ribs/anchors was also determined to greatly influence mechanical interlock load transfer at the element-grout interface. For example, the slight variation in the deformed rib geometry between the rebar and CT-Bolt elements was measured to have a significantly greater impact on bond efficiency than the inclusion of the soft, HDPE sheath of the CT-Bolt (which effectively disassociated the grout annulus).

The FOS measurements indicated that when mechanical interlock is the predominant bond mechanism, load development length from the source of coaxial load is relatively independent of loading
magnitude after 25 kN has been reached. In addition, the interface shear stress distribution associated with mechanical interlock was measured to be non-uniform and, therefore, an average bond stress per length of

Figure 4-23: Top: Coaxial Load-displacement response curves for all reinforcement elements. Bottom: Coaxial Load-deformation response curves for all reinforcement elements. Displacement has been determined from the measured actuator stroke and the LVDTs. Deformation has been determined from the coaxial strain measured along the given element with the FOS. Note: The deformation scale is 10% of the displacement scale. For the cable specimens, short and long embedment length refers to the 750 mm and 1500 mm samples, respectively.
In comparison, when a frictional-dilational mechanism governs bond behaviour, load development length was strongly dependent on the magnitude of the element will not provide a representative bond capacity. In comparison, when a frictional-dilational mechanism governs bond behaviour, load development length was strongly dependent on the magnitude of

Figure 4-24: Comparison of the load transfer response between the rebar, cable, CT-Bolt, and D-Bolt reinforcement elements. Top: Comparison of the coaxial load distributions measured along the various reinforcement element types at 100 kN of applied load. Bottom: Comparison of the interface shear stress distributions of the various reinforcement element types at 100 kN of applied load. Note: The rebar, cable, CT-Bolt, and D-Bolt load distributions and shear stress distribution correspond to test specimens RS40-500, CS49-1500, CTS49-1000(2), and DS49-1500, respectively (refer to Table 4-2).
load. The mechanism also resulted in relatively uniform bond distribution being measured (as it is primarily frictional). This insight into the mechanistic behaviour of each reinforcement element was not discernable from the conventional, load-displacement response curves.

An additional outcome that is worthy of consideration when conducting coaxial pull tests with longer specimen lengths (such as those in this study) is the relative difficulty to associate the measured displacement with the various components of the test apparatus. For example, the deformation response within the elastic limit of the given element were generally 10% of the displacement response (Figure 4-23). Accordingly, the displacement measurements (even after correcting for apparatus deformation and element slip) would substantially underestimate the shear stress generated at the element-grout interface. A similar finding was also discussed by Salcher and Bertuzzi (2018) when comparing the bond capacity determined from a collection of in-situ pull tests (with long encapsulation lengths) and the bond capacity determined from laboratory experiments (with short encapsulation lengths) and was attributed to the accumulation of deformations of the entire pull test system.
4.6 References


Chapter 5 Augmenting the in-situ rock bolt pull test with distributed optical fiber strain sensing

5.1 Introduction

The coaxial pull test is a commonly practiced procedure for assessing the in-situ load capacity of tendon support elements, including rock bolts, cables, and other ground anchor variations. The procedure involves installing the support element in agreement with the normal operational procedures/protocols at the given site and subsequently applying coaxial load to the support element in a monotonic or cyclic manner. Load is often applied using a hydraulic ram assembly, comprised of (i) a hollow plunger hydraulic cylinder, (ii) a coupling apparatus to connect the support element to the hydraulic ram, (iii) load bearing components, and (iv) load and displacement measurement equipment. An example pull test assembly is displayed in Figure 5-1.

There are several standards (ASTM, 2013a; CEN, 2013; Lardner and Littlejohn, 1985) that recommend procedures to be followed for the support element pull test. Main factors covered in these standards include pull test setup, loading procedure, required accuracy and precision of the measurement techniques, and reporting of load-displacement data. Discussion is also provided in terms of the role or purpose of the in-situ pull test with respect to desired outcomes for an assessment test or an acceptance test. Assessment tests are performed in order to evaluate the in-situ load transfer efficiency of a particular support element and seek to determine characteristic features of the support system, including:

1) the working capacity (the magnitude of load corresponding to the onset of non-linearity or yield);
2) the ultimate capacity (the maximum magnitude of load that can be withstood by the support element);
3) the stiffness of the support system (load versus displacement), and;
4) the bond strength (the shear stress capacity at the interface between the support element and confining material) (Hadjigeorgiou and Tomasone, 2018; Hagan et al., 2014).
The amalgamation of these relative performance features allows recommendations to be made on support element type, size, spacing, quantity, length, and design load at a given project. In contrast, acceptance tests are conducted in order to verify that a support element is functioning as per the design or that it has been installed properly (within a quality assurance framework). In both cases, the pull test is often regarded as a short-term evaluation; however, modifications can be made for long-term assessment (ASTM, 2013b).

The pull test is generally a simple and inexpensive test to conduct; however, there are limitations regarding the pull test’s ability to function as a support performance characterization technique or as a quality assurance / quality assessment tool. One major constraint is the external nature of measurement techniques during a test. The vast majority of in-situ pull tests only measure load and displacement at the support element’s head or along a portion of the support element that is accessible at the excavation periphery. This is particularly problematic for pull tests that are conducted on support elements that are either fully or partially coupled to the ground mass by means of cement grout or resin encapsulation within a borehole. Along the encapsulated length of such support elements, the load and bond stress distribution

Figure 5-1: Example in-situ pull test unit in an underground mine (Modified after Nicholson, 2016).
may not necessarily be uniform (Benmokrane et al., 1995; Blanco Martín et al., 2011; Hyett et al., 1992; Li and Stillborg, 1999; Liu et al., 2017). Accordingly, in the absence of easily identifiable pull out (or slip) during a pull test, it is very difficult to accurately quantify the bond efficiency at the support element-grout interface.

The bond strength is necessary to ascertain as it is recognized to heavily influence the overall performance of a support element and it will directly dictate the critical encapsulation length required to reach ultimate load carrying capacity of the support element (Li et al., 2016). It is recommended that pull tests being conducted to assess bond strength should consider an encapsulated length that is often much shorter than that specified by the project design (ASTM, 2013a; Bawden et al., 1992; Hutchinson and Diederichs, 1996). This helps promote shear failure (i.e., slip) at the interface between the support element and its confining material, as opposed to failing the element itself in tension. Nevertheless, this still requires the bond strength to be averaged over the given encapsulation length. Under laboratory pull test configurations (ASTM International, 2010), the load transfer efficiency along the axis of various encapsulated support elements has been investigated using multiple discrete sensors such as foil-resistive strain gauges (Chen, 2014; Grasselli, 2005; Jalalifar, 2006; Rong et al., 2004; Serbousek and Signer, 1987); however, the use of such technologies in-situ has been limited. This has been attributed to the difficult working environments affiliated with in-situ pull tests, restricting instrumentation due to delicacy and setup time (Nicholson, 2016).

There is a benefit to be gained from a sensing method that can measure the load distribution along an encapsulated support element during an in-situ pull test. However, wide implementation of such a measurement technique would require it to generate minimal impact to the standard pull test procedure in order to reduce or eliminate conformance. Within this context, a technique using distributed optical fiber strain sensing (DOS) is presented as a sensing based methodology to augment the assessment of ground anchors/support elements during in-situ pull tests. The DOS technique is robust, requiring no alteration to the support element installation procedure or additional handling awareness and is capable of measuring a
virtually continuous load distribution along a given support element (a measurement obtained every 0.65 mm along the axis of the support element). This is demonstrated to remarkably increase the comprehension of support element behaviour during in-situ pull tests by quantifying load development length and the shear stress distribution along fiber optic instrumented rebars installed in an underground mine.

5.2 DOS Pull Test Arrangement

The technique to augment the in-situ pull test is centered around the application of a commercially available DOS technology form Luna Innovations (Luna Innovations, 2019a). The given technology utilizes optical frequency domain reflectometry (OFDR) to measure the Rayleigh backscatter component of light along the length of a standard, low-cost optical fiber (OF) (Glombitza and Brinkmeyer, 1993; Soller et al., 2005). In comparison to other commercially available quasi-distributed fiber Bragg grating systems (Luna Innovations, 2019b; Meltz et al., 1989) and other DOS solutions (Horiguchi and Tateda, 1989; Niklès et al., 1996), Rayleigh-OFRD has been found to be particularly fit for application with common ground support elements (Chapter 2 - Forbes et al., 2018). The most enticing feature of this sensing system is its ability to measure strain every 0.65 mm along the length of a fiber optic sensor (FOS), which corresponds to potentially thousands of strain measurement locations when coupled with a support element. However, the method to couple a FOS with a support element is non-trivial. For use in-situ, and to achieve wide acceptance within industry, a FOS instrumented support element must be able to be installed in the same manner as a normal support element and should remain to function (mechanistically) as a normal support element. The FOS-support element coupling method will vary depending on the support element type, but the conceptual benefit of measuring a virtuously continuous strain profile along a support element is indisputable. The following sections describe how FOSs were coupled with rebar support elements and how an in-situ pull test unit was modified to test the FOS instrumented rebars in an underground salt mine.

5.2.1 Rebar Specimens

This study considered two sizes of rebar that were sourced from DSI Underground Canada Ltd. The general parameters of the rebars are provided in Table 5-1. Each rebar was instrumented with a single polyimide
coated OF having a diameter of 155 µm. The instrumenting procedure involved embedding and straightening the OF along the bottom surface of a pair of diametrically opposed square grooves (2.5 mm wide by 2.5 mm deep) that were machined along the length of each rebar (Hyett et al., 2013; Iten, 2011; Zhao et al., 2018). A semicircle groove with the same width and depth was also milled at 0.19 m from the toe end of the rebar to connect the two grooves. The OF was then encapsulated within the grooves using a LORD Fusor 108b metal bonding adhesive. This bonds the OF to the rebar and also provides a protective barrier. Figure 5-2 displays a general schematic of the FOS along the length of the rebar.

At the head end of each rebar a 6 mm diameter, 28.5 mm long centered hole was drilled in order to feed the OF from the groove down into the core of the rebar where a male OF connector could be fitted. This is a necessary procedure in order to allow a nut to be threaded onto the head end of the rebar for installation purposes without risking damage to the FOS. It also allows for a protective cap to be threaded onto the rebar to protect the OF connector during handling and installation (Figure 5-3). This instrumenting procedure ensures that no portion of the FOS is directly exposed at any stage other than when connecting it with the DOS measurement unit.

![Figure 5-2: General layout of the FOS along a rebar. A single 155 µm diameter optical fiber is embedded along the length of diametrical opposed grooves machined along the rebar. The FOS initiates with an FOS connector at the head of the rebar, is looped within a milled semicircle 0.19 m from the toe end of the rebar, and is terminated near the head end threads. The active sensing length is defined as the length of the FOS that runs straight along the rebar and is the length analyzed further in this study. Not to scale.](image-url)
Table 5-1: Rebar size and strength parameters.

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<th>Value 2</th>
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<td>Minimum Yield Strength (MPa)</td>
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</tr>
<tr>
<td>Minimum Tensile Strength (MPa)</td>
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<td>620</td>
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<tr>
<td>Elastic Modulus (GPa)</td>
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5.2.2 Modified Pull Test Apparatus for DOS

Pull test units will often use a coupler piece to connect a given support element to an extension rod that will pass through a loading frame and hydraulic cylinder (Figure 5-1). The extension rod will be fastened at the head of the hydraulic cylinder to transmit load to the support element while the loading frame bears off the ground mass. Several modifications to the conventional in-situ pull test unit were made by the author to accommodate the FOS. Referring to Figure 5-4A, the male OF connector at the head of the rock bolt must be connected with an OF lead cable to then connect with the DOS measurement unit. This necessitates an

Figure 5-3: A – Example FOS instrumented rebars and the DOS measurement unit. B – Example stainless steel protective cap used to cover the OF connector during handling and installation.
aperture for the OF lead cable to exit the coupler piece and the load frame. Accordingly, custom coupler pieces and a loading frame were manufactured, which are displayed in Figure 5-4B and Figure 5-4C, respectively.
Table 5-2: FOS instrumented rebar variations. Expansion anchors were sourced from DSI Underground Canada LTD (73 mm length for 35 mm borehole) and were threaded onto the toe threads of specified rebar prior to installation. J-LOK resin cartridges were sourced from Jennmar USA (31.75 mm diameter, 762 mm length, 120 second gel time) and were inserted into the borehole prior to installing and spinning specified rebar. The calculated encapsulation fill position is referenced from the head of the rebar and is a volume calculation that assumes the resin completely fills the toe end of the borehole prior to filling the annulus between the rebar and borehole wall.

<table>
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<td>Anchor Method</td>
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<tr>
<td>Calculated Encapsulation Fill Position (mm)</td>
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<td>1480</td>
<td>640</td>
</tr>
</tbody>
</table>

In addition to the FOS, a digital measurement system comprised of a in-line pressure transducer and a magneto-strictive displacement sensor (YieldPoint Inc., 2018) was used to measure the applied load and stroke of the hydraulic cylinder, respectively. These two instruments were connected with a handheld data acquisition. As shown in Figure 5-4C, the displacement sensor is directly mounted on the hydraulic cylinder. This deviates from standard (ASTM, 2013a) and suggested method (Lardner and Littlejohn, 1985) procedures that recommend the support element displacement to be measured from a stable reference position (as the ground mass bearing the pull test unit may compress during a test). However, the utilized digital measurement system is portable and convenient for use in-situ and it autonomously records load and displacement during a pull test. In comparison, standard (ASTM, 2013a) and suggested method (Lardner and Littlejohn, 1985) procedures discuss manual readout and recording of load and displacement, necessitating many small loading increments in order to capture a full load-displacement profile. Santos et al. (2015) have demonstrated an automated pull test unit designed to meet or exceed standards recommended for monitoring load and displacement during in-situ pull tests; however, this technique is
much more burdensome and intricate to set up and has thus far been unproven within an operational underground environment. The complete modified pull test apparatus for DOS is shown in Figure 5-5.

![Modified pull test apparatus for operation with DOS.](image)

**Figure 5-5: Modified pull test apparatus for operation with DOS.**

### 5.3 Pull Test Results

A series of in-situ pull tests were conducted on various FOS instrumented rebars at an active domal salt mine located in southern Louisiana, US. Selected load-displacement results and strain distributions are presented for three rebars as overviewed in Table 5-2. Each rebar was installed into a freshly drilled horizontal borehole and torque tensioned to 50 kN using a bolting jumbo. For consistency of the ground mass across each pull test, all FOS instrumented rebars were installed along a 25 m section of a drift sidewall. A minimum 3 meters of spacing was provided between adjacent rebars that were pull tested.

The 50 kN tension applied by the bolting jumbo was released with a hand wrench prior to setting up the pull test unit. A minimum 25.4 mm length of the coupler piece was threaded onto the given rebar (Figure 5-4B). After the pull test unit was fully assembled (Figure 5-4C) a 5 kN preload was initially applied...
to seat the pull test assembly with the sidewall and remove any slack between constituent components (also suggest by ISRM (Lardner and Littlejohn, 1985)). Load and displacement were zeroed at the 5 kN preload. If loading was performed cyclically, load was not fully released. This prevented resetting or shifting of the pull test unit between loading cycles. The maximum applied load was limited to 90 kN for the 19.05 mm diameter rebars and 125 kN for the 22.22 mm diameter rebars. These load values were selected to avoid failure of the given rebar’s head threads during testing, which would dictate the ultimate load capacity of the rebars tested. Load was applied and released using a hydraulic hand pump.

5.3.1 Load-Displacement Measurements

Applied load and support element head displacement are the traditional measurements taken during an in-situ pull test. Using the digital instrumentation, hydraulic cylinder pressure and stroke were logged at a rate

![Figure 5-6: Load-displacement response curve measured for FOS_02. Load was applied in five cycles. A solid line indicates the loading domain of the load cycle, while a dashed line indicates the unloading domain. Theoretical elastic deformation of the pull test assembly at a given load has been removed from the displacement measurements.]()
of 1 Hz. The former is readily converted to load and was visualized in real-time to assist in applying the load during each pull test. Figure 5-6 displays a selected load-displacement response curve measured for FOS_02 (Table 5-2). This curve is the result of linearly interpolating between the logged load and cylinder stroke measurements. Load was applied to each FOS instrumented rebar at approximately 50 kN per minute; however, it was difficult to unload in a similarly controlled manner using the pressure release knob on the hand pump. This resulted in portions of the load being released much faster than the 1 Hz logging interval. Accordingly, the unloading domain of each load cycle displays an artificial hysteresis. Referring to Figure 5-7, each load cycle can be further separated into a reloading segment and loading segment (except for the first load cycle). The reloading segment of the subsequent load cycle is believed to provide a more

![Figure 5-7: Load-displacement response of load cycle 3 for FOS_02 (refer to Figure 5-5). The reloading segment transitions to the loading segment at the maximum applied load of the previous load cycle.](image-url)
accurate representation of the unloading behaviour, elastic recovery of the given rebar, and the displacement at the head of the rebar. Large unrecovered displacement after each loading segment is primarily attributed to compression of the ground mass. On the contrary, an almost fully elastic response resulted from each reload segment (refer to load cycle 2 in Figure 5-6). This behaviour has also been noted under laboratory cyclical pull tests (Feng et al., 2018).

Across all pull tests the load-displacement response of the reloading segment was stiffer than the loading segment. A summary of the secant stiffness from digital load and displacement measurements across all pull tests is presented in Table 5-3.

Table 5-3: Mean secant stiffness determined for the reloading and loading segment for all pull test load cycles.

<table>
<thead>
<tr>
<th>Number of Samples</th>
<th>Mean Stiffness (kN/mm)</th>
<th>Sample Standard Deviation (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reload Segment</td>
<td>10</td>
<td>23.28</td>
</tr>
<tr>
<td>Load Segment</td>
<td>12</td>
<td>11.04</td>
</tr>
</tbody>
</table>

5.3.2 Rebar Strain Distributions

The benefit of looping the OF sensor in order to measure strain along diametrically opposed sides of the rebar is that the coaxial strain can be distinguished from the bending moment induced strain at any position along the active sensing length \( i \) according to Equation 5-1.

\[
\varepsilon_{ coaxial_i } = \frac{\varepsilon_{ side 1, i } + \varepsilon_{ side 2, i } }{2}
\]

Previous experience of the author indicates that it is extremely difficult to apply a purely coaxial load to a support element, even under controlled laboratory conditions (Forbes et al., 2015). Any misalignment between the rebar and the pull test unit (e.g., from bearing off an uneven surface) will result in a component of bending. While a support element will generally be expected to take on a combination of coaxial and
bending moment induced strain from in-situ ground mass loading throughout its serviceability life, it is most informative to observe only the coaxial strain (or bending moment compensated strain) under pull test loading.

Each FOS instrumented rebar was monitored at 1 Hz with the DOS sensing unit during pull tests. Monitoring rebar strain was triggered simultaneously with the digital instrumentation for comparison with load and displacement measurements. Figure 5-8 displays coaxial strain distributions measured along the active sensing length of FOS_03 at selected levels of applied load. There are two apparent loading

![Diagram of rebar with free and anchored lengths](image)

**Figure 5-8:** Strain distributions measured along the active sensing length of FOS_03 at selected applied loads.
behaviour regions along the length of the rebar. Between the head of the rebar and position of resin encapsulation (from 0 m to 0.78 m) a uniform strain distribution is measured. This identifies this segment of the rebar to have acted as an end loaded member or as a free length. From the position of resin encapsulation to the toe end of the rebar (from 0.78 m to 1.8 m) the measured strain exponentially decays from the uniform strain level to zero towards the end of the active sensing length. This identifies this segment of the rebar to have acted as the anchored length, encapsulated in resin. The transition point between the free length and the anchored length of rebar was consistent across all levels of applied load;

Figure 5-9: 100 με scale view of strain distributions measured along the encapsulated length of FOS_03 at selected applied loads. A Hampel filter has been used to reduce the low-strain measurement noise for visualization purposes (solid distribution versus shaded distribution).
however, the length of rebar mobilized within the resin encapsulation was found to increase with applied load (Figure 5-9).

As shown in Table 5-2, three installation methods were considered between the FOS instrumented rebars. Figure 5-10 displays a comparison of the strain distributions measured along the active sensing lengths of FOS_01, FOS_02, and FOS_03 at an applied load corresponding to 300 MPa for the given rebar diameter. The measured strain distributions clearly indicate a variation in the load mobilization along the rebar depending on the installation method. FOS_01 was coupled to the ground mass using an expansion anchor at the toe end of the rebar, which is beyond the active sensing length of the FOS. Accordingly, a solely uniform strain distribution was measured, indicating the rebar was acting as an end loaded member (or free length). As previously discussed for FOS_03, partial resin encapsulation differentiates the loading behaviour of the rebar into a free length and anchored length. The strain distribution for FOS_02, bonded to the ground mass using a single resin cartridge, identifies the resin encapsulation to have reached 1.41 m from the head of the rebar. The strain distribution for FOS_03, bonded to the ground mass using two resin cartridges, identifies an increased encapsulation length with resin encapsulation reaching 0.78 m from the head of the rebar. The FOS determined encapsulation lengths agree with predictions based on volume calculations (Table 5-2). Furthermore, the average strain magnitude measured along the free length of the rebar compares within ±5% of the strain magnitude predicted by elastic theory.

**5.3.3 Rebar Bond Stress Distribution**

In the absence of support element pull out (or slip) during a pull test, the average shear stress ($\bar{\tau}$) across the encapsulated length ($L$) at a given load ($P$) can be calculated from Equation 5-2 (Farmer, 1975)

$$\bar{\tau} = \frac{P}{2\pi rL}$$

where ($r$) is the effective radius of the support element. It has been reported that the shear stress distribution at the support element-grout interface will only be uniform at very short encapsulation lengths, less than
4 - 5 times the support element diameter (Benmokrane et al., 1995; Hyett et al., 1992). Such short encapsulations lengths are very rarely trialed in-situ, meaning the shear stress distribution will not be uniform along a support element during an in-situ pull test, even in the presence of pull out.

![Diagram showing free length and anchored length for FOS_01, FOS_02, and FOS_03](image)

**Figure 5-10**: Strain distribution comparison between FOS_01, FOS_02, and FOS_03. Top: Sketches depicting the installation variations between each FOS instrumented rebar. Bottom: Strain measured along the active sensing length of each FOS instrumented rebar at an applied load equivalent to 300 MPa (85 kN for FOS_01 and FOS_03 and 115 kN for FOS_02).
Equation 5-2 can be rewritten to determine the mean shear stress between adjacent strain measurement positions along a FOS (Equation 5-3)

\[ \bar{\tau}_i = \frac{rE}{2\Delta x} (\varepsilon_{\text{coaxial},i+1} - \varepsilon_{\text{coaxial},i-1}) \]  

where \( \Delta x \) is the spatial resolution of strain measurements and \( E \) is the Young’s modulus of the support element. Assuming that the interface slip is equal to the displacement of the FOS instrumented support element, the shear stress distribution can be directly determined from the strain distribution at a given pull load.

Figure 5-11 displays a comparison between the shear stress distributions along FOS_03 determined according to Equation 5-2 and Equation 5-3 at selected applied loads. The DOS measurements clearly define a non-uniform shear stress distribution across the anchored length of the rebar. Equation 5-2 was also found to significantly underestimate the magnitude of the peak shear stress. It is observed that the position of the peak shear stress (from Equation 5-3) did not correspond to the transition point between the free length and the anchored length of the rebar. Instead it was measured at a position within the anchored length and was found to progress further away from the head of the rebar with applied load. However, the position along the rebar where shear stress began to develop remained constant across (at 0.78 m, as found for the strain distribution) up until 90 kN, indicating the onset of plastic behaviour at the bond interface. It should be noted that partial encapsulation of the rebar resulted in a longitudinally unconfined boundary condition at the resin fill position (i.e., the start of the anchored length). Accordingly, the peak shear stress would not be expected to occur at start of the anchored length as is often visualized in analytical studies of pull tests within elastic loads (Hyett et al., 1996; Li and Stillborg, 1999; Ren et al., 2010). Beyond the position of peak shear stress a quadratic-shaped attenuation of interface stress was measured along the rebar. For FOS_03, shear stress did not extend fully to the toe end of the rebar at any of the applied pull loads.
5.4 Discussion

5.4.1 FOS Instrumentation Procedure

The instrumenting procedure presented in this research involves removing material (i.e., grooves) from the support element to house the FOS. This introduces a cost in terms of creating the sensor and inherently

Figure 5-11: FOS_03 shear stress distributions at the support element-resin interface for selected loads. The dashed line corresponds to Equation 2 and the solid line corresponds to Equation 3 (from the FOS strain profile). A 25-point moving average (i.e., 16.25 mm interval) was applied to the DOS measurements in order to reduce the amplification of measurement noise introduced through numerical differentiation.
reduces the load carrying capacity of the support element by reducing the cross-sectional area and flexural rigidity. It also restricts the applicability of the instrumenting procedure to support elements of a certain diameter or wall thickness (if the core is hollow). Nevertheless, the robustness of the FOS instrumentation is necessary for routine use beyond controlled studies. The FOS instrumented rebars considered in this research were successfully installed within an underground salt mine using a conventional bolting jumbo. No special instructions were given to the operator(s) other than specifying the anchor method (Table 5-2). The FOSs operated well immediately after installation and during pull test loading. A key benefit is that no part of the FOS is exposed at any time other than when connecting the OF lead cable.

