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MEASURING, UNDERSTANDING, AND IMPROVING THE PERFORMANCE OF FULLY GROUTED RESIN BOLTS IN UNDERGROUND COAL MINES

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by

Aditya Mishra

B.Tech., Indian School of Mines, 2013

A Thesis

Submitted in Partial Fulfillment of the Requirements for the

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THESIS APPROVAL

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AN ABSTRACT OF THE THESIS OF

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MAJOR PROFESSOR: Dr. Anthony J.S. Spearing

Despite progress in research related to ground control, roof falls remain a major problem leading to injuries and fatalities in underground coal mines. Around 68 million rock bolts are installed annually in coal mines in the USA (Tadolini and Mazzoni, 2006), of which 68% of primary supports in coal mines are fully grouted resin rebar bolts. A fully grouted rock bolt is one that is fixed inside the drill hole by means grout, and is fully encapsulated throughout the length of the bolt. Production in underground coal mines is achieved by the combined processes of excavation and ground support, carried out in a cycle. Primary support is that which is installed on cycle during production. Secondary support is that which is installed in addition to primary support, *i.e.*, out of the cycle, and typically in intersections where the effective span is much larger making extra support both appropriate and necessary. Although safe and effective, there is still considerable opportunity to improve the performance of fully grouted resin rebar. The objective of this thesis is to measure and study key installation parameters of fully grouted resin bolts to optimize their performance. This will be accomplished by conducting a series of lab tests, developing a new small-scale test method, performing *in situ* tests, and numerical modeling.

With a research collaboration between the Department of Mining and Mineral Resources Engineering at Southern Illinois University Carbondale (SIUC) and Orica Americas funded by the Illinois Coal Research Institute (ICCI), almost two hundred pull tests (ASTM F432) were conducted mainly at the Orica North American Innovation and Technology Center in Bowerston OH, but also at two underground coal mines in the Illinois Basin. Laboratory testing consisted of rock bolt pull tests in a basically homogenous and isotropic material (concrete block) and hardness tests by modifying a drill press by varying key installation parameters. *In situ* testing consisted of rock bolt pull tests to at two different mines (Mines A and B). These were analyzed and optimized to better understand the performance of grouted rock bolts. Pull tests were conducted at SIUC's Coal Research Center high-bay laboratory in Carterville, IL to investigate the effect of glove fingering when installing #5 and #6 rebar. Finally, a series of tests were performed to study the behavior of axial load shedding (creep) with time in grouted, tensioned rock bolts using U-cells (Mitri, 2012) and conventional load cells followed by a basic numerical modelling on the pull test.

It was found that spin time and rotational speed play significant role to obtain a more effective fully grouted rebar system. An eccentric action was added to the wrench used for installing the bolts had no significant impact on the pull out load at in situ conditions but showed increase in the pull out load, when the installation procedure was altered. Resin hardness can be used as parameter to predict the pull out load of fully grouted rock bolt system and a significant creep was observed in the fully grouted tensioned rock bolts.

ACKNOWLEDGMENTS

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

INTRODUCTION

The topic of this thesis applies to the field of ground control, which is a collective term used to regulate and prevent the collapse or failure of mine openings by applying aspects of rock mechanics. The 20th century has seen many advances in the field of ground control (Gardner, 1971), one of which was the introduction of rock bolts. However, ground control problems typified by roof falls remain a major concern in underground coal mines.

Figure 1 and Figure 2, from data compiled by the National Institute for Occupational Safety and Health (NIOSH), provide insight into the severity of rock fall injuries and fatalities in US coal mines. It can be seen that the number of injuries and fatalities are decreasing with time due to the concern and efforts of mine owners and management, regulations imposed by the US Mine Safety and Health Administration (MSHA), and advancements in the field of ground control. However, the ultimate goal for all ground control applications is zero injuries and zero fatalities. Most roof falls occur as a result of key block failure (Molinda, 2010). Thus, there is a need to understand the design, application, and performance of roof support systems to reduce roof falls and hazards associated with them.

In the 1950s, rock bolts became widely accepted as a reinforcing element in coal mine roofs (Mark, 2002). There was, however, a need to understand the behavior of rock bolts in that capacity. Thanks to significant developments in ground control engineering, between 1950 and 1970 there was a 35% increase in percentage of total bolts installed that were fully grouted. This happened because mechanical bolts were no longer considered an adequate support for stabilizing weak roof conditions (Mark, 2001). Each year, rock bolts with an installed cost of approximately $500 million (Mark, 2001) are installed in US coal mines covering about ten thousand miles of underground entries.

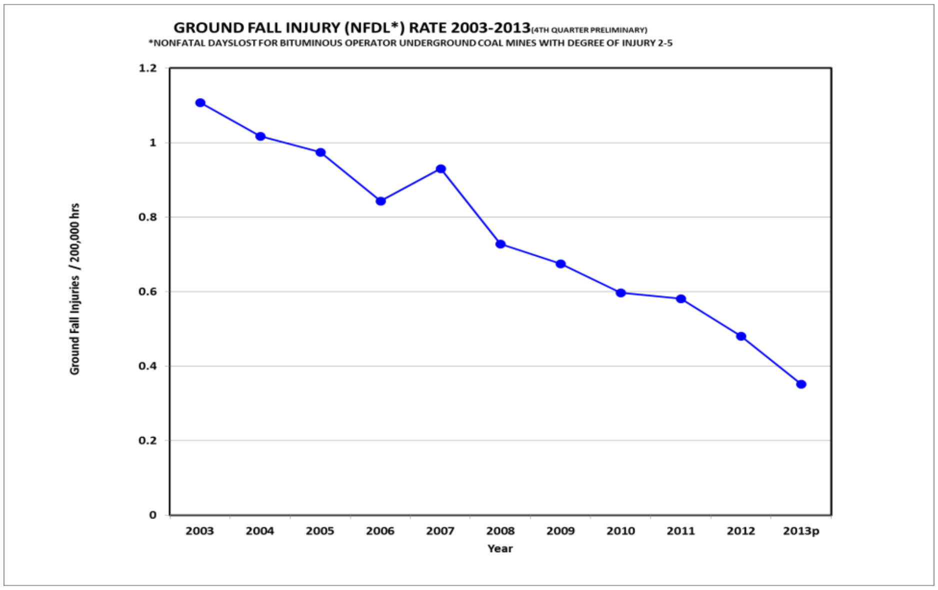


Figure 1: Rock fall injuries from 2003 to 2013 (NIOSH, 2013).

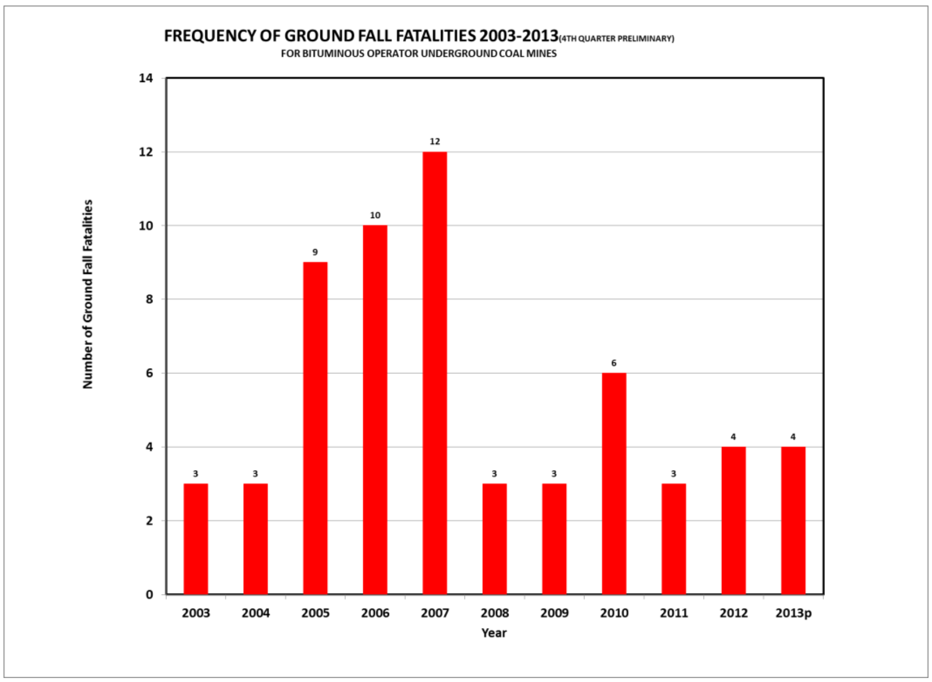


Figure 2: Rock fall fatalities from 2003 to 2013 (NIOSH, 2013).

Rock bolts are the first line of defense to protect workers from hazardous falls of ground by generating a stable immediate roof and rib structure. They are installed as soon as possible after excavation of a mine opening. Based on the time of installation, rock bolts in coal mines are categorized as primary and secondary supports. Primary supports are installed as part of the normal production cycle of excavation and support. Secondary supports are intentionally added to assist the primary support upon the requirement of additional roof support. Secondary support is usually installed out of the normal production cycle and typically in intersections where the effective span is much larger.

On the basis of installation load, rock bolts are categorized as active and passive bolts. Active bolts are tensioned upon installation, whereas passive bolts develop tension only in response to ground movement. The loading in both systems acts to reinforce the mine roof so as to significantly decrease the probability of rock falls. On application of tension to a rock bolt (active), compressive forces are generated upon roof rock layers earlier than in the case of passive bolts. The objective is to stabilize loose roof rock layers, which increases frictional forces and reduces tensile forces in the rock. On the other hand, passive bolts are installed with minimal tensile force producing localized bolt reactive loads after small movements of ground (vertical sag, shear along a bedding plane, or dilatation of a roof layer buckled by horizontal stress). Both active and passive rock bolts create a system of high stiffness resulting in high capacities and reducing excessive deflections caused by anchor slip (Mark, 2001).

Based upon geology and stress regimes, four kinds of rock bolt reinforcement mechanisms have been identified (Figure 3):

- Skin control support: In a strong, self-supporting massive roof, cracks and/or joints can create occasional loose rocks (key blocks). Thus, relatively short rock bolts are usually sufficient to prevent local falls of the immediate roof layer, but not sufficient to prevent major falls.

- Suspension support: In areas where a much stronger roof overlies a weaker immediate roof layer, rock bolts are anchored into the stronger overlying strata and act to suspend the weaker layer in a traditional dead weight loading design.
- Beam building support: When no self-supporting stronger bed is within easy reach for creating a suspension support, bolts tie multiple weaker immediate roof layers together creating a beam by generating high friction on bedding planes. To attain an effective beam mechanism, higher support densities (number of roof bolts installed per area) are required.
- Supplemental support: In extreme conditions where the roof is very weak and/or stress is very high, rock bolts may not be able to prevent roof failure from progressing beyond a reasonable anchorage horizon. In these areas, cable bolts, truss bolts, or standing supports may be necessary to support the dead-weight load of the affected roof strata.

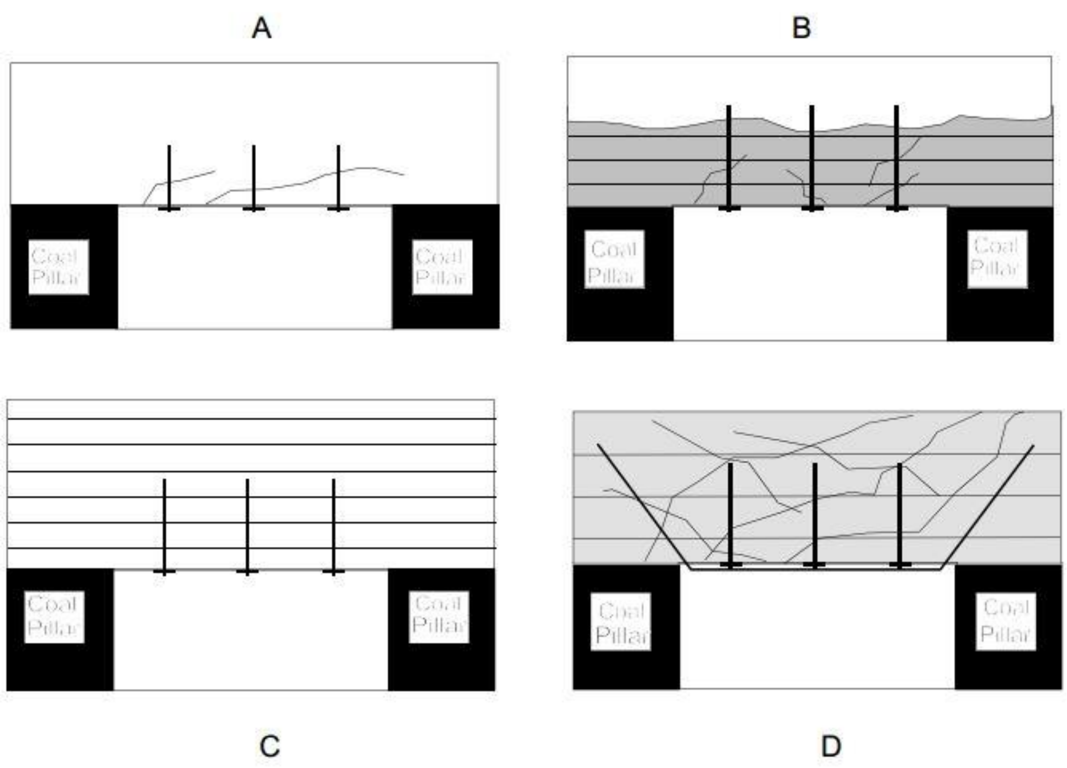


Figure-3: Rock bolt support mechanisms: (a) simple support, (b) suspension, (c) beam building, and (d) supplemental support in failing roof.

The work reported in this thesis is concentrated on parameters that affect the anchorage mechanism of fully grouted resin rebar. Resin was introduced to US coal mines in 1971 and commercial production started in 1973 (Minova, 2006). Early resin cartridges contained polyester resin, which was very expensive and had slow gel times making its application limited to very difficult ground conditions. By 1974, costs had been reduced by 30%, and as the value of the dollar increased, it was possible to buy six times more resin in 2006 than in 1974 (Blevins and Campoli, 2006).

Upon installation, rupture and shredding of the resin cartridge initiates a chemical reaction involving two separate components, which when mixed, begin to harden. The larger, darker component is the resin mastic and the smaller, less dense component is the catalyst. They are separated by a thin plastic film. The mastic is a mixture of limestone filler and additives that accelerate and stabilize the chemical reaction and suspend the solids. The catalyst is a mixture of benzoyl peroxide, limestone filler, water, glycol, and additives. Both components do not react with the rock or rebar, and offer protection to the steel from corrosion. Manufacturers provide specific diameters of cartridges, the volume of which is controlled by length. Rotation of the bolt during installation mixes the two components transforming the liquid-gel mixture into a solid providing an effective anchorage (mechanical key) between the rock anchor and rock around the hole. Gel time is the amount of time required for the mixture to start to solidify, after which any further mixing could damage the partially set resin causing a loss of strength.

The following are definitions related to resin cartridge mixing (Minova, 2006):

- Set time: The time at which the mixed system has reached eighty percent of its ultimate strength.
- Cure time: The time taken by the mixed system to reach its ultimate strength.
- Spin time: The length of the time for which the bolt must be spun to ensure an effective mix of the two components within the cartridge. This length of time must also ensure that the film around the cartridge is adequately destroyed at a certain speed range.
- Hold time: The approximate length of the time that the operator must wait after spinning before the active bolt can be tensioned or the bolter boom can be disengaged from the bolt head.

Rebar diameter is identified with a number that describes eighth inch fractions. For example, #6 rebar denotes a diameter of 6/8". Annulus is half of the difference between hole diameter and rebar diameter. For example, #6 rebar in a 1" diameter hole form an annulus of 1/8". The length of the hole drilled in the roof to install the bolt is 1" longer than the bolt length minus the length of the bolt head and the thickness of the bearing plate used. A simple procedure is followed to install the fully grouted resin rebar. First, the roof bolter operator inserts a resin cartridge (sometimes two when needed) of sufficient length to fill the annulus into the hole followed by the rebar. During insertion of the rebar, it is rotated for a fixed amount of time (*i.e.*, spin time) to shred the resin cartridge for efficient mixing of resin components. The rebar is then held against the roof for a certain period of time (*i.e.*, hold time). The resistance offered by the rock bolt to ground movement is achieved by load transfer from rock to resin grout at the grout-rock interface followed by load transfer from grout to bolt at the grout-bolt interface. These interfaces are called contact surfaces.