A limitation in demonstrating the wider potential of the FOS technique in this research was the ultimate load capacity of the rebar head threads, which restricted pull test loads below the working capacity of the rebar body. However, the instrumenting procedure is easily altered to accommodate a forged head support element (Figure 5-12), which would allow loading beyond working capacity (i.e. yielding) of the support element. Chapter 3 (Forbes et al., 2017) demonstrated that the DOS measurement unit is capable of measuring high gradient strains in excess of 15,000 µε along a similarly instrumented rebar. This is beyond the working capacity of the majority of tendon support elements, but possibly not adequate to

![Figure 5-12: Alteration of head connector for forger head support elements. A – Forged head rebar and example protective cap with external threads. B – View of female OF connector (male OF connector also applicable) recessed within a centered hole and internal threads for protective cap.](image-url)
measure the ultimate capacity of a support element due to the large deformations associated with the onset of yielding. In this regard the FOS technique is most applicable for support element assessment or acceptance tests in loading range of the support element’s working capacity.

5.4.2 Support Element Assessment

The 0.65 mm spatial resolution of the DOS interrogator provides a detailed description of the load distribution along a FOS instrumented support element during an in-situ pull test. As demonstrated for FOS_03, the load distribution and interfacial shear stress along an encapsulated support element (with a realistic encapsulation length) will not be uniform when subjected to coaxial loading, necessitating high spatial resolution measurements. Load and displacement measurements solely at the head of the support element head, therefore, are not capable of measuring this mechanistic behaviour. Furthermore, the load behaviour along an encapsulated support element will vary depending on the in-situ ground mass conditions as well as the material properties and dimensions of the grout and support element (Aziz, 2004; Benmokrane et al., 1995; Canbulat et al., 2005; Hyett et al., 1992; Kılıç et al., 2003; Li et al., 2016; Mark et al., 2002; Spang and Egger, 1990). This research has established that a FOS instrumented support element is able to quantify the rate of load attenuation along a support element and also ascertain the encapsulated length of support element that is mobilized during an in-situ pull load. Another benefit of the FOS procedure is that support element deformation (not to be confused with displacement) can be determined by numerically integrating the measured strain distribution. As shown in Figure 5-13 for FOS_03, this can be used to estimate deformation at the support element head or at a specific position along the support element.

Many in-situ pull test campaigns report a significantly less stiff load-displacement response in comparison to laboratory studies. This feature is often attributed to the uncertainty associated with the measurement of bolt head displacement (Blanco-Martín, 2012; Nicholson, 2016; Salcher and Bertuzzi, 2018; Serbousek and Signer, 1987). Figure 5-14 presents a comparison of the load-displacement response and the corresponding stiffness of FOS_03 as measured by the digital displacement sensor and the DOS measurement unit. Load-displacement behaviour is presented for both the rebar head and the encapsulation
Displacement measurements from the digital sensor are also distinguished between the load and reload cycle (Figure 5-7).

Figure 5-13: FOS_03 coaxial deformation distribution referenced from the toe end of the rebar at selected applied loads. The dashed line segment of each distribution provides an estimate of displacement at the head of the rebar. This has been calculated by determining the average stiffness along the free length of rebar.

Referring to Figure 5-14, the anchor length responded to load in a stiffer manner than the rebar head. There is also a large variance in the stiffness determined from each displacement measurement condition. This can be explained by considering the constituent components of measured displacement. The load segment and reload segment displacements that were measured by the digital sensor resulted from the
deformation of the ground mass, slip at the rebar-grout and/or grout-ground interface, and the deformation of the rebar. As discussed in Section 5.3.1, each reload segment induced a relatively elastic response in comparison to the large unrecovered displacements of the load segments. Thus, a less stiff response was measured for the load segments. In contrast, the DOS measurements only capture deformation of the rebar, and therefore, measured the stiffest response.

When considering which load-displacement response best describes the in-situ behaviour of the support element, two circumstances can be discussed: 1) for use with convergence confinement methods
and 2) for quantifying bond-slip behaviour and comparing with analytical and laboratory results. When considering the former, it is important to recognize that the in-situ pull test only simulates an anchor length along a support element (i.e., where the support element moves relative to the ground mass). In-situ, loading is generated by ground mass movements along a pick-up length of the support element (i.e., where the ground mass moves relative to the support element) (Freeman, 1978). Therefore, it is prudent to consider a measurement of stiffness that incorporates deformation of the ground mass. However, when considering bond stress behaviour at the support element-grout interface, the DOS measurement of support element deformation is most applicable, as this is often assumed to equal the interfacial slip (Blanco Martín et al., 2011; Ma et al., 2013; Ren et al., 2010).

5.4.3 Installation Quality Control and Assurance

Many underground projects now use mechanized installation equipment that install, grout, and tension the given support element. This can increase productivity and reduce worker exposure to unsupported ground, but it also requires periodic inspection and pull testing. A particular concern is often grout encapsulation and grout quality. Once the support element is installed, it is very difficult to visually inspect any segment not directly at the borehole collar, especially when fully encapsulated. This research has demonstrated that a pull test using the FOS procedure can identify the grout fill position. The strain distribution measured along the support element can also be used to detect deficiencies in the grout through the observation of uncharacteristic anchor length behaviour. However, since an encapsulated length less than one meter is required to fail many support elements (Hutchinson and Diederichs, 1996; Li et al., 2016; Serbousek and Signer, 1987), the FOS may not necessarily detect a grout deficiency towards the toe end of the support element (e.g., in the case of a fully encapsulated, multi-meter support element). Villaescusa et al. (2008) have discussed that the worst consistency of the encapsulation material is often at the toe end of the support element and that full assessment can be accomplished through over-coring techniques. Within this context, a combined FOS instrumented support element pull test and over-coring procedure can be used to provide a very complete description of the support installation quality and performance.
5.5 Conclusions

This research has demonstrated that high spatial resolution DOS can be used to augment the support element in-situ pull test. Through a series of pull tests conducted on FOS instrumented rebars at an underground salt mine, it has been shown that a continuous strain profile can be measured along a coaxially loaded support element. The strain profile can be used to distinguish the borehole encapsulation length and also quantify load development length of a support element. Furthermore, both support element deformation and the interfacial shear stress distribution can be derived. Of significance is that the support element instrumenting procedure completely protects the FOS during handling and installation of the support element. No additional care was required when handling the instrumented support element and no procedural changes were required for installation. The FOS instrumented support element remained to function as a normal support element and operated as expected under applied pull test loads.
5.6 References


CEN, 2013. Execution of special geotechnical work.


Chapter 6  Measuring the in-situ response of tunnel support using high spatial resolution optical fiber strain sensing

6.1 Introduction

Underground excavation projects will often use borehole installed reinforcement elements (e.g., rock bolts, cables, friction sets) to control excavation induced ground mass displacements and maintain a safe excavation for personnel and equipment. It is well established that the stability of a relatively massive or discontinuous ground mass can be greatly improved with the use of reinforcement elements (Azuar, 1977; Dight, 1982; Fairhurst and Singh, 1974), but the selection of the most appropriate type or combination of reinforcement is not necessarily a simple process. The systematic design and selection of excavation reinforcement requires an understanding of the anticipated ground mass behaviour, the expected failure mechanisms, and the predicted excavation damage or magnitude of displacements (Hoek et al., 2000; Kaiser et al., 1996; Oliveira, 2018). While in-situ site investigation and laboratory testing can greatly assist in characterizing the ground mass and its anticipated interaction with reinforcement, it is difficult to assess or measure the response of a reinforcement element in-situ. This is especially true for reinforcement elements that are fully encapsulated with a cement grout or resin, or those that are described as being continuously mechanically coupled (CMC) with the ground mass (Thompson et al., 2012; Windsor, 1997).

The load distribution along a CMC reinforcement element will be heavily influenced by the distribution of ground mass displacements surrounding the element. In heterogeneous, jointed, and fractured ground masses, this may result in a load distribution that is reflective of multiple discontinuity movements at various depths from the excavation (Bjornfot and Stephansson, 1984; Hyett et al., 1996; Li and Stillborg, 1999; Pells, 2002). This behaviour cannot be accurately inferred from a measurement of the reinforcement element segment that is visible at the excavation periphery. In order to accurately distinguish the influence of multiple ground movements along a CMC element, many small gauge-length discrete measurements are required along the element (Martin et al., 2000; Windsor, 1993), especially without a
prior knowledge of ground movement locations. It must also be recognized that ground displacements may not necessarily be coaxial with the reinforcement element. Instead they will most likely result in a combination of coaxial and bending moment induced loads. The latter induces a nonuniform stress distribution over the cross section of the reinforcement element. Therefore, it is necessary to determine the orientation of ground displacements relative to the reinforcement element in order to ascertain the maximum magnitude of moment induced load.

Measurement of the complete three-dimensional reinforcement response, consequently, demands large sensing resources. On a mine or tunnel project-scale, this cannot practically or cost-effectively be achieved using conventional sensing technologies, such as: load cells, displacement transducers, or electrical strain gauges. Within the scope of measuring the full response of CMC reinforcement elements, several researchers have trialed alternative and novel sensing technologies such as guided ultrasonic waves (Beard and Lowe, 2003; Lee et al., 2012; Steblay, 1987) and fiber optic strain sensing, FOS (Hyett et al., 2013; Iten and Puzrin, 2010; Nellen et al., 2000). Chapter 3 (Forbes et al., 2017) demonstrated a high spatial resolution FOS technique that was capable of measuring the three-dimensional strain and displacement response of CMC reinforcement elements when subjected to laboratory experiments that simulated expected in-situ loading mechanism, including: coaxial, bending, and differential shear loading. The following research extends on the use of this technique to reinforcement elements in working tunnel projects. Alterations and improvements to protect and install FOS instrumented reinforcement elements in-situ are presented and the benefits of measuring complete reinforcement behaviour for ground reinforcement design, quality assurance, and optimization are discussed.

6.2 Fiber Optic Strain Sensor Instrumentation Procedure

The FOS technique discussed in Chapter 3 (Forbes et al., 2017) considers embedding a 155 µm diameter polyimide coated optical fiber (OF) within three lengthwise machined grooves along a reinforcement element. Referring to Figure 6-1, the three grooves are spaced equally at 120 degrees from each other around the circumference of the reinforcement element (or in a delta-configuration). A pair of semicircle
Figure 6-1: Schematic drawing of the FOS. Three grooves are machined along the length of a reinforcement element. The lengthwise machined grooves are approximately 2.5 mm deep by 2.5 mm wide and are equally spaced at an angular distance of 120 degrees ($\varphi$). The active sensing length refers to the length between the pair of machined loops that connect the grooves in a clockwise direction from the head of the reinforcement member. A single connecting loop is located at the positions “a” and “b” from the head, defining the active sensing length. The FOS connector and end termination are both situated outside of the active sensing length at opposing ends.

grooves at opposing ends of the element connects the lengthwise grooves in a clockwise pattern from the head of the element. This allows a single OF to be run between all three machined lengths below the exterior profile of element. The OF is bonded to the reinforcement element by encapsulating it with a proprietary metal bonding adhesive. The embedding and encapsulating procedure provides a protective barrier for the glass OF (Hyett et al., 2013; Iten and Puzrin, 2010; Nellen et al., 2000) and also positions the OF at a relatively constant distance from the centroid of the reinforcement element (as many reinforcement elements have a geometrically varying profile). At the head of the reinforcement element the OF is terminated with a connector. The type of connector is dependent on the interrogation technology that is used to measure strain along the OF. This research solely considers the use of a commercially available distributed optical fiber strain sensing (DOS) technology available from Luna Innovations (Luna Innovations, 2017). The given DOS technology measures the Rayleigh backscatter component of light travelling along the core of an OF in order to resolve strain at spatial increments as low as 0.65 mm (Luna Innovations, 2016a; Soller et al., 2005); however, the discussed FOS technique is not limited to this specific technology. It should also be noted that the FOS technique is applicable to many other types OF coatings and additional protective layers, but the polyimide coating has been found to be ideal for strain transfer.
(e.g., Brault, 2018; Brault et al. 2019a; Inaudi et al., 1996; Regier and Hoult, 2014; Weisbrich and Holschemacher, 2018).

Following the procedure outlined in Chapter 3 (Forbes et al., 2017), the strain measured along the three OF instrumented lengths can be compared to determine the principal (or maximum) strain along the reinforcement element at 0.65 mm intervals. The maximum strain profile can be differentiated into coaxial and bending moment induced strain components and be used to derive the orientation of bending moment inducing strain around the circumference of the reinforcement element (i.e., the ground mass movement vector). Coaxial and transverse deformation of the reinforcement element can also be determined from the strain components.

6.3 Sensor Alterations and Improvements for In-Situ Application

The vast majority of reinforcement elements are installed using a mechanized bolting jumbo. This installation procedure often involves inserting and spinning a reinforcement element into a borehole (e.g., to expand a mechanical anchor, to apply pre-tension, or to mix resin) and requires a socket, or similar fastening mechanism, on the jumbo to connect with the head of the reinforcement element. The FOS technique described in Chapter 3 (Forbes et al., 2017) was demonstrated through laboratory experiments and incorporated a multi-meter long segment of OF lead that extended from the head of the reinforcement element (to connect with the DOS interrogation unit). For use in-situ, the lead would obstruct the connection between the jumbo and the head of the reinforcement element. Accordingly, the OF lead was replaced with a partially recessed FOS connector, which was covered with a stainless-steel protective cap (Figure 6-2). A 6 mm diameter hole houses the FOS connector at the head of the element and a 2.5 mm diameter hole provides an aperture for OF from the connector to extend into one of the lengthwise grooves. The given instrumenting procedure ensures that no portion of the FOS is directly exposed at any stage of handling or installation other than when connecting it with an external OF lead. Most importantly, it allows an OF instrumented reinforcement element to be installed in the same manner as a normal reinforcement element (which is necessary to capture realistic mechanistic behaviour).
Several research efforts have used early versions of the described FOS technique for laboratory experiments and in-situ studies of fully grouted rebar (Beneteau et al., 2019; Jessu et al., 2016; Kostecki, 2019; Kostecki and Spearing, 2018; Snell et al., 2017). These studies corroborated that the FOS technique was able to measure the three-dimensional behaviour of a CMC rebar element when subjected to various laboratory loading configurations. The strain measured with DOS was also found to compare well with theoretical predictions and with strain measurements from other sensing technologies, especially within elastic loading limits. In addition, the FOS technique was successfully installed by various mechanized procedures in-situ. However, concerns were raised about the longevity of the FOS technique in the underground environment. In some instances, a FOS would only provide reliable strain measurements for several days after being installed, even though other sensing technologies determined that the given FOS instrumented reinforcement element was well within the operating limits of the DOS interrogator.

Figure 6-2: FOS connector configuration at the head of a reinforcement element. A: Example stainless steel protective cap threaded onto the element’s head, covering the FOS connector. B: View of a FOS connector after installation of the reinforcement element (protective cap removed).
It is believed that the longevity of the FOSs could have been increased through improved monitoring protocols; primarily by improving procedures to clean the FOS connections (i.e., dust and humidity mitigation) and by improving lead wire management and protection. Nevertheless, alterations to the FOS configuration, specifically at the terminations of the FOS, were made in order to improve the signal integrity along a FOS. The following improvements are incorporated in the FOSs that are discussed in this research.

### 6.3.1 Sensor Connection Improvements

The selected DOS technology that was applied in this research determines strain by measuring the shift of the Rayleigh backscatter spectra along the core of an OF. This requires a complex cross correlation to be conducted between the reflected spectrum along a segment of an OF at a strained state and at a reference state (Froggatt and Moore, 1998). Part of this procedure involves an identification algorithm that samples the first 20 cm of the FOS in order to match it with the reference state. Ideally the first 20 cm of a FOS should be subjected to very little strain in order to accomplish this identification procedure. Experience by the author has found that strains in excess of 500 µε along the first 20 cm of a FOS will often result in a failed identification of the FOS, and thus no strain measurement.

The manufacturer of the DOS interrogation unit suggests creating a strain relief segment at the beginning of a FOS (Luna Innovations, 2016a). In the laboratory setting, this can easily be accomplished by leaving a free (i.e. unbonded) looped segment of OF at the beginning of a FOS. However, this is much less trivial to accomplish with a reinforcement element in-situ. As previously described, free OF at the head of a reinforcement element will prevent mechanized installation. Exposed fiber at any other point along the reinforcement element is also likely to be damaged during handling or installation. A possible solution is to connect a short length of bundled OF after the reinforcement element has been installed and reference this segment of OF as the start of the FOS; however, this is not recommended as a best practice by the author. Any length of exposed OF in the underground environment will be prone to damage. Therefore, a
An unbonded loop of OF at the head of the FOS would potentially negate the efforts taken to protect the majority of the FOS (i.e., by embedding and encapsulation within the machined grooves).

Referring to Figure 6-3, a procedure was developed to place an unbonded segment of OF along the first 20-30 cm of a reinforcement element by deliberately slackening the OF within the centered hole that houses the recessed FOS connector. The OF is run from the centered hole at the head of the reinforcement element into a length of stainless-steel tubing (1.75 mm in diameter) that is positioned at the bottom of a machined groove. This leaves the given length of OF completely unbonded from the reinforcement element between the FOS connector and the far end of the stainless-steel tubing (where it is then encapsulated with an adhesive in the groove) while simultaneously situating the slackened OF below the exterior profile of the reinforcement element for protection. It should be noted that the centered hole at the head of the reinforcement element is intentionally drilled to a length less than 5 cm to ensure that a residual threaded length at the head of the reinforcement element can sustain design specified load from a nut and face plate assembly (if present). For the FOSs discussed in this research, the OF was slackened to allow approximately

![Figure 6-3: Schematic drawing of an example FOS head construction for in-situ application. A 6 mm diameter, 5 mm long centered hole at the head of the reinforcement element is used to recess a FOS connector. OF extending from the FOS connector is slackened within the centered hole and run through a 2.5 mm diameter angled hole into a stainless-steel tube (1.75 mm diameter, 0.25 mm wall thickness, 20 cm length) positioned at the bottom of a machined groove. The OF is unbonded to the reinforcement element between the FOS connector and the distal end of the stainless-steel tube.](image-url)
1 % strain (i.e., 10,000 µε) at the head of the reinforcement element prior to the unbonded OF segment experiencing strain.

The primary benefit of constructing the head end of the FOS in this described manner (e.g., in comparison to connecting a loose bundle of OF after the reinforcement element has been installed) is that the FOS starts and ends within the reinforcement element. Accordingly, if the external lead wire is damaged at any stage of a monitoring program it can simply be replaced with little to no impact on the integrity of the FOS. Additionally, the free length of fiber can be used to identify the presence of a temperature gradient at the head of the element. This can be used to assist in temperature compensation of strain measurements. However, if a highly varying temperature profile is anticipated along the length of the reinforcement element, a more complete temperature compensation scheme should be considered (e.g., Mohamad et al., 2011).

### 6.3.2 Sensor Termination Improvements

Rayleigh scattering is a spontaneous loss mechanism arising from random fluctuations of the index of refraction along the core of an OF. This is an elastic scattering process (i.e., no energy transfer to the OF) and, therefore, the power of the backscattered signal is proportional to (less than) the power of the incident signal (Bao and Chen, 2012). Consequently, the signal to noise ratio is very sensitive to back reflections of the incident light at the far end of the FOS. A FOS using the given DOS technology should ideally be end terminated with a low-reflection surface or a very high return loss termination piece. At minimum, a -70 dB return loss should be achieved to prevent spurious signals from interfering with the Rayleigh backscatter measurement (Luna Innovations, 2016b).

Initial end termination pieces for the FOS described in Chapter 3 (Forbes et al., 2017) were constructed by coating the end of the OF (i.e., the end of the FOS) with an index matching liquid. While this was found to be suitable for short-term laboratory experiments, the effectiveness of this termination was observed to be highly variable over longer periods (weeks to months) with and without strain applied to the encapsulated OF. A more stable return loss for in-situ FOSs was created by constructing a new
termination piece with multiple signal absorption steps. Referring to Figure 6-4, the primary component of the termination is a “coreless” OF segment that is fusion spliced onto the end of the FOS. The core of the OF composing the FOS and the coreless OF segment have essentially the same effective index of refraction, but the coreless OF segment has a much larger cross section for the signal to propagate within. Accordingly, forward travelling light is both absorbed and scattered when transecting the coreless OF segment, resulting in a weaker reflected signal at the end (Kosinski, 1993). The splice region is protected with a heat shrink sleeve 1 mm in diameter. A 1.75 mm diameter stainless steel tube covers the splice and the coreless segment (which inhibits strain transfer). To further reduce the intensity of the reflected signal, the end of the coreless segment is encapsulated with an index matching gel inside the stainless tube. This also acts as another signal attenuation region. The end of the stainless tube is capped with an index matching epoxy. The full

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Figure 6-4: Schematic drawing of the FOS end termination assembly, including a detail view of the fusion splice between the OF (that composes that active sensing length of the FOS) and the coreless OF. Section AA is displayed at a 5:1 diameter to length scale for visualization purposes. The length of the end termination is the length of the stainless-steel tube, which is 5 cm for the FOSs discussed in this research.
termination assembly is approximately 5 cm in length and 1.75 mm in diameter, which allows it to be encapsulated within a machined groove along a reinforcement element in the same manner as the length of OF composing the FOS.

6.3.3 Lead Wire Protection and Management
Lead wire protection and management is vital to measurement signal strength and efficient monitoring in an underground project. With DOS, an OF is used for the lead. The light guiding construction of the OF lead is essentially the same as the OF used for the FOS. While the spectral properties of light propagating through the OF lead are not necessarily measured, the FOS measurement signal can certainly be affected while travelling through the lead length. Since the given DOS technology measures back reflections of the scattered incident light, a degraded OF lead will both diminish the forward propagating incident signal that is sent from the interrogation unit (producing a weaker backscatter signal) and then further diminish the back reflected measurement signal on its return path.

Ideally, an OF lead should be subjected to minimal vibration and strain or temperature change. While this is not entirely preventable under most circumstances, there are many OF distributors that offer OF constructions with various protective and insulating layers that are aimed at isolating the OF core. There is a vast availability of such OF constructions because fiber optics have predominately been developed for the communications industry where the primary focus has been on preventing external perturbations (such as strain) from altering the transmitted signal.

There is not a quintessential procedure or amount of lead protection and management. Various factors, including: the site conditions, ease of access to the FOS, monitoring interval, and monitoring duration will heavily influence lead requirements. Nevertheless, it is suggested that the number of connections/disconnections made between a FOS and a lead be kept as small as possible in the underground environment. OF connectors are very sensitive to environmental dust and moisture. FOS instrumented reinforcement element will also often be located in an inaccessible, or lift required, location, such as the tunnel crown. Under ideal circumstances, one connection between the lead and the FOS would be
established and maintained rather than continually connecting and disconnecting. This connection would be tested for quality and adequately protected; thus, limiting the exposure of the FOS connector. It is often possible to position the opposing end of the OF lead (for connection with the integration unit) at a more accessible location. Accordingly, the connection between the lead and the DOS interrogation unit is more apt for multiple connections, if necessary. In general, an inverse relationship should be followed between the number of FOS connections and the harshness of in-situ conditions (i.e., airborne dust, accessibility, etc.). Further discussion on lead protection and management is discussed in more detail for three in-situ applications of the FOS technique in the following sections.

6.4 In-Situ Results
Selected results from three in-situ applications of the discussed FOS technique are presented within this section. A summary of the reinforcement element instrumented by in-situ application is given in Table 6-1. The FOS instrumented reinforcement elements were installed using the normal mechanized procedures at each site. However, they were not necessarily installed as the primary reinforcement.

6.4.1 Coaxial Pull Testing
The coaxial pull test is a commonly practiced procedure for assessing in-situ reinforcement elements (ASTM, 2013; Lardner and Littlejohn, 1985). The procedure involves installing a reinforcement element in agreement with the normal operational procedures at the given site and subsequently applying a coaxial load, often with the use of a hydraulic ram assembly (Figure 6-5A). While this testing procedure does not necessarily replicate the true loading response of the given reinforcement element, it can provide very useful insight into the characteristic behaviour of the element, including, but not limited to: the load capacity, the elongation capacity, the load-displacement response, and the interfacial bond capacity (when a CMC element is considered). Understanding such reinforcement behaviour is greatly beneficial to reinforcement design and optimization, making the in-situ pull test an essential procedure at many projects. The pull test is also a very convenient procedure to trial the FOS technique in-situ as will be discussed in this section.
Table 6-1: Overview of the FOS instrumented reinforcement elements by in-situ application. The active sensing length, which is defined by Position a and Position b from the head of the reinforcement element, is as described in Figure 6-1. HDPE refers to high-density polyethylene. The modified cross-sectional area includes the removed groove material but does not account for loop positions.

<table>
<thead>
<tr>
<th>Reinforcement Element</th>
<th>Acceptance / Pull Tests</th>
<th>Umbrella Arch Reinforcement</th>
<th>Crown Reinforcement in Stratified Ground</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material</td>
<td>Galvanized Steel Rebar with HDPE Sheath and Expansion Anchor</td>
<td>Hollow Steel Threadbar</td>
<td>Black Finish Steel Rebar with HDPE Sheath and Expansion Anchor</td>
</tr>
<tr>
<td>Thread Size</td>
<td>M24</td>
<td>R32-360</td>
<td>M24</td>
</tr>
<tr>
<td>Threaded Length (mm)</td>
<td>120 (head) 160 (toe)</td>
<td>3000</td>
<td>90.0 (head) 160 (toe)</td>
</tr>
<tr>
<td>Nominal Diameter (mm)</td>
<td>22.0</td>
<td>29.5</td>
<td>22.0</td>
</tr>
<tr>
<td>Internal Diameter (mm)</td>
<td>-</td>
<td>14.5</td>
<td>-</td>
</tr>
<tr>
<td>Modified Cross Sectional Area (mm²)</td>
<td>360</td>
<td>500</td>
<td>360</td>
</tr>
<tr>
<td>Length (m)</td>
<td>2.40</td>
<td>3.00</td>
<td>4.50</td>
</tr>
<tr>
<td>Yield Strength (MPa)</td>
<td>640</td>
<td>590</td>
<td>640</td>
</tr>
<tr>
<td>Ultimate Strength (MPa)</td>
<td>800</td>
<td>710</td>
<td>800</td>
</tr>
<tr>
<td>Elastic Modulus (MPa)</td>
<td>$200 \times 10^3$</td>
<td>$200 \times 10^3$</td>
<td>$200 \times 10^3$</td>
</tr>
<tr>
<td>Position a (m)</td>
<td>0.160</td>
<td>0.350</td>
<td>0.200</td>
</tr>
<tr>
<td>Position b (m)</td>
<td>2.00</td>
<td>2.70</td>
<td>4.20</td>
</tr>
<tr>
<td>Active Sensing Length (m)</td>
<td>1.84</td>
<td>2.35</td>
<td>4.00</td>
</tr>
<tr>
<td>Additional Notes</td>
<td>HDPE corrugated sheath has a nominal diameter of 32.5 mm and a 2 mm wall thickness.</td>
<td>Self-drilling, injectable spile with sacrificial drill bit</td>
<td>HDPE corrugated sheath has a nominal diameter of 32.5 mm and a 2 mm wall thickness</td>
</tr>
</tbody>
</table>
6.4.1.1 Installation and Pull Test Procedure

Many CMC reinforcement elements are installed as passive members, meaning they are mobilized by movements of the surrounding ground mass. The installation timing and positioning of reinforcement within an excavation can greatly effect the magnitude of load mobilization (e.g., Carranza-Torres, 2004; Peila and Oreste, 1995). In general, excavation advances will induce ground mass displacements within proximity to the active tunnel face. An appropriate assessment of the FOS technique includes measuring a reinforcement element’s response to such ground movements; however, the tunnel face is not an ideal location to trial a FOS instrumented reinforcement element without interruption to ongoing construction operations (e.g., mucking, shotcrete application, scaling, reinforcement and support installation). In addition, excavation induced ground mass displacements may be complex depending on the in-situ ground conditions and may occur over a monitoring interval that is difficult to fully capture (i.e., a very fast or a very long duration). The pull test, on the other hand, allows an immediate and controlled load to be applied to a reinforcement element, and as a result, a FOS instrumented reinforcement element does not to be installed near an active excavation front in order to guarantee load mobilization.

Figure 6-5: FOS instrumented CT-Bolt in-situ pull test. A: Pull test assembly mounted onto the instrumented bolt. B: View of the modified coupling nut used to exit the OF lead and connect the instrumented bolt with the extension rod (and hydraulic ram).
Referring to Table 6-1, FOS instrumented CT-Bolts were installed into the sidewall of an excavation at an active tunnel project in Ashfield shale (e.g. Bertuzzi, 2017; Pells, 2002). The FOS CT-Bolts were installed using the automated, mechanized jumbo procedure at the project. This followed no human contact with a FOS CT-Bolt after it was loaded onto the multi-bolt carousel of the jumbo. The full mechanized installation procedure involved drilling a borehole, fully inserting the bolt into the borehole, spinning the bolt in order to expand the expansion anchor and apply a predetermined preload, and fully encapsulating the bolt within the borehole using a cement grout (via a grouting boom). This was the same procedure followed for the non-instrumented CT-Bolts at the project. The FOS CT-Bolts were installed through a steel-fiber reinforced shotcrete layer into a competent section of Ashfield shale at position approximately 25 m from the tunnel face. Minimal lead protection was required because of the short-term nature of the pull tests being conducted. A 5 m long OF patch cable was selected to temporarily connect the FOS with the DOS interrogation unit during installation of the FOS bolts and during pull tests.

6.4.1.2 Installation and Pull Test Results
Two initial acceptance tests were conducted within the context of assessing the suitability of the FOS technique in-situ: 1) Mechanized installation acceptance and 2) Reinforcement response measurement through induced loading (i.e. through pull tests). This project represented the first mechanized in-situ application of the FOS head construction detailed in Figure 6-3. Therefore, it was pragmatic to test each FOS after individual aspects of the installation procedure, prior to conducting pull tests. Strain measurements were taken with the DOS interrogator at three stages of installation: 1) Prior to placing the FOS CT-Bolt onto the jumbo, 2) After the jumbo had inserted and preloaded the bolt within the borehole (i.e., spun the bolt), and 3) after the bolt was grouted. Referring to Figure 6-6, the baseline measurement was taken with the FOS CT-Bolt positioned straight along the tunnel floor. Correspondingly, relatively zero strain was measured along the active sensing length. The install measurement was taken after the bolt was inserted into the borehole and spun in order to open the expansion anchor and apply preloaded. This resulted in the strain profile along the bolt uniformly increasing to approximately 750 µε. The post grouting
Figure 6-6: Mechanized installation acceptance test strain profiles. The baseline measurement was recorded prior to installing the instrumented CT-Bolt, with the bolt positioned flat on the tunnel floor. The install measurement was recorded immediately following the bolt being inserted into the borehole with the jumbo and spun in order to expand the end anchor and tension the bolt. The grout measurement was recorded after encapsulating the bolt with a cement grout. Note: Tensile strain is taken as positive.

measurement was recorded approximately 10 minutes after the install measurement. Accordingly, an equivalent strain profile was measured. With the given bolt cross-sectional area, 750 µε is equal to approximately 54.0 kN of preload. This was in close agreement with the 50 kN (± 5 kN) of preload specified to the jumbo operator and provided validation for the preload procedure at the project by quantifying load along the bolt. The shape of the install and grout strain profiles were also consistent with the conceptual behaviour of an end loaded member (or free length), where the bolt was loaded between the expansion anchor at the toe end of the borehole and the faceplate assembly at the borehole collar.
Following successful measurements of the FOS CT-Bolts during installation, a pull test assembly was connected to the head of selected bolts (Figure 6-5A). Pull test assemblies will often couple a reinforcement element to an extension rod. This rod will extend through a hollow plunger hydraulic cylinder such that load can be applied to reinforcement element by bearing off the surrounding ground mass. As the head of the CT-Bolt housed the FOS connector, it was necessary to manufacture a coupling nut with an aperture for the OF lead (Figure 6-5B). The given pull test assembly was positioned on the FOS CT-Bolts six days after grouting and load was applied using a hand pump and recorded from a dial gauge.

Figure 6-7 (top) presents the coaxial strain profiles measured along the active sensing length of a bolt at 50 kN intervals of applied load. There are two apparent behaviour regions to the strain profiles measured along the bolt: 1) A region of load attenuation (from 0 m to approximately 1.00 m), and 2) A region of relatively low, uniform strain (from 1.00 m to the toe end of the bolt). The first region corresponds to section of the bolt that acted as the anchor length where load was transferred from the bolt to the ground mass through the cement grout annulus. It is observed that the shape of the strain profile along the anchor length changes from a decaying exponential-like distribution at lower loads (50 kN, 100 kN) to a profile at higher loads (200 kN, 250 kN) that follows a semi-linear decay near the position of applied load (i.e., the head of the bolt) and transitions to an exponential decay where strain approaches zero. This behaviour would suggest partial decoupling towards the head end of bolt; however, there was no evidence of definitive decoupling along the active sensing length. At all levels of applied load there is little to no change to the position of the transition point between the two bolt behaviour regions. This opposes the presence of a decoupling front (e.g., Li and Stillborg, 1999; Ren et al., 2010) at the levels of applied load considered in the pull test.

It is well established that the shear strength at the interface between the CMC element and its confining material is comprised of three mechanisms: adhesion, mechanical interlock, and friction, which are lost in sequence as deformation compatibility is lost (Benmokrane et al., 1995; Li and Stillborg, 1999; Serbousek and Signer, 1987). The interfacial shear strength (or bond strength) is also known to directly
dictate the critical encapsulation length required to reach ultimate load of the reinforcement element (Li et al., 2016). Previous studies have established that most reinforcement elements will require under 0.750 m
of encapsulation to fail at the shank (e.g., Hutchinson and Diederichs, 1996; Li et al., 2016; Serbousek and Signer, 1987; Vlachopoulos et al., 2018). Accordingly, it was unanticipated that 50 kN of applied pull load would be measured to mobilize 1.00 m of the CT-Bolt and that at higher loads, which were well beyond yield of the steel, the anchor length did not increase.

Although the anchor length remained relatively constant at all levels of applied load, the remaining length of the bolt was measured to experience a uniform strain increase with each additional 50 kN load interval. While this was limited to a total of approximately 75 µε, it was not an inconsequential amount of strain. For example, Figure 6-7 (bottom) displays the bolt deformation profiles ($\delta$), not to be confused with bolt displacement, determined by numerically integrating the pull test coaxial strain profiles ($\varepsilon_{coaxial}$) from the end of the active sensing length, Equation 6-1.

$$\delta = \int \varepsilon_{coaxial} \, dx$$  \hspace{1cm} 6-1

The linear deformation trend between 1.00 m and the end of the bolt suggests that this segment of the bolt was subject to a uniform tension and, therefore, that the expansion anchor was mobilized at the toe end of the bolt. Mobilization of the bolt length beyond the anchor length would generally be unanticipated without definitive evidence of decoupling for a CMC element (Blanco Martín et al., 2011; Ma et al., 2013; Ren et al., 2010). However, mobilization of the expansion anchor can be accounted for by considering the construction of a CT-Bolt. Unlike conventional fully grouted bolts, a CT-Bolt includes a 2 mm thick HDPE sheath that effectively separates the encapsulating grout into two separate grout annuli: 1) A grout annulus between the rebar and the sheath and 2) A grout annulus between the sheath and the borehole. The HDPE sheath provides the steel rebar with an additional protective barrier to corrosion, but it also inherently influences the load distribution of the bolt by creating a preferential coaxial shear failure plane; as evident by the expansion anchor being mobilized during the pull tests.
The pull tests indicated that the in-situ load transfer efficiency of a CT-Bolt at the given site was less than half of what has been reported for fully grouted rebar in the laboratory setting; particularly from results that have indicated critical encapsulation lengths under 0.45 m for rebars that have been grouted into concrete forms or steel pipes (Li et al., 2016; O’Connor et al., 2019; Vlachopoulos et al., 2018). Nevertheless, laboratory pull tests conducted on FOS CT-Bolts that were cement grouted (0.40 water:cement) into 1.00 m long steel pipes (60.5 mm outer diameter, 5.59 mm wall thickness) confirmed the measured in-situ behaviour is not attributed to the ground mass. Figure 6-8 displays a comparison

![Figure 6-8: In-situ versus laboratory pull test comparison. Coaxial strain profiles are presented for a 2.4 m long in-situ installed CT-Bolt and a CT-Bolt grouted within a 1.00 m long steel pipe (60.5 mm outer diameter, 5.59 mm wall thickness) in the laboratory. Strain profiles are presented at 50 kN, 100 kN, 150 kN, and 200 kN of applied load. Note: Tensile strain is taken as positive.](image-url)
between the strain profiles measured during the laboratory pull test and the in-situ pull test at selected loads. It is apparent that the full 1.00 m grouted length of the laboratory CT-Bolt was mobilized in a similar manner as the in-situ bolt. The strain distributions are also very similar, especially at lower loads. The deviance between the laboratory and in-situ strain distributions at higher loads is attributed to the limited embedment length of the laboratory specimen (since the expansion anchor was mobilized at 2.40 m in-situ) and also operator error in recording an accurate applied load in-situ. The consistency between the shapes of the in-situ and the laboratory strain distributions affirms that the HDPE sheath negatively impacts the load transfer between the rebar and the ground mass, although not to an extent that the bolting system is entirely dependent on the toe end expansion anchor. Further discussion regarding the laboratory pull test apparatus is provided by Vlachopoulos et al. (2018).

6.4.2 Umbrella Arch Response to Tunnelling
According to Oke et al. (2014), an umbrella arch is a temporary support arrangement that forms a structural umbrella around the excavation from the insertion of longitudinal support members that are installed from within the tunnel, above and around the crown of the tunnel face. Structural elements composing the umbrella arch are installed prior to the first pass of the excavation and are founded on the previous tunnel support round (if present) and the ground mass ahead of the tunnel face. This arrangement provides reinforcement to the ground mass at and ahead of the tunnel face and distinctively reinforces the unsupported span immediately behind the tunnel face during excavation advance. The latter is an advantageous feature of the umbrella arch in comparison to other pre-support or pre-improvement techniques (e.g., face-bolting). Of interest to this research is the application of spile elements to compose the structural umbrella at a recent shallow tunnel project (approximately 5 m over cover) through lacustrine clays (e.g., Elwood and Martin, 2016). Further details on the tunnel project are described in this chapter and by Forbes et al. (2018).
6.4.2.1 FOS Instrumented Spile Installation

Spiles are defined as reinforcement elements with a length smaller than the height of the excavation and are often installed at angles fanning outwards at 5°-40° from the tunnel alignment (e.g., Gschnitzer and Goliasch, 2009; Oke et al., 2014; Trinh et al., 2007). Along the tunnel section designated to install FOSs, 6 m long spile elements were installed at 6° angles with every 1.00 m advance (resulting in approximately 5.00 m of overlap). Each spile was composed of two 3.00 m long R32 hollow threadbars (refer to Table 6-1) that were connected with a threaded coupling nut (150 mm length). All spiles at the tunnel were installed as self-drilling members using sacrificial drill bits. The installation procedure involved drilling in the first 3.00 m length of threadbar at a position around the crown of the tunnel, connecting on the second 3.00 m length of threadbar, drilling the final 3.00 m length, and cement grouting the spile within its self-drilled borehole. During the drilling process, water was pumped through the hollow core of the threadbar in order to flush drilling debris. Once fully inserted, cement grout was pumped through the core, effectively toe-grouting the spile within its self-drilled borehole. Referring to Figure 6-9A, each spile was drilled through a lattice girder positioned approximately 0.50 m from the tunnel face. The head of the spile at the lattice girder was then completely cast by a steel-fibre-reinforced-shotcrete layer prior to the subsequent excavation advance.

There were several challenges to address with respect to applying to the FOS technique to the self-drilling spiles at this project. Firstly, each spile was composed of two separate lengths of threadbar. An obvious solution would be to instrument a solid 6.00 meter length of threadbar; however, this was not possible due to the limited stroke of the jumbo. Secondly, the hollow core of both threadbars were required to pump water and grout through the spile during installation. As previously discussed, an essential aspect of the FOS technique is the recessed FOS connector that is centered at the head of the element. This would inherently block water and grout flow (in addition to the FOS connector being susceptible to damage). Lastly, the head of the spile was designated to be covered with a shotcrete layer shortly following installation. Thus, limiting connection availability with an OF lead.
Two FOS instrumenting approaches were investigated with regard to the discussed challenges: 1) Instrument a 6.00 m long threadbar, hand insert the element into a pre-drilled borehole, and grout the element using a grout tube and 2) Instrument the 3.00 m long threadbar segment nearest to the excavation and follow the regular installation procedure. The latter was ultimately selected since the former would alter the installation procedure and potentially impact the mechanistic behaviour of the spile, in addition to interrupting operations at the tunnel face. However, conforming with the self-drilling installation procedure was a non-trivial undertaking, even for a single 3.00 m threadbar segment. There were no practical

Figure 6-9: Spile installation and lead cable protection. A: View of a lattice girder and spile round. A FOS spile is also positioned on the jumbo to be installed near center span of the crown. B: Connection between the OF lead cable and the FOS connector. A protective cap covers the connection at the head of the spile after completing the connection. C: Flexible plastic conduit positioned over top of the head of the FOS spile, secured using a hose clamp. The OF lead cable runs through the conduit for protection. D: Positioning of the OF lead cable and plastic conduit assembly around the perimeter of the excavation (along the lattice girder) for improved accessibility during monitoring.
alternatives to the positioning of the FOS connector that would not leave the connector prone to damage during installation. Therefore, an installation coupler was designed to permit water and grout to bypass the FOS connector during installation.

Referring to Figure 6-10, the installation coupler was composed of a 280 mm long steel conduit (42.26 mm outer diameter) with 80 mm threaded lengths on both ends (R32 thread). One end of the coupler was for an instrumented threadbar and the opposing end was for an additional length of threadbar that the jumbo socket could connect with. The 120 mm coupler length between the two threaded sections provided a cavity (35.5 mm inner diameter) for water or grout to circumvent the protective cap covering the FOS connector and then enter the core of the instrumented threadbar through a pair of 9.50 mm diameter inlets. Referring to Figure 6-11, two slots on diametrically opposed sides of the instrumented threadbar were milled to allow water or grout to reach the inlet holes. This modified segment of the threadbar was situated on the tunnel side of the lattice girder (Figure 6-10B) and, therefore, was not anticipated to negatively impact the load carrying capacity of the spile as a whole.

Figure 6-10: FOS spile installation coupler. A 280 mm long steel pipe (42.6 mm outer diameter) acts as a coupling nut (80 mm thread length) for a FOS spile and an additional threadbar length necessary to connect with the jumbo. A 120 mm unthread length across the center span of the coupler (35.5 mm inner diameter) provides a cavity for water or grout to flow around the protective cap housing the FOS connector and into the 14.5 mm diameter core of the instrumented spile through 9.50 mm diameter inlets.
The installation coupler allowed the 3.0 m FOS instrumented spile segment to be installed in conformance with the self-drilling procedure at the tunnel. Both water and grout flowed well through the spile as was evident by grout flow reaching the borehole collar. It is possible that a similar procedure could have been followed to extend the FOS to the deeper threadbar segment; however, it was pragmatic to first test if the FOS would survive the harsh self-drilling installation procedure prior to adding a secondary connection and handling complexity.

Figure 6-11: Grout inlet at the head of the FOS spiles. A pair of 9.50 mm diameter holes were drilled on diametrical opposed sides at approximately 60.0 mm from the head of the spile to allow water or grout to enter the hollow core. These holes are beyond the FOS connector construction (i.e., Figure 6-3), which was blocked off using a 10.0 mm layer of metal bonding adhesive. A slot, 10.0 mm wide and 4.50 mm deep (max depth from circumference), was milled from the head of the spile to the grout inlet holes. The milled slots were necessary to allow water or grout to reach the inlet holes, which were positioned along the threaded section of the installation coupler (refer to Figure 6-10) during installation.
6.4.2.2 OF Lead Management

Following installation of a FOS instrumented spile, the installation coupler was removed in order to access the FOS connector. This involved removing the protective cap that was used during installation and then connecting an OF lead cable (Figure 6-10B). A separate protective cap with a cord grip at its end was then thread onto the head of the spile, overtop of the FOS connection. This assembly isolated the FOS connection, preventing unintentional damage by hitting the FOS connector or pulling on the OF lead. Referring to Figure 6-9C, a flexible plastic conduit was used to protect the OF lead cable. This conduit was positioned onto the head of the spile and fastened using a hose clamp. The protected lead cable was then run along the perimeter of the excavation (following the lattice girder) to an easily accessible height and brought back several meters to the previous excavation round (Figure 6-9D). It was necessary to complete this lead cable procedure immediately following the spile installation round as a shotcrete layer would subsequently completely cover the FOS spile and lattice girder. While this limited access to the FOS connector, it did provide additional protection for the connector and the lead cable assembly.

6.4.2.3 FOS Measurements

Instrumented spiles were installed over several excavation advance positions in the tunnel and were installed in proximity to the mid span of the tunnel crown. Measurements with the DOS interrogator were taken, at minimum, with every excavation advance that was within 6 m from the install position of an instrumented spile. Four measurement plots are displayed for the active sensing length along a selected spile, which have been determined from a comparison of the three sensing lengths along the spile (as discussed in Chapter 3):

1) The resolved absolute principal (or maximum) strain (Figure 6-12);

2) The resolved bending moment induced strain component referenced to the top alignment of the spile (Figure 6-13 top);

3) The resolved coaxial strain component (Figure 6-13 bottom), and;
4) The resolved orientation of bending moment inducing strain around the circumference of the spile (Figure 6-14).

Figure 6-12: Absolute maximum strain measured along the active sensing length of an instrumented spile at selected measurement dates. The baseline (i.e., surface) measurement was taken prior to installing the instrumented spile with it laying flat on the tunnel floor. The Install Reading was taken immediately after installing the spile and lead cable assembly. The 1st Reading and 2nd Reading correspond to the first and second excavation advances, respectively. The 1 Month Reading corresponds to a measurement approximately one month after installation. The approximate position of the tunnel face along the spile for the Install Reading, 1st Reading, and 2nd Reading are indicated by the dashed lines. Note: Tensile strain is taken as positive.
Figure 6-13: Moment induced strain and coaxial strain components determined along the active sensing length of the instrumented spile at selected measurement dates. Top: Bending moment induced strain corresponding to the top alignment of the spile. Bottom: Coaxial strain acting through the cross-section of the spile. The approximate position of the tunnel face along the spile for the Install Reading, 1st Reading, and 2nd Reading are indicated by the dashed lines. Note: Tensile strain is taken as positive.
The selected measurement dates corresponded with a baseline measurement taken before installation of the spile, a measurement taken after the first excavation advance (1st Reading), a measurement taken after the second excavation advance (2nd Reading), and a measurement taken approximately one month after installation. The measurements taken after the first two excavation advances were taken immediately following the excavation (i.e., prior to tunnel support installation). The position of the tunnel face during the one month measurement was approximately 30.0 m from the installation position of the given spile.

6.4.2.4 Mobilized Spile Strain Observations

The most common analytical approaches for studying the loading behaviour of an umbrella arch element is through treatment of the element as a simple beam or elastically founded beam that is subjected to a distributed load (e.g., Harazaki et al., 1998; John and Mattle, 2002; Oke et al., 2016; Oreste and Peila, 1998; Peila and Pelizza, 2003; Volkmann and Schubert, 2007; Zhang et al., 2014). It is also recognized that the primary loading condition (or loading boundary) of the umbrella arch element will be expected to change throughout its serviceability life as a result of tunnel construction, which includes progressively excavating underneath the element and installing various tunnel support measures (e.g., steel-sets, shotcrete, concrete liner). A mechanistic response representative of this can be distinguished along the instrumented spile by inspecting the bending moment induced strain profiles (Figure 6-13 top) and the orientation profiles (Figure 6-14) for the first two excavation advances, detailing bending moment to have been mobilized over increased lengths of the spile.

Referring to the measurement taken after the 1st excavation advance, a parabolic strain profile was measured along the top alignment of the spile. The spile carried negative (i.e., contractive) strain within the unsupported span between the tunnel face (at approximately 1.70 m) and the position of the tunnel face during spile installation (approximately 0.70 m) that was counteracted by positive (i.e., tensile) strain along the founded sections of the spile. This behaviour is indicative of the longitudinal stress redistribution away from the unsupported span to the previous support round and the ground mass ahead of the tunnel face through bending of the spile. It is observed that the material strength and stiffness as well as the coupling
The first 0.70 m of the spile (i.e., between the girder and tunnel face during installation, Figure 6-9C) was completely cast in a 150-200 mm thick steel-fibre-reinforced-shotcrete layer prior to the first excavation advance, whereas the toe end of the spile was founded in the clay ground mass. Accordingly, the bending moment reversal at the head of the spile, which was relatively fixed in the shotcrete layer, occurred over a condition between the spile and its founding material greatly impacted the spile’s mechanistic response.

Figure 6-14: Orientation of the bending moment inducing strain around the circumference of the spile with respect to the top alignment of the spile (i.e., 0° refers to load acting vertically downward on the top profile of the spile). The approximate position of the tunnel face along the spile for the Install Reading, 1st Reading, and 2nd Reading are indicated by the dashed lines. Note: Positive angles refer to a clockwise orientation from the top alignment.
shorter distance than the toe end of the spile, which was embedded in the clay. The latter is a comparatively less constraining boundary condition.

The second excavation advance extended this behaviour to a further distance along the spile. As shown in the bending moment induced strain profile (Figure 6-13 top), the length of the spile that carried a negative (contractive) bending strain extended to the end of the active sensing length and, accordingly, the counteracting positive strain is not measured near the toe end of the spile. This is corroborated by the orientation profiles (Figure 6-14). After the first excavation advance there is a clear inflection point (where the bending moment reverses orientation by approximately 180°) within the ground mass ahead of the tunnel face (at approximately 2.15 m along the spile). This provides a rough indication of the stability position ahead of the face where the spile was solely founded in the ground mass, as opposed to being actively loaded by ground deformations in the direction of the excavation. Accordingly, this provides an estimate of the stability surface ahead of the tunnel face. After the second excavation advance, this inflection point is no longer observed at the end of the spile and this additional length of the spile is subjected to contractive bending strain (or what is essentially a distributed load along the top alignment of the spile).