Assessment of resin anchorage systems is done by performing pull tests standards on installed rock bolts. The pull test objective is to measure working and ultimate capacities of a grouted rock bolt anchor. Ultimate capacity is the load on the anchor system at which the rock bolt system fails and working capacity is the load on the anchor system at which failure begins, marked by significantly increased displacement. A pull test will only show the failure load of a full-column bolt, unless installation is very poor. Thus, short encapsulation pull tests are performed as defined in ASTM F432. Pull tests performed with shorter encapsulation length (typically not less than 12”) is a qualitative approach to measure anchorage system effectiveness in a particular rock mass. During the pull test, load is applied on the bolt in regular increments of 1 ton. Deformation of the system is recorded at each load increment. As long as the system is in the elastic zone, the normal change in deformation can be measured with each increment. Failure in rock bolt pull testing is marked by inability of the system to sustain a stable or increasing load without a rapid increase in deformation. Data collected from these tests can be used to select a suitable anchor system for a particular immediate roof and determine the performance of various installation parameters.

This thesis also addresses glove fingering, which occurs when the plastic cartridge wrapper remains substantially intact around the hardened resin. The term glove fingering refers to the resin cartridge plastic partially, or completely, encasing a length of the bolt, typically with unmixed resin filler and catalyst remaining within the cartridge. The effect of glove fingering can significantly decrease (up to 30% (Campbell, 2003)) resin-grouted rock bolt performance as it reduces the mechanical key at the resin-rock interface causing a decrease in expected anchorage capacity of the rock bolt. Initial assessment of glove fingering assessment is reported; however, definitive conclusions have not yet been drawn.

LITERATURE REVIEW

During the late 1940s and early 1950s, rock bolting was introduced to the mining industry. It was a major development in the field of ground control (Gardner, 1971). In general, a rock bolt is a type of roof and rib support for internally stabilizing rock excavations by transferring load from unstable rock masses to confine and much stronger rock masses (suspension support). From a technical viewpoint, rock bolts replaced timbers and other standing supports and greatly reduced the number of roof falls accidents (Mark, 2001).

As rock bolts became more widely used as primary supports, proven methods needed to be developed to evaluate factors that affect bolt anchorage and to estimate the relative safety and efficiency of proposed rock bolt installations. One of the first efforts (Barry, 1954) related to rock bolt research in underground mines used a torque wrench to determine tension in rock bolts and introduced a new application of existing equipment, later to be known as a pull test apparatus (Figure 4). The new equipment with its test procedure was developed to study rock bolt anchorage capacity in both sandstone and shale roof, which could not be satisfactorily determined using methods that existed at the time. Results from pull tests and tests using a torque wrench were compared with each other to form an effective analysis showing anchorage capacities.

The analysis of test results showed that anchorage capacities up to the yield load of bolts could be developed in either (sandstone or shale) rock type. The author concluded that anchorage characteristics of the test bolt can be determined from pull test data by examining the slope of the curve produced by plotting load recorded from hydraulic jack pressure vs. relative bolt displacement. The author also concluded that pull tests conducted with a hydraulic jack were preferred over tests conducted with a torque wrench because they provided more data for making the comparison, *i.e.*, a full load vs. deformation curve. Although, an effective comparison was made between the two techniques, no further tests were done to study the effectiveness of parameters involved in rock bolt installation. From 1960 to 1970, research focused on mechanical shells, which is not relevant to this thesis.

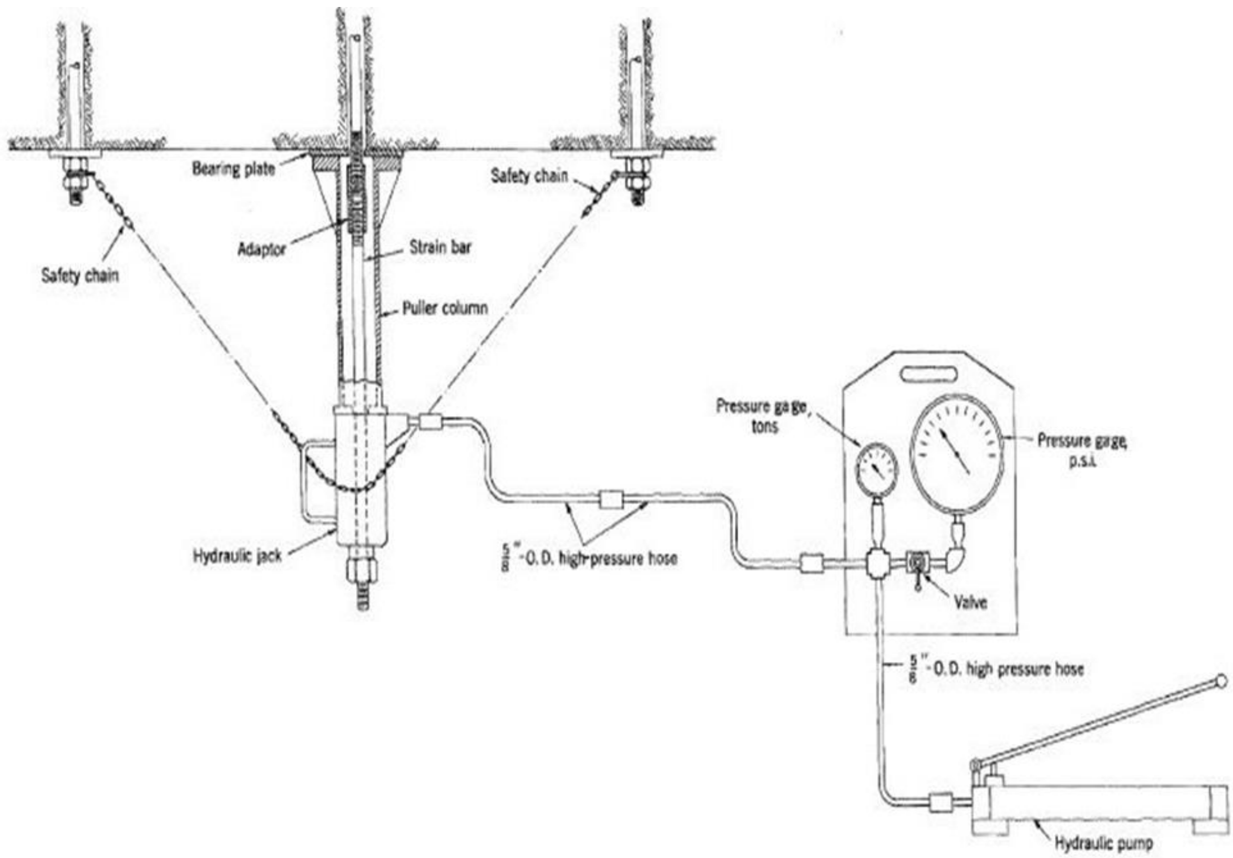


Figure 4: Pull test apparatus.

With the introduction of resin to provide anchorage in rock bolts, Caverson and Parker (1971), conducted tests to compare mechanical and resin anchorage. These tests were done with 1" glass resin capsules, #6 (0.75") full column resin bolts, #5 (0.625") mechanical anchors, and #5 (0.625") mechanical anchors with resin. Results indicated that for a 6' bolt length for both mechanical and resin bolts, the installation time for resin bolts was 1.5 times the installation time for mechanically anchored bolt whereas, at that time, the resin assembly cost three times that of mechanically anchored bolts.

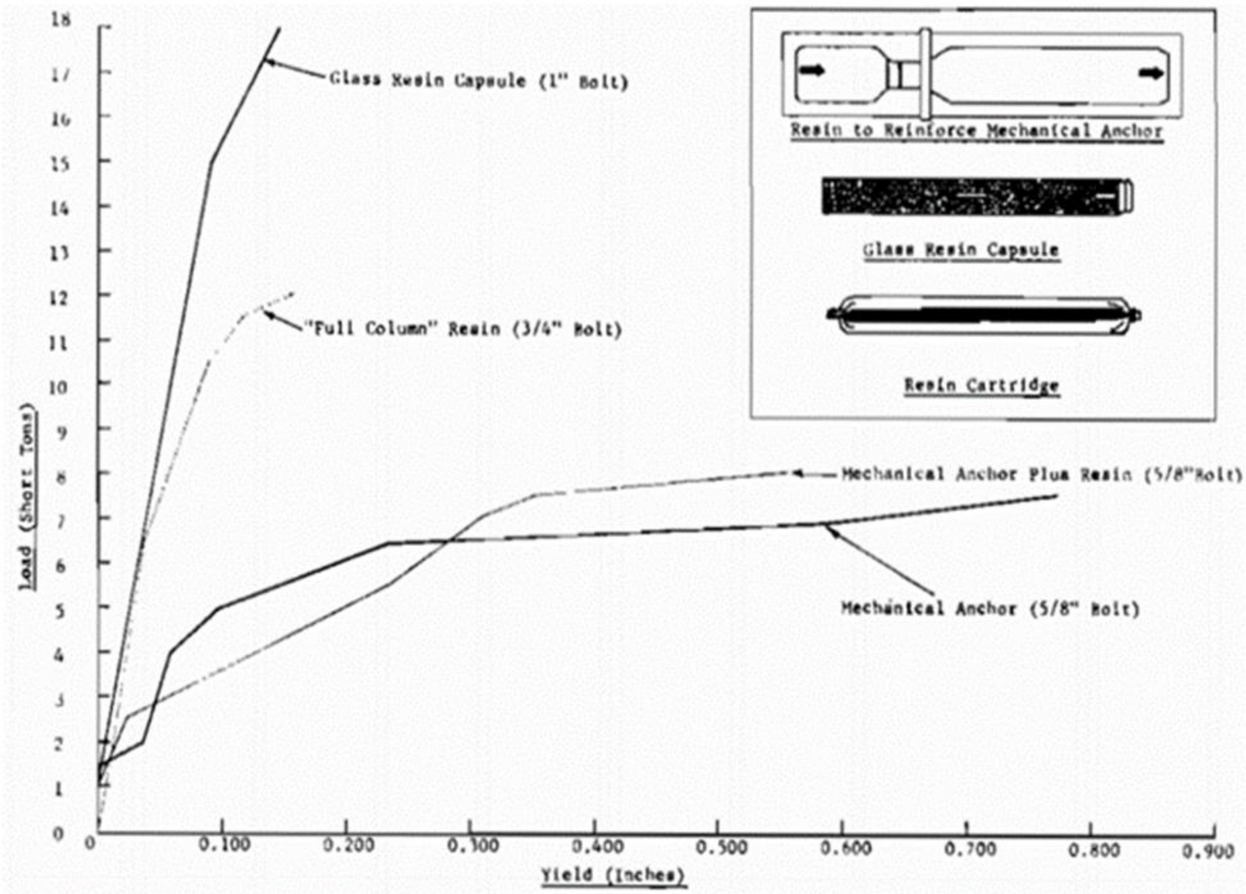


Figure 5: Comparison of resin anchorage to mechanical anchorage.

On the basis of pull tests (Caverson and Parker, 1971), the authors found no significant difference in results obtained for holes in shale or sandstone where in all cases bolts were pulled to destruction. They found that stress at the bearing plate is reduced by resin as opposed to mechanical shells. The tension on a mechanical bolt, which induces high compressive and destructive stresses (depending on plate thickness), at the rock-plate interface was not induced when resin was used. This is important when associated with weak immediate roof. On further analysis, tightening a mechanical bolt generated high lateral compressive forces at the anchor-rock interface, as the load is concentrated at anchors, whereas no such tendency was observed on resin anchorage due to distribution of the load throughout the entire length of the resin column. The presence of moisture and blast vibrations also affected the performance of mechanical anchors whereas resins exclude moisture (after hardening) and air from the bolt hole and absorb most of the blast vibrations. These tests conducted by the authors confirmed better performance of the resin anchorage system as opposed to the mechanical anchorage system (Figure 5). However, the quality of the resin anchorage after each installation was not tested.

A major contribution to the field of ground control using rock bolts was made by T.J. Freeman (1978). It was an investigation on the performance of fully bonded rock bolts using instrumented bolts in the Kielder Experimental Tunnel. The tunnel had eight sections: by drilling and blasting and four excavated by road header. Rock bolts with polyester resin cartridges were used in three of the sections. The instrumented bolt was a 0.98” diameter, hollow reinforcing bar with a read-out unit that measured longitudinal strain distribution along the bolt. Differential rock movements were measured by extensometers and the load on the bolt was measured by electrical resistance strain (ERS) gauge readings that convert strain into an electrical signal from which shear stresses were derived. Figure 6 illustrates the position of the four strain gauges placed along the length of the bolt at regular intervals.

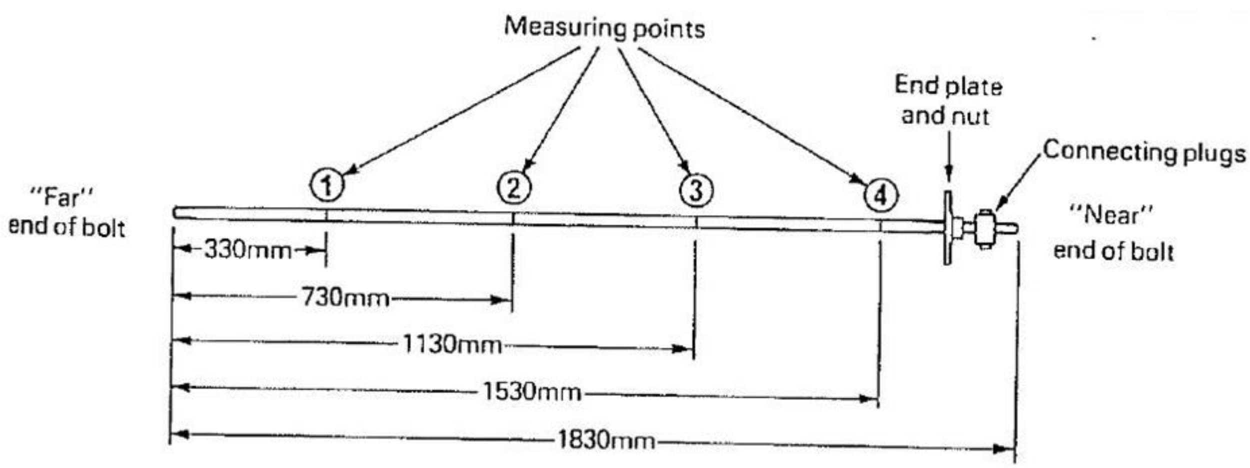


Figure 6: Details of the instrumented rock bolt.

Freeman concluded that the behavior of rock bolts was best described in terms of anchor length and a pick-up length (Figure 7). Anchor length of instrumented bolts varied from about 20 percent to 70 percent of bolt length. It shortens with continued rock movements and can be increased by providing tension, but at the expense of higher loading in the bolt, which may make failure of the bolt more likely. Upon loading, shear stresses induced on the bolt have opposite directions along the anchor length and the pick-up length. The point at which the shear stress on the bolt is zero is called the neutral point.

In 1984, load transfer mechanics for fully grouted rock bolts in coal mines were investigated by Serbousek (1984) at the US Bureau of Mines (BOM) Spokane (WA) Research Center. The objective was to understand and interpret results obtained from pull tests performed regularly in both the laboratory and in mines. Laboratory pull tests included variations in bolt length, hole diameter, and grout type. Bolts were tested in hole diameters of 1” and 1-3/8” with polyester resin. Instrumented bolts (Figure 8) of 0.75” diameter (#6) were designed to measure the distribution of forces along the length of the bolt as load was applied. In addition, strain gauges in opposing slots were used to track bending during installation. The author of this thesis suggests that may not be effective in the case when bending may occur in the third direction.

Limited pull tests were conducted on rock bolts inserted in cement blocks to investigate load transfer mechanics of full column bolts grouted with polyester and epoxy resin. It was observed that the curve gradient (stiffness) increased with an increase in applied forces indicating no adhesion between bolt and grout. Rather, it is the mechanical interlock that transfers force from the steel to the grout (Figure 9). It was concluded that the mechanical interlock between components is the most important parameter as far as reinforcing grouted rebar bolt system strength is considered. Moreover, it was established that pull tests can determine the yield point of a rock bolt, but they do not confirm the development of full anchorage throughout the total length of a grouted bolt. While these tests confirmed the significance of a mechanical interlock in the anchorage system of resin-grouted bolts, the effect of resin mixing was not tested.