There was very little strain that was measured to have been mobilized along the active sensing length of the spile in the subsequent excavation advance rounds. The loading orientation around the spile remained relatively constant and the bending moment induced strain steadily decreased from the 2nd Reading to the 1 Month Reading, where it then stabilized. Accordingly, only the first 2 m of excavation advance were found to mobilize significant strain along the spile. This is not an unanticipated result since the overlapping of spiles with subsequent excavation advances would be expected to limit the load transfer responsiveness of spiles that are further away from the tunnel face (Harazaki et al., 1998; Oke et al., 2016; Peila and Pelizza, 2003). However, it must be acknowledged that only the first 3 m segment of the spile was instrumented. It is not believed that the deeper segment of the spile was subjected to significant strain (i.e., of greater magnitude than that presented) with following excavation advances since the first 3 m segment, which was closer to the excavation profile, was measured to be relatively unaffected.
Nevertheless, it is possible that the final 1 m of the spile, which would be the top spile in the overlapped formation, could have experienced a localized end moment that was not captured by the instrumented segment.

Bending moment is considered to be the most critical umbrella arch design criteria (e.g., Oke, 2016; Song et al., 2013). Therefore, it was unanticipated that a comparable magnitude of coaxial strain to bending moment induced strain would be measured along the spile (Figure 6-13). However, unlike the bending moment induced strain, the coaxial strain mobilized along the spile was considered to be relatively independent of the excavation advance rounds. The coaxial strain measured after the installation of the spile remained relatively constant after the first and second excavation advances. In comparison, the same excavation advance rounds mobilized substantial bending moment induced strain along the spile. As previously discussed for the mechanized installation of the CT-Bolt (refer to Figure 6-6), the uniform coaxial strain profile measured along the spile indicates that a free length of the spile was subjected to tension between the lattice girder, at the head of the spile, and some further point along the spile, outside of the active sensing length. The minor bending moment induced strain measured after installing the spile provides some indication that the coupler between threadbar segments may have fixed within the borehole and induced the uniform tension along the instrumented spile segment, but this cannot be verified. Nevertheless, the spile was not measured to be mobilized by or to significantly resist ground deformations parallel with the tunnel drive.

6.4.3 Measuring Crown Reinforcement Effect in thickly bedded ground

In a discontinuous ground mass a CMC reinforcement element can be expected to take on a load distribution that is reflective of a number of localized discontinuity movements (Bjornfot and Stephansson, 1984; Hyett et al., 1996; Li and Stillborg, 1999), which may act coaxial and/or transverse to the element’s axis. In general, this will result in a combination of coaxial loads and bending moments being mobilized along the reinforcement element. An example ground mass that exhibits such discontinuous behaviour is the Hawkesbury Sandstone unit in the greater Sydney region (Herbert and Helby, 1980). This is a near
horizontally bedded Triassic sandstone deposited in 1-5 m thick layers, typically 2 m, massive or cross bedded, with occasional siltstone bands (e.g., Bertuzzi, 2014; Pells, 2002). A relatively high horizontal in-situ stress field is known to be persistent in the Sydney region (Macklin et al., 2014; McQueen, 2004). The combination of the high horizontal in-situ stress and the low shear strength and/or low shear stiffness at sub-horizontal beddings has been discussed to substantially increase crown stress concentrations when excavating in the Hawkesbury Sandstone, resulting in both brittle failure (i.e., spalling and slabbing of intact material) as well as differential shear deformations across pre-existing and newly formed discontinuities (De Ambrosis and Kotze, 2004; Hestermann et al., 2017; Pells, 2002, 1980). A CMC tunnel reinforcement element installed in such ground conditions must have the load carrying capacity to suspend/anchor structurally controlled instabilities and provide resistance to localized differential shear displacements across transecting discontinuities (Bertuzzi and Pells, 2002; Oliveira and Paramaguru, 2016). It is also critical that the element has enough deformation capacity to withstand potentially sudden dilational zones from stress induced failure (Oliveira and Diederichs, 2017).

Referring to Table 6-1, FOS instrumented CT-Bolts were installed as temporary support at an active tunnel operation in the Hawkesbury Sandstone unit. The bolts were installed using the same mechanized procedure as discussed for the CT-Bolt pull test, including a designated 50 kN of preload and full encapsulation using cement grout. The CT-Bolts were installed overtop of a steel-fibre reinforced shotcrete layer at the excavation periphery. A 5-10 m long OF patch cable was connected to the FOS and bundled at the head of the CT-Bolt when not being used for measurement. This necessitated a boom lift to unbundle the OF lead down to the floor level in order to take measurements with the DOS interrogator. Strain measurements from a selected CT-Bolt that was installed midspan in the crown of an arch shaped excavation (100 m² approximate face area) at a section of the tunnel with approximately 60 m of cover are presented and discussed in the following section.
6.4.3.1 Mobilized CT-Bolt Strain Observations

Figure 6-15 displays the resolved absolute principal (or maximum) strain along the active sensing length of the CT-Bolt corresponding to measurements described in Table 6-2. Figure 6-16 displays the resolved coaxial and bending moment induced strain components for the same measurement dates. It should be noted that the FOS was tared (i.e., zeroed) after the mechanized installation procedure. Accordingly, the strain induced by the preloading procedure (refer to Figure 6-6) was removed from the baseline measurements shown in Figure 6-15 and Figure 6-16. This was performed in order to solely examine the strain mobilized along the CT-Bolt as a result of ground displacements. Bedding and/or discontinuity locations that were

![Diagram of CT-Bolt strain observations](image)

**Figure 6-15: Absolute maximum strain measured along the active sensing length of a FOS CT-Bolt.** The location of sub-horizontal discontinuities as observed using a borehole camera in an adjacent borehole following the installation of the CT-Bolt are shown as black dashed lines. Note: Tensile strain is taken as positive.
identified by a borehole camera in an adjacent borehole are also indicated in the strain profile plots (black dashed lines).

Table 6-2: Summary of FOS CT-Bolt strain measurements detailing: the distance of the CT-Bolt from the tunnel face, the time duration from the installation date of the FOS CT-Bolt, and additional information associated with the measurement (including auxiliary excavation activity near the FOS CT-Bolt).

<table>
<thead>
<tr>
<th>Measurement</th>
<th>Distance from Tunnel Face (m)</th>
<th>Duration Since Installation (Days)</th>
<th>Additional Information</th>
</tr>
</thead>
<tbody>
<tr>
<td>Baseline</td>
<td>4.9</td>
<td>0</td>
<td>- Measurement zeroed following installation of the CT-Bolt</td>
</tr>
<tr>
<td>1st Reading</td>
<td>11</td>
<td>4</td>
<td>- Tunnel face advance</td>
</tr>
<tr>
<td>2nd Reading</td>
<td>16.05</td>
<td>6</td>
<td>- Tunnel face advance</td>
</tr>
<tr>
<td>3rd Reading</td>
<td>46.65</td>
<td>19</td>
<td>- Tunnel face advance</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Widening of an adjacent excavation conducted</td>
</tr>
<tr>
<td>4th Reading</td>
<td>N/A</td>
<td>142</td>
<td>- Cross passage excavation completed parallel to the CT-Bolt</td>
</tr>
</tbody>
</table>

The measurements taken over an approximately 5 month period indicated that the CT-Bolt was subjected to a combination of coaxial and bending moment induced strain during excavation advance, profile widening of an adjacent excavation, and cross passage excavation conducted orthogonal to the element. The latter was measured to mobilize the most significant strain along the element; however, the cross passage excavation occurred during a 123 day interval between the 3rd Reading and the 4th Reading. This is a substantially longer interval than between the prior measurements where less strain was mobilized.

Referring to the bending moment induced strain profiles (Figure 6-16 bottom), the FOS was found to be very accurate at identifying the locations of bedding partings and/or discontinuities relative to the excavation periphery (at approximately 1.61 m, 2.27 m, 3.22 m, and 3.57 m along the CT-Bolt). At each discontinuity a distinct shear couplet was measured, which is observed as a compressive and extensile bending moment induced strain development that mirrors on opposing sides of a shearing discontinuity. This measured behaviour is consistent with a dowel reinforcement effect, which is typically observed as an
Figure 6-16: Coaxial strain and moment induced strain components determined along the active sensing length of the FOS CT-Bolt. Top: Coaxial strain acting through the cross-section of the CT-Bolt. Bottom: Bending moment induced strain corresponding to one side of the element’s most eccentric external fiber (i.e., one of the pair of external fibers subjected to maximum moment induced strain, furthest from the neutral axis). The location of sub-horizontal discontinuities as observed using a borehole camera in an adjacent borehole following the installation of the CT-Bolt are shown as black dashed lines. Note: Tensile strain is taken as positive.
S-like or double bend of the element (Ferrero, 1995; Grasselli, 2005; Jalalifar, 2006; Pellet and Egger, 1996; Spang and Egger, 1990) as opposed to direct guillotining (Stillborg, 1994). The shape of the bending moment induced strain profiles at a shear plane are also consistent with the measured strain response of a grouted CT-Bolt subjected to a double shear test configuration in the laboratory (Chapter 3).

Figure 6-17 presents a comparison between the in-situ measured bending moment induced strain profile and the orientation of load with measurements from a laboratory sheared bolt. The laboratory results correspond to approximately 1.75 mm of differential shear displacement and were selected to match with the maximum bending moment induced strain with the in-situ measurements. The in-situ measurements correspond to the 4th Reading for the shear plane located at 3.22 mm along the CT-Bolt. The borehole camera indicated this location to be the thinnest shear plane (although it was unable to accurately quantify the thickness) and, therefore, was most apt for comparison with the laboratory arrangement (a 3 mm thick, frictionless shear plane). For both the in-situ and the laboratory measurements, an inflection of the bending moment induced strain occurs within proximity of the shear plane. The strain inflection corresponds with an approximately 180 degree rotation of the load orientation around the element. This identifies that diametrical opposed sides of the CT-Bolt were mobilized on opposing sides of the shear plane and, therefore, shear displacement was being resisted by the element (essentially increasing the cohesion of the discontinuity). The maximum bending moment for all four in-situ identified shear planes was located between 70.0 mm and 42.0 mm from the center of the shear plane and was negligible beyond 190 mm. Comparatively, this is a further distribution of moment along the element than measured in the laboratory arrangement (Figure 6-17 top), but the laboratory shear plane was also a thin, idealized discontinuity, and was essentially non-dilating.

Referring to Figure 6-16 (top), at each discontinuity coaxial strain was mobilized at a near-equal or greater magnitude of strain than that which was induced by bending moment. The coaxial strain profiles exhibit multiple maxima that are located at the discontinuities. The maxima positions have been conceptually discussed as neutral points (Freeman, 1978), where the interfacial shear stress between the
Figure 6-17: Shear response comparison between in-situ and laboratory measurements. The selected in-situ strain profile and load orientation correspond to the 4th Reading at the shear plane located at approximately 3.22 m along the CT-Bolt (refer to Figure 6-16). The laboratory results correspond to a cement grouted CT-Bolt subjected to approximately 1.75 mm of differential shear across a thin (3 mm thick), frictionless shear plane. Top: Bending moment induced strain measured along the top alignment of the CT-Bolt. Bottom: Orientation of the bending moment inducing strain around the circumference of the CT-Bolt with respect to the top alignment of the element (i.e., 0° refers to load acting vertically downward on the top profile of the element). Note: Tensile strain is taken as positive and a positive orientation angle refers to a clockwise rotation from the top alignment.

reinforcement element and the confining grout is zero and the tensile stress of the element is maximum. On
opposing sides of the neutral point exists a pick up length, where the element resists ground mass movement towards the excavation (considered to be positive interfacial shear stress), and an anchor length, where the element anchors to deeper seated ground (considered to be negative interfacial shear stress) (e.g., Hyett et al., 1996; Li and Stillborg, 1999). This mechanistic behaviour is evident in Figure 6-18, which shows a comparison between the coaxial strain ($\varepsilon_{\text{coaxial}}$) profile from the 4th Reading and the corresponding shear stress ($\bar{\tau}$) distribution determined according to Equation 6-2 (Farmer, 1975)

$$\bar{\tau}_i = \frac{rE}{2\Delta x} (\varepsilon_{\text{coaxial},i+1} - \varepsilon_{\text{coaxial},i-1})$$  \hspace{1cm} 6-2

where $r$ is the effective radius of the reinforcement element, $E$ is the Young’s modulus of the element, and $\Delta x$ is the spatial resolution of strain measurements (i.e., 0.65 mm). The discontinuities located at 1.61 m and 3.22 m along the element clearly experienced a reversal in the sense of interfacial shear stress as discussed by Freeman (1978). This behaviour is consistent at the other discontinuities, but due to the overlapping of induced strain from adjacent discontinuities is less evident (Bjornfot and Stephansson, 1984; Hyett et al., 1996). The collocation of peak coaxial strains with the position of discontinuities along the CT-Bolt indicates the presence of localized ground mass bulking and/or dilation at the discontinuities; in addition to differential shear displacement.

At the time of the 4th Reading, a borehole camera in an adjacent borehole indicated 1.5 ± 0.50 mm of differential shear displacement at that the discontinuity 1.61 m along the element. This was compared with the CT-Bolt’s measured strain response by converting the bending moment induced strain ($\varepsilon_{\text{bending moment}}$) to deflection ($w$) of the element according to Equation 6-3

$$w = \int \frac{\varepsilon_{\text{bending moment}}}{r} \, dx^2$$  \hspace{1cm} 6-3
The bending moment induced strain between 1.41 m and 1.79 m was considered to be the segment of the reinforcement element mobilized by the differential shear displacement at the given discontinuity. Applying Equation 6-3 to this segment of the FOS measurement indicated the CT Bolt had deflected approximately 1.62 mm across the discontinuity, which is in good agreement with the borehole camera estimate.

A similar approach was also followed to estimate the amount of coaxial bulking/dilation by determining the coaxial deformation along the entire element according to Equation 6-1 (Figure 6-19). The position of discontinuities along the CT-Bolt becomes much less evident when comparing the deformation and strain profiles. It is also a subjective procedure to determine the amount of bulking/dilation that is introduced through numerical differentiation. Tensile strain is taken as positive.

Figure 6-18: Comparison between the shear stress distribution at the reinforcement element-grout interface and the coaxial strain corresponding to the 4th Reading. Positive shear stress denotes slip of the ground mass (towards the excavation) relative to the reinforcement element while negative shear stress denotes the opposite. Note: A 25-point moving average (i.e., 16.25 mm interval) was applied to the shear stress distribution in order to reduce the amplification of measurement noise.
specifically associated with a particular discontinuity. For example, the total deformation of the CT-Bolt after the 4th Reading was approximately 4.00 mm (i.e., the deformation observed at the excavation periphery). Referring to the coaxial strain profile, the discontinuity at 1.61 m appears to have induced a relatively uniform level of strain between 1.57 m and 1.65 m. This segment corresponds to the length of the element where interfacial shear stress reverses (essentially under uniform tension) and is in between the positions of maximum moment on opposing sides of the discontinuity. The deformation of the element over this segment, according to the measured coaxial strain, is approximately 210 µm. This is substantially less than the total deformation; however, the mobilized coaxial strain at this discontinuity is also a result of the differential shear displacement (e.g., Pellet and Egger, 1996). Accordingly, the coaxial strain measurement

Figure 6-19: Coaxial deformation of the CT-Bolt referenced to the end of the active sensing length. The location of sub-horizontal discontinuities as observed using a borehole camera in an adjacent borehole following the installation of the CT-Bolt are shown as black dashed lines. Note: Tensile deformation is taken as positive.
will overestimate discontinuity bulking/dilation. Nevertheless, the FOS technique provides a means to estimate the magnitude and the direction of ground mass displacements surrounding an excavation and it is able to readily distinguish the location of discontinuities.

6.5 Discussion

6.5.1 Application of the FOS Technique
Three in-situ applications of the FOS technique have demonstrated that FOS instrumented reinforcement element can be installed in conformance with various in-situ mechanized installation procedures at active tunnel projects. The robustness of the FOS design, particularly in maintaining the complete FOS assembly below the external profile of each reinforcement element, was central to successful installation and permitted even very austere installation procedures to be followed (i.e., the self-drilling spile installation procedure). Improvements to the FOS connector and the end termination assembly were found to maintain signal integrity after installation of the FOS and enabled multi-month monitoring durations (with the FOSs still operational at the conclusion of monitoring). The importance of obtaining a complete understanding of the reinforcement installation procedure/equipment and the tunnel construction and operation procedures at the design stage of each FOS cannot be understated. This information was crucial for providing adequate protection to each FOS configuration and also for implementing an appropriate monitoring procedure at the given project.

Lead wire protection and management was addressed differently for each presented application. The solution demonstrated for the spiles, involving casting a protected lead within a shotcrete layer and positioning the connection at an accessible junction, is deemed to be the most appropriate for long term monitoring in tunnels. This provides substantial protection for the lead after it has been installed and the connection can be easily accessed throughout progress during tunnel construction and many years afterwards (also discussed by Wagner et al., 2019). While none of the selected in-situ applications employed a continuous or automated monitoring routine, the shotcrete-cast in place lead is the procedure that is recommended.
It should be acknowledged that the discussed FOS technique requires modification of the given reinforcement element to house the OF within machined grooves. This inherently reduces the coaxial and the moment carrying capacity of the element and also limits the technique to reinforcement elements that have dimensions which can accommodate the FOS head assembly and grooves. Nevertheless, these procedures are necessary to adequately protect the glass OF without the addition strain transfer-inhibiting protective layers. There are an increasing number of application examples of high spatial resolution DOS in the construction industry (e.g., Adebola et al., 2020; Barrias et al., 2018; Bersan et al., 2018; Brault et al., 2019b; Lienhart et al., 2019; Madjdabadi et al., 2017; Monsberger et al., 2018). The challenge in transitioning this technology from research centric exercises to industry practice will not be in demonstrating the potential insight that can be gained by measuring excavation performance with high spatial resolution DOS, but in designing FOSs that can be installed and monitored in a manner that conforms with current construction operations at the project (as demonstrated with reinforcement elements in this research).

6.5.2 Measuring the Mechanistic Behaviour of Reinforcement Elements

The high spatial resolution capability (i.e., 0.65 mm spacing between strain measurements) of the DOS interrogation unit permitted strain distributions to be measured along the entire length of tunnel reinforcement elements. This has proven to provide substantial insight into the load transfer mechanisms between the element and surrounding ground mass. When a FOS instrumented reinforcement element was mobilized by a relatively continuous or a singular ground deformation feature, the load development length could readily be quantified. For example, the in-situ pull test identified that approximately 1.00 m of the CT-Bolt will be mobilized by a coaxial load inducing source (refer to Figure 6-7 top and Figure 6-16 top). Similarly, the bending moment induced strain profiles that were measured along the spile element during excavation advances detailed that bending moment was mobilized beyond 1.00 m ahead of the tunnel face (refer to Figure 6-13 top). In both cases, the measured strain distributions quantified the load transfer efficiency of the particular reinforcement element, which will be controlled by the element type/profile and
in-situ ground conditions (e.g., Hyett et al., 1992; Kılıç et al., 2003; Li et al., 2016; Yazici and Kaiser, 1992). The ability to readily quantify this behaviour can be used to justify alterations to or to optimize the design length of the given element.

Under discontinuous ground mass behaviour, the FOS technique was able to accurately identify and locate ground movement features away from the excavation and was also able to differentiate localized ground mass movements into bulking/dilation and differential shear. Referring to the CT-Bolt results (Figure 6-16), differential shear was measured to induce a concentrated bending moment localized to within 0.20 m of the transecting discontinuity. Bulking/dilation of a discontinuity induced a relatively more distributed coaxial strain along the CT-Bolt; however, the combination of multiple discontinuities resulted in a highly variable strain profile. An alternative sensing technique with a coarser spacing between measurements points (e.g., an array of electrical strain gauges) would not able to detail such a varying strain distribution (e.g., Figure 6-15) and would be prone to omission of localized loading features (such as the shearing discontinuities) as well as erroneous interpretation of load/strain along the element. While endoscopic borehole cameras can be used to obtain a sense of internal ground movements, the conversion process from ground displacement to mobilized load of a nearby reinforcement element is non-trivial due to the combination and interaction of multiple load inducing ground movements along the element.

It is a critical advancement to be able to locate and quantify the maximum magnitude of strain that is acting along a reinforcement element. From a quality assurance perspective, measurement of the full strain distribution can be used to provide a greater confidence that a reinforcement element is loaded within design limits, yield strengths, and longevity thresholds, but also that the element is responding mechanistically as designed for. For example, the spile strain measurements confirmed the UA was mobilized with the unsupported span directly behind the advancing tunnel face and the CT-Bolt strain measurements confirmed the elements provided dowel reinforcement across transecting discontinuities. Both of these behaviours cannot be readily identified from the excavation profile (i.e., convergence measurements), but are essential to verify that the reinforcement was responsible for or was involved in
facilitating stress redistribution around the excavation. The FOS technique has been proven to capture these responses and can be directly compared against reinforcement design.

6.5.3 Measuring Ground Deformations Surrounding the Excavation

This research has primarily discussed the capability of the FOS technique to measure distributed and localized coaxial and bending moment induced strain along a reinforcement element. The response of a passive reinforcement element is dictated by the movement of the surround ground mass. Therefore, the strain measurements can also be used to ascertain the inherent behaviour and the condition of the ground mass surrounding the excavation. Referring to the crown installed CT-Bolt results (Figure 6-16), active discontinuities are readily identified along the reinforcement element. It was also shown that both the magnitude and the orientation of movement across a discontinuity and at the excavation periphery can be estimated. Furthermore, as evident by the spile measurements, it is apparent that the FOS technique can be used to investigate the ground mass response ahead of the tunnel face (a concept also discussed by Vlachopoulos and Forbes (2018).

6.6 Summary

This research has discussed several improvements and advancements that were made to be able install and monitor strain along in-situ tunnel reinforcement elements that were instrumented using the FOS technique presented in Chapter 3 (Forbes et al., 2017). FOS instrumented CT-Bolts and spiles were successfully installed in conformance with the mechanized installation procedures at the given project, which included a forbidding self-drilling procedure for instrumented spiles. Improvements to the FOS connector and the end termination assembly were found to increase and maintain the measurement signal quality and permitted multi-month long monitoring programs to be conducted using a high spatial resolution DOS interrogator. The FOS technique is not limited to the given interrogation unit, but it was determined to provide unprecedented insight into the in-situ mechanistic response of tunnel reinforcement elements by capturing strain measurements every 0.65 mm along the length of the element (i.e., a nearly continuous strain distribution).
Selected results that were presented for instrumented CT-Bolts demonstrated that the full reinforcement effect (often considered a dowel effect) of a grouted element transecting an active discontinuity could be measured with the FOS technique. This included the differentiation of coaxial and bending moment induced strain along the element and also the orientation of bending moment (i.e., the ground mass movement vector). When subjected to coaxial load, a minimum load development length of 1.00 m was mobilized along the CT-Bolt. Contrarily, differential shear was found to induce a bending moment that was localized to within 0.20 m of the discontinuity. The influence of multiple active discontinuities at various distances from the excavation periphery could be distinguished and the locations compared well with borehole camera measurements.

Spile strain measurements confirmed that the primary reinforcement mechanism of the UA element consisted of a longitudinal stress transfer from the unsupported span to both the previous tunnel support round and the ground mass ahead of the tunnel face. Accordingly, the spile was determined to act as an elastically founded beam, mobilized by bending moment. The spile strain measurements also established that the FOS technique can be used to gain insight into the ground mass response (and condition) ahead of the excavation face.

The presented in-situ applications demonstrated that the FOS technique was able to measure the magnitude and the position of maximum strain along an instrumented reinforcement element, in addition to capturing highly variable mechanistic behaviour. This can be used to more confidently compare the in-situ response of a reinforcement element with the specified reinforcement design. The FOS technique has been proven to have fundamental value as a reinforcement assessment tool, and it can be used to verify that the reinforcement is operating within design thresholds at a project.
6.7 References


213


214


215


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Chapter 7 An in-situ monitoring campaign of a hard rock pillar at great depth within a Canadian mine

7.1 Introduction

Creighton Mine is an active underground nickel and copper mine owned and operated by Vale in Sudbury, Ontario, Canada. The mine is situated in the south-east region of the Sudbury Igneous Complex; which was emplaced via the Sudbury Event meteorite impact and impact melt process approximately 1.85 billion years ago (Dietz 1964; Krogh 1984; Deutsch et al. 1995). Ore body mineralization predominantly follows the contact between the granite-gabbro footwall rocks and norite hanging wall, the basal unit of the Sudbury Igneous Complex. With ore reserves discovered at depths exceeding three kilometers, the current operations as part of the Creighton Deep development are actively mining at depths greater than 2.5 km. This results in in-situ stress ($\sigma_{\text{max}} = 3\sigma_1 - \sigma_3$) to strength ($\sigma_c$ – unconfined compressive strength) ratios greater than unity (Bawden and Coulson 1993; Malek et al. 2008; Morissette et al. 2017b) and, correspondingly, a high risk of severe rockbursting with deep burst volumes (Barton et al. 1974). In maintaining economic viability and adequate safety at the mine’s current depth, and with deeper mine development, an engineering challenge exists in the form of level layout, ore body sequencing, and ground control practice. The latter requires an understanding of the anticipated rock mass behaviour, expected failure mechanisms, and anticipated damage (Kaiser et al. 1996). In the case of Creighton Mine, the presence of several late-stage faults (referred to as shear zones), which are typically schistose and biotite-rich, provide an additional complexity in determining the relationship between excavation-scale stability and the stress redistributions and seismicity induced from bulk mining processes. In such rock mass conditions that are susceptible to rockbursting and bulking ground, Kaiser (2014) notes that the displacement demand on the ground support (a combination of dynamic and pseudo-static factors) is most relevant. However, it is difficult to numerically predict the bulking process and related elevated straining of a supported rock mass with the
current capabilities of continuum, discontinuum, and hybrid numerical techniques (Kaiser 2014, Vazaios et al. 2018).

In recent research efforts that addressed the ground support demands at Creighton Mine, Groccia et al. (2016) conducted a comprehensive monitoring study of the bulking factor at several locations on the 7910 level (2.41 km depth) of the mine. In their study, the depth of failure from the excavation periphery was measured via: core photograph examination, borehole camera logging (within split sets), core logging, optical televiewer logging, and ultrasonic velocity logging. A wide range of bulking factors (1.4% to 41%) were quantified and attributed to local structure and seismicity. In addition, the progressive development of failure depth was measured. However, the time intensive nature of the measurement techniques (i.e. the physical requirement to take measurements) resulted in long durations between measurements (months). Consequently, little insight was provided into the affect of daily mining processes and microseismic events on excavation stability. Multi-point rod extensometers, while providing a coarser spatial resolution than the previously discussed techniques, offer a robust and temporally advantageous alternative to measure rock mass displacements. Additionally, by installing an array of such sensors it has been demonstrated that complex failure mechanisms within rock masses can be discerned (e.g., Spearing et al. 2013). Within this context, the research highlighted in this paper presents the findings from a recent mine-by pillar experiment on the 8010 level (2.44 km depth) of Creighton Mine. The project consisted of installing a combination of multi-point rod extensometers, borehole pressure cells (BHPC), and custom fiber optic-based strain sensors to measure the rock mass displacements within a sill drift pillar. Similar to the experiment conducted by Walton et al. (2016), the pillar response was measured throughout mine-by development rounds and production operations on the 8010 level. However, the research effort presented herein has a greater emphasis placed on measuring larger rock mass displacements associated with the geometric non-fit (Kaiser and Cai 2012) during the transition from competent to fractured rock, and then fractured to broken rock. The measured pillar displacements are also used to comment upon the efficacy of rockburst-resistant ground control measures and also the impact of mine-scale structure (i.e., shear zones) on excavation stability.
7.2 Background – Creighton Deep

The 8010 level of Creighton Mine is located at an approximate depth of 2.44 km. Mining on the level primarily focuses on the 461 ore body, which is entirely hosted within the granitic footwall rock mass (refer to Figure 7-1). In general, the host granite-gabbro formation is blocky to massive: RQD (Deere 1963) ≈ 100%, RMR (Bieniawski 1989) = 60-75, GSI (Marinos and Hoek 2000; Hoek et al. 2002) = 75-85, and has an estimated unconfined intact compressive strength of 240 MPa and a Young’s modulus of 60 GPa. The in-situ stress field at this depth in the mine is provided in Table 7-1. The high in-situ stress to rock mass strength ratio categorizes the expected excavation behaviour as stress-driven brittle failure of the intact rock in the form of heavy rockbursting (Barton et al. 1974; Kaiser et al. 2000).

Rockbursting, according Kaiser et al. (1996) is defined as “damage to an excavation that occurs in a sudden and violent manner and is associated with a mining-induced seismic event.” This failure type can be further classified into various mechanisms, including: strainbursting, face/pillar-bursting, and shear/fault-slip (Ortlepp 1997; Kaiser and Cai 2012). There is also a distinction made between rockbursts that are directly mining-induced (coincident with the seismic source) and those that are dynamically triggered by mining (remote from the seismic source) (Hudyma and Heal 2007; Kaiser 2017). At Creighton Mine, seismicity has been identified since 1934 at depths of 0.70 km; however, it is the more recent developments at depths greater than 2.00 km that has associated rockbursting and related seismicity with day-to-day mining activities (Malek et al. 2008). This has promoted the evolution of bulk mining techniques and ground control practice to best avoid rockbursts (Cai and Champaigne 2009), but when necessary, accommodate the high levels of mining induced stress. Regarding the primary focus of avoiding rockbursts, development and production activities are distanced from highly stressed rock mass regions through a combination of destressing techniques (e.g., O’Donnell 1999) during sill development and modifications to traditional vertical retreat mining procedures. A slot-and-slash method is currently used to mine steeply dipping ore bodies. This is a pillarless open stoping technique performed in a top-down and centre-out (or V-shaped) sequence in order to concentrate stress into the abutments (rather than leave stressed pillars
within the ore body). In addition, the top sills are developed subsequently beneath or within the previously mined and backfilled stopes of the level above (Yao et al. 2014). These measures reduce the direct exposure of equipment and personnel to rockbursts, but they do not eliminate the occurrence of rockbursts within the mine. Accordingly, rockburst compatible ground support systems (that have dynamic capacity) are required to improve the stability and reinforce the rock mass (Brady and Brown 2006), retain broken rock blocks and tie back retaining elements, and contain dynamic loads (Kaiser et al. 1996).

Figure 7-1: Plan view of the proposed 8010 level (2.4 km depth) as of January 2016. The pillar of study was located between the 6230 and 6245 sills. The instrumented segment of the pillar is highlighted in red. Stopes taken throughout the monitoring period are also shown in the ore body. FW refers to footwall (Modified after Punkkinen et al. 2018).
Table 7-1: Estimated in-situ stress field on the 8010 level of the Creighton Mine (from measurements taken by Bawden and Coulson 1993; Malek et al. 2008; Morissette et al. 2017b).

<table>
<thead>
<tr>
<th>Principal Stress</th>
<th>Magnitude (MPa)</th>
<th>Trend (Degrees)</th>
<th>Plunge (Degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Major ($\sigma_1$)</td>
<td>113-132</td>
<td>270</td>
<td>10</td>
</tr>
<tr>
<td>Intermediate ($\sigma_2$)</td>
<td>89.0</td>
<td>000</td>
<td>0</td>
</tr>
<tr>
<td>Minor ($\sigma_3$)</td>
<td>65.0-71.0</td>
<td>090</td>
<td>80</td>
</tr>
</tbody>
</table>

The support system employed on the 8010 level evolved following rockbursts experienced while mining the 461 orebody on upper levels (7680 level – 2.34 km, to the 7910 level – 2.41 km). From experiencing an increased frequency of rockburst events with increased depth of mining, a priority was placed on developing a first-pass (and single-pass) dynamic support system that would provide immediate stability to the excavation and also mitigate exposure to a potential rockburst event. Much of the evolution in terms of reinforcement of the rock mass at the mine involved the progression from stiff support tendons, that are prone to localized loading and brittle post-peak behaviour (e.g., fully grouted rebar and mechanically anchored rockbolts), to support tendons that can distribute load and have a ductile post-peak behaviour (e.g., friction coupled and dynamic rock bolts). Localized loading and undesired angular orientations of the support tendon at the excavation periphery is avoided by using a dome face plate that is coupled to the support tendon with spherical washers. Additionally, a single #0/0-gauge 300 mm mesh square is placed under the face plate to distribute load more efficiently from the face plate to the screen or shotcrete surrounding the excavation (Yao et al. 2014). Ground control measures have also been optimized through the qualitative and quantitative assessment of support demands at various locations throughout the mine (e.g., Yoa et al. 2009). This has led to recommendations of location-specific, as well as modifiable, support systems. For example, the support demand required in the top sills that are developed underneath or within the previously backfilled stopes will vary from that of drift and intersection development in the granitic footwall rock mass. However, a ground control challenge still exists in determining how the rock mass displaces with continued mining activity over time, and correspondingly, how the load and elongation
capacities of the support system are consumed. This brings forth concepts of excavation vulnerability or susceptibility to rockbursts and other stability issues (e.g., Heal et al. 2006; Heal 2010; Kaiser and Cai 2013; Duan 2016).

In the systematic design/selection of ground support, each potential failure mode must be considered (i.e., the support system must be designed to withstand a given depth of failure and corresponding static and/or dynamic loads within a given safety margin depending on the in-situ stress and rock mass conditions). If designed correctly, the support system when first installed can be considered to have a relatively high safety margin or low probability of failure. However, with increased mining activities (i.e., increased extraction ratios and mining induced stresses) the load and elongation capacities of the support system will be mobilized. With continued loading, a ground support system that is brought up to or near a state of unstable equilibrium will be highly susceptible to failure/collapse through a coincident or remote seismic event. In this sense, it is critical to understand the mechanisms by which various support members are mobilized and the interaction between static and dynamic support member capacities.

Stress-driven rock mass fracturing and bulking have the potential to result in localized displacement demands at the upper limit and approaching the capacity of a particular support system. Regarding rockbursts, stiff support may inadvertently increase an excavation’s vulnerability to strainbursting by preventing a gradual notch formation to develop. Contrarily, if the support system does not provide sufficient radial resistance, the rock mass may lose its ability to self support (e.g., through ravelling of the stress-fractured arch). Kaiser (2017) remarks that a well-reinforced rock mass, even with the presence of stress fractures, can provide significantly more confinement than a support system alone. In such a reinforced rock mass, it is important to consider that the individual members of a multi-component support system will undertake unique loading characteristics. It cannot be assumed that each member will equally load or expend its capacity. This provides an inherent complexity in quantifying the rock mass response and stability of a reinforced rock mass. Directly measuring the load and/or displacement response of individual members of a multi-component support system would require significant sensing resources; and
may not capture the excavation response in its entirety (e.g., if the depth of failure extends past the given support members). Accordingly, there is a sensing resource advantage to measuring the displacement response of a reinforced rock mass (e.g., with multi-point extensometers) and inferring the load/displacement response of the support system, and its constituent members (e.g., through load-displacement response curves), as well as the full excavation response to mining induced stresses and dynamic disturbances within the mine. This has been performed at Creighton Mine within the framework of understanding contributing factors to excavation vulnerability.

7.3 Mine-By Monitoring Experiment

7.3.1 Monitoring Location and Objective

The location of the mine-by monitoring project was along a sill drift pillar on the 8010 level (approximate mine level depth of 2.44 km) at Creighton Mine. Referring to Figure 7-1, the pillar was hosted within the granite-gabbro footwall rock formation situated between the 6245 and 6230 sills. Sill geometry on the level consisted of a 4.88 m wide by 4.88 m high excavation with vertical sidewalls and rounded shoulders at approximately 3.65 m from the floor. Pillars between adjacent sills had a specified width of 7.31 m, and accordingly, a width to height ratio of 1.5. Ground control for sill excavations of the 8010 level included, but was not limited to: 2.44 m long, 22 mm diameter D-bolts (Li 2010) in the crown extending down to the upper walls (3.05 m from the floor), 2.0 m long, 46 mm diameter welded ring friction sets in the sidewalls, #4-gauge galvanised welded steel-wire mesh along the sidewalls (and back when required), and a layer shotcrete layer along the sidewalls that covers the steel-wire mesh and friction sets (Punkkinen and Yao 2007; Yao et al. 2014, Punkkinen et al. 2018). Single #0/0-gauge 300 mm mesh squares were installed under 150 mm square washer plates with the D-bolts and friction sets to prevent localized failure of the #4-gauge steel mesh and help hold the steel mesh prior to shotcrete. When necessary, an additional #4-gauge welded steel-wire mesh is installed over top of the shotcrete based on observations that installing the steel mesh within shotcrete significantly reduces the elongation capacity of steel mesh (i.e., through localized failure) and its ability to restrain loose blocks at the excavation (Punkkinen and Mamidi 2010). In this
manner, the primary role of the external steel mesh is to contain fractured rock and spalled shotcrete at the periphery.

Rock mass displacement and pressure sensors were installed along an approximately 15.24 m lengthwise segment of the pillar on May 4th, 2016. This instrumented pillar segment ranged from 7.62 m from the pillar nose at the 0002 footwall drift and 12.5 m from the 461 ore body (Figure 7-1). A combination of multi-point rod extensometers, BHPCs, and custom fiber optic strain sensors (FOS) were installed from within the 6245 sill on May 4th, 2016 and extended across the entire width of the pillar (Figure 7-2). All instrumentation was installed within freshly-drilled horizontal boreholes at 2.44 m from the floor and cement toe-grouted until the borehole was filled. Borehole inclination was measured to be approximately 5 ± 5 degrees for all boreholes except for extensometers EXTO_08 and EXTO_09, which had measured inclinations of 10 ± 5 degrees. At the time of sensor installation, the 6230 sill had yet to be developed. Excavation round lengths and dates for the development of this sill are presented in Figure 7-2. The toe-end of the sensors were situated within proximity to the sidewall of the 6230 sill. This necessitated an alteration to the current precondition or “destressing” practice employed at Creighton Mine, which specifies the charging of preconditioning holes drilled ahead of the future face as well as in the upper and lower walls. Accordingly, the requirement for sidewall destressing was removed for the 6230 sill to best protect the toe-end of the sensors. As part of this procedural change, re-entry into the 6230 sill was restricted until it could be verified that all adverse seismic activity had decayed to background levels following an excavation advance. Face destressing protocols were not altered.

On February 28th, 2017 four additional FOS were installed from within the 6230 sill (Figure 7-3). These sensors were installed prior to mining of the 6307 stope at the end of the pillar (Figure 7-1). Referring to Figure 7-2 and Figure 7-3, two additional comments about the level layout and geologic structure should be noted. Unlike the proposed 8010 level layout (Figure 7-1), the 0002 footwall drift that is west of the 6245 sill was shifted approximately 15.5 m north-west (towards the ore body) along the 6230 sill. This was
implemented due to encountered stability issues when mining subparallel with the northeast-southwest orientation of a steeply dipping geological feature.

Twelve persistent fault zones containing schistose material (locally termed shear zones) have been identified in the Creighton Deep development (Coulson 1996; Morissette et al. 2017a). The most seismically active of these faults is the Plumb Shear Zone (PSZ), receiving its name due to its strong association with identifying the presence, shape, and distribution of the plumb ore body (i.e., the 461 ore...
The PSZ is characterized by steeply dipping schistosity containing sub-vertical biotite mineral foliation (mostly healed) and varies in thickness from centimeters to several meters (Malek et al. 2008). (Seidler 2008). The PSZ is characterized by steeply dipping schistosity containing sub-vertical biotite mineral foliation (mostly healed) and varies in thickness from centimeters to several meters (Malek et al. 2008).
High magnitude seismic events and extensive excavation damage (i.e., > 100 tonnes of displaced rock mass) are often attributed to lock-up/release and fault slip mechanisms along shear zones; however, it has been debated that the shear zones only indirectly promote seismicity by governing stress realignment in the footwall rock mass (Snelling et al. 2013). Nevertheless, a recent back analysis of seismicity experienced at Creighton Mine from 2000 to 2013 determined that the majority (>75%) of high damage rockbursts (i.e., > 10 tonnes of displaced rock mass) occurred within 15 m of the nearest shear zone, with the PSZ being accountable for the highest number of events (Morissette et al. 2017a). Referring to Figure 7-3, the PSZ has been mapped to transect the pillar of study in the vicinity of extensometers EXTO_08 and EXTO_09.

Within the context of the monitoring the given pillar at Creighton Mine, three primary objectives were identified:

1) Measure the supported rock mass displacement response to mining activities on the 8010 level;

2) Investigate the impact of transecting mine-scale structure (in specific, the PSZ) on excavation stability and induced seismicity, and;

3) Determine the efficacy of ground control and destressing measures on the 8010 level (in specific, the effect of eliminating wall preconditioning during the 6230 sill development).

Further details on the installed instrumentation and the 8010 level mining operations that were conducted during the monitoring program are provided in the following subsections.

7.3.2 Sensing Technique

Three sensing technologies were installed to monitor the pillar response during mine-by development and 8010 level production operations: 1) multi-point rod extensometers, 2) rock mass displacement and support strain measuring FOSs, and 3) vibrating-wire BHPCs. A general overview of these sensing technologies
and comments on their installations in the given pillar at Creighton Mine is presented within the following subsections.

7.3.2.1 Multi-Point Rod Extensometers
An array of six multi-point rod extensometers (YieldPoint Inc 2017) was installed from within the 6245 sill on May 4, 2016. The head of each extensometer housed six variable induction displacement coils that allowed coaxial displacement to be measured at six specified locations along the full length of the extensometer body (totaling 36 measurement locations). The six measurement locations along each extensometer were distinguished by serrated aluminum anchors that anchored within the cement grouted borehole. Each measurement location (or anchor point) had a 150 mm stroke (with 25 mm maximum contraction and 125 mm maximum elongation selected for this experiment), 0.01 mm measurement resolution, and better than 0.5 % full stroke accuracy. The anchor point arrangement for the extensometers installed into the pillar is presented in Figure 7-4. Extensometer boreholes were drilled an approximate total length of 6.50 m. This allowed the head anchor (closest to the 6245 sill) to be recessed 0.40 m to 0.50 m within the pillar, which also situated the 6th anchor at a similar depth from the 6230 sill when factoring in overbreak. Recessing the extensometer head was considered a necessity for protecting the head assembly at later stages in the mine development when the 6245 sill would be used for mucking. A one-hour logging interval was set for each extensometer to prolong the battery life of 4-channel loggers (YieldPoint Inc 2014), realizing that the instrumentation may not be accessible during certain stages of the 8010 level progression.

![Figure 7-4: Multi-point borehole extensometer anchor locations referenced to the head anchor (closest to the 6245 sill).](image)

7.3.2.2 Distributed Optical Strain Sensing (DOS)
Advancements in data logging and transmission techniques for current multi-point rod extensometers allow for real-time, high-temporal-resolution monitoring of rock mass displacements. However, the limited
number of practical sensors along a single extensometer inherently limits the spatial resolution of measurements. This is extremely disadvantageous for monitoring brittle rock mass behaviour as an array of discrete measurement points or zones will be prone to misinterpretation and possibly omission of localized displacements and fracturing. One interesting alternative to measuring rock mass displacements is the consideration of DOS. There are currently several commercially available DOS technologies which use an optical time domain reflectometry/analysis, optical frequency domain reflectometry, or optical correlation domain reflectometry/analysis technique to interrogate strain along the length of a low-cost silica optical fiber. Of the listed techniques, a Rayleigh backscatter-based optical frequency domain reflectometer offers a solution to measure strain at a spatial resolution of 0.65 mm (Luna Innovations 2017), providing an unprecedented opportunity to monitor the pillar response at Creighton Mine. However, transferring this technology to the mining environment is a non-trivial development (Inaudi et al. 1996; Heasley et al. 1997). A challenge exists in protecting the 250 μm diameter optical fiber from localized failure, while simultaneously not over-dampening the displacement transfer between the FOS and brittle rock mass response (i.e., negating the spatial resolution benefit of using DOS).

A monitored mine-by experiment focused on a sill drift pillar on the 7910 level at Creighton Mine (the production level immediately – 30.5 m – above the 8010 level) measured displacements that were less than 15 mm over an eight-month monitoring period. This included mine-by development and stoping at the end of the pillar (Walton et al. 2016). The highest magnitude displacements (> 10 mm) were measured within 1 m from the pillar sidewalls. Lesser magnitude displacements were measured within proximity to the pillar core (< 5 mm measured). The larger magnitude displacements (10-25 mm of elongation) are well within the range of the multi-point rod extensometers selected for this experiment. Therefore, the design of the FOS was focused on measuring submillimeter displacements, especially towards the core of the pillar. Two FOS configurations were initially conceived: 1) a coaxial sensing configuration and 2) a shape sensing configuration (as in Chapter 3 - Forbes et al. 2017), with both configurations suspending an optical fiber along a core assembly (Figure 7-5). The core assembly was then housed within a 25.4 mm outer diameter
flexible plastic conduit and cast within a urethane rubber (Figure 7-6). The FOS were designed to connect with a 1.75 m long PVC conduit to recess the sensor towards the core of the pillar and avoid larger displacements at the pillar wall as measured by Walton et al. (2016). Both sensor configurations were calibrated and limit tested up to 20 mm (i.e., four times the anticipated displacement magnitude). However, concerns that localized, sub-vertical shear displacements (which would have not been measured/identified by the extensometers installed during the 7910 level pillar study) would potentially damage the sensors warranted the installation of a more heavily protected FOS. This was approached by instrumenting a 7-strand steel cable bolt (as discussed in Chapter 2 - Forbes et al. 2018). The sensor design consists of replacing the central strand of the cable bolt with an instrumented stainless steel tube. This stainless tube was instrumented with a single optical fiber sensor that was centered and encapsulated using a two-part epoxy resin (Figure 7-7). Four 4.75 m long optical fiber instrumented cable bolts were installed into horizontal boreholes (maximum inclination measured to be 4.5 ± 2.5 degrees) from the 6230 sill and cement.

Figure 7-5: Core assembly of the optical fiber displacement sensors. Left - Coaxial Sensor: Tensioning of the optical fiber to be situated along the centroid of the assembly. Right – Shape Sensor: Tensioning of the optical fiber to be situated along three lengths of the assembly in a delta (i.e., 120-degree) configuration.
grouted. This length situates the sensors past the center of the pillar, but not fully to the 6245 sill (Figure 7-3).

7.3.2.3 Borehole Pressure Cells
In addition to the multi-point rod extensometers and FOS, three BHPCs (Geokon Inc 2014) were installed to compare the induced rock mass displacements and the stress change within the pillar. A flatjack type pressure cell was selected for the experiment, which is predominately sensitive to stress in one plane (Sellers 1970). Accordingly, BHPC_62 and BHPC_64 (Figure 7-2) were installed with flatjacks at perpendicular orientations: horizontally orientated to measure stress changes in the vertical plane and vertically orientated to measure stress changes along the pillar axis, respectively. These BHPC were situated approximately 1 m

Figure 7-6: Example optical fiber sensor. The 25.4 mm outer diameter flexible plastic conduit houses the optical fiber core assembly which has been centered and fully cast using a urethan rubber compound.
from the 6230 sill. BHPC_60 was installed into a 9.75 m long borehole from the 6245 sill to situate the flatjack pressure cell approximately in the middle span of the 6230 sill. This BHPC was installed with its flatjack vertically orientated to measure stress changes in plane with 6230 development rounds. Ideally, an additional BHPC would have been installed with the flatjack parallel to the axis of the pillar; however, difficulties were encountered in maintaining borehole integrity from the nose of the pillar.

7.3.3 Monitoring Program

The monitoring program of the 8010 level instrumented pillar ran from May 19, 2016 to March 30, 2017. During this period the 6230 sill was developed past the instrumented section of the pillar, the footwall drift west of the 6245 sill was repositioned and developed along the 6230 sill, and level-located stoping operations were conducted on the 6327, 6320, 6347 and 6307 stopes (refer to Figure 7-1, Figure 7-2, and Figure 7-3). Information regarding the size, duration, and induced seismicity from development and production rounds were compiled to compare with the measured rock mass displacements and stress changes. A summary of mining operations during this period is presented in Table 7-2, which has been amalgamated from Punkkinen et al. (2018). Multi-point rod extensometers and BHPCs were logged continuously at 1 hr intervals using battery powered logging devices. A long-term battery powered logging solution was not available for the DOS technology at the time of this experiment. Additionally, the unavailability of reliable power during the 8010 level development necessitated physical measurements to

Figure 7-7: Optical fiber instrumented plain strand cable bolt construction using 15.24 mm nominal diameter steel strand (Forbes et al. 2018).
Table 7-2: Summary of 8010 level mining operations. Note: BIE refers to Blast Induced Event (After Punkkinen et al. 2018).

<table>
<thead>
<tr>
<th>Date</th>
<th>Location</th>
<th>Event</th>
<th>Comment</th>
<th>Size (10^9 g)</th>
<th># BIE</th>
<th>BIE Magnitude</th>
<th>Hours to clear</th>
</tr>
</thead>
<tbody>
<tr>
<td>2016-05-25</td>
<td>6327</td>
<td>Production</td>
<td>Raise</td>
<td>1.44696</td>
<td></td>
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</tr>
<tr>
<td>2016-05-26</td>
<td>6327</td>
<td>Production</td>
<td>Raise</td>
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</tr>
<tr>
<td>2016-05-30</td>
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<td>2</td>
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<tr>
<td>2016-06-02</td>
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<td>Production</td>
<td>Stope</td>
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<td>3.5</td>
</tr>
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<td>6327</td>
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<td>Crown</td>
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<td>2.9 Mn</td>
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<tr>
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</tr>
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<td>2016-08-16</td>
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<td>Square</td>
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<tr>
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<td>2.785965</td>
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<td>Crown</td>
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be taken with the optical fiber based sensors. In this regard, the multi-point rod extensometers provided high temporal resolution measurements of rock mass displacements within the pillar while the FOS (being used as an inspection tool) provided high spatial resolution measurements.