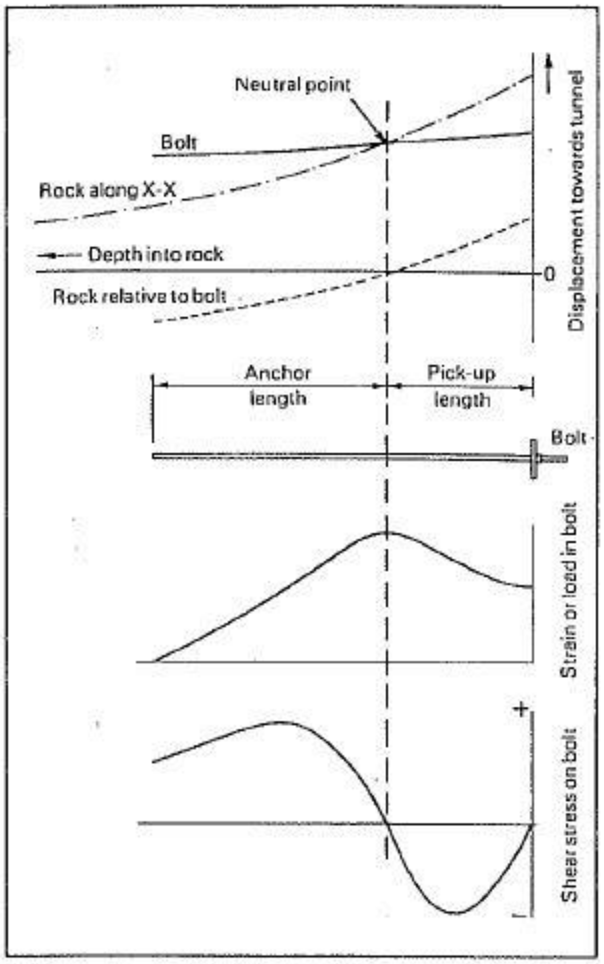


Figure 7: Anchor length and pick-up length of the bolt.

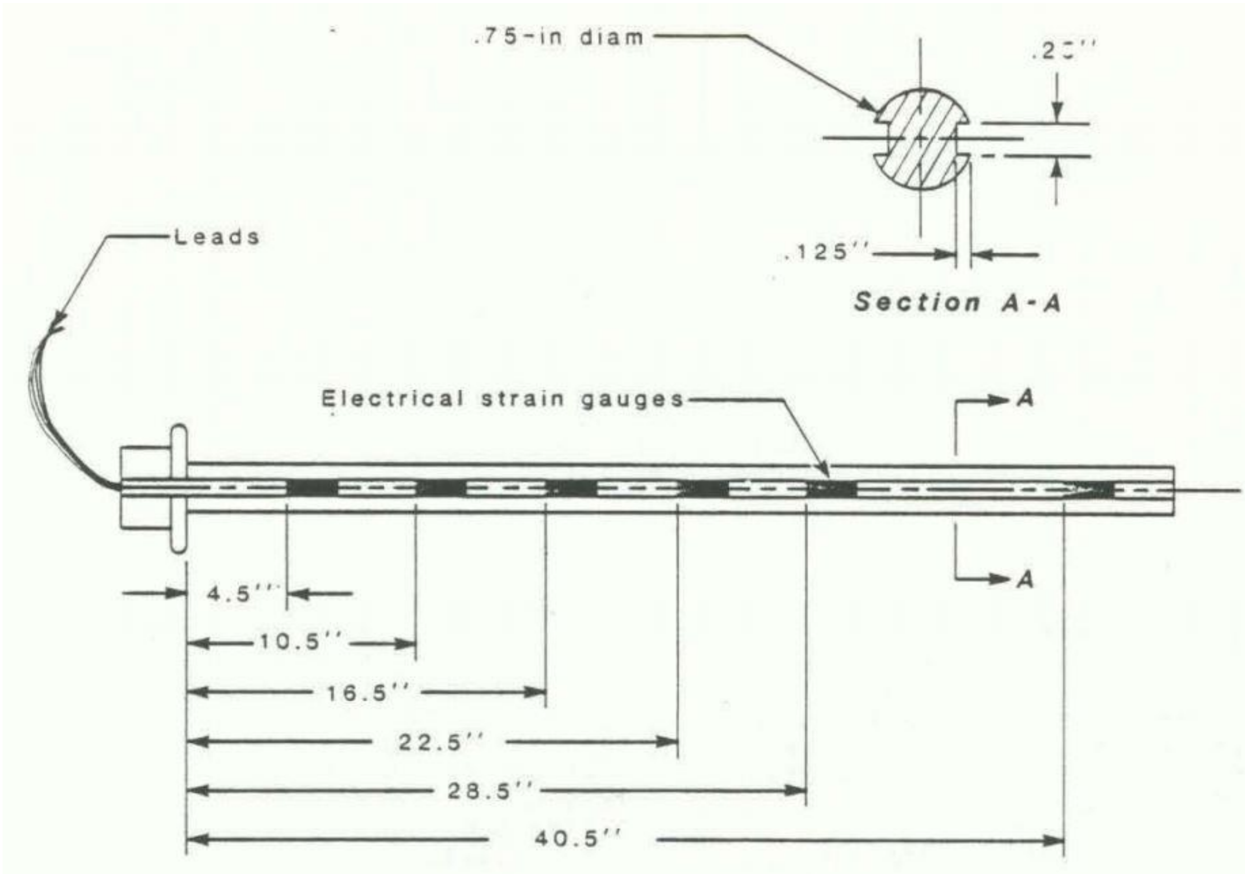


Figure 8: Positioning of strain gauges along the bolt.

Even though fully grouted resin rock bolts were widely used by the 1980s, their anchorage capacity still required investigation. In 1987, the BOM undertook a small project using expertise within the Spokane (WA) and Denver (CO) Research Centers to determine the cause of excessive glove fingering with a subsidiary goal of determining effective length (the difference between bolt length and the length of glove fingering) of resin-grouted bolts (Pettibone, January 1, 1987). Pettibone also investigated different installation variables such as spin time, hole depth, hole diameter, and bolt diameter by installing a series of bolts in concrete blocks. Each installation parameter was tested at two extremes, *e.g.*, 4' and 2' bolts, no spin after insertion and over-spin after insertion, *etc*. Pull tests were conducted for each parameter at both extremes to analyze the performance of resin-grouted rock bolts. Regarding spin time, for both installation conditions a low pull-out load was observed. It was concluded that under-spinning resin leads to inefficient resin mixing whereas over-spinning leads to destruction of the partially cured resin. It was deduced from these tests that manufacturer-recommended installation procedures should be followed. However, no testing was done to evaluate optimizing spin time for resin-grouted bolts. Pull tests performed to analyze glove fingering length observed that under-spun bolts resulted in full or partial glove fingering while very little, if any, glove fingering was observed when bolts were over-spun.

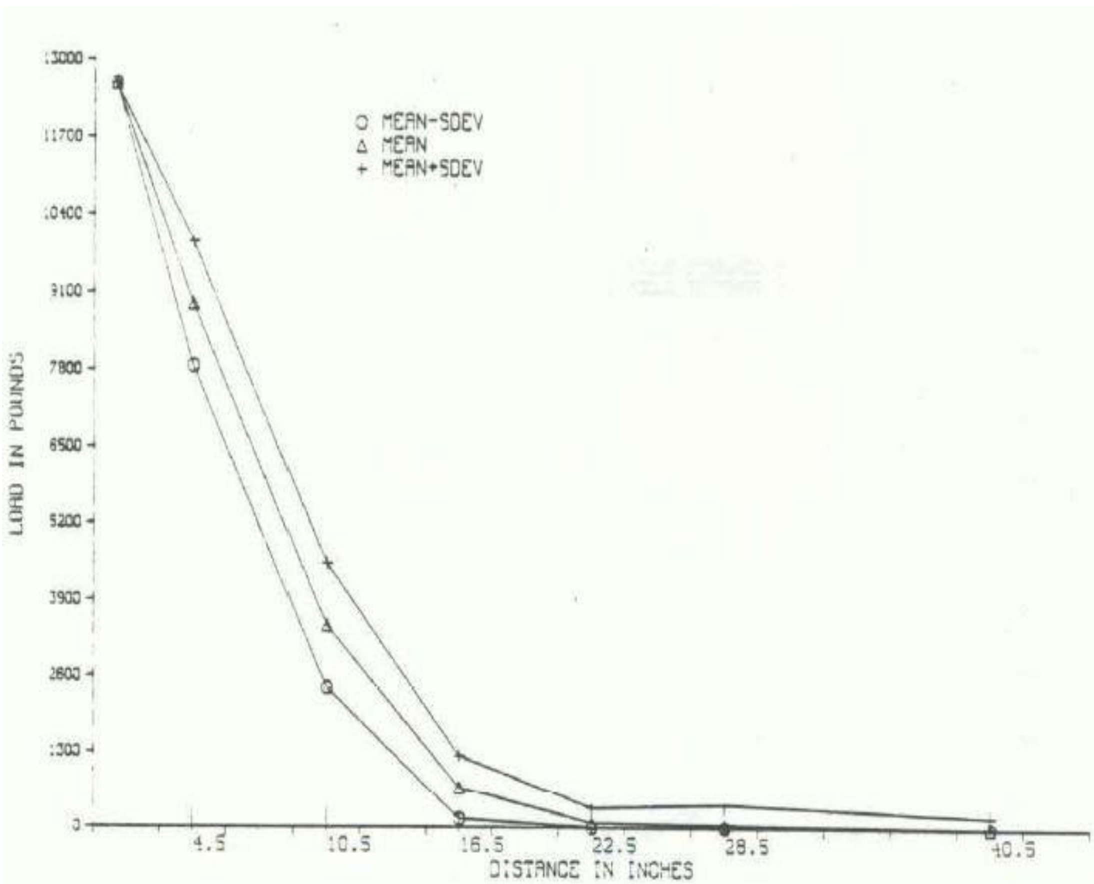


Figure 9: Load distributed over the length of the bolt.

Cincilla (1986) conducted a field study on the effect of grout volume on anchorage strength of grouted bolts in a coal mine roof strata. Tests were conducted at thirteen sites in ten underground coal mines having shale, top coal, sandstone, and limestone as four general immediate roof rock lithologies. Grade 40, 4’ long, #6 rebar having a minimum ‘rated’ yield point of 8.8 tons was used with encapsulation lengths of 12”, 18”, 24”, and 48” For an efficient analysis, Cincilla developed a ‘pass/fail’ criterion, *i.e.*, bolts were considered good if the maximum pull-out load was greater than the yield of steel with bolt displacement of less than 0.2”. An indexing property known as effective column length was established based on 90% or more individual test results that met the ‘pass’ criteria. It was concluded that under axial loading conditions, the rebar dominates loading characteristics of the system. From results of large-scale pull tests, it was evident that installation parameters for fully grouted resin rebar play an important role in resin anchorage. However, no further tests were conducted to validate that conclusion.

To get a clearer picture on the mechanics of grouted rock bolts, Signer (1988) conducted laboratory and field tests to determine the bolt-grout-rock interaction behavior in both elastic and post-yield phases. The test procedure was similar to tests conducted by Serbousek (1984), but the objective of these tests was to determine the load distribution over the length of the bolt due to the rock movement. Furthermore, tests were also conducted to examine time dependent properties of polyester as well as gypsum grouted bolts. The standard pull test procedure was used for the application of axial load to study load-transfer for both laboratory as well as field tests. Over fifty pull tests were performed in the laboratory with variations in bolt length, diameter of hole, and grout type installed in concrete blocks as was done before (compared to tests done in 1984). Bolt lengths were 1’, 2’, and 4’. Variations in diameter of hole and grout type resulted in no statistical variations for load transfer rate. However, the author of this thesis regards that as an incorrect conclusion because annulus thickness plays a major role in determining resin mixing quality. It has been found that for a #6 rebar installed in competent roof, optimum annulus thickness is 1/8” for a 1” diameter hole (Ulrich, 1989). Fourteen pull tests were conducted in four coal mines situated in Colorado, Illinois, and Pennsylvania, all having shale as immediate roof. Due to roof variations and zones of weaker rock, a significant variation in load transfer was observed.

Signer created a finite-element model to simulate testing performed in the laboratory as well as field tests and checked for equilibrium and compatibility to validate it. To approximate laboratory tests, he introduced a slip element in the model to simulate effects of movement at grout-rock and bolt-grout interfaces. Lastly, laboratory tests were conducted on six bolts of varying length (1', 2' and 4') installed in 1" diameter holes by applying axial loads of 9,000; 13,000; 18,000; and 23,000 lbs. with hydraulic rams. The change in load was measured by strain gauges. The objective of these tests was to study time dependent properties (creep) of grouted bolts. The load was measured for a period of one month until any significant change in rate of load change was no longer observed. Creep was determined by observing the rate of load change per day per load at each station (Figure 10). They concluded that both field and laboratory tests showed that 90% of the load was distributed along twenty-four inches of the bolt length starting from the bolt head and was within the elastic range. Furthermore, the load was primarily transferred through mechanical interlock beyond which the shear strength of rock or grout, whichever is weaker, would determine system failure. That failure occurs when shear strength is replaced by residual strength of the bar. It was also established that weaker rock and/or poor installation requires a larger load transfer length. However, no significant creep was observed.

Though resin annulus thickness was identified as one of the most important installation variables in the late 1960s, an extensive study was not performed until Ulrich (1989) did so at the BOM. For this study, #6 forged-head, fully grouted resin bolts of 1' and 2' lengths were installed in concrete blocks. Hole diameters of 1", 1-1/8", 1-1/4", 1-3/8", and 1-1/2" were drilled to depths of 13” for 1’ bolt and 25” for 2’ bolt leaving a 1” gap between the end of the bolt and the end of the hole after bolts were installed. Bolt performances in different test conditions were calibrated by pull tests. From an analysis of pull test results, it was found that bolt performance was superior for the case with a 1” hole and a 1/8” annulus (Figure 11). To investigate mixing quality, the concrete block was broken away from bolts so that grout mixing quality could be visually inspected. A significant decrease in quality of grout mix and axial stiffness was found as annulus thickness increased. There was no record of any other installation parameters (*e.g.*, spin time and rotational speed) evaluated in this testing.

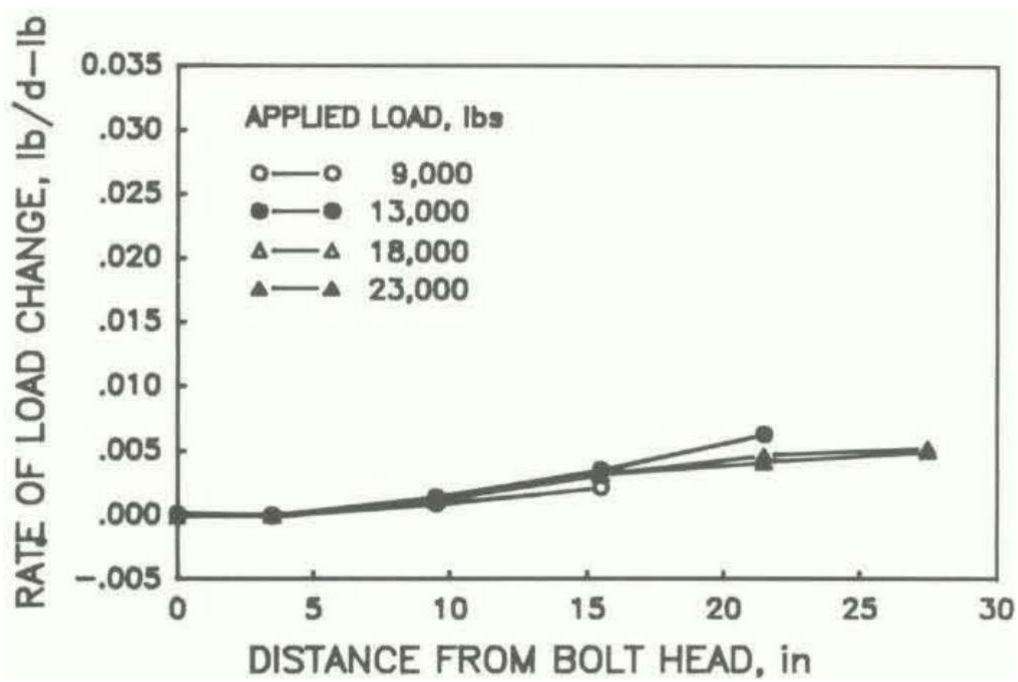


Figure 10: Creep tests results.