7.4 Monitoring Results

The following section presents an overview of the displacement, strain, and pressure response measured across the sill drift pillar. Refer to Figure 7-2 and Figure 7-3 for the positioning of sensors.

7.4.1 Multi-Point Rod Extensometer Results

Approximately 7500 measurements were logged for each anchor location of the six installed multi-point extensometers. These measurements were continuously updated into time series plots of the rock mass displacement. Figure 7-8 displays the displacement measurements for each extensometer with selected dates and events annotated. It should be noted that the third anchor location was taken as the reference position for displacement measurements. This anchor for each extensometer was located at the approximate mid span of the pillar. Therefore, each displacement-time series plot presents the rock mass displacements approximately referenced from the pillar center, where positive displacements are in the direction of the 6230 sill and negative displacements are in the direction of the 6245 sill.

As of September 21st, 2016, specific anchor locations were either damaged or had their displacement range exceeded (this has been illustrated with a transition from a solid to dashed line in the plots of Figure 7-8). A clear example of this in the measurement data is shown for EXTO_09 within the period of September 20th, 2016 to September 24th, 2016. During this period, the 6230 sill was advanced past EXTO_09 (the extensometer closest to the ore body). Following the 6230 sill advance on September
20th, 2016, the displacement of anchor 6 (i.e., closest to the 6230 sill) displaces significantly over a one-hour period (i.e., the logging rate) and then continues to displace up to September 24th, 2016. However, the induced displacement accompanying the advance round on September 24th, 2016 exceeded the displacement range of anchor 6. After noticing this in the measurement data, an investigation in the newly excavated 6230 sill found anchor 6 from extensometer EXTO_09 visible (extending out) near the shoulder of the excavation. A summary of encountered measurement issues or limitations for all extensometers is presented in Table 7-3. As a majority of these issues were within several months of the 6230 development, it was selected to present the displacement measurements of the entire pillar width only up to December 21st, 2016.

In general, the most significant displacements were measured over the period of August 26th, 2016 to October 1st, 2016 by the head, 5th, and 6th anchor positions (nearest to the pillar sidewalls). This corresponds with the substantial loss of confinement to the instrumented pillar (especially at the sidewalls) as the 6230 sill was advanced past each extensometer. Mining induced stress changes from level production blasts (i.e., stoping) and footwall drift development along the 6230 sill (e.g., November 26th, 2016) also induce noteworthy displacement measurements. An examination of the displacement-time series plots reveals a time-dependent nature to the measured rock mass displacements. Following selected mining activities (Table 7-2), there is an immediate increase in the magnitude of measured displacement over the first logging interval (i.e., 1-hour). Displacements then continue to accumulate at a decaying rate until a new position of stability is reached. Similar to cumulative seismicity and seismicity rate analysis for re-entry protocols (e.g., Vallejos and McKinnon 2011), the time-dependent displacement behaviour can be analyzed by conducting a 1-hour linear regression of the measured displacements to obtain velocity-time series profiles for each extensometer anchor position (Figure 7-9). The velocity profiles can then be used to quantify the time-duration of mining activity induced displacements by determining when the velocity returns to the noise floor or the background velocity measurement (approximately ±0.01mm/hr) (Figure 7-10). To determine the time-duration from the velocity time-series profiles, a Hampel filter was employed
Figure 7-8: Measured anchor displacement profiles referenced to the third anchor position (approximately the center position of the pillar) from May 19th, 2016 to December 21st, 2016 for each digital extensometer. Positive displacements are in the direction of the 6230 sill while negative displacements are in the direction of the 6245 sill. A dashed line displacement profile indicates an issue or limitation with the measured displacements (refer to Table 7-3) (Modified after Punkkinen et al. 2018).
to filter out possible low velocity occurrences (i.e., below the noise floor) that were not indicative of displacement stabilization for a given anchor position. It was also necessary to analyze the absolute velocity profiles in order to capture instances where bulking or extension (positive displacement) was followed by contraction (negative displacement), or vice versa. The time-durations of anchor displacement following a mining activity for all extensometer anchor positions are presented in Figure 7-11 (left plot). This plot only displays instances where anchor displacement was induced via a mining activity. The shortest time-duration that could be determined was 1-hour, as per the logging rate.

From the time-duration of mining activity induced anchor displacement, two other measures of the rock mass response were quantified:

1) The immediate anchor displacement, which refers to the displacement measured in the first hour (or one logging interval) after a mining activity was conducted, and;

2) The cumulative anchor displacement, which refers to the measured displacement over the determined time-duration.
The difference between the cumulative displacement and the immediate displacement is considered to be the extended displacement.

A comparison of the immediate and extended anchor displacements for all mining activities which induced displacement are presented in Figure 7-11 (right plot). This categorizes a total of 583 instances of mining activity induced anchor displacement across all extensometers. 488 instances (83.7%) are binned between 0 mm and 1 mm (i.e., sub-millimeter displacement). Furthermore, 362 instances (62.1%) are within the range of 0 mm and 0.1 mm. Referring to the time-duration plot, 238 instances (40.4%) are categorized as single hour events. An increased logging rate would most likely have distinguished more induced...
displacement events and would have provided a more accurate event time-duration to compare against seismic measurement arrays at the mine; however, the 1-hour logging rate was necessary to prolong battery life. Nevertheless, the extensometer data set clearly distinguishes that in over half of the instances where displacement was induced via a mining activity on the 8010 level, displacements within the granitic pillar accumulated for multiple hours post activity. In some instances, the extended displacement was substantially more than the immediate displacement. To further examine this time-dependent response

Figure 7-9: Displacement-time series profiles (top) and corresponding velocity-time series profiles (bottom) for selected anchor positions of extensometer EXTO_08 from September 10th, 2016 to September 28th, 2016.
across the pillar, the extensometer data was categorized by anchor position (or position within the pillar). A comparison of the time-duration and the cumulative displacements induced via mining activities are presented in Figure 7-12 for the six anchor positions.

In general, an increased number of multi-hour time-durations were measured towards the pillar sidewalls (i.e., anchor positions: Head, 1, 5, and 6). The magnitude of induced anchor displacement was also larger towards the pillar sidewalls. These observations were anticipated considering the variability of confining stresses spanning the pillar (lowest at the walls) and is expected to have been even more pronounced if the 6th anchor position for all extensometers had continued to operate for all of the investigated mining activities. However, there was an unexpected number of large displacements and multi-hour time-durations induced at the anchor positions nearest to the pillar core (i.e., anchor positions: 2 and 4). Although these events were found to primarily correspond with sub-millimeter cumulative displacements, in the several instances the cumulative displacement of anchor positions 2 and 4 exceeded 1 mm. A further inspection of these irregularities revealed that all cumulative displacements over 0.50 mm

Figure 7-10: Visual explanation of event time-duration determination from an example velocity-time series profile. IOLV refers to an interval of low velocity which is not indicative of displacement stabilization.
for anchor positions 2 and 4 were attributed to EXTO_08. Referring to Figure 7-8, the symmetry of the displacement-time series profile of EXTO_08 for anchors 2 and 4 suggests that displacement extended across the entire pillar in the region surrounding this extensometer. Therefore, to more accurately understand the pillar response in this region, the displacement measured at each anchor position was converted to strain between adjacent anchor positions (removing the reference position necessity of displacement measurement data). Figure 7-13 presents a comparison between the strain-time series profiles for EXTO_08 and EXTO_11 for a period of the 6230 mine-by (September 10th, 2016 to September 28th, 2016). The strain-time series profile of EXTO_11 is characteristic to the response of the other extensometers, in that little strain is measured within the vicinity of the pillar core (i.e., Anchors 2:3 and Anchors 3:4). However, the strain-time series profile of EXTO_08 is unique, clearly indicating significant strain, and therefore displacement, across the entire span of the pillar. This inconsistency is attributed to the presence of the PSZ, which transects the pillar in proximity to EXTO_08 (and also the toe-end of EXTO_09), as further investigated in the discussion section of this article.

Figure 7-11: Histogram plots displaying the frequency distribution (log-scale) of mining activity induced extensometer anchor displacements (for all extensometers). Left plot: Time until anchor displacement stabilizes following a mining activity. Right plot: Comparison between Immediate and extended anchor displacement. An increased bin-width resolution for displacements between 0 mm and 1 mm is presented within the right plot. Note: There are three events not shown in the left plot (for visualization purposes): individual instances with time durations of 40, 49, and 57 hours.
Figure 7-12: Histogram plots displaying the frequency distribution (log-scale) of mining activity induced displacement (cumulative) and displacement time-duration classified by extensometer anchor position. Anchor positions: Head, 1, and 2 are located approximately 0.40 m, 1.77 m, and 2.99 m from the 6245 sill. Anchor positions: 6, 5, and 4 anchor are located approximately 0.40 m, 1.16 m, 1.92 m from the 6230 sill.
DOS Results

Unlike the extensometer measurements, the FOSs required a constant power source in order to record strain measurements using the selected DOS technology. During the early stages of the monitoring program, while the 8010 level was still in development, power was only available for short intervals. This prevented the configuration of a continuous logging routine for DOS measurements. Accordingly, manual measurements were required for all of the DOS measurements.

The initial DOS measurements are associated with FOS_01 and FOS_02. A baseline reading was taken for these sensors post-grouting on May 20th, 2016. Three other measurements were taken, albeit with several issues being encountered, as summarized in Table 7-4. Figure 7-14 displays the strain profiles measured for FOS_01 and FOS_02 as well as the displacement profiles that have been obtained by integrating the strain profiles from the head of the sensor for FOS_01 (due to the broken sensor length) and the toe of the sensor for FOS_02 (i.e., displacements from the core of the pillar). The FOS displacement profiles compare well with the displacement profiles measured by the extensometers that are immediately adjacent to each FOS. Both FOS_01 and FOS_02 measure similar displacement magnitudes as the

Figure 7-12 – Continued
extensometers; however, the strain profiles obtained with DOS provide substantially more insight into the response of the rock mass across the pillar. Referring to the plot for FOS_02, the sub-millimeter magnitude displacement measured across the pillar is observed to be the result of numerous, localized dilational and contrac tional strain events, indicating fracturing to be the dominant mode of yield across the pillar. This

Figure 7-13: Strain-time series profiles for extensometers EXTO_08 and EXTO_11. Strain profiles correspond to the strain at the midpoint between two adjacent anchor positions of the given extensometer. Positive strain indicates extension and negative strain indicates contraction. A dashed line indicates an issue with the measured displacements (refer to Table 7-3).
behaviour is consistent with previous findings that detail the spalling limit to dictate the lower bound
damage initiation threshold across of the pillar, even at its highest confinement near the core (Diederichs
2003; Walton et al. 2016). This behaviour is also apparent for the strain profile of FOS_01 that was
measured at a later date (and progression of the mine-by), for which an order of magnitude greater level of
strain and displacement is measured. It is unfortunate that a measurement could not be obtained for either
FOS on September 16th, 2016, as this date corresponded to a position of the 6230 excavation face prior to
mine-by of the FOSs. Referring to Figure 7-8, the three subsequent 6230 advances induced substantial
displacements across the pillar, much past those anticipated from Walton et al. (2016). These displacement
magnitudes exceeded the sensor design range and locally failed both sensors.

Table 7-4: Summary of measurements taken for FOS_01 and FOS_02.

<table>
<thead>
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<th>Date</th>
<th>Description of Reading/Issue</th>
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<td>May 20th, 2016</td>
<td>Baseline measurement taken post grouting for FOS_01 and FOS_02.</td>
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<tr>
<td>July 16th, 2016</td>
<td>First measurement taken for FOS_02. Power inconsistency prevents a measurement for FOS_01.</td>
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<tr>
<td>September 16th, 2016</td>
<td>Unable to obtain consistent power. No measurement taken for FOS_01 or FOS_02.</td>
</tr>
<tr>
<td>September 26th, 2016</td>
<td>Measurement taken for FOS_01 and FOS_02. Both sensors have locally broken; however, FOS_01 provides strain data for approximately 1 meter of its initial 2.4m length.</td>
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</table>

As a consequence of the unanticipated high magnitude rock mass displacements, it was prudent to
drastically increase the durability of the DOS technique for a secondary monitoring campaign, even though
this would dampen the transfer of rock mass displacements. This was approached by developing a technique
that coupled the optical fiber sensor with a 7-strand steel cable bolt (refer to Chapter 2). Four FOS Cables
were installed and grouted from the 6230 sill on February 28th, 2017. The installation date for these sensors
was selected in order to measure the pillar’s response to mining the 6307 stope. The measured strain profiles
from the un-plated, plain-strand instrumented cable bolts: FOS Cable 1 and FOS Cable 4, are presented in
Figure 7-15 for the period of March 1st, 2017 to March 6th, 2017 (corresponding to three mining operations
Figure 7-14: FOS strain and displacement profiles compared against interpolated extensometer displacement profiles for selected monitoring periods. The FOS displacement profiles have been obtained by integrating the given strain profile (from the 6245 sill for FOS_01 and from the toe of the sensor near the pillar core for FOS_02). For comparison with the extensometer displacement measurements, the measured anchor displacements have been averaged between extensometer EXTO_08 and EXTO_15 (i.e., the immediately adjacent extensometers to the FOSs). Extensometer anchor displacements have been referenced to the first anchor position for FOS_01 and the third anchor position for FOS_02. Note: Negative strain is contractional and positive strain is extensile.
outer six strands due to the helical construction of the cable. For cable loads under 225 kN, a 20,000-25,000 kN/m/m conversion can be used to convert the strain recorded with DOS to the load of the full cable (Hyett et al. 1997; Forbes et al., 2018). In addition, the FOS strain profiles represent the support member strain, which is not equivalent to the rock mass strain (Hyett et al. 1996). Accordingly, the displacements measured by the adjacent extensometers are most informative from a mechanistic standpoint (i.e., how rock mass displacements induce support member strain). Unfortunately, the displacement capacity was exceeded for

Figure 7-15: FOS cable strain profiles and their corresponding cable displacement profiles for the monitoring period of March 1st, 2017 to March 6th, 2017. Negative strain is contractional and positive strain is extensile.
many anchors of the adjacent extensometers at this stage of the monitoring program, limiting comparison with extensometer measured displacement and cable stretch, which was determined by integrating the cable strain profiles from the toe end of the sensors (i.e., near the pillar core).

The strain (or load) mobilized along the cable is a result of slip between the rock and cable. This generates shear stresses (traction) at the cable-to-grout interface. When observing the cable strain profiles, peaks correspond with localized dilational features across the pillar and decoupling between the cable and rock mass. Conversely, troughs indicate sections along the cable where a degree of compatibility with the rock mass has been maintained (refer to Figure 7-16). Comparing the strain and displacement profiles from both cables, it is apparent that FOS Cable 1 has been subjected to a much less continuous load distribution, where numerous dilational and contractional locations are distinguishable, asserting that the displacement of the pillar sidewall is the result of numerous discrete rock block displacements internal to the pillar. In comparison, the strain profile of FOS Cable 4 is the result of a relatively distributed rock mass displacement profile.

Figure 7-17 presents the shear stress (τ) profiles for FOS Cable 1 and FOS Cable 4 according to the following relationship based on elastic beam theory derived by Farmer (1975):

$$
τ_n = \frac{r_b E_b}{2} \left( \frac{ε_n - ε_{n-1}}{x_n - x_{n-1}} \right)
$$

Where $r_b$ is the radius of the cable, $E_b$ is the Young’s modulus of the cable, and $ε$ and $x$ is the strain and position at the location $n$ along the cable. Referring the shear stress profile for FOS Cable 4, two distinct sections are discernable: 1) A pickup length (where the rock mass is slipping from the cable, resulting in a negative shear stress) and 2) An anchoring length (where the cable is slipping from the rock mass, resulting in a positive shear stress). In between these two sections is the neutral point, corresponding to the position of no relative displacement between the rock mass and cable and also the peak cable load (or strain).
Referring to the shear stress profile for FOS Cable 01, the pickup length and anchoring length sections are only observable as relative regions, with no clear (or singular) neutral point location along the cable. The two cable profiles denote the stark difference in the displacement response of the rock compositions each cable was installed into: the granitic footwall rocks (FOS Cable 1) and the PSZ (FOS Cable 4), as in agreement with displacement trends of the extensometers (Figure 7-8). The relatively

Figure 7-16: Example un-plated cable bolt coaxial displacement and load distribution response to coaxial rock mass displacements. Two conditions are considered (following the analytical solutions proposed by Hyett et al. 1996 – refer to sketches at top): Left - A relatively distributed rock mass displacement profile resulting in a displacement $U_{ex}$ at the excavation periphery; Right – Discrete rock mass displacements at various distances from the excavation periphery ($U_{r1}$, $U_{r2}$, and $U_{rn}$) summing to a displacement $U_{ex}$ at the excavation periphery.

Referring to the shear stress profile for FOS Cable 01, the pickup length and anchoring length sections are only observable as relative regions, with no clear (or singular) neutral point location along the cable.

The two cable profiles denote the stark difference in the displacement response of the rock compositions each cable was installed into: the granitic footwall rocks (FOS Cable 1) and the PSZ (FOS Cable 4), as in agreement with displacement trends of the extensometers (Figure 7-8). The relatively
The distributed load distribution of FOS Cable 4 suggests the PSZ is heavily damaged with no apparent rock blocks. Furthermore, this implies that shearing governs the yielding process of the rock mass across the PSZ span of the pillar.

Figure 7-17: Shear traction (stress) profiles obtained from the March 6\textsuperscript{th}, 2017 measurements of the FOS cables. Negative tractions denote slip of the rock mass while positive tractions denote slip of the cable.
7.4.3 BHPC Results

The installation procedure for the BHPCs included installing and orientating the BHPC flatjack within a borehole, cement grouting the BHPC, and pressurizing the BHPC to allow subsequent changes in the rock stress to be measured. According to Sellers (1970), the stress change measured by the BHPC within a relatively soft grout inclusion can be used to determine the stress change of a rock mass using the following relationship that provides an estimate of the stress reduction factor (SRF) within the hollow inclusion.

\[
SRF = \frac{(D + A')}{2}
\]

Where

\[
D = \frac{(\lambda'_2 + \mu'_2)(\lambda'_1 + 2\mu'_1)}{(\lambda'_1 + \mu'_1)(\lambda'_2 + \mu'_2 + \mu'_1)}
\]

\[
A' = \frac{4\mu'_2}{\mu'_2 + 3\mu'_1}
\]

\[
\lambda' = \frac{vE}{(1 + v)(1 - 2v)}
\]

\[
\mu' = \frac{E}{2(1 + v)}
\]

and \(E\) is the Young’s modulus, \(v\) is the Poisson’s ratio, and the subscripts 1 and 2 refer to the rock mass and the grout, respectively.

Considering the previously listed properties for the footwall rocks and a 0.4 w:c grout (Young’s modulus = 12 GPa, UCS = 60 MPa at full strength, Poisson’s ratio = 0.185 – Hyett et al. (1992)), the BHPC stress change will represent approximately 32% of the rock mass stress change. Therefore, a pre-
pressurization level of 20 MPa for each BHPC was selected to accommodate approximately ±50 MPa of rock mass stress change. A factor in selecting this pressurization level was the integrity of the cement grout (i.e. to not significantly damage the grout during pre-pressurization).

The rock mass stress change for BHPC_64 and BHPC_62 measured over the entire monitoring period as well as during the mine-by period are presented in Figure 7-18. The baseline reading for stress change was taken from August 25th, 2016, corresponding to the stabilization of the BHPCs post pre-pressurization. There are three apparent regions of the pressure-time series plot: 1) The mine-by duration, 2) Mining of the 6347 stipes, and 3) Mining of the 6307 stipes. Similar to the extensometer data series, interpretation has primarily been constrained to the monitoring period before December 22nd, 2016, which coincides with the final mining operation of the 6347 stope. Following this event, the pressure in both BHPC_64 and BHPC_62 reached their minimum range (effectively no pressure remaining in the cell) and subsequent results fall outside of the sensors’ calibration. The results from BHPC_60 have not been presented in Figure 7-18 as there were issues with pre-pressurizing this instrument.

A strong relationship between the 6230 mine-by and stress change is evident for both BHPCs. Referring to the mine-by round locations shown in Figure 7-2, the vertical stress (BHPC_62) rises with each mining activity and peaks with the advance on September 20th, 2016. For 9 hours following this advance, stress exponentially decreased a total of 20 MPa. Subsequently, vertical stress decreased linearly at a lower gradient. This date also corresponded with three mining induced seismic events being registered. The stress change along the pillar axis (BHPC_64) initially followed a similar trajectory as the vertical stress up to September 20th, 2016. However, it is not until the 6230 advance on October 1st, 2016 that a peak stress was reached (even though this BHPC is situated within the same advance position as BHPC_62), at which point stress decreased rapidly in a semi-linear fashion. Both BHPCs experience a rapid loss of pressure following a peak measurement that was over 50 MPa. It is pragmatic to coincide this pressure decrease with damage to the grout (and surrounding rock mass). The immediately adjacent extensometer anchor locations to the BHPCs both measured rock mass strain in excess of 1% synchronous with the
pressure decrease. Accordingly, the applicability of rock mass stress change in comparison to the original baseline on August 25th, 2016 must be treated with speculation. However, it is compelling to observe that following the complete mining of the 6347 stope on December 22nd, 2016 (and deconfinement at the ore

Figure 7-18: Pressure-time series plots for BHPC_62 (aligned to measure vertical stress change within the pillar) and BHPC_64 (aligned to measures stress change along the pillar axis). Top: Rock mass stress change throughout the entire monitoring duration. Bottom: Rock mass stress change during the mine-by phase. Selected event dates are identified (refer to Table 7-2). A dashed line pressure profile indicates missing measurement data.
end of the pillar) there was an immediate drop in the stress measured along the pillar’s axis (BHPC_64), which also coincided with two 2.0 Mn events being measured.

7.5 Discussion

The original experiment outline sought to monitor the hard rock pillar response to 8010 level mining activities, primarily focusing on the: 1) the mine-by operation of the 6230 sill and 2) mining of the 6307 and 6308 stopes that were aligned with the pillar axis. However, prior to mining the 6307 or 6308 stope, many of the installed sensors had reached their sensing range. Accordingly, emphasis has been placed on examining the supported rock mass behaviour over the period of May 2016 to January 2017. Mining activities of significance during this monitoring period included: 1) mining of the 6327, 6328, and 6347 stopes, 2) mine-by of the 6230 sill, and 3) 0002 footwall development along the pillar of study (Table 7-2). These mining activities are used as the basis to further discuss the measured displacements across the pillar and to comment on the efficacy of ground control measures.

7.5.1 Rock Mass Displacement and the Influence of the PSZ

The extensometer displacement-time series profiles (Figure 7-8) detail a time-dependent nature to displacement stabilization across the pillar following a mining activity. It is apparent that the extensometers within the vicinity of the PSZ (EXTO_08 and EXTO_09) measured larger magnitude displacements that also prolonged after mining activities. EXTO_08 and EXTO_09 measured over 60 mm of displacement at specific anchor locations (or positions within the pillar) by October 10th, 2016. In comparison, the next closest measurement of displacement was made by EXTO_10, measuring 31.9 mm of displacement for anchor position 5. It is also apparent that displacement extended across the entire width of the pillar within the region of the PSZ. Figure 7-19 displays selected strain rate contour profiles across the instrumented segment of the pillar. These profiles have been obtained by linearly contouring the strain measurements between all adjacent extensometer anchor locations along the pillar. The strain rate magnitudes correspond to the change in strain measured over the determined mining activity time duration (i.e., Figure 7-10), unique for each extensometer anchor location. Two strain rate contour profiles are presented:
1) June 6th, 2016: Mining of the 6327 crown; and, 

2) September 20th, 2016: 6230 sill advance round.

Figure 7-19: Microstrain (µε) contour profiles. Left: Strain induced from mining the 6327 crown on June 6th, 2016. At this stage of mining the 6230 sill was not developed. Right: Strain induced from the 6230 sill development round on September 20th, 2016. This mine-by-round corresponded with daylighting of the PSZ, situated near EXTO_08 and EXTO_09, as depicted in the geological overlay. Anchor position 6 measurements have been omitted from the strain contours due to missing data. Tensile strain is taken positive. Approximate distances of anchor positions from the 6245 sill: A – 1.09m, B – 2.38m, C – 3.52m, D – 4.51m, E – 5.35m.

These dates were selected as they depict the contrasting behaviour of the PSZ and the granitic footwall rocks before and after the 6230 sill was developed (August-to-October 2016). Accordingly, the extensometer measurements associated with the June 6th, 2016 mining activity do not correspond to that of a sill drift pillar, but instead, a sidewall at the boundary of the level (which would have much more
confinement than a pillar). Conversely, the measurements associated with the September 20th, 2016 mining activity correspond to the pillar response (refer to Figure 7-1 and Figure 7-2).

Mining of the 6237 stope on June 6th, 2016 induced the most measured strain of all the mining activities prior to the 6230 mine-by. At the head of the extensometers, along the 6245 sill wall, more strain was measured within the vicinity of the mapped shear, but the most significant contrast between the shear zone and the footwall rocks is evident towards the toe-end of the extensometers (Anchor Position E, approximately 5 m from the 6245 sill sidewall). Only EXTO_09 measured significant strain. In addition, only EXTO_08 and EXTO_09 measured strain at the mid span of the pillar. Referring to Figure 7-1, this indicates, at minimum, a mining level connectivity of the PSZ, as the 6327 stope is aligned with the eastern-adjacent 6270 sill. This substantiates previous findings that detail the PSZ to heavily influence the distribution of stress across the mining level (Malek et al. 2008; Snelling et al. 2013; Morissette et al. 2017a).

The 6230 development round taken on September 20th, 2016 coincided with daylighting of the PSZ across the instrumented pillar. In this instance, it is observed that the contour profile agrees well with the geological section in defining the extent of the PSZ, although the relatively large distance between EXTO_08 and EXTO_15 does exaggerate the strain between anchor locations of the two extensometers. There is a significant contrast between the pillar’s strain response in the vicinity of the PSZ and of the footwall rocks (i.e. towards the nose of the pillar). Large rock mass strains are evident along a majority of the pillar sidewalls; however, it is only in the vicinity of the PSZ that strain extends across the entirety of the pillar. Although the PSZ has been reported as healed (Coulson 1996), the selected strain contours and the displacement trends of EXTO_08 and EXTO_09 indicate that the PSZ is highly active during mining development and production. The lack of seismicity registered for the large displacements of the PSZ also suggests that this rock mass unit within the pillar is at a residual state (acting aseismically). This corroborates observations from the FOS cable bolts, which indicated that the PSZ is heavily damaged and that shear governs its yield behaviour, while the granitic footwall rocks displayed a fracture driven failure
process. This can be further extended to indicate that the time-dependent nature of the PSZ is similar to that which is often considered creep, being a shear strain or distortion process. Contrarily, the footwall rocks experience stress relaxation through tensile crack initiation, propagation, and accumulation (Diederichs 1999; Paraskevopoulou 2016).

The stiffness transition between the PSZ and the granitic footwall rocks is of significance to safety. As discussed by Diederichs (2018), with excavation advance through a heterogenous rock mass with contrasting stiffnesses, energy will initially be stored in the softer unit (in this study, the PSZ). As critical stress is reached in the softer unit, it is then transferred to the stiffer unit (the metavolcanic footwall rocks). Conceptually, this will result in increased tangential stresses within proximity of the material transition zone and, consequently, an increased rockburst susceptibility at and near the excavation face when passing through this zone. Referring to Table 7-2, three mining induced seismic events with a peak magnitude of 1.2 corresponded with the September 20th, 2016 sill advance round that transected the PSZ (the largest magnitude registered during the 6230 mine-by). This aligns with previous findings that the vast majority of rockburst in Creighton Deep occur within the vicinity of mine-scale shear zones (Morissette et al. 2017a), but not necessarily as a result of slip (Snelling et al. 2013).

It is important to acknowledge that the displacement measurements correspond to the supported rock mass, and that ground support has a substantial role in stabilizing crack growth following a mining activity. Without support it can be expected that geometric correction in the form of the spalling and crack propagation would undoubtedly result in excessive tangential stresses near the excavation periphery and induce a propagation of instability towards the core of the pillar in the form of slabbing and potentially strain bursting of thicker slabs (Martin and Christiansson 2009; Diederichs and Martin 2010). Therefore, in highly stressed hard rock conditions, similar to those experienced at Creighton Mine, there is a great benefit to maintaining the baggage zone of yielded rock in proximity of the excavation periphery such that a feedback pressure (i.e., confinement) can be generated and progressive damage can be choked off (Diederichs et al. 2004; Diederichs 2007; Walton et al. 2016). In this regard, scaling should only be
performed following initial excavation or if further damage necessitates. Rehabilitation reinforcement methods should be favoured, recognizing that the role of ground support in such mining situations is not to entirely restrict periphery displacements or the geometry of the excavation.

Assessing the timing and extent of the required reinforcement requires a strong understanding of constituent support element load-displacement behaviour, which can certainly be achieved using instrumented support elements. However, it is possible to estimate that state of an existing support system with excavation displacement information, as discussed in the following section.

7.5.2 Stability of the Supported Rock Mass

An understanding of an excavation’s vulnerability to rockbursting is a critical aspect to safety and performance in seismically active mines. There are several empirical and semi-analytical rockburst risk assessment systems (e.g., Kaiser et al. 1996; Albrecht and Potvin 2005; Heal et al. 2006; Heal 2010) that take into consideration several indexed factors, such as: rock mass stress, excavation geometry, rock mass structure/features, and ground support effectiveness. It is recommended to reassess indexed factors after notable seismic events are experienced; however, the ground support factor, or the remaining capacity of the support system, is often overlooked until a degree of failure is observed. In view of the rock mass displacements measured at Creighton Mine, it is argued that the initial support index factor determined after installing the ground support system (during drift development) would drastically underestimate the remaining capacity of the support system at a later time stage (e.g., during major stoping operations). It is proposed that the evolving state of the support system is perhaps the most salient factor to be quantified on a daily (or continuous) basis. A seismically prone excavation is not necessarily a rockburst prone excavation if the support system maintains sufficient residual capacity following a seismic or displacement inducing event. Kaiser et al. (1996) discussed that the load-displacement characteristics of a support system’s constituent support elements can be used to assess support consumption, or remaining support capacity. This concept is demonstrated using the extensometer displacement data from the pillar study and characteristic load-displacement profiles for three selected support tendons.
Figure 7-20 displays common coaxial load-displacement profiles for a 20 mm cement grouted rebar, 46 mm friction set, and 22 mm resin grouted D-Bolt from various laboratory tests (Blanco Martin et al. 2011; Li 2012; Li 2017; Vlachopoulos et al. 2018). The corresponding load and elongation safety margins ($SM$) that have been determined according to Equation 7-7 are also presented

$$SM = 1 - (1/FS)$$

where $FS$ is the factor of safety for load or elongation calculated using the peak load and corresponding elongation (except for the coaxially loaded friction set for which a maximum elongation of 200 mm (Kaiser et al. 1996) was selected). The SM profiles represent the static capacities of the support members. As noted by Potvin and Wesseloo (2013), it is difficult to ascertain the dynamic capacity of an entire support system; however, an estimate of the available energy absorption of an individual support member can be obtained by determining the potential work from the load-displacement profiles (i.e., the area under the load-displacement profile).

As an example of how the measured rock mass displacements can be used to assess the state of a support system, the displacements measured by EXTO_11 have been examined along both the 6230 sill and 6245 sill for three selected dates (Table 7-5). The listed displacement values for the 6230 and 6245 pillar sidewall correspond to the maximum displacement measured by anchors 5 or 6 and Head or 1 respectively (i.e. anchor locations within a typical bolting horizon and near the excavation periphery). To provide a better estimate of the displacement of the 6245 sidewall, it is assumed that the 6245 sill experienced a similar displacement history as that measured for the 6230 during mine-by (at minimum). Therefore, the displacement listed for the 6245 sidewall is taken as the sum of the displacement experienced by the 6230 sidewall post mine-by (November 6th, 2016) and the given 6245 sidewall measurement. Assuming that the measured displacement (which is coaxial to the pillar’s core) is approximately the displacement experienced by individual support tendons, the displacement values listed in Table 7-5 can be used to quantify the static load and elongation SM as shown in Figure 7-20 (bottom).
Figure 7-20: Load-displacement profiles (from Blanco Martin et al. 2011; Li 2012; Li 2017; Vlachopoulos et al. 2018) (top) and corresponding SM profiles (bottom) for three support element types: 20 mm cement grouted rebar, 46 mm friction set, and 22 mm resin grouted D-Bolt. In the SM plots the solid line corresponds to the elongation SM per excavation displacement and the dashed line corresponds to the load SM per excavation displacement. A depiction of the support element elongation and load consumption is shown using measured displacements of the 6230 and 6245 sidewalls from EXTO_11 (as per Table 7-5). The three selected dates coincide with: 1) September 11th, 2016 - Installation of support from the 6230 sill, near-parallel with the selected extensometer, 2) November 6th, 2016 – Completion of the 6230 sill, and 3) December 22nd, 2016 – Final production blast of the 6347 stope.
Table 7-5: 6230 and 6245 sidewall displacement measurements from extensometer EXTO_11 at selected measurement dates. September 11th, 2016 is referenced as the installation date for the support system aligned with EXTO_11. November 6th, 2016 corresponds to the completion of the 6230 sill. December 22nd, 2016 corresponds to the final mining operation of the 6347 stope. For simplicity it has been assumed that all support typologies have been pretensioned to 50 kN or approximately 2.5mm of elongation.

<table>
<thead>
<tr>
<th>Date</th>
<th>6230 Sidewall Displacement (mm)</th>
<th>6245 Sidewall Displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>September 11th, 2016</td>
<td>2.50</td>
<td>15.37</td>
</tr>
<tr>
<td>November 6th, 2016</td>
<td>13.19</td>
<td>44.11</td>
</tr>
<tr>
<td>December 22nd, 2016</td>
<td>33.42</td>
<td>83.21</td>
</tr>
</tbody>
</table>

As the pillar sidewalls displace, the load and elongation capacity of the given support member is consumed in manner that is dependent on the specific load-displacement characteristics. For example, if a grouter rebar member was to have been installed in either the 6230 or 6245 sidewall, Figure 7-20 indicates that by the end of the 6230 sill development (November 6th, 2016) there would be no remaining support capacity in either sidewall (i.e., failure of the support member). In comparison, the D-Bolt and Friction Set members (which were the support elements used on the 8010 level) both still have remaining capacity for managing static (and dynamic) loads. This endorses the decision at Creighton Mine to move away from stiff support tendons such as grouted rebar in such high stress and high bulking developments. This assessment of the support member’s residual static capacity using displacement measurements can be used to aid in the selection/timing of re-support measures. In addition, a similar procedure that considers the remaining energy absorption SM with empirical energy demanded charts (e.g., Diederichs 2018) would allow for a prediction of the potential rockburst magnitude that can be withstood. However, this analysis does require displacement measurement to be aligned with the axis of the support element. Surveyed displacements of the excavation periphery could be used to assess support installed in the shoulders and
back, but internal rock mass displacements (such as those provided by the extensometers and FOSs in this study) are crucial to evaluating performance along a support element’s length.

It is important to appreciate the role of non-coaxial displacements and surface support in dictating the ultimate capacity of the support system. Previous studies (Heal 2010; Potvin and Wesseloo 2013) have found the dynamic capacity of a support system to be less than that of the composite support members and that instead it is controlled by the weakest link, often the shotcrete or mesh at the excavation periphery. Non-coaxial displacements will also be anticipated to significantly reduce the elongation capacity of a support element in the form of lock-up (Groccia et al. 2016), preventing the full member length from being mobilized. Nonetheless, it is apparent that the pillar displacements measured at Creighton Mine necessitate high load and elongation capacity support members (e.g., D-Bolts and Friction Sets). As demonstrated, the capacity of stiff support members (e.g., grouted rebar) would quickly be expended and would increase the potential risk to a rockburst by building stress at the excavation periphery and releasing energy in a brittle manner. In contrast, high static elongation support elements enable excavation strategies that allow for increased levels of controlled excavation deformations, which ultimately reduces the dynamic demand (i.e., allow the rock mass to fracture and bulk to a certain degree). This is corroborated by a general trend of decreased excavation damage per rockburst in recent years at Creighton Mine (since moving away from stiff support such as grouted rebar) even though the mine has progressed deeper (Morissette et al. 2017b). An observation in the displacement data that confirms this concept is the general progressive deformation trend that was measured along both the 6230 and 6245 sidewalls, even though the 6230 sill was not subjected to wall precondition.

7.5.3 Comments on Seismicity
Over the course of the monitoring campaign there were no significant displacement events (i.e., above discernable background noise) that did not correlate directly with a mining activity. This is primarily attributed to the locality of the instrumentation, which is restricted to approximately 15.25 m of one pillar on the 8010 production level. In comparison, the mine’s seismic array spans across many levels within the
mine. For this reason, an extensive analysis was not conducted between the mine wide spatial distribution of seismicity and measured displacements. However, it is worthwhile to mention that the displacement-time trends per mining activity are very similar in profile to regular temporal seismic response profiles (Vallejos and McKinnon 2011; Woodward et al. 2017), which often monitor seismic events lower than Mn -2. In a recent seismicity study at Creighton Mine, seismic decay time was found to exceeded 8 hours in only 10% of cases (Vallejos and McKinnon 2011). Referring to Figure 7-11, a similar trend for the time-duration of displacements was determined. In this regard, the measured displacements of the pillar are predominantly associated Mn < 0 events, which often comprise over 99% of mine seismicity and are well correlated in time and space with mining activities (Hudyma et al. 2017).

7.6 Summary

An array of six multi-point rod extensometers, six FOSs, and three BHPCs were installed across a 15.24 m lengthwise segment of a hard rock sill drift pillar at a depth of 2.44 km in Creighton Mine. Throughout an approximately ten-month long monitoring duration, from May 2016 to March 2017, the pillar’s displacement, strain, and pressure response were measured, specifically focusing on the induced rock mass response to 8010 level mining activities, including: 1) mine-by of the pillar, 2) footwall development along a segment of the instrumented pillar, and 3) stoping operations aligned with, and adjacent to, the pillar’s axis. Due to the high in situ stress ratio, large displacements in the form of bulking at the pillar sidewalls were anticipated and the relatively massive granitic-gabbro host rock was expected to deform in a brittle failure process. The variety of sensing technologies installed across the span of the pillar allowed for this anticipated behaviour to be measured, allowing dilation at the microstrain level to be visualized and differentiated from large rock mass dilation/bulking at the centimeter scale.

The extensometers provided excellent temporal resolution for rock mass displacement measurements throughout the monitoring campaign (one-hour logging interval over 10 months). Using these measurements, a process was developed to identify the time-duration of displacements induced from mining activities (analogous to seismic decay time) by analyzing velocity time-series profiles. This process
indicated that for 83.7\% of the cases where displacement was induced at an extensometer anchor position, the magnitude was less than one millimeter (and 62.1\% instances were less than 0.1 millimeter). The process also indicated a time-dependent nature to the rock mass displacements. Over half of the induced displacements were found to prolong for more than one hour. In general, larger magnitude displacements and longer displacement time-durations were measured at the pillar’s sidewalls than at the core. An exception was measured in the vicinity of the PSZ, a several meter-thick fractured zone of steeply dipping schistosity, that extended across a segment of the pillar. Through the PSZ, the largest displacements were measured (over 80 mm cumulative), the longest time durations were determined (57 hours), and displacement was measured through the core of the pillar. A linearly contoured strain plot between all measurement positions of the six extensometers provided a very useful depiction of the PSZ, defining its extent as per the geological section.

In comparison with the extensometer measurements, the FOSs provided superior spatial resolution, detailing substantially more insight into the displacement response across the pillar. Within the granitic-gabbro footwall rocks, highly localized, brittle behaviour was captured. Contrarily, within the PSZ, shear dominated behaviour was observed. This indicated that the highly time-dependent nature of the PSZ is akin to a shear strain or distortion process while the footwall rocks is associated with a fracture driven process. The strong agreement between the displacement magnitude measured with the extensometers and the FOSs (especially at the submillimeter scale) highlights the vast potential for further use and development of the FOS technique to measure localized brittle rock mass behaviour and define regions of contrasting damage zones (e.g., the footwall rocks in comparison to the PSZ). There is also significant benefit to be gained from continued use of DOS with ground support members, such as the cable bolts instrumented in this experiment, as the entire load distribution across the support member is measured. This provides an improved understanding of the support member’s load-displacement characteristics and could be used to augment support system assessment procedures, such as the SM assessment demonstrated for grouted rebar, friction sets, and D-Bolts.
The use of support members with combined high load and static elongation capacities (e.g., the D-Bolt) greatly contributed to excavation stability and mitigation of rockbursts on the mining level. The continual measurement of displacements at the sidewalls of the pillar demonstrated the capability of the support system to allow the rock mass to bulk and dissipate stress through controlled excavation displacements, while maintaining excavation integrity. Although the pillar sidewall of the 6230 sill was not subjected to preconditioning, it is apparent that the support system did accommodate stress relief and baggage zone development. However, the seismicity and large displacements measured after daylighting the PSZ indicates that an agile excavation approach is required when crossing transition zones of contrasting material stiffness in highly stressed rock, even when using a rockburst compatible support system.
7.7 References


269


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270


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272


Chapter 8 Conclusion and Outlook

8.1 Summary of Research
Distributed optical fiber strain sensing (DOS) has been demonstrated to significantly advance the assessment of tendon reinforcement elements used in underground excavations. This thesis has presented the development and application of a sensing technique using Rayleigh optical frequency domain reflectometry (ROFDR) to measure the continuous strain distribution along various reinforcement elements at spatial increments as low as 0.65 mm. By arranging the fiber optic sensor (FOS) in a delta configuration, the continuous strain distribution can be further distinguished into coaxial strain and bending moment induced strain components (and the related orientation of bending). A compelling benefit of this procedure is that the maximum strain experienced at any location along the given reinforcement element can be derived. This is highly advantageous for application in-situ because there is often a degree of uncertainty associated with the locations(s) and direction(s) of ground mass displacements.

The sensing technique was initially demonstrated through a series of laboratory and controlled in-situ studies that considered variations of a coaxial pull test arrangement (as well as a limited number of double shear plane experiments). These experiments established that DOS can be used to substantially improve the ability to measure the load transfer characteristics of continuously mechanical coupled (CMC) reinforcement elements. In particular, DOS was used to quantify the coaxial load experienced by a given element as a function of distance from the load inducing source. This information can be directly used to validate analytical treatment of bond development and bond capacity at the interface between the support element and the confining material. Previously, such studies have relied on inferring this behaviour through external measurement procedures. This thesis has identified that this is not a suitable assumption for reinforcement elements that resist load in a non-uniform manner across their anchoring length, such as fully grouted rebars.

The process of transitioning the optical fiber-based sensing technique towards in-situ application involved designing a procedure to instrument reinforcement elements in a manner that protected the FOS
while simultaneously permitting conformance with the reinforcement standard operating procedures at the given project. This included consideration of site-specific handling and installation practices of the reinforcement elements (such as mechanized, self-drilling procedures) and accessibility for monitoring. Accordingly, a robust sensor arrangement was centered around maintaining the complete FOS assembly below (or within) the external profile of each instrumented reinforcement element. In this sense, the focus was placed on ensuring that the FOS was never directly exposed to the harsh conditions of an underground construction project. Furthermore, it was integral that the instrumentation arrangement, in particular the FOS connector (positioned at the head of the reinforcement element), did not necessitate special handling considerations or alterations to the typical reinforcement installation procedure (e.g., to be suitable for installation with a bolting jumbo). By accomplishing this, instrumented reinforcement elements were able to be installed as part of the primary support round and, thus, be used to measure how load was mobilized along the given element during the immediate excavation rounds following installation (which is arguably when the highest demand is placed on the temporary reinforcement, or when temporary reinforcement is solely relied upon).

This thesis has established that the discussed FOS technique can be installed in-situ and can provide unprecedented insight into the response of a reinforced excavation. From a quality assurance perspective, it is a critical advancement to be able to locate and quantify the maximum magnitude of strain along a reinforcement element as this can be used to provide increased confidence that the given element is functioning within its designed capacity and/or longevity threshold. However, the DOS measurement of the full strain distribution was most informative in being able to identify mechanism(s) by which reinforcement facilitated stress redistribution around the excavation. As examples of the mechanisms measured in this thesis, these included: 1) Measuring the mobilization of bending moment along spile elements that were installed in an umbrella arch arrangement and 2) Measuring the dowel reinforcement effect (a combination of coaxial load and localized bending moment) provided by CT-Bolts that transected differentially shearing discontinuities. It is in this sense that DOS measurements provided an unparalleled
approach to confirming that the installed reinforcement was mechanistically behaving in a manner that was consistent with the analytical and/or numerical treatment of reinforcement during support design.

While this thesis has presented several examples where DOS was employed in multi-month long monitoring programs, DOS was never able to be used as a continuously logged monitoring solution. This is primarily attributed to the power requirements, limited sensing length, and limited number of sensors that can simultaneously be measured when using the selected ROFDR technology. However, this did not inherently reduce the merit of using DOS in-situ. When incorporated into a multi-sensor monitoring program (such as the Creighton mine study – Chapter 7), the fundamental value of DOS was in its use as an inspection tool. Conventional sensing technologies (such as borehole extensometers) can readily provide high temporal resolution measurements for correlation with excavation activities and other underground occurrences. The advantage of DOS was in using its high spatial resolution to define the mechanisms controlling the overall response of the excavation. In the example of the hard rock rib pillar at Creighton mine, this included distinguishing a shear dominated residual displacement response for a heavily damaged fault zone from a fracture dominated mode of yield for a relatively massive granite formation. In comparison, extensometer measured displacements could identify a difference in the magnitude of displacements associated with the two material units during mining but could not provide insight into the rock mass failure mechanisms responsible for the measured variation.

8.2 Major Contributions
This thesis has specifically focused on the use of DOS for monitoring and assessing the performance of reinforcement elements in-situ. However, it must also be acknowledged that during the timespan of this research effort there have also been many other research application examples that have presented the use of DOS within the construction industry, in general. As a reflection of the author’s personal experience with using DOS, the major challenge in transitioning this technology from research centric exercises to industry practice will not be in further demonstrating the potential benefits of incorporating DOS measurements into a construction project. Instead, the most significant challenge will be in demonstrating
that DOS can be installed and monitored in an efficient and simple manner that can conform with standard operating procedures and not impede the construction process. Accordingly, the truly novel advancement that has been contributed by this thesis is the development of an FOS arrangement, which allows unparalleled assessment of reinforcement elements in-situ and, crucially, does not interfere with the overall functionality of a reinforcement element (i.e., handling, installation, and mechanistic response). The value of the FOS technique within a research framework of reinforced excavations is substantial. The major contributions of this thesis follow.

1) Developed a fiber optic sensor configuration and accompanying analysis to determine the maximum strain distribution along tendon reinforcement elements: By situating a single FOS in a delta configuration (i.e., 120 degree circumferential spacing around the element) a comparative analysis allows strain measured along the given element to be distinguished into coaxial and bending moment induced strain components and also determines the orientation of bending moment around the circumference of the element. This further permits the maximum strain to be derived at any position along the element. Accordingly, both the orientation(s) and location(s) of ground mass movements (inducing strain) do not need to be known beforehand in order to be accurately measured.

2) Developed a sensor construction and protection design to allow FOS instrumented reinforcement elements to be installed in conformance with standard operating procedures in-situ (i.e., handling and installation): The sensor arrangement maintains the complete FOS construction (consisting of a 155-242 µm diameter optical fiber, strain relief connector segment, and high return loss end termination piece) below the external profile of each reinforcement element. For bar elements, this involved machining lengthwise square grooves to house the sensor. For cable strands, this involved replacing the central wire with an instrumented tube. Crucially, the FOS started and ended within
the reinforcement element. Thus, damage to and/or replacement of exposed lead wires or the DOS unit would not negatively impact the FOS.

3) **Enhanced measurement techniques for laboratory and in-situ coaxial pull test assessment**: Coaxial pull test application of DOS permitted load to be measured as a function of length from the loading source. This allows derivation of the bond stress distribution and the deformation distribution. Accordingly, pull test specimens are not limited to relatively short anchor lengths because non-uniform load development and failure can be measured. Similarly, DOS can be used to augment quality assessment and/or acceptance testing of reinforcement elements during in-situ pull tests by slightly modifying the pull test assembly.

4) **Demonstrated an unprecedented assessment of reinforcement response mechanisms in-situ**: In-situ measurement of the complete strain distribution allowed mobilized reinforcement and stress redistribution mechanisms to be identified throughout the underground construction process. Additionally, the magnitude and position(s) of maximum strain mobilized along a given element was readily identified. This is incredibly valuable for confirming that the empirical/analytical/numerical technique(s) used to design the reinforcement system have evaluated an appropriate response in addition to verifying that a reinforcement element is operating within design thresholds of the project. Accordingly, DOS can be used to quantifiably justify alterations or optimizations to support design.

5) **Demonstrated the potential to use distributed optical fiber sensing to measure ground mass displacements and damage surrounding an excavation**: The high spatial resolution capability of DOS was perhaps most unparalleled when inspecting discontinuous ground mass behaviour. The location(s), relative direction(s), and displacement magnitude(s) of pre-existing and stress-driven
discontinuities could readily be inferred. This included distinguishing dilational and bulking ground mass displacements from differential shear across discontinuities. Within the context of a hard rock reinforced excavation, submillimeter magnitude displacements at the excavation boundary were measured to be the result of numerous contractional and dilational features within the rock mass. Measuring such in-situ behaviour of ground mass movement and damage is unprecedented within the geomechanics community.

8.3 Summary of Reinforcement Response Mechanisms

In the development and application of the FOS technique, general reinforcement response characteristics and mechanisms were measured. This thesis does not include a rigorous parametric study or definitively investigate a particular aspect that influences reinforcement behaviour. Nevertheless, there are many identified behaviours that are believed to be of current significance to the progression and validation of analytical and numerical treatment of reinforcement within excavation design (albeit limited to the reinforcement elements types, specific testing apparatuses, induced loading ranges, ground mass conditions, and excavation procedures discussed in this thesis) A summary follows.