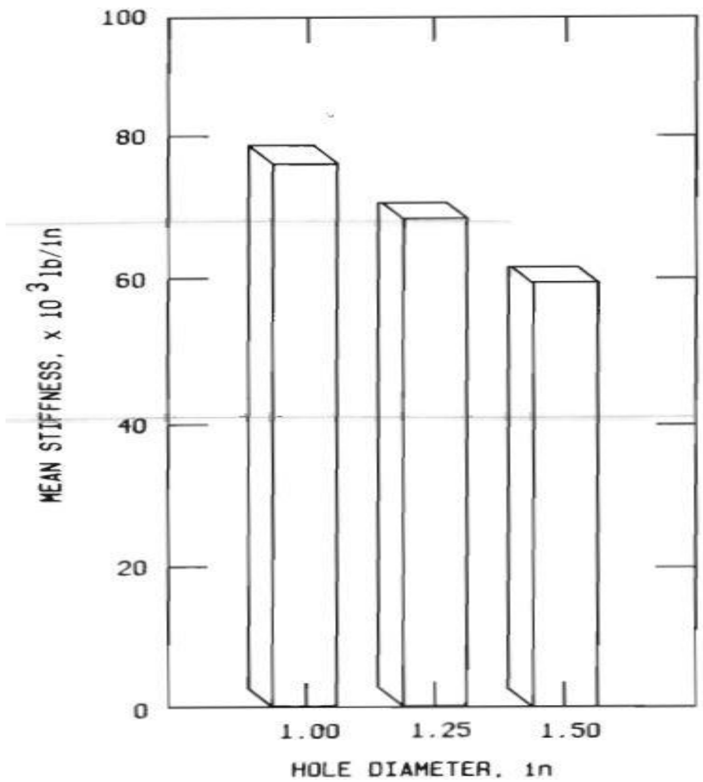


Figure 11: Mean stiffness vs. hole diameter.

This thesis also consists of tests to compare the performance of cable bolts when installed with an eccentric wrench as opposed to a standard wrench. During the 1990s, cable bolts were introduced in underground coal mines. Due to their ease of installation and higher yielding capacity vs. grouted rebar, they were used as secondary supports to replace standing supports such as cribs and post columns. In 1996, a series of pull tests were conducted on resin-grouted cable bolts at the BOM Spokane (WA) Research Center (Martin, 1996). Grade 270K, low-relaxation, bulbed cable bolts were used (Figure 12). They were approximately 6’ in length with a diameter of 0.6”. The ultimate strength of these cable bolts was 26 tons. Bolts were tested by five repeatable tests at 18”, 24”, 30”, and 36” encapsulation lengths. The objective of these tests was to find the critical encapsulation length for which cable bolts could withstand a 26-ton axial load. Bolts with 18” and 24” encapsulation failed at lower axial loads. It was concluded that 36” encapsulation was the critical encapsulation length. This test showed that pull tests on resin-grouted cable bolts embedded in concrete blocks can successfully determine axial load and displacement. It was observed that the cable bolt fails at either grout, strand, or by achieving ultimate load capacity (Figure 13). There was no effect on load carrying capacity observed when hole diameter was varied from 1” to 1-3/8”.

Figure 12: Typical bulb in a conventional cable bolt.

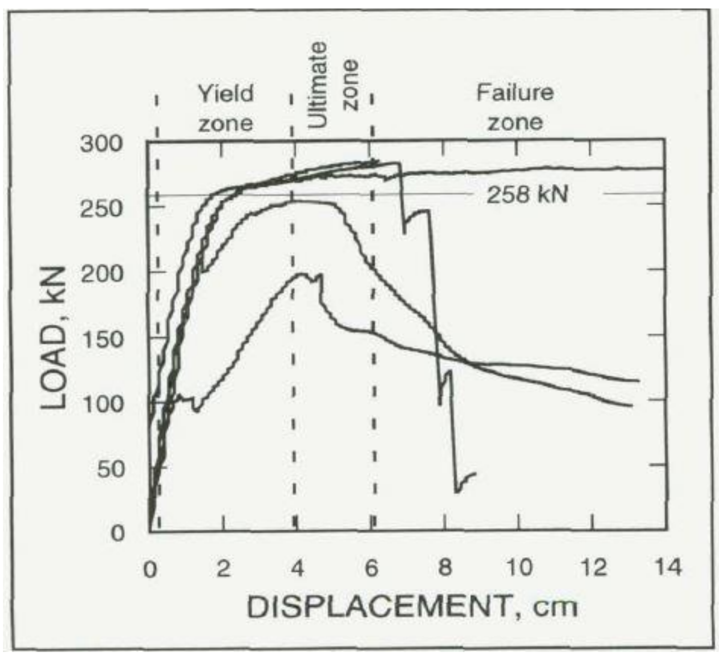


Figure 13: Load vs. displacement for cable bolts with 3’ encapsulation in 1” diameter hole.

To evaluate resin bolting design criteria, a hybrid boundary finite-element model was formulated to model rock bolt reinforcement installed in roof strata with bedding planes. The model analyzed stress distributions around a rectangular opening varying parameters such as the ratio of entry width to mining height, the number of bolts installed, the number and location of bedding planes, and *in situ* stresses (Peng and Guo, 1991). A stress analysis was performed to investigate the effects of fully grouted roof bolts on the state of stress in bolted mine roof strata. Bolt parameters used in the model were three to six bolts in a row with bolt lengths varying from 24” to 104”. Borehole diameter was varied from 1” to 1.25” and steel rod diameter was varied from 0.75” to 1.0”. In this analysis, rock bolt installations were assumed to be uniformly distributed over the whole length of the opening, which may not be correct in reality. It was concluded that the stress distribution was always irregularly distributed along the bolt length assuming that rocks around the opening are in a state of plain strain after bolts are installed, which may not be correct for smaller lengths of a tunnel. It was deduced that the maximum axial load occurs at the mouth of the hole and moves upward into the roof. The load is mostly distributed in the lower segment of the bolt, but fluctuates and reduces in magnitude towards the upper end of the bolt. However, an individual bolt (probably encapsulation length) reinforces the immediate roof in an inhomogeneous manner. The authors also concluded that bolt length should be longer than 2’ to utilize constraining force and clamping action to the fullest. To effectively prevent bed separation among roof strata layers, the distance from the end of the bolt to the nearest plane of weakness along the length of the bolt should be longer than 1’. In case of weak, laminated immediate roof, a suspension bolt should extend through all layers into a stronger and massive overlying roof. A numerical modeling study was conducted in this thesis to determine the effect of grout properties on the pull-out load when mixing parameters (*e.g.*, spin time and rotational speed) are varied.

Peng (1994) studied the problem of bolting in weak roof. According to him, weak roof refers to those strata immediately overlying the coal seam that cannot be effectively supported by normal roof bolting patterns. Typically, weak roof strata include clay shale, thinly laminated shale, shale with closely spaced joints, laminated sandstone, and massive sandstone with weak cementing materials. However, laminated roof strata are characterized as weak whenever there are weak materials filled between beds of immediate roof. A resin-grouted rock bolt uses the friction between the resin annulus and the rock wall of the drill hole to prevent vertical movement of the surrounding strata. Lateral movement, however, is resisted by the hardened resin filling up the hole along with the bolt and developing a stiffer system. The performance of the resin bolt is significantly reduced whenever a gap, even a minute one, is created between the resin annulus and the rock wall as a result of strata movement. The major problem in bolting is weathering of weak roof after exposure to air and water, which results in roof strata breaking off around bolts causing a loss of integrity in the beam effect created by bolting. Also, resin bolts will no longer be effective if the contact between roof rock and resin is lost, *i.e.*, removal of the mechanical key. Then, resin bolts must be used along with roof channels or roof mats (wire mesh). Weathering can also be greatly reduced by using resins having two speeds (fast and slow), where fast-cured resin is used to anchor the top and slow-cured resin is at the bottom building a high-tension (active) bolt and creating a solid composite beam where air cannot penetrate.

For an effective analytical study on rock bolt systems, a model was developed in which a smooth resin-anchored bolt was modeled with one-dimensional truss elements dominated by axial load behavior and the resin annulus was modeled with axisymmetric, isoperimetric elements (Guo *et al.*, 1997). It was assumed that the shear stress acting on the bolt will equally act at the inner surface of the resin annulus whereas shear stress acting at the resin/rock interface will eventually be transferred to the mine roof. This model was then incorporated in a hybrid boundary finite-element model. The model considered effects of rock inhomogeneity, bedding planes, bolting patterns, and different mine geometries. A single bolt was modeled far from the roof's surface. This model was similar to the pull test where a short segment of the bolt is grouted in a large block of rock with no displacement induced actively by the rock itself and a pulling load applied at the head of the bolt. The pattern of shear-stress distribution at the resin/rock interface is similar to those in pull tests. Thus, the purpose of putting the resin anchor far from the roof surface is to simulate the pull-out action like that in the laboratory. A study of the numerical model showed that the distribution of shear stress at the resin/rock interface varies significantly with resin modulus where the resin modulus is calibrated to the mixing of resin, which is in agreement with tests conducted in this thesis.

Li (2000) examined different case studies and briefly explained the load bearing process of fully coupled smooth bolts loaded in continuously deformed rock masses, blocky rock masses, and pull tests. The objective of the study was to interpret data from these different sources to help explain modes of axial loading under the three conditions. He concluded that the position of the peak load moves toward the far end of bolts with increasing rock deformations in continuously deformed rock whereas in a blocky rock mass several peaks in axial load may be observed, depending upon joint locations intersected by the bolt. As far as pull tests were concerned, decoupling first acts at the loading point and moves toward the far end of the bolt as the pull-out load increases. This data is represented in Figure 14 and Figure 15.

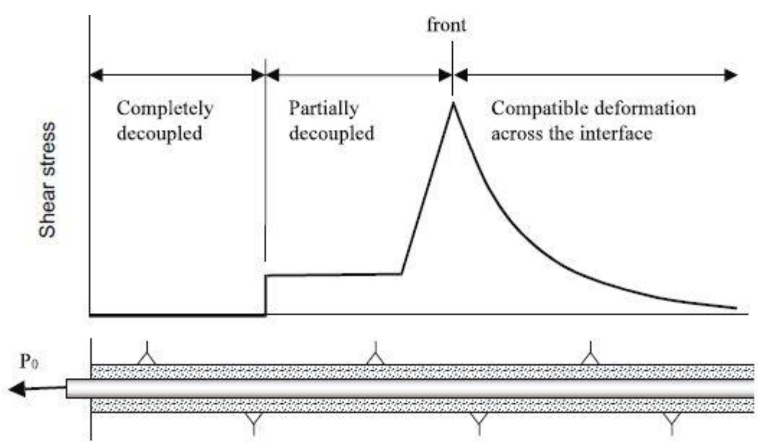


Figure 14: Distribution of shear stress over the length of a fully grouted bolt during a pull test.

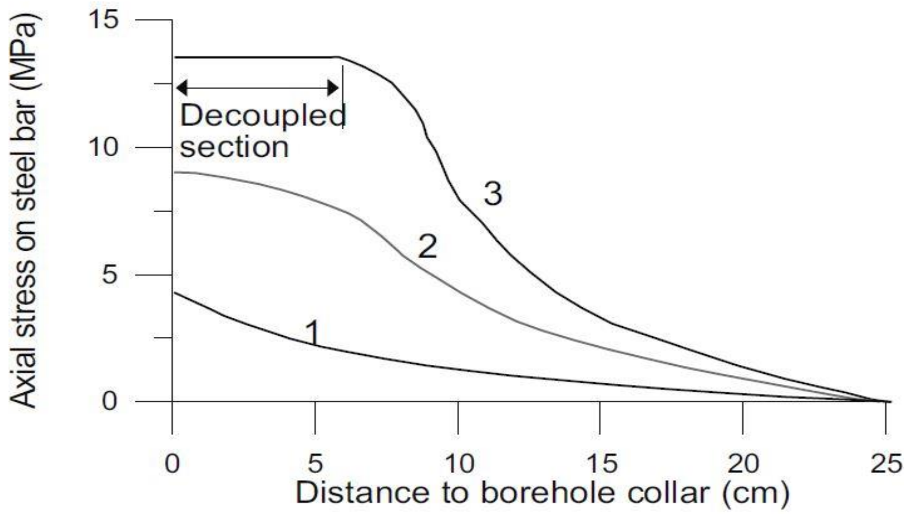
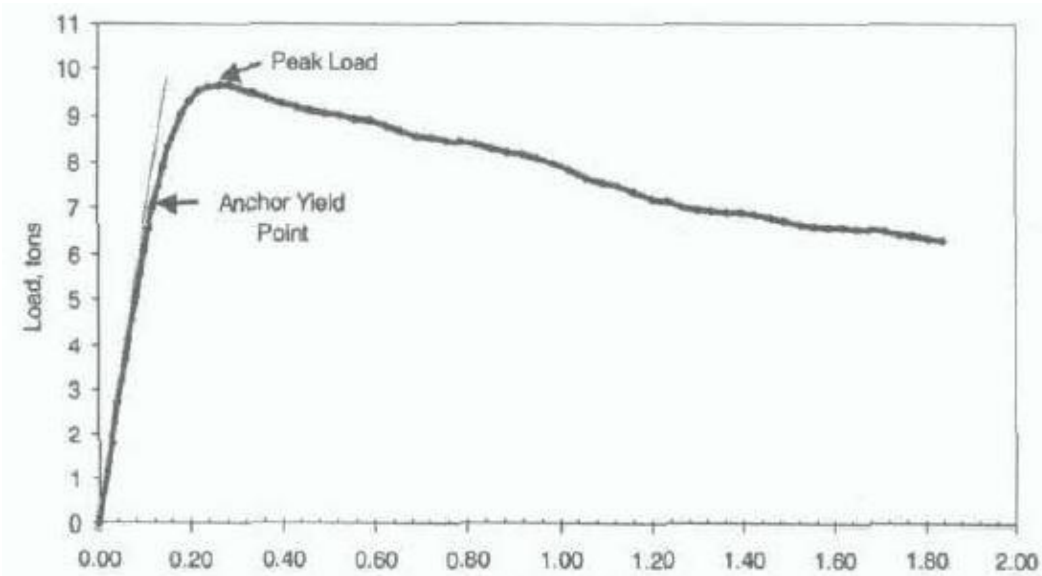


Figure 15: The axial load in a fully grouted bolt at different levels of applied load.

Though standard pull tests were widely used, no information was obtained about the anchorage near the top of bolt. In 2003, short encapsulation pull tests were conducted in NIOSH's Bruceton (PA) Safety Research Coal Mine and at underground mines in Pennsylvania and West Virginia (Mark *et al.*, 2003) where a total of fifty-six rock bolts, comprising #5 and #6 bolts in 1" diameter holes, were pulled with the objective to evaluate the effect of hole depth. Bolts were installed at a speed of 500 rpm and a 6-ssecond spin time resulting in 50 revolutions for each bolt. From laboratory tests, the authors postulated that the load deformation curve during the pull initializes with a linear trend, but as resin failure at the lower portion of the anchor progresses, the trend deviates from the straight line (Figure 16) and the anchor starts to slip, after which it carries only 70% of the peak load over 1.0" to 2.0" of additional deformation.



(inches)

Figure 16: Load characteristics of a pull test on a #5 bolt.

A study (Ray *et al.*, 2012) conducted at three room-and-pillar coal mines in the Illinois Basin and one longwall mine in Colorado to evaluate the performance of fully grouted tensioned rebar and fully grouted passive rebar showed that under similar geo-mining conditions, no significant superiority is observed, when these bolting systems are compared with each other. Tests were also conducted on partially grouted rebar with mechanical shells. Initial readings at the two room-and-pillar mines were delayed as data loggers were not intrinsically safe. Active tensioned rebar showed loss of initial load, but the exact reasons for such losses are still not clear although they have been postulated as creep.