1) *Coaxial load mechanisms of fully grouted rebars:* Rebars were determined to resist coaxial load primarily through a mechanical interlock mechanism (e.g., Benmokrane et al., 1995). This corresponded with an exponential decay of load and bond stress at the rebar to grout interface from the position of loaded to the unloaded end. A critical embedment length required to fail the rebar shank was measured to be between 0.40-0.45 m. In an unexpected finding, load development length was found to be predominately independent of the magnitude of applied load after 5% of the elastic load capacity was reached (i.e. full load development was achieved at very low coaxial loads and did not further develop with increased load). The latter statement is limited to the elastic loads studied in this thesis, but the results are able to confirm that adhesion has little influence on bond strength.
2) Coaxial load mechanisms of fully grouted plain strand cables: Plain strand cables were determined to resist coaxial load primarily through a frictional-dilation mechanism (e.g., Fuller and Cox, 1975). The low torsional rigidity of a cable strand resulted in the bond failure mechanism transitioning from dilational-slip and/or shearing of grout flutes near the position of applied to non-dilational unscrewing towards the unloaded end (transitioning around 0.25 m from the position of load). The bond stress distribution of an unconstrained cable was relatively uniform. An embedment length of 2.5 m was estimated as the length required to fail the cable element. Load development is strongly dependent on the magnitude of applied load.

3) Comparison of coaxial load development: In order, the shortest to longest load development lengths for the reinforcement elements considered in this thesis follow: 1) Rebar (0.40-0.45 m), 2) CT-Bolts (1.25 m with an end expansion anchor), 3) D-Bolts (> 1.0 m, heavily influenced by position of first anchor), and 4) Plain strain cable bolts (2.5 m). It should also be noted that while coaxial load was measured to distribute over multiple anchor positions along the D-Bolt, the smooth bar section between the position of applied load and the adjacent anchor will fail prior to the following smooth bar segment experiencing yield (i.e., the first anchor acts as an obstacle to stress progression along the bolt).

4) Shear and dowel reinforcement effect: Elements transecting differentially shearing discontinuities (both during laboratory experiments and in-situ studies) were measured to resist shear in a manner that mobilized coaxial load and bending moment along the element (akin to plastic hinge or dowel behaviour – Spang and Egger, 1990). Maximum bending moment was measured within 0.07 m of the center of the discontinuity plane and bending moment was negligible beyond 0.20 m. Coaxial load development was not readily quantified but was similar to that determined through pull testing.
5) *Umbrella arch behaviour*: Spile elements installed in an umbrella arch arrangement within the crown of a tunnel were measured to longitudinally redistribute stress away from the unsupported span to both the previous support round and ground at and ahead of the tunnel face. Strain was primarily mobilized during the first two meters of excavation advance and bending moment was confirmed to be the most critical design criteria. The spiles were not found to significantly resist ground movement parallel with the tunnel drive (i.e., very little coaxial strain was induced during excavation advance).

6) *Rockburst support*: While in-situ rockburst reinforcement elements were not directly measured as part of this thesis, borehole extensometers and FOSs were used to measure the response of a reinforced rock mass (that included rockburst reinforcement elements). Rock mass displacement measurements indicated that reinforcement elements with high static elongation availability and static load capacity were essential for allowing an overstressed rock mass to bulk and dissipate stress gradually through controlled excavation displacements. This is discussed in opposition to stiff reinforcement, which would aim to resist excavation deformation and, therefore, concentrate stress.

### 8.4 Limitations of Current Research

#### 8.4.1 DOS Sensing Limitations

The high spatial resolution capability of the ROFDR interrogator used throughout this thesis undoubtedly enhances the assessment of reinforcement elements. However, there are measurement trade-offs related to obtaining a submillimeter spatial resolution. Specifically, at the time of conducting the studies in this thesis only one FOS could be continuously monitored by the DOS interrogator, the maximum sensing length was limited to 20 m, and the maximum lead length to connect a sensor with the interrogator was 50 m. Within the context of a mine or tunnelling project, 50 m is a very limited lead length to setup a permanent logging
location for multiple elements that may be positioned throughout the development. Yet, even when conducting local, periodic measurements, the DOS interrogator requires clean power to operate. A simple demand, but not always available in underground construction projects.

The Rayleigh backscatter signal measured by the ROFDR interrogator is a relatively weak signal. Accordingly, damage to any position along the FOS will often result in a spurious reflection that will overpower the entire measurement signal (essentially negating the entire FOS). Furthermore, the presence of dust and humidity can greatly reduce the measurement signal. This necessitated a strict, and sometimes tedious, connection procedure in order to establish strong measurements (inherently requiring trained operators). Additionally, the ROFDR does not differentiate between strain and temperature. Therefore, if temperature is anticipated to be highly variable (temporally or spatially), compensation techniques are required (adding to post-processing procedures).

Within the context of measuring reinforcement, the strain sensing range of DOS (between 1.0-2.0%) can be limiting for mining applications. Several reinforcement element types, such as the D-Bolt (Li, 2010), have been designed to deform well over 1.0% throughout their support life. In addition, discontinuous ground mass displacements, particularly in the form of differential shearing, can result in highly localized reinforcement strains that exceed the strain sensing range within proximity of the discontinuity plane. In such instances, a measurement drop (or no measurement) may be recorded locally (over several centimeters), while the majority of the element is well within the sensing range. This can create a degree of ambiguity as to whether a particular segment of the element is under substantial localized strain or if the measurement drop is the result of a weakened measurement signal (e.g., from a dirty connection).

An additional limitation of the ROFDR interrogator that must be acknowledged is the upfront cost of purchasing the measurement unit (> $100,000 USD). However, if DOS measurements can be used to justify optimizations to the ground support system or identify/prevent a potential underground instability then this cost can readily be rationalized within the scope of a project budget. Furthermore, the FOSs are
relatively inexpensive to construct (the optical fiber costing roughly $0.10/m) and there is virtually no limit to how many sensors can be interrogated by a single DOS unit (although not simultaneously).

8.4.2 Limitations of the FOS Technique
The sensor arrangement presented in this thesis requires physical modification to the given reinforcement element in order to house the FOS. In the case of bar reinforcement elements, this has been accomplished by machining grooves along the element. While this inherently adds a cost to the instrumenting process, the most limiting aspect of this modification procedure is that it cannot be practically conducted at the construction site (in most circumstances). Logistically, this may require reinforcement elements to be return shipped from the construction site to the location of machining and instrumenting for compliance reasons. This, in turn, can add a significant expense to the instrumenting procedure, but can also result in long lead times (which is particularly problematic when monitoring demand arises in the form of unanticipated construction issues or safety concerns). However, it is possible that the cable sensor design, considering an instrumented central wire, could be finalized at the construction site.

A crucial consideration if an instrumented reinforcement element is to be used as part of the primary support round is the impact of the element modifications on the coaxial load and bending moment capacity of the element. This may necessitate a larger element and/or higher grade of steel to be considered in order to ensure that design loads can be sustained.

8.5 Recommendations for Future Work
It has been established in this thesis that high spatial resolution DOS is a very powerful technology for characterizing the mechanistic behaviour and structural response of elements integral to the construction of underground infrastructure. It is very difficult to conceptualize a displacement or strain based monitoring program that would suffer from an increased number of measurement locations. Accordingly, there are a vast number of potential applications for DOS and, more specifically, ROFDR, beyond monitoring reinforced excavations. The following recommendations for future work have been limited in scope to the behaviour of reinforcement and reinforced excavations as well as improvements to the FOS technique.
1) *Laboratory and controlled studies of reinforcement elements:* It is recommended that the FOS technique be further applied in a series of coaxial pull test parametric studies for various reinforcement element types (e.g., borehole size, confinement pressure/stiffness, surface irregularity, and load development under sustained loading). This will undoubtedly provide substantial data for the calibration of analytical and numerical reinforcement models and also provide practical insight for practitioners. Modifications to the FOS arrangement should be investigated to avoid measuring localized strain outside of the grouted length (which yields first) and primarily focus on measuring only strain along the grouted length. This should increase the loading extent that FOS can be used for.

2) *Full support system monitoring experiment:* The scope of monitoring reinforcement elements in this thesis was limited to a single type of reinforcement element at a given project. It would be tremendously valuable to instrument a complete ground support system (e.g., multiple reinforcement elements such as rebar and cables as well as the support members retaining load at the excavation periphery, such as shotcrete) in order to ascertain how these various elements mobilize load individually but also in collaboration to support the excavation.

3) *Incorporation of DOS within ground support optimization frameworks:* The focus of ground support design and optimization is more frequently being associated with empirically driven probabilistic approaches to limit equilibrium and numerical methods (e.g., Potvin et al., 2019). Generally, as a mine or tunnelling project advances more reliable data regarding support performance becomes available and opportunities to optimize support design arise. The FOS technique is appropriate as an inspection tool for such optimization frameworks and can be used to
validate of falsify critical assumptions of the support design (i.e., ensure the design adequately accounts for the reinforcement response mechanism).

4) *In-situ study on excavation damage zone:* While this thesis was predominately associated with reinforcement elements, use of DOS in a hard rock mass demonstrated that submillimeter magnitude displacements could be measured within the boundary of the excavation and that the displacements could be attributed to numerous dilational and contractional features. There is significant potential in further exploring the application of the FOS technique for measuring the extent of excavation damage and the transition depths of excavation damage zones (e.g., Perras et al., 2010) surrounding an excavation, especially within the context of deep geological repositories.

5) *Compact FOS development:* The FOS cable bolt design has significant potential for further use as a sensor that can be assembled in-situ. As demonstrated in this thesis, the central strand of a cable is replaced with an instrumented stainless-steel tube that houses the FOS. There is potential to further use this technique with hollow core self drilling elements (such as the spile elements in this thesis) by inserting a similar compact FOS into the core of the element post drilling, but prior to grouting. This would require significantly less instrumenting effort (i.e., no machining of grooves) and would also allow a stock order of FOSs to be provided to a construction site. These could then be installed and used as demanded by excavation conditions. Furthermore, investigating the use of multi-core optical fibers (e.g., Moore and Rogge, 2012) in order to extend the principal strain analysis described in Chapter 3 to cable bolts is recommended.

6) *Automating measurement processing:* Post-processing strain measurements is currently a time-intensive process that can involve measurement noise filtering and interpolating between measurement drop locations in order to conduct principal strain analyses. Automation of this
process for real time display of maximum strain and bending moment orientation would greatly assist user operation of the FOS technique in-situ. Furthermore, the analysis should be progressed to distinguish torsion from the three sensing arms.

Beyond these recommendations, it is also important to recognize that the DOS and other related optical fiber-based sensing technologies are developing at rapid pace in order to meet the demands of various industries, including underground construction. The ROFDR interrogator, which during the time of conducting this thesis was limited to measuring one FOS (without an additional beam splitting unit) can now accept eight sensors and accommodate 200 m lead wire lengths. In the near future, the price and physical size of optical fiber sensing technologies will continue to decrease while the measurement capabilities will increase. Furthermore, as these systems progress, the FOS technique developed in this thesis will remain compatible and is currently applicable to other optical fiber based strain sensing technologies, such a fiber Bragg gratings and other DOS technologies. Currently, ROFDR provides the highest spatial resolution of commercially available technologies.

8.6 Published Contributions

The scientific contributions that have been produced as part of this research effort are represented in Chapters 2-7. Full citations of the contributions follow:

8.6.1 Articles Published in Refereed Journals (3)


8.6.2 Articles Accepted by Refereed Journals (1)

1) **Forbes, B.**, Vlachopoulos, N., Diederichs, M.S., Aubertin, J. 2019 Augmenting the in-situ rock bolt pull test with distributed optical fiber strain sensing. *International Journal of Rock Mechanics and Mining Sciences*. IJRMMS_2019_911. (Accepted pending minor revisions)

8.6.3 Articles Submitted to Refereed Journals (1)


8.6.4 Articles in Preparation (1)

1) **Forbes, B.**, Vlachopoulos, N., Diederichs, M.S. 2019. Insight into the coaxial load distribution of fully grout rebar, cable bolt, CT-Bolt, and D-Bolt reinforcement elements. TBD.
8.7 References


Appendix A - Additional In-Situ Development and Application Material

A1 FOS Construction and Protection (Bar Elements)

The following appendix section provides additional information on the FOS connector identification segment and the high return loss end termination that were described in Chapter 6. Figure A-1 displays additional construction views of the FOS connector segment and Figure A-2 displays additional construction views of the end termination piece.

Figure A-1: Assembly of the FOS identification segment.
Figure A-2: Assembly of the high return loss end termination piece. Note: The presented end termination is longer than the one described in Chapter 6.
A2 Initial Handling and Acceptance Testing

This appendix section presents more information on selected handling and acceptance tests that were conducted during development of the in-situ FOS arrangement. Three primary tests are covered:

1) Controlled installation – Installation of FOS instrumented reinforcement element by hand, pre-load applied by hand wrench, grouting with a nozzle system;
2) Acceptance of FOS assembly with mechanized installation equipment, and;
3) Dry run installation with a mechanized bolting jumbo – Installation of a FOS instrumented reinforcement element.

The assessment of the FOS reinforcement element for each of the selected handling tests was performed by visually inspecting the sensor and connecting it with to the DOS measurement unit. The latter confirmed that the sensor was still operational and was conducted after each step in the installation procedure (i.e. insertion into borehole – connect to analyzer – pre-tension – connect to analyzer – grout – connect to analyzer). A more detailed overview the various handling tests follows.

Controlled Installation

The controlled installation handling tests consisted of inserting two of the FOS instrumented CT-Bolts by hand into horizontally drilled boreholes. The bolts were then subjected to pre-tension with the use of a hand impact wrench (Figure A-3). One of the bolts was also grouted using a nozzle system with the hemispherical dome of the bolt (Figure A-4).

The FOS were assessed at each stage of the controlled installation. This considered connecting the optical sensor to the DOS interrogator prior to being inserted into the borehole, post borehole installation, post pre-load, and post grouting. In all cases the FOS was found to be successfully operating. The installed FOS were also subject to shotcrete post controlled installation; however, due to accessibility limitations, only a visual inspection was conducted. An example strain reading from the FOS identified as FOS_02 is
presented in Figure A-5. Both the strain captured along the entirety of the FOS (i.e. three sensing lengths) as well as the coaxial strain captured along the element are presented. As displayed in the right plot, the magnitude of strain induced by the hand impact wrench corresponds to approximately 0.70 tonnes.

Figure A-3: Controlled installation of the FOS instrumented reinforcement element. A) Insertion of a FOS by hand into a borehole. B) Pre-tensioning of a FOS element using a hand impact wrench. C) Identifications of the FOSs installed.

Figure A-4: FOS head assembly acceptance with a grouting boom (grout nozzle).
Acceptance of the FOS Construction

Acceptance testing consisted of inserting the FOS instrumented reinforcement element into the installation socket and the grouting boom of a mechanized bolting jumbo (Figure A-4 and Figure A-6(A)). Again, the DOS bolt was tested after each step of the dry run. This considered connecting the DOS bolt to the optical analyzer prior to being inserted into the dolly, post insertion into the dolly, and post insertion into the grouting boom. The head protection piece for the DOS bolt was found to have adequate room in both the dolly and grouting boom.

Dry Run Installation

Dry run installation consisted of subjecting the FOS instrumented reinforcement element to the standard reinforcement handling and installation procedure at the construction project. For the selected project, this included installation into the borehole, pre-load torqueing, and grouting with a Robodrill bolting jumbo (Figure A-6).
At the time of the FOS installs, the construction crew in the tunnel was halfway through a bolt support round. The construction crew were instructed to install the FOS reinforcement elements in the same manner as normal reinforcement elements (i.e., continue the current procedure they were following). FOS Figure A-6: Tunnel installation of the FOS technique. A) Insertion of a FOS instrumented element into the dolly of the bolting jumbo. B) Grouting of a FOS element using the grouting boom on the bolting jumbo. C) Post pre-tension and grouting view of FOS instrumented element.

Figure A-7: Identification of the FOSs installed in the tunnel with the Robodrill bolting jumbo.
were installed immediately following the support. In total five FOS were installed approximately 10 m behind the excavation face (Figure A-7).

The FOSs were connected to the DOS interrogator and assessed both post-tensioning and post-grouting. In all cases the FOS were found to be operational. Figure A-8 presents the total strain measured across the length of the reinforcement elements for FOS_03 and FOS_07 at various stages of the dry run installation tests.

![Figure A-8: Total strain measured along the reinforcement element during various stages of the installation handling tests. Left – Strain measured across FOS_03 before installation and post-tensioning. Right – Strain measured across FOS_07 before install, post-tensioning, and post grouting.](image)

The strain measured for FOS_03 after pre-tensioning the reinforcement element is particularly interesting as pre-load determined by the strain measured along the element is about half, if not less, of what the support design had specified (approximately 5 tonnes). However, during the pre-loading stage (i.e., torqueing) of FOS_03 it was noted by the jumbo operator that the reinforcement element may not have been torqued adequately and should retorqued (or a new bolt should be installed if part of the actual support round). However, it decided to leave this FOS instrumented element at the given level of tension in order to compare it with the other FOS elements bolt’s, such as FOS_07, which were believed to have been
installed with an adequate level of pre-load. As displayed for FOS_07, the design specified pre-load was confirmed for FOS_07, demonstrating DOS has potential value as a QA/QC tool.

An additional consideration during dry run installations was the durability of the protective cap used to cover the FOS connector. Several wall thicknesses and materials were tested including aluminum, steel, and stainless steel. While all variations were successful, certain protective caps were damaged (in turn, damaging the FOS connector) when manufactured from aluminum and steel (Figure A-9). Accordingly, stainless steel was selected as the material for the protective caps in all subsequent applications.

Figure A-9: Example damaged aluminum protection cap. A – FOS connector damaged at the head of the reinforcement element. B – View of damaged protection cap and FOS connector
A3 Example Periodic Monitoring Protocol Used In-Situ

The following appendix section provide example FOS measurement and monitoring protocols used as guidelines were operators at various in-situ projects.

*Note a comprehensive description of the recommended procedures for operating the ODiSI-B interrogator is provided in the users manual offered by LUNA Innovations ([https://lunainc.com/product/odisi/](https://lunainc.com/product/odisi/)). The following procedures offer a condensed summary.*

**Summary of Operating the DOS Measurement Unit (for reinforcement assessment)**

1) **Start up:**
   - Power the ODiSI-B unit
   - Once a green power and orange source light is lit on the ODiSI B unit, power the computer and monitor
   - Note: the ODiSI-B unit must be powered prior to powering the computer

2) **Connecting FOS to the measurement unit:**
   - Sensors are connected to the ODiSI-B unit through the 50m stand-off cable.
   - Before making a connection, it is crucial that both the optical lead connectors (male) and standoff connector (female), and FOS connector (female) are cleaned diligently.
   - Cleaning the FOS connector and standoff connector: remove the dust cap, clean socket with bulkhead cleaner (two clicks) (Figure A-10).
   - Cleaning the optical lead connector: remove the dust cap, clean the socket using the reel-based cleaning pad (swipe same direction) (Figure A-11).
   - Connection should be made promptly after connector cleaning.
   - It is suggested to clean the female connection first, the male connection second.
Free cable (i.e. unspooled from the standoff and lead cable) should be kept as straight as possible to establish the highest signal quality.

Figure A-10: Cleaning process for the FOS connector (i.e., female, LC connector) at the head of a FOS instrumented reinforcement element. Follow the same procedure for the connector at the end of the 50m stand-off cable for the DOS measurement unit.

Figure A-11: Cleaning pad used to clean the optical lead cable connector (male).
3) **Identifying a connected FOS:**

- Double click ODiSI-B v5.1.0 to open.
- ODiSI-B will automatically identify a connected sensor (inability to identify the sensor may indicate a damaged or broken sensor).
- Moving the interrogation unit may misalign the laser. In such a case a warning message will appear prompting realignment of the system. Accept to realign. Click okay when notified that alignment has been achieved.
- Upon successful identification of the connected sensor ODiSI-B should look similar to Figure A-12.
- To observe the strain profile along the attached sensor click Start Operation (as in Figure A-12).
- ODiSI-B provides a real time visualization of the microstrain (y-axis) along the sensor length (x-axis). The scale of the plot, line type, and many other configurations can be adjusted while the system is running – this will not affect the measurement.
- Save tare will zero the current strain profile to use as a base-line for future readings (a significant tare name should be given – it is not recommended to tare any readings).
- To change the connected sensor click Stop Operation and then disconnect the current sensor (remember to place dust caps back on the sensor and standoff cable to maintain the system).
- Follow the specified sensor connection procedures whenever connecting a new sensor.
- Click Identify Sensor once the new sensor has been connected to reinitiate the identification process.
4) **Observing strain along a connected FOS:**

- To observe the strain profile along the attached sensor click Start Operation.

- ODiSI-B provides a real time visualization of microstrain (y-axis) along the sensor length (x-axis). The scale of the plot, line type, and many other configurations can be adjusted while the system is running.

- To enable sample logging click Stop Operation. At this stage enable sample logging settings can be selected.

5) **Sampling strain data:**

- When opening ODiSI-B, Enable Processed Data Logging will not be selected, and only raw data is recorded to the raw buffer.
- Navigate to Data Logging > Processed Data Logging Settings to configure save data location (Figure A-13).

- At this stage there is an option to process data simultaneously with logging (recommended) using the Processed Data File tab or record only raw data (.odb file) for post-processing later.

- First enter an appropriate Basename for the processed data (e.g. NCX_05 – Figure A-13)

- A time stamp is automatically included at the end of the chosen sensor reading name and therefore, will not overwrite pre-existing readings.

- Select the appropriate storage location for the measurement.

- Maximum file size can be left to 0 (by default this measures indefinitely)

- Strain readings along the sensor can be triggered or sampled periodically according the logging rate selected (Figure A-13) – it is recommended to log at 1Hz and to take a minimum of 10 measurements (10 seconds).

- To begin sample logging, make sure Enable Processed Data Logging is selected, then click Start Operation. This will create a file under the name specified and save all readings to it (until Stop Operation is clicked).

- The current number of measurements is shown as the # Scans in Sample.

- Stop Operation stops the current measurement session and saves the file as an .odb file or .txt file depending on logging selection.

- The measurement file should be confirmed in the selected save location after the reading is taken.
Summary of Sensor “Keying” Procedures

1) Open ODiSI-B Sensor Configuration software.
2) Click Acquire to view the sensor trace (Figure A-14).
3) Enter (type in) X1 and X2 values provided for the connected FOS:
   - The X1 and X2 values correspond to distances (from the FOS connection) along the connected FOS that defined the length of the optical fiber which will be considered the active sensing length. The initial Rayleigh signature will be stored for future strain measurements.
   - These are determined during the sensor manufacturing stage to fall within the length of the given reinforcement element (which may have multiple sensing lengths)

Figure A-13: Overview of strain measurement procedure. Navigate to Data Logging > Processed Data Logging Settings. Set a Basename for the Processed Data File, Maximum Size (if left at zero the file will write indefinitely), and Logging Rate (set to 1 Hz). Select apply changes. Select Enable Processed Data Logging. Selecting start operation will then initiate logging of strain. For further information refer to the ODiSI-B user manual.
Note: lead wire lengths will also be visible in the sensor trace. Therefore, when keying a FOS, care should be taken to ensure the lead wire length is the same as that used during manufacturing (or the X1 and X2 are adjusted to reflect the change in lead wire length). Once the sensor has been keyed, the same lead wire or lead wire length does not need to be used to identify the sensor in the ODiSI-B measurement program.

4) Enter the appropriate sensor name click Save Sensor

Figure A-14: Example sensor trace in ODiSI-B Sensor Configuration. In this example X1 and X2 have been set to 3.937 and 16.507, respectively (this refers to meters away from DOS stand-off connection).

Suggested Installation and Measurements Procedure

1) Key the FOS prior to install:
   - After keying, thread on a stainless-steel protective cap for installation.
2) Following installation, clean any grout/shotcrete/water/dust from the sensor protection cap.

3) Remove the stainless-steel protective cap:
   - Check the FOS connector has not been damaged (Figure A-15).
   - If the connector is dirty, wipe clean and use compressed air (recommended).

4) Connect the lead cable to the FOS connector:
   - Thread cable through hollow protective cap.
   - Clean FOS connector (Figure A-10).
   - Clean lead wire connector (Figure A-11).
   - Establish connection (ensure clean connection: no dust into FOS connector, lead wire connector tip does not come into contact with the walls of the FOS connector).
   - Thread on hollow protective cap.
   - Connect the opposing end of the optical lead cable to the stand-off cable of the DOS measurement unit (following the same cleaning procedure as the FOS connector).
   - It is recommended to not disconnect the lead wire from the FOS connector (i.e., this connection should be established once and maintained). Accordingly, consider appropriate protection considerations for the lead wire length.

5) Strain measurements:
   - Record post installation measurements.
   - If the measurement signal is not stable, it may be necessary to re-key the FOS.

6) Protection between measurements:
   - Post measurement, disconnect the OF lead wire from the DOS measurement unit and protect the lead wire connector and loose length in an appropriate manner depending on the site conditions.

7) Subsequent measurements:
   - Follow procedures previously discussed.
Figure A-15: Example images of a functioning and damaged FOS connector post installation.
A4 Additional In-Situ Pull Test Components

The following appendix section provides additional information on the components necessary for conducting the in-situ coaxial pull-tests presented in Chapter 5. This includes additional details of the modified pull test unit (Figure A-16) and the digital instrumentation (Figure A-17) used in conjunction with DOS.