In 2012, Mitri developed a new strain gauge instrument known as the U-Cell for use in underground mines (Mitri, 2012). The U-cell is a load cell device that can be coupled to the threaded end of a bolt prior to installation. It is interfaced with digital signal processing technology to provide quantified output data at the immediate underground location. It measures axial strain at the head of the bolt. The objective of these tests was to calibrate the U-cell for a specific rock bolt and test its potential for accurate monitoring of deformation caused in the rock bolt due to tensioning at the grout-to-plate interface only. It was concluded that the U-cell can provide a methodology for determining axial loading of rock bolts. It accurately monitors both strain and displacement over a limited grout-to-plate interface, which can be compared with numerical models if needed.

RESEARCH OBJECTIVES

The overall objective of this thesis is to measure, understand, and improve the performance of fully grouted rock bolts in underground coal mines. To accomplish this objective, it is necessary to understand how the performance of a fully grouted rock bolt is measured. The performance of a fully grouted rock bolt is measured in terms of anchoring capacity of the bolt system as described by a load vs. displacement curve. The resistance offered by the rock bolt is achieved by load transfer from rock to grout at the grout-rock interface followed by load transfer from grout to bolt at the grout-bolt interface. These two interfaces are referred to as contact surfaces. As the anchoring medium for grouted bolts is the polyester resin (grout), installation that results in a weaker grout would not be able to transfer load from rock to bolt as effectively. Failure due to poor grout is called anchorage failure.

Efforts to improve/optimize the performance of fully grouted rock bolts were accomplished through completing the following research objectives:

Objective 1: Measure and optimize anchorage capacity obtained with fully grouted rock bolts by evaluating effects of resin (grout) mixing parameters.

Grouted rock bolt anchorage can be evaluated using a standard pull test where an installed rock bolt is hydraulically pulled while concurrently measuring bolt displacement. Pulling continues until the rock bolt anchor fails in shear at the anchor-rock interface. Thus, the first objective of this research is to measure and evaluate grout anchorage obtained for various resin mixing parameters and correlate results with pull-out loads. Resin mixing parameters include rotational speed, bolt diameter, set time, hold time, and cure time.

Objective 2: Measure and evaluate the effect of using eccentric installation tools vs. normal installation mechanisms on anchorage capacity achieved for commercial support systems.

In 2003, one of the major manufacturers of rock bolt products developed the eclipse bolt system (Campoli, 2003) where the center of the bolt head is offset by 1/8" from the centerline of the bolt. The objective was to introduce a whipping action when spinning the bolt, thereby improving resin mixing. Tests on the eclipse system conducted by the manufacturer achieved higher pull-out loads leading the manufacturer to conclude that a more consistent and uniform mixing can be obtained, which increases bolt performance. Following the same principle, a prototype installation tool called an eccentric wrench is being developed to introduce the whipping action to any type of rock bolt by means of a weight welded on one side of the wrench stem. Thus, the second objective of this research is to measure and evaluate the effect of using eccentric installation tools during installation of commercial anchorage systems (*i.e.*, #5 and #6 rebar and cable bolts).

Objective 3: Measure and optimize resin hardness by evaluating effects of varying installation parameters such as spin time, rotational speed, and bolt diameter.

Shore hardness test is a convenient and inexpensive method of comparing the hardness of rocks. The Shore hardness of a specimen is determined from the rebound height of a diamond or tungsten-carbide tipped hammer released from rest onto a horizontal smooth surface (Rabia and Brook, 1979). The greater the Shore hardness of the material, the greater resistance it has to deformation indicating the greater the strength of the material. In a rock bolt system, greater resin hardness will indicate a higher coefficient of friction between the rock and the resin indicating anchorage improvement and resulting in higher pull-out loads. Failure of the mechanical key (Figure 17) at the grout-rock interface is a cause of lower anchorage of fully grouted rock bolt systems, which is thought to be caused by lower grout hardness. Measuring pull-out loads with pull tests is time consuming and expensive as it takes at least four people to conduct one. Therefore, it is hypothesized that a cost effective and inexpensive experiment can be designed using Shore hardness as an indicator of the pull-out load obtained by varying key installation parameters. Thus, the third objective of this study is to measure and understand the effect on cured resin hardness of varying spin time, rotational speed, and bolt diameter using a simple test that gives results comparable with pull-out loads obtained from actual pull tests.

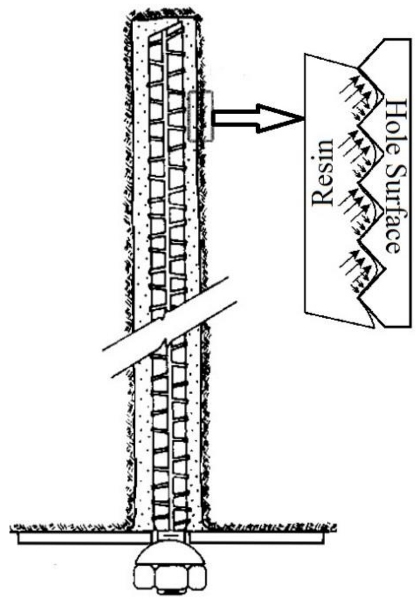


Figure 17: Failure of mechanical key at grout-rock interface.

Objective 4: Measure and understand creep behavior in grouted, tensioned rebar.

Creep is the tendency of a solid material to permanently deform under the influence of mechanical stress as a function of time (Brooks, 2014). Shear stress is generated at the grout-rock interface as a result of ground movement due to the distribution of frictional forces along the length of the grout-rock interface. Studies conducted by Ray *et al.* (2012) comparing active vs. passive grouted rebar showed loss of initial load for tensioned rebar indicating it had no superiority over fully grouted passive rebar. It is hypothesized that the loss of initial load is due to failure of the mechanical key at the grout-rock interface, usually involving the rock (weaker) material. This reduces frictional forces, which in turn will reduce the anchorage performance of the grout (*i.e.*, effective load transfer from rock to bolt). Thus, the fourth objective of this study is to measure and understand creep behavior of grouted, tensioned rebar.

Objective 5: Measure and understand the effect of glove fingering on the pull-out load.

Glove fingering occurs when the plastic wrapper surrounding the unmixed resin remains intact around the hardened resin. Glove fingering is a concern as it results in loss of contact (*i.e.*, load transfer) between grout and rock at the grout-rock interface, thereby reducing shear strength at the interface along the affected length. To address this issue, the fifth objective of this research is to measure and understand the effect of glove fingering on pull-out load.

Objective 6: Develop a method to predict pull-out load for varying grout properties (obtained by different mixing mechanisms) of fully grouted rock bolts using FLAC3D numerical modeling.

Numerical modeling of pull tests on fully grouted bolts installed in a homogeneous, isotopic material such as cement can be performed to predict pull-out load and compare it with actual results. In attempting this, the sixth objective of this study is to develop a FLAC3D numerical model to simulate the effect of grout shear stiffness on pull-out load of fully grouted rock bolts. Results of this modeling will be compared with laboratory pull test results to validate and calibrate the model.

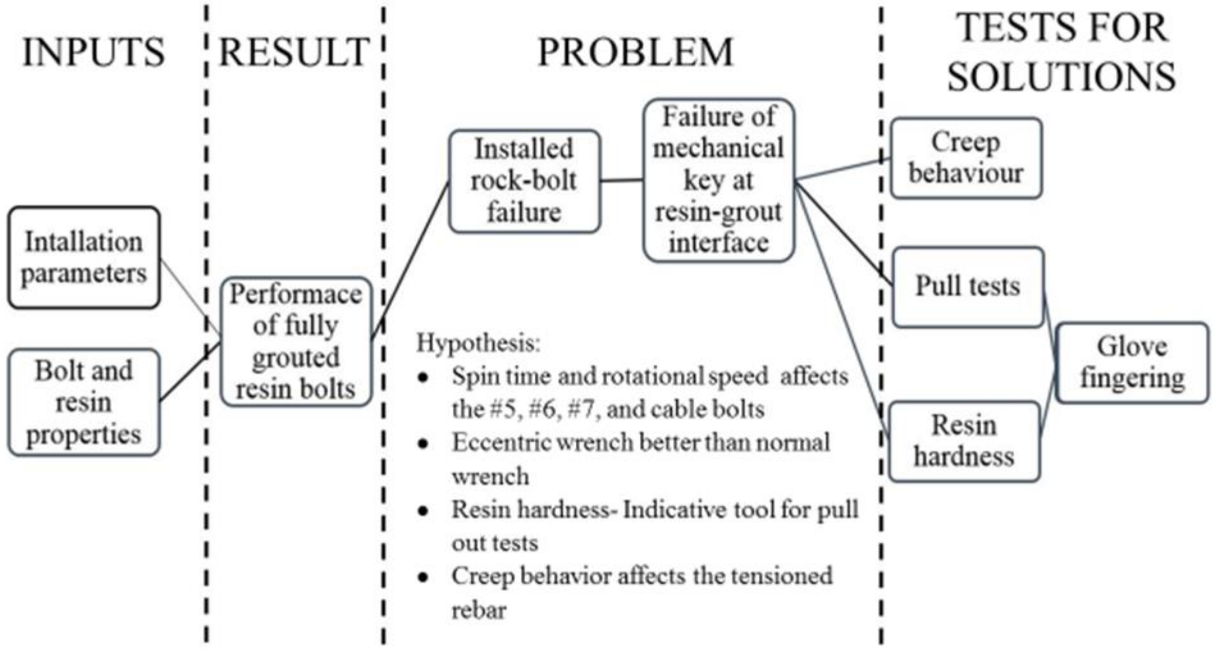


Figure 18: Thesis objectives.

Figure 18 depicts how the overall objective will be realized through completing each objective previously outlined. Results acquired from tests can be analyzed to acquire optimized installation parameters leading to the achievement of higher system anchorage at no extra cost. For example, assume that the pull-out load achieved by a 4' bolt anchored with a resin encapsulation length of 1' is 10 tons. If the same pull-out load can be obtained by a 4' bolt anchored with a resin encapsulation length of 0.5', it can be deduced that by using optimized installation parameters, the effective length of the bolt can be increased (Figure 19). Underground coal mines can use this information to improve the performance of fully grouted rock bolts by which a safer environment with higher productivity can be created at no additional cost. Results from creep tests can help in evaluating a comparative performance of fully grouted passive and active rebar. Fully grouted active tensioned rebar cost 10% more than fully grouted passive rebar (Spearing and Gadde, 2011). Thus, if the loss of initial load for an active bolt system can be explained resulting in it having no superiority over a fully grouted passive system, a significant amount of money can be saved by mine operators.

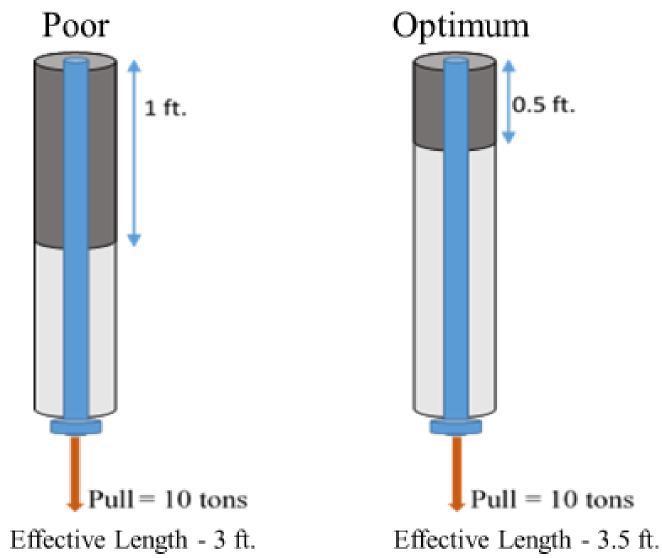


Figure 19: Effective length obtained after optimization.

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HYPOTHESES

This thesis describes large- and small-scale laboratory tests designed and conducted to understand the performance of fully grouted passive and active rock bolts. Along with laboratory pull tests, a series of pull tests were conducted in two coal mines (*i.e.* Mines A and B). These tests were required to test the following hypotheses:

- Spin time and rotational speed during installation significantly affect the performance of #5, #6, and #7 fully grouted rebar, and cable bolts.
- Improved performance can be obtained with eccentric wrench installation tools vs. normal installation procedures.
- Resin hardness can be used as an indicator in determining pull-out load behavior of partially grouted rebar.
- Significant creep occurs in tensioned (active) rebar affecting its performance.
- Glove finger can negatively impact support system performance. Minimizing or eliminating glove fingering can be accomplished by understanding and following optimized mixing procedures.

TECHNICAL TASKS

To develop a better understanding of fully grouted rock bolt performance and to achieve the previously stated objectives, the following tasks were undertaken:

- Performed large-scale laboratory pull tests with #5 (0.625” diameter), #6 (0.750” diameter) and #7 (0.875” diameter) partially grouted passive rebar while varying encapsulation length, spin time, rotational speed, and length of bolt upon installation.
- Conducted *in situ* pull tests with #5 and #6 fully grouted passive rebar installed with normal and eccentric installation procedures.
- Developed a new test to understand the mixing of resin using a small-scale modified drill press calibrated on the Shore hardness scale. If proven reliable, this new quick and repeatable laboratory test will expedite future research.
- Conducted creep tests by studying the axial load behavior with time of fully grouted tension rebar using conventional hollow load cells and U-Cells upon installation.
- Investigated glove fingering by conducting drill rig tests with #5 and #6 fully grouted passive rebar.
- Developed a basic FLAC3D numerical model to simulate pull tests.

LARGE-SCALE LABORATORY PULL TESTS

The anchorage performance of fully grouted rock bolts can be determined by conducting controlled pull tests. The pull test is conducted to quantitatively measure the anchorage performance of resin-grouted rock bolts and pull tests are designed to measure working and ultimate anchorage capacities and the anchorage stiffness of rock bolts. Standard pull tests could not be performed on fully grouted bolts because pulling forces seldom extend more than 18-24 inches up into the resin column thereby measuring only the strength of the rock bolt itself (Serbousek, 1984). Thus, in order to determine the performance of grouted rock bolts, pull tests conducted for this research were done mainly with a short encapsulation length, *i.e.*, short encapsulation pull test (SEPT). All pull tests were conducted in compliance with ASTM F432.

Large-scale laboratory testing consisted of pulling almost two hundred rock bolts installed following the same procedure used to install rock bolts in underground coal mines. After installation, the bolt head is pulled with a hydraulic jack in regular increments of one ton of load until the onset of anchorage failure occurs (Figure 20). Anchorage failure is marked by a sudden increase in displacement measured at the rock bolt head with no load increase.

Spearing and Mishra (2014) described the testing done for this research as follows:

> *"Short encapsulation tests can be done either in the laboratory or under in situ conditions, but in situ tests are difficult to arrange and interpret due to the following reasons:*
>
> - *The immediate coal roof properties vary considerably over even short distances.*

- *The rock bolts cannot be installed reliably at right angles to the top. This induces other stresses in the support during pull tests that are not pure tensile in nature.*
- *The tests can adversely impact coal production on a mine.*
- *Ready access in and out from the site is difficult.*
- *Mine personnel must be present at all times.*

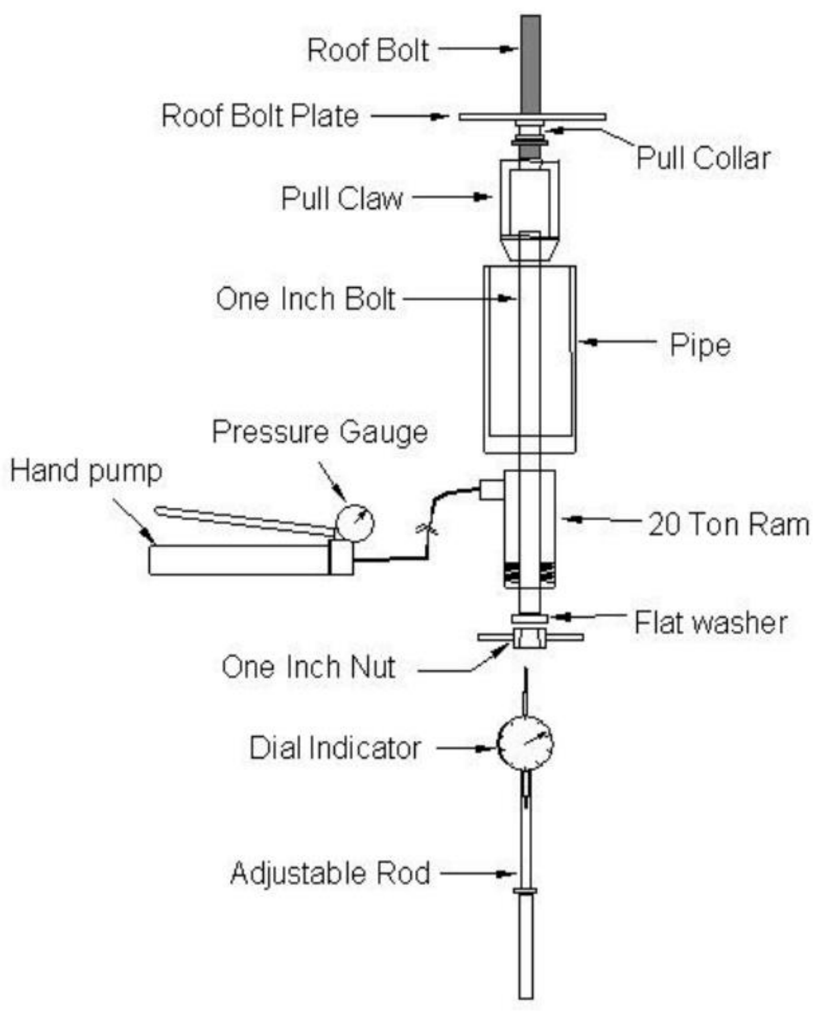

Figure 20: Exploded diagram of pull test

For these reasons, it is more reliable, safer, quicker and easier to conduct full-scale tests in a laboratory environment with a lesser cost compared to in situ tests, if one is readily accessible, for evaluation of rock bolt performance.

"The tests were conducted at Orica North American Innovation and Technology Center in Bowerston, Ohio. Rebar were grouted in cement blocks, a homogenous and isotropic material. The Orica facility offers the advantage to install 4-ft-long rock bolts in concrete blocks by following standard installation procedure. In addition, the bolter is fully instrumented, and the following important parameters can be accurately set for each test:

- *Resin insertion (spin) speed can be varied and set for each test in 5 rpm increments, if needed.*
- *Resin spin (insertion) direction can be set for clockwise, counterclockwise, or both, if needed (e.g., clockwise first, then counterclockwise to tension a rock bolt).*
- *Resin mix (spin) time can be set at any value in 1-second increments.*
- *Resin hold time can be set at 1-second intervals.*
- *Torque can be set at 5-ft-lb increments, up to 500 ft.-lbs. maximum.*

"These effective test controls facilitate the bolt installation and, therefore, mitigate any operator-related variability that, otherwise, could be encountered in an underground scenario. Further, to ensure reliable results, at least three tests were performed for each set of the aforementioned parameters."

Tests were conducted on #5, #6, and #7 rebar with the following installation parameters:

- Rebar Length: 18”, 39”, and 69”.
- Hole Diameter: 1” for #5 and #6 rebar, 1.125” and 1.375” for #7 rebar.
- Encapsulation Length: 12”, 24”, and 48”.
- Spin Time: 2, 3, 4, 5, 6, 7, and 9 seconds.
- Rotational Speed: 300, 390, 540, and 675 rpm.
- Resin Type: LIF20, LIF35, H35, and DSI 30seconds. The LIF and H stands for Low insertion force and H stands for high viscosity resin. The numbers 20,30 and 35 stands for the manufacturer recommended cure time.

To obtain 1’ encapsulation length in a 1” diameter hole for 1’ bolt length, the resin cartridge was prepared using Equation 1:

Cartridge Length (inches) = $[(13\times HoleD^2)-(12\times BoltD^2)]/TubeD^2$ ……………...…..….(1)

where HoleD is hole diameter, BoltD is bolt diameter, and TubeD is resin cartridge diameter.

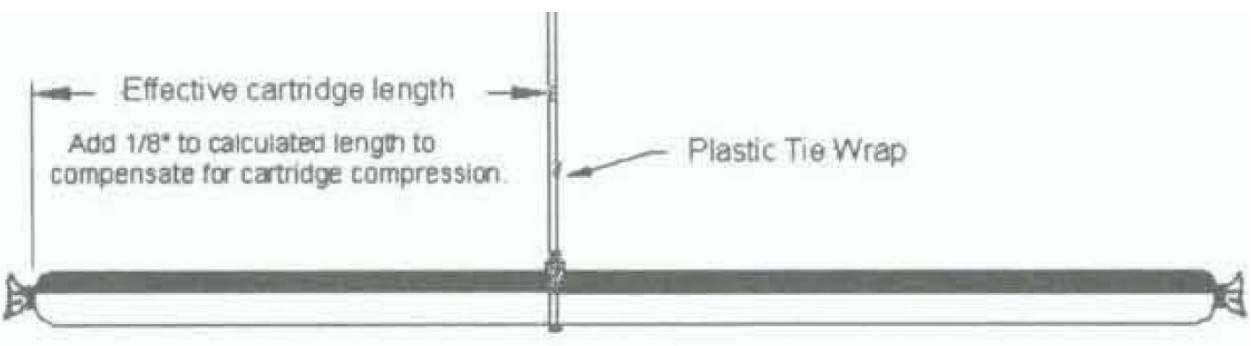

Figure 21: Effective cartridge length.

The cartridge was cut to the calculated length. An additional 1/8” was added to the calculated length before tightening it with the zip tie to allow for compression of the resin cartridge. Then excess tie wrap and resin were cut and removed (Figure 21).

Bolt length was also adjusted to compensate for the pull collar and the bearing plate thickness. Hole length was determined by assembling the bolt, pull collar and bearing plate, measuring the distance from the bearing plate to the end of the bolt, and adding 1”. The extra 1” at the end was to allow for the plastic cartridge wrap when shredded by the bolt and is the preferred practice in mines too.

Pull tests were generally conducted one hour after installation of bolts unless otherwise noted. After assembling the pull test gear, the claw was seated on the pull collar (Figure 22) and the nut was tightened as much as possible by hand. The ram was then loaded to one ton. Next, the displacement gauge and adjustable rod were added in line with the bolt axis so that the bolt would be pulled directly down the displacement gauge (Figure 23). Once everything was in place, the dial on the gauge was set to zero and the bolt was loaded 1-ton increments. The load from the gauge on the hydraulic pump was recorded as well as deformation shown on the displacement gauge. This process was repeated until the system yielded (*i.e.*, started to pull out).

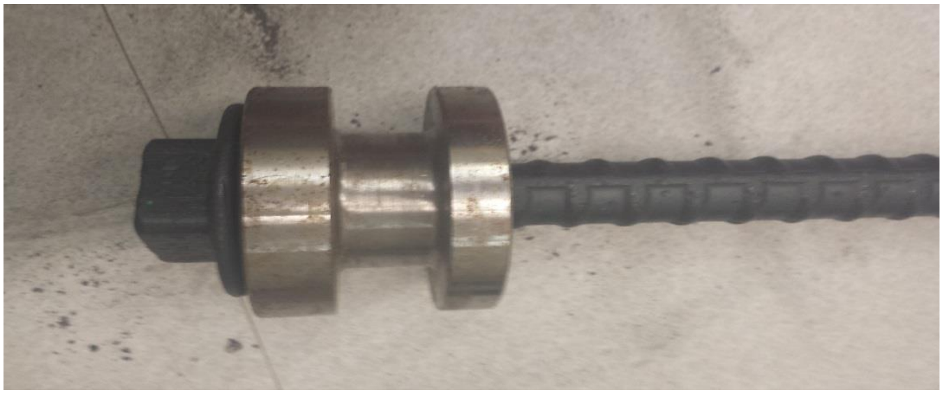

Figure 22: Pull collar attached to the bolt head

Throughout each test, the entire assembly, including the bolt, was supported with safety chains and tests were conducted following all Orica specified safety precautions. According to the bolt manufacturer, ultimate tensile strength of #5 bolt is 27,900 lbs. (14 tons) and #6 bolt is 39,600 lbs. (20 tons). Thus, for safety reasons, tests were terminated at 12 tons for #5 bolts and 16-17 tons for #6 bolts.

Figure 23: Pull tests at Bowerston Ohio Research Facility.

It was very clear from these tests that #6 rebar outperformed #5 rebar when installed in a 1” hole (Table 1 and Table 2). Tests also showed that when 10-20 seconds of gel time was allowed, 450-600 rpm appears to be the optimum rotational speed. A faster or slower speed at this range would affect resin mixing; however, on the basis of bolt diameter, #6 rebar performs better when installed at speeds of 450-700 rpm, whereas #5 rebar performs best when installed at speeds of 390-575 rpm. As far as spin time is considered, the best performance (i.e., 90%

capacity range) can be achieved when resin mixing is 2.5-5.0 seconds for #6 rebar and 3.5-4.5 seconds for #5 rebar.

Table 1: Pull test results for #6 rebar.

Number of Tests	**Bar Diameter**	**Bar Length (inches)**	**Hole Diameter (inches)**	**Resin Type**	**Encapsulation Length (inches)**	**Spin Time (seconds)**	**Rotational Speed (rpm)**	**Average Peak Load (tons)**	**Load Variation (tons)**	**Average Peak Displacement (inches)**
3	#5	18	1.0	LIF35	12	5	540	7.7	7-9	0.057
3	#5	39	1.0	LIF35	12	4	540	10.7	10-11	0.151
3	#5	39	1.0	H35	12	6	540	10.0	10.0	0.261
3	#5	39	1.0	H20	12	5	540	11.0	11.0	0.134
3	#5	69	1.0	LIF35	12	4	540	9.3	8-10	0.179
3	#5	69	1.0	H35	12	6	540	10.0	10.0	0.177
3	#5	69	1.0	H20	12	4	540	10.7	10-11	0.128
3	#5	69	1.0	LIF20	48	4	540	10.4	8-11	0.134
2	#5	69	1.0	LIF20	48	5	540	10.4	8-11	0.134
4	#5	39	1.0	H20	12	2	540	5.5	3-7	0.135
3	#5	39	1.0	H20	12	3	540	7.7	7-9	0.124
3	#5	39	1.0	H20	12	4	540	10.7	10-11	0.151
4	#5	39	1.0	H20	12	7	540	6.3	5-8	0.092
4	#5	39	1.0	H20	12	9	300	4.8	3-10	0.103
4	#5	39	1.0	H20	12	7	390	5.3	3-9	0.091
3	#5	39	1.0	H20	12	4	540	10.7	10-11	0.151
4	#5	39	1.0	H20	12	3	675	8.0	6-11	0.169

Table 2: Pull test results for #5 rebar.

Number of Tests	Bar Diameter	Bar Length (inches)	Hole Diameter (inches)	Resin Type	Encapsulation Length (inches)	Spin Time (seconds)	Rotational Speed (rpm)	Average Peak Load (tons)	Load Variation (tons)	Average Peak Displacement (inches)
3	#6	39	1.0	LIF20	12	4	540	16.0	16.0	0.179
3	#6	39	1.0	H35	12	6	540	15.0	15.0	0.157
3	#6	39	1.0	H20	12	3	540	15.7	15-17	0.158
3	#6	39	1.0	H20	12	7	540	13.0	12-15	0.140
3	#6	39	1.0	H20	12	2	540	14.0	12-16	0.142
3	#6	39	1.0	H20	12	9	300	12.3	8-15	0.153
3	#6	39	1.0	H20	12	7	390	13.7	13-15	0.151
3	#6	39	1.0	H20	12	4	675	15.7	15-16	0.159
3	#6	69	1.0	LIF20	12	4	540	14.7	14-16	0.231
3	#6	69	1.0	H35	12	6	540	13.7	10-16	0.210
3	#6	69	1.0	LIF20	48	4	540	12.0	8-14	0.158

To find the optimal spin time and rotational speed, test results were compared for different combinations of rotational speed and spin time while maintaining the total number of rotations constant at 36, which is within the range recommended by the manufacturer to achieve optimum mixing of the resin and catalyst (Figure 24 and Figure 25). Tests were also analyzed for different spin times at 540 rpm, which is the standard speed of roof bolters in underground coal mines. These SEPTs measured displacements at the bolt head. On application of axial load (pull) on the head, displacement measured at the head was a combination of deformation of the free length of rebar and displacement of the resin at the resin-concrete interface due to marginal failure of the mechanical key. Thus, a lower bond displacement would indicate a stiffer rock bolt system. To compare tests, bond displacement was calculated from test result data using the following formulae:

Head Displacement = Deformation of free length of rebar + Bond Displacement …… (2)

$BD = H - [(L-L_E) \times P / E_S]$…..…………………………………………..……….….. (3)

where BD is bond displacement, H is total displacement at the bolt head, L is length of the rebar, L_E is encapsulation length, P is stress on the bolt, and E_S is modulus of elasticity of the rebar. According to ASTM A615 and A706, the modulus of elasticity for Grade 60 rebar is 29×10^6 psi.

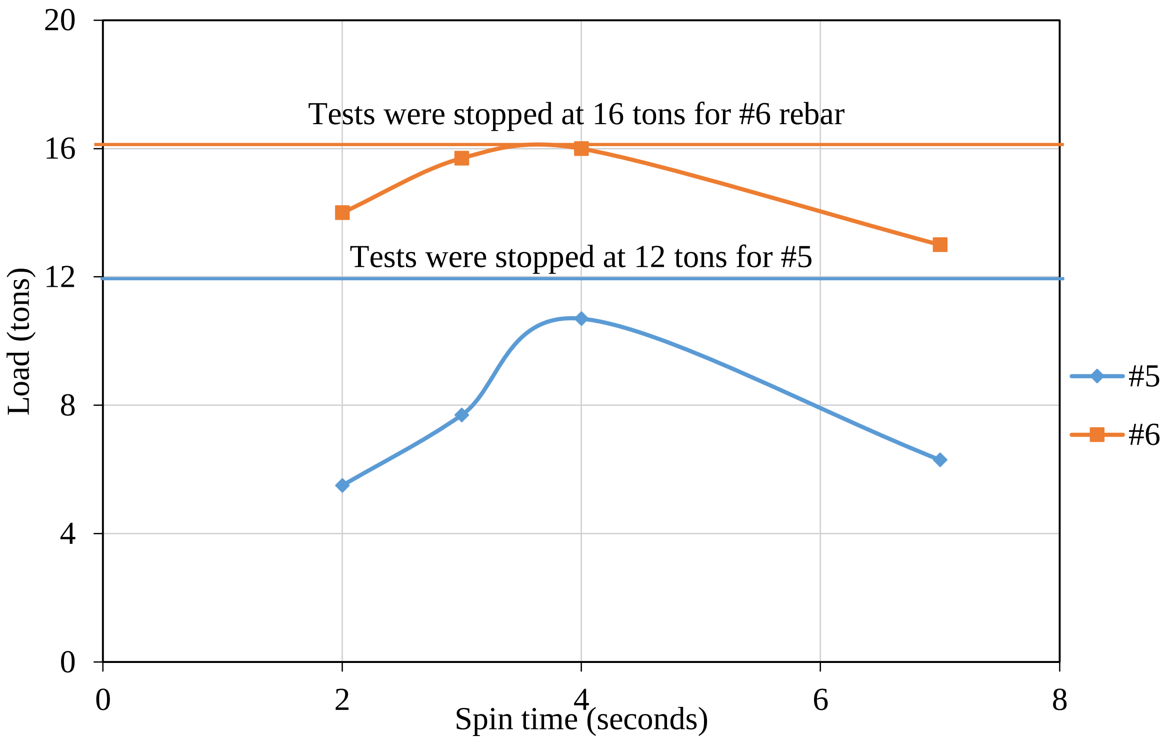

Figure 24: SEP T results at 54 0 rpm.

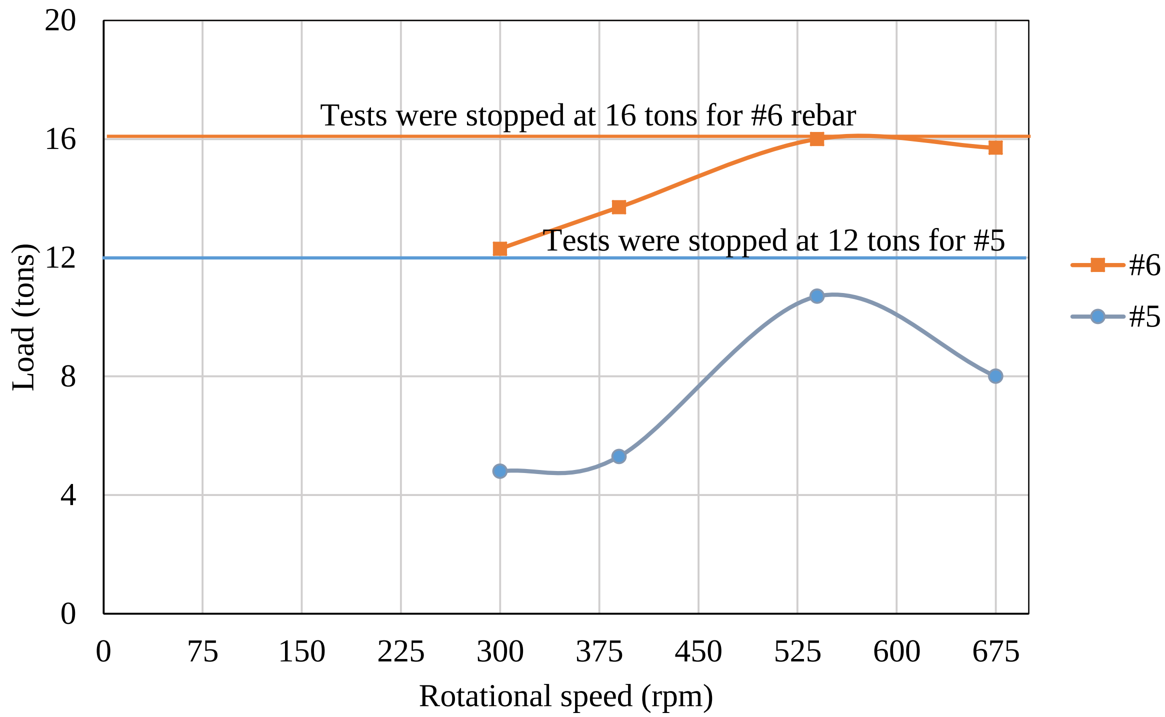

Figure 25: SEPT results at constant rotations (36) and varying spin times and speed.

Figure 26 and Figure 27 show bond displacement curves for tests with 12" encapsulation length. Similar trends were found for both #5 and #6 rebar. Bond displacement shown in these figures is the displacement of grout at the grout-concrete interface. For the same length of encapsulation, the less bond displacement at failure, the stiffer the system. Hold time for pull tests with #5 rebar was 10 seconds for all tests except when H35 resin was used. For H35 resin, hold time was 15 seconds based on the manufacturer's recommendation. Table 2 clearly shows that when varying spin time while keeping all other variable constant, pull-out load increased as spin time increased from 2 to 4 seconds, but decreased as spin time increased from 4 to 7 seconds indicating optimality at 4 seconds. Detailed pull test results are plotted in the Appendix (Figures 62-69).

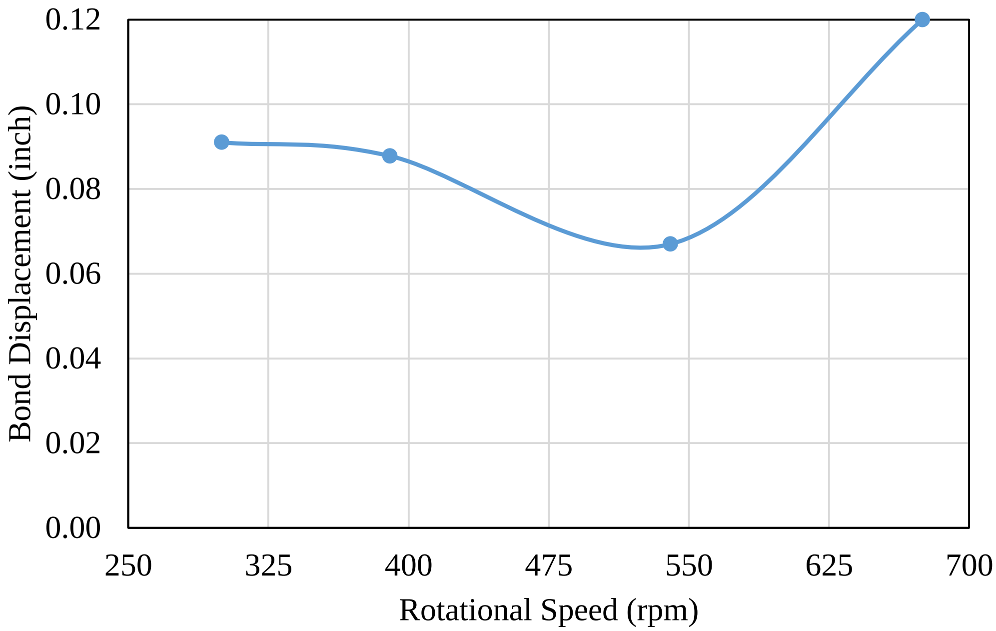

Figure 26: Bond displacement during #6 rebar pull tests.

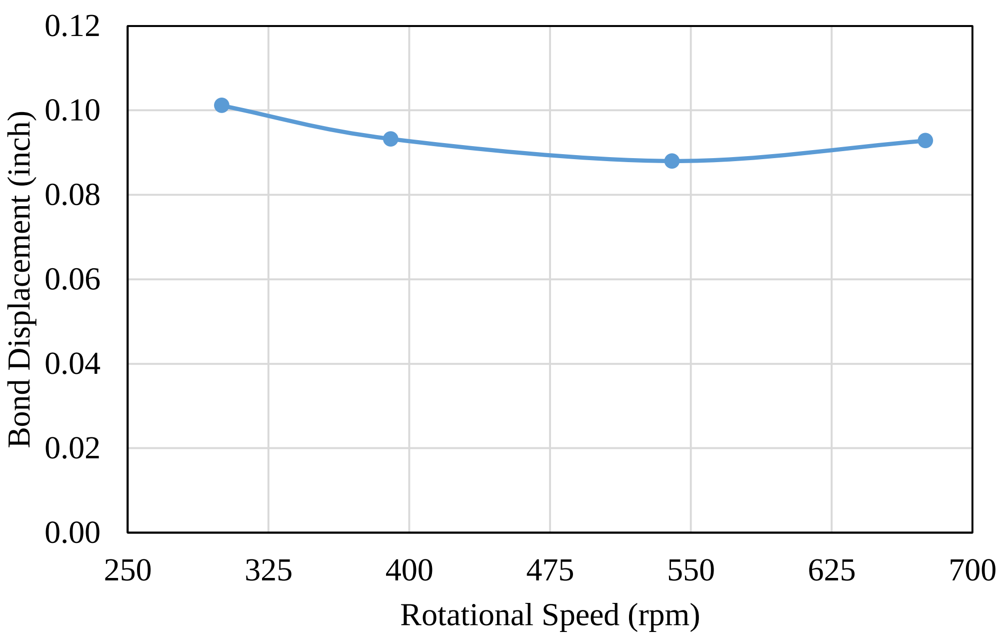

Figure 27: Bond displacement during #5 rebar pull tests.

Data was statistically analyzed as shown in Tables 3-7 to determine the significance of spin time and rotational speed. Single-factor Analysis of Variance (ANOVA) was performed on #5 rebar pull test data using Design Expert software. As shown in Table 3, the model F-value when analyzing spin time at 540 rpm rotational speed is 30.37, which is significant. There was only 0.01% chance that an F-value this large (*i.e.*, 30.37) could occur due to noise. If the variable A represents spin time, then both A and A^2 are significant. A two-factor ANOVA performed with spin time (A) and rotational speed (B) as variables found that the A-B interaction effect was significant (Table 4). This interaction is the equivalent of number of rotations, which is found by multiplying spin time (in seconds) by rotational speed (in revolutions/minute).

Table 3: ANOVA for #5 rebar response surface quadratic model.

Source	**Sum of Squares**	**Degrees of Freedom**	**Mean Square**	**F-value**	**p-value Prob > F**	**Remarks**
Model	92.00	2	46.00	30.37	< 0.0001	significant
Spin Time (A)	39.10	1	39.10	25.82	< 0.0001	
A^2	86.73	1	86.73	57.26	< 0.0001	
Residual	25.75	17	1.51			
Lack of Fit	4.67	3	1.56	1.03	0.4082	not significant
Pure Error	21.08	14	1.51			
Total	117.75	19				

Table 4: Single-factor ANOVA for #5 rebar interaction effects.

Source	**Sum of Squares**	**Degree of Freedom**	**Mean Square**	**F-value**	**p-value**
AB (Rotations)	55.50	1	55.50	9.39	0.0052
Residual	147.69	25	5.91		
Total	203.19	26			

Table 5: ANOVA for #6 rebar response surface quadratic model.

Source	**Sum of Squares**	**Degrees of Freedom**	**Mean Square**	**F-value**	**p-value Prob > F**	**Remarks**
Model	62.74	2	31.37	15.06	0.0005	significant
Rotational Speed (A)	57.68	1	57.68	27.70	0.0002	
A^2	57.01	1	57.01	27.37	0.0002	
Residual	24.99	12	2.08			
Lack of Fit	5.66	2	2.83	1.46	0.2770	not significant
Pure Error	19.33	10	1.93			
Total	87.73	14				

Table 6: Confidence interval for #6 rebar mean pull-out load.

Response	**Mean**	**Median**	**Std Dev**	**SE Mean**	**95% CI low**	**95% CI high**	**95% TI low**	**95% TI high**
Peak Load	10.57	10.57	1.44	0.58	9.29	11.85	3.89	17.25

Table 7: Confidence interval for #5 rebar mean pull-out load.

Response	**Mean**	**Median**	**Std Dev**	**SE Mean**	**95% CI low**	**95% CI high**	**95% TI low**	**95% TI high**
Peak Load	10.3585	10.3585	1.23073	0.396396	9.52216	11.1948	5.23026	15.4867

The statistical analysis was followed by a regression analysis of significant model terms. Empirical equations in terms of significant factors can be used to make predictions about the pull-out load for given levels of each factor; however, these equations should not be used to determine the relative impact of each factor because coefficients are scaled to accommodate factor units and the intercept is not at the center of the design space. For example, the equations cannot describe the effect of single factor on the pull-out load. With A representing spin time (in seconds) and B representing rotational speed (in rpm), empirical equations are as follows:

For #5 rebar:

Peak Load = $-6.81 + 7.58 \times A - 0.81 \times A^2$…………………………..…………………… (4)

For #6 rebar:

Peak Load = $11.86 - B \times 8.08 \times 10^{-3} - 1.97 \times A + 6.88 \times 10^{-5} \times A \times B$……………………….… (5)

PULL TESTS IN UNDERGROUND COAL MINES

Pull tests at the Bowerston facility resulted in several important results. It was also important to conduct tests under *in situ* conditions to measure and understand the behavior of pull-out load, since those conditions are far consistent than that in the laboratory. In addition, they were also needed to evaluate the effect of the eccentric wrench during installation of bolts. It was intended that four different sets of tests would be conducted at *in situ* conditions on #5 and #6 rebar, and cable bolts with varying encapsulation lengths. Tests were conducted in two underground coal mines identified as Mine A and Mine B. Technical staff from Orica were present at all times during these tests and provided pivotal assistance. Bolts were installed close to each other to try to reduce the effect of immediate roof lithology change. All pull tests were conducted in out-by areas; however, test site selection was random. Tests were completed in such a manner that installations with the eccentric wrench were adjacent to those with the conventional wrench. In addition to rebar, pull tests were also conducted with cable bolts since they are widely used as secondary bolts in underground coal mines.

Underground Pull Tests at Mine A

Test conditions were as follows:

- #5 Grade 60 rebar was used in 48” lengths. Rock bolts had forged heads.
- Spin time was 4 seconds.
- Hold time was 10 seconds.
- The same bolter and operator were used for every installation.
- Rotational speed could not be recorded.
- Temperature was not recorded.
- Minova H10 resin was used in 23-mm (0.90”) diameter, 11.5” long cartridges.

- Drill hole diameter was 1” and length was one inch longer than effective rebar length (46.75”); *i.e.*, less the forged head, plate, and test collar.
- All rock bolts were pull tested one hour after installation.
- Rock bolts supplied by the manufacturer were offset from the bolt head. Thus, all the rock bolts had a resin restraint made by wrapping insulation tape around the bolt (to a thickness of at least ⅛ inch) 12” from the end of the bolt to prevent undesirable eccentric action of the bolt.
- All bolts were loaded to one ton and displacement was set to zero at the beginning of each test.

Table 8 depicts average results for the eclipse bolt, the standard bolt installed with a conventional wrench, and the standard bolt installed with an eccentric wrench. It can be seen that tests conducted with standard bolts performed approximately the same as those tested in the Bowerston facility. It was observed that these bolts were offset to some extent. The technical staff present made the logical suggestion that taping all offset bolts would restrict the eccentric action as well as would also restrict the resin to flow towards the head while spinning during the SEPT. The eclipse bolt and the standard bolt installed with the eccentric wrench performed about the same as the standard bolt installed with the conventional wrench holding equal load, which was unexpected. A possible explanation of this performance may be due to the tape wrapped on the bolt, which may have restricted the whipping action.

Table 8: Average results for pull tests conducted at Mine A.

Test Type	**Load (tons)**	**Displacement (inches)**
Eclipse Bolt	9.3	0.183
Standard Bolt	9.3	0.182
Standard Bolt (eccentric wrench)	9.3	0.248

Installed bolts were also analyzed individually to evaluate bolt stiffness consistency during pull tests. The Appendix (Figures 70-80) shows load-displacement curves. It is important to know that displacements shown in these figures are combined displacements of the bond (resin column) and deformation of the free length of the bolt since bolts are not fully encapsulated. Consistent anchorage performance can be seen for standard bolts, eclipse bolts, and standard bolts installed with an eccentric wrench.

Figure 28 shows average results of all tests at Mine A. Pull-out load is plotted vs. bond displacement (*i.e.*, grout displacement at the grout-rock interface). The stiffest system was the standard bolt followed closely by the eclipse bolt whereas the weakest system was the standard bolt installed with an eccentric wrench. It can be deduced from these results that even though the eclipse bolt with its eccentric motion performed similar to the standard bolt, the whipping action induced by it and the eccentric wrench is not significant enough to increase resin mixing quality and more controlled tests may be needed to draw firm conclusions. It can be seen that all bolt systems remain in their elastic zone. The modulus of elasticity was calculated from average results obtained from load vs. bond displacement curves and is shown in Table 9. The very low elastic modulus for the standard bolt installed with the eccentric wrench may be more the result of poorer roof conditions than other mechanical or installation factors.

Table 9: Elastic modulus determined from pull tests at Mine A.

Support System	**Elastic Modulus (GPa)**
Standard Bolt	48.66
Eclipse Bolt	44.70
Standard Bolt (with eccentric wrench)	27.70

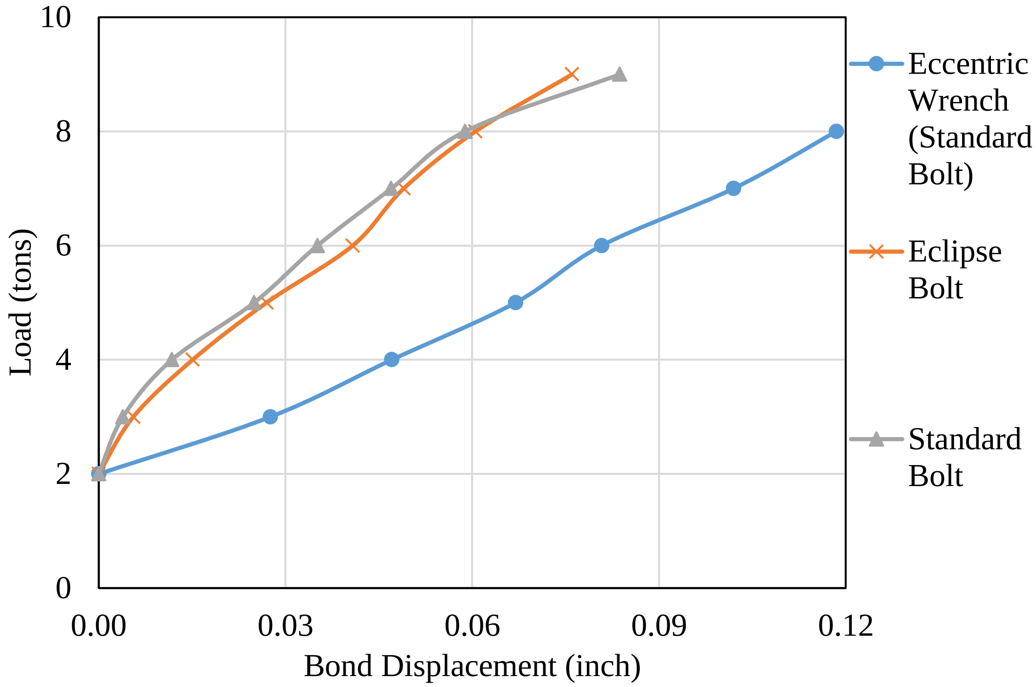

Figure 28: Average results of pull tests at Mine A.

Underground Pull Tests at Mine B – Test 1

At Mine B, conventional double-bulbed cable bolts and newly developed indented cable bolts (Figure 29) with no bulbs were tested. Both types of cable bolts were installed with a conventional wrench as well as with the eccentric wrench. Bolt performance was analyzed by plotting load-displacement curves. These tests were performed with the objective of comparing conventional installation to eccentric installation and to evaluate the effectiveness of the whipping action in cable bolts.

Figure 29: Indented cable bolt

Test conditions were as follows:

- Both types of cable bolts had lengths of 72” and a nominal diameter of 0.6”.
- Resin encapsulation length was 30” (2.5’). This length was selected to fully encapsulate one cable bulb. The presence of the second bulb makes displacement readings unrealistic as that bulb elongates and closes under load.
- Spin time was planned for 6 seconds.
- Hold time was 30 seconds.
- Rotational speed could not be recorded.
- Temperature was not recorded.
- The same bolter and two operators (one on each side) were used for every installation.
- Minova VLIF50 (Very Low Insertion Force) resin was used in 23-mm diameter, 30” long cartridges.
- Drill hole diameter was 1” and length was 69.75” (*i.e.*, less the forged head, plate, and test collar and with 1” extra at the back of the hole).
- All rock bolts were pull tested 1.25 hours after installation. This was to accommodate the number of cables tested in as short a time as possible.
- The cable bolts are less stiff than the rebars. Thus, all bolts were loaded to two tons and displacement was set to zero at the beginning of each test.

On inspection of the test site, it was observed that the immediate roof was very weak and fractured with fractures going about 1’ up into the immediate roof. Moreover, indented cable bolts were installed at a weaker site compared to where bulbed cable bolts were installed based on visual observations.

Table 10 presents average pull test results. It can be deduced from these results that 30" encapsulation length exceeded cable bolt capacity providing no comparison on the basis of pull-out load. Individual test results depict that these tests were consistent. The small deviation within the results may have been caused by lithology variability as well as the strength of the roof.

As per the cable bolt manufacturer, elastic modulus values for bulbed and indented cable bolts are 10,080 ksi (low due to the presence of the bulb) and 28,282 ksi, respectively. Elastic moduli values were used to calculate system bond displacement. Results from these calculations, shown in Figure 30, indicate that the bulbed cable bolt performed better than the indented cable bolt. The performances of indented cable bolts installed with a conventional wrench, bulbed cable bolts installed with a conventional wrench, and bulbed cable bolts installed with an eccentric wrench were similar. The performance of indented cable bolts installed with an eccentric wrench was worse. However, on comparing installation procedures, conventional wrench installations resulted in much stiffer systems than eccentric wrench installations. The modulus of elasticity was calculated from average results obtained from load vs. bond displacement curves and is shown in Table 11.

Table 10: Average results for cable bolt pull tests conducted at Mine B.

Test Type	**No. of Tests**	**Load (tons)**	**Displacement (inches)**
Indented cable bolts	3	No failure at 20 t 1 cable retested and failed at 24t	0.684 Not measured
Indented cable bolts (eccentric wrench used)	2	No failure at 20 t 1 cable retested and no failure at 25t	1.057 Not measured
Bulbed cable bolts	4	3 tests - No failure at 20 t 1 test - Failure at 19t 1 cable retested and no failure at 25t	0.629 0.693 Not measured
Bulbed cable bolts (eccentric wrench used)	3	No failure at 20 t 1 cable retested and no failure at 25t	0.594 Not measured

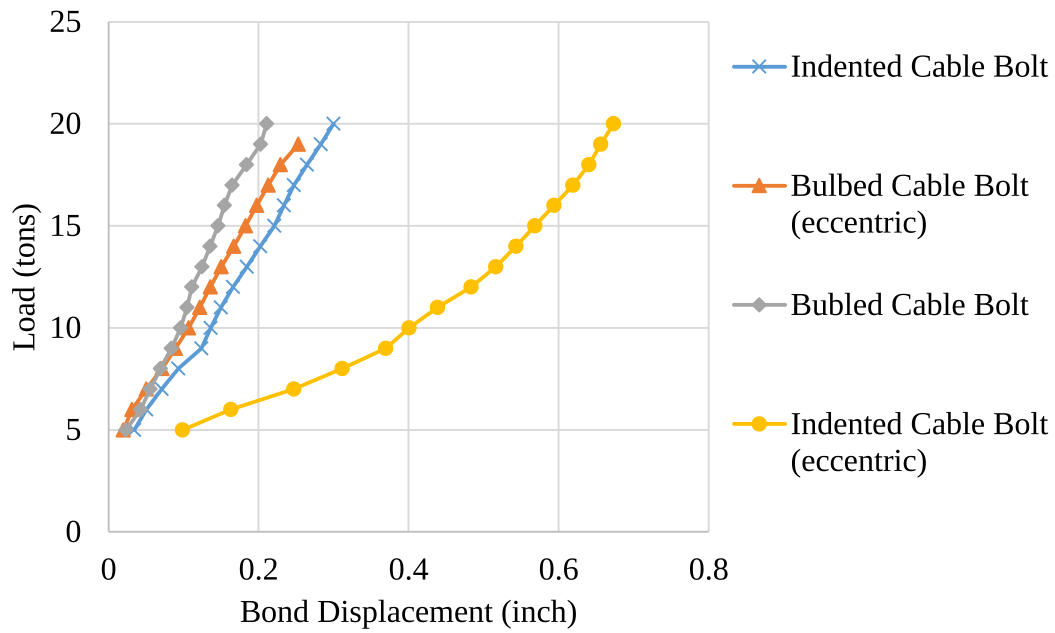

Figure 30: Average results of cable bolt pull tests at Mine B.

Table 11: Elastic modulus determined from cable bolt pull tests at Mine B.

Support System	**Elastic Modulus (GPa)**
Indented Cable Bolt	83.21
Indented Cable Bolt (eccentric installation)	37.89
Bulbed Cable Bolt	123.33
Bulbed Cable Bolt (eccentric installation)	89.52

Underground Pull Tests at Mine B – Test 2

The second series of tests in Mine B consisted of installing indented cable bolts manufactured with either button or welded tips at the end. Testing compared the performance of these two products to see the whether the welded tip would perform equally or better than the button tip because the button tip costs more than the welded tip to the manufacturer Also, testing was required for the newly developed indented cable bolts manufactured with bulbs before they could be certified for use in mines. Finally, conventional and eccentric installation methods were again compared.

Test conditions were as follows:

- Spin time was planned for 6 seconds.
- Hold time was 30 seconds.
- The same bolter and operator were used for every installation.
- Rotational speed could not be recorded.
- Temperature was not recorded.
- Minova VLIF50 resin was used with 23-mm diameter cartridges and a resin encapsulation length of 20.5”. This was as small as possible while still completely covering the bulb with the resin column.

- Drill hole diameter was 1” and length was 70.25” (*i.e.*, bolt length less the forged cable head, plate, and test collar plus 1” extra at the back of the hole).
- All bolts were loaded to two tons and displacement was set to zero at the beginning of each test. This is higher than the rebar because rebar is stiffer than cable bolts and the initial loading eliminates the error caused by sudden loading to the system.

For these tests, a miscommunication occurred with the manufacturer and pull tests had to be conducted at different times after installation. The grip used with the hydraulic puller had a maximum width, which was slightly less than the forged head diameter on cable bolts. Thus, cable bolts specifically meant to be pull tested were specially ground down at the factory before delivery and installation. This did not happen in this case so it took lots of time to fit the grip in the hydraulic puller as a result of which, testing time had to be increased significantly.

The inclusion of a bulb increased pull-out load performance by 20%. Individual tests produced consistent force-displacement curves. Installation with an eccentric wrench resulted in low anchor stiffness, which was also shown in previous tests. The indented cable bolt with double bulbs installed by conventional means outperformed other bolts on the basis of stiffness (Figure 31).

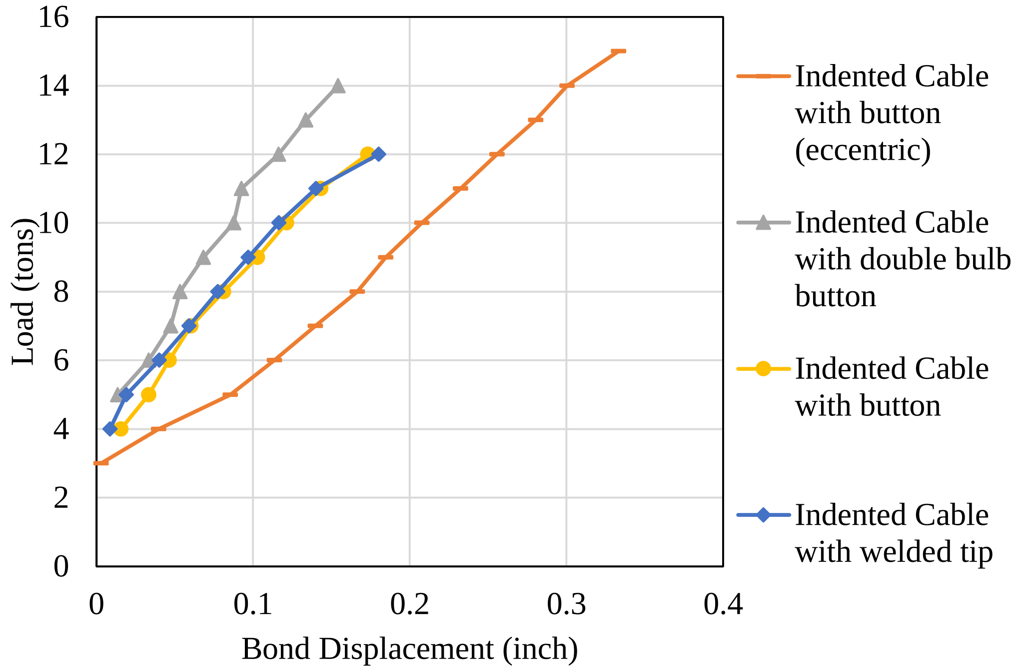

Figure 31: Average results from second series of pull tests at Mine B.

Five tests resulted in sudden, violent failures, which could indicate a cable strand failure or failure of the mechanical key between the rock and the resin column; however, neither could be confirmed. These bolts were pulled again, but that resulted in a much lower peak load. Premature failure of these bolt may be caused by poor or inefficient resin mixing resulting in mechanical key failure and low stiffness. Two tests, 14 and 15, caused the gauge to fall to the ground due to the violent nature of the failures. Thus, the installation procedure was changed such that the bolt was rotated an inch or two prior to the head and plate reaching the roof thereby providing some eccentric action, followed by spinning the head and plate against the roof. Results from pull tests with this new procedure (Figure 32) were very encouraging, but more testing is needed to be definitive about the performance. Better performance was achieved when indented cable bolts were bulbed due to taking advantage of both bulb and indent at the same time. Comparing installation methods, results suggest a stiffer system with higher pull-out loads was obtained with eccentric installation vs. conventional installation.

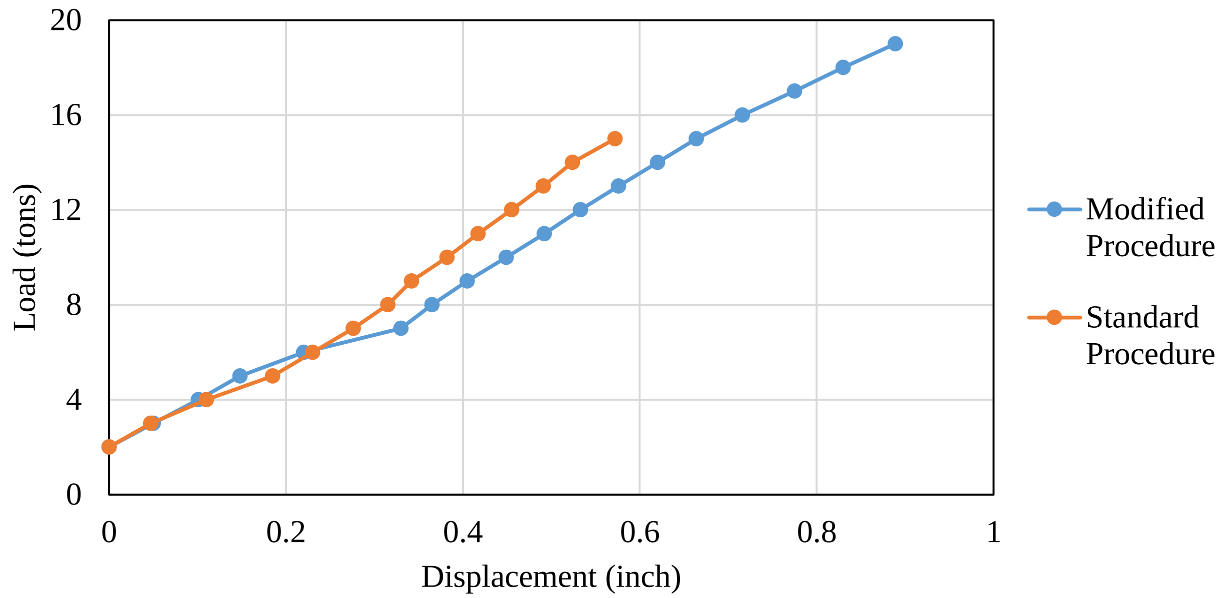

Figure 32: Pull test results for cable bolts installed with modified installation procedure.

Underground Pull Tests at Mine B – Test 3

SEPTs were performed with #5 rebar and indented cable bolts to compare installation with the eccentric wrench vs. the conventional wrench. Table 12, Figure 33 and Figure 34 show average results. Cable bolts and #5 rebar installed with the eccentric wrench performed about the same as those with conventional installation, which indicates no significant effect from the eccentric wrench installation. It was also surprising to note that the best performance was achieved with a sub-optimum resin annulus. The reason for premature failure in Test 11 can be explained by poor installation (inadequate spin time) resulting in poor resin mixing. However, spin time inadequacy in eccentric wrench installations is compensated for by whipping action, which explains better performance for eccentric installations vs. conventional installations at the same spin time. None of the #5 rebar failed at 10 tons, but it can still be deduced that conventional installation provided a stiffer system than eccentric installation.

Table 12: Average results for Mine B – Test 3 pull tests.

Test Type	Test No.	Time before Test (mins)	Load (tons)	Average Load (tons)	Displacement (inches)
Indented cables with button at the end (eccentric installation)	2 4 6	62 60 60	8 5(#) 13	8.66	0.215 0.114 0.278
Indented cables with a button at the end	7 9 11	60 60 60	4 4(#) 2(#)	3.66	0.098 0.107 **
#5 Rebar (eccentric installation)	8 10 12	60 60 60	10* 10*(#) 10*	10*	0.244 0.218 0.232
#5 rebar	1 3 5	60 60 60	10* 10*(#) 10*	10*	0.202 0.205 0.205

Bolts were held for five additional seconds
* No failure observed during testing
** Premature failure during initial loading due to poor installation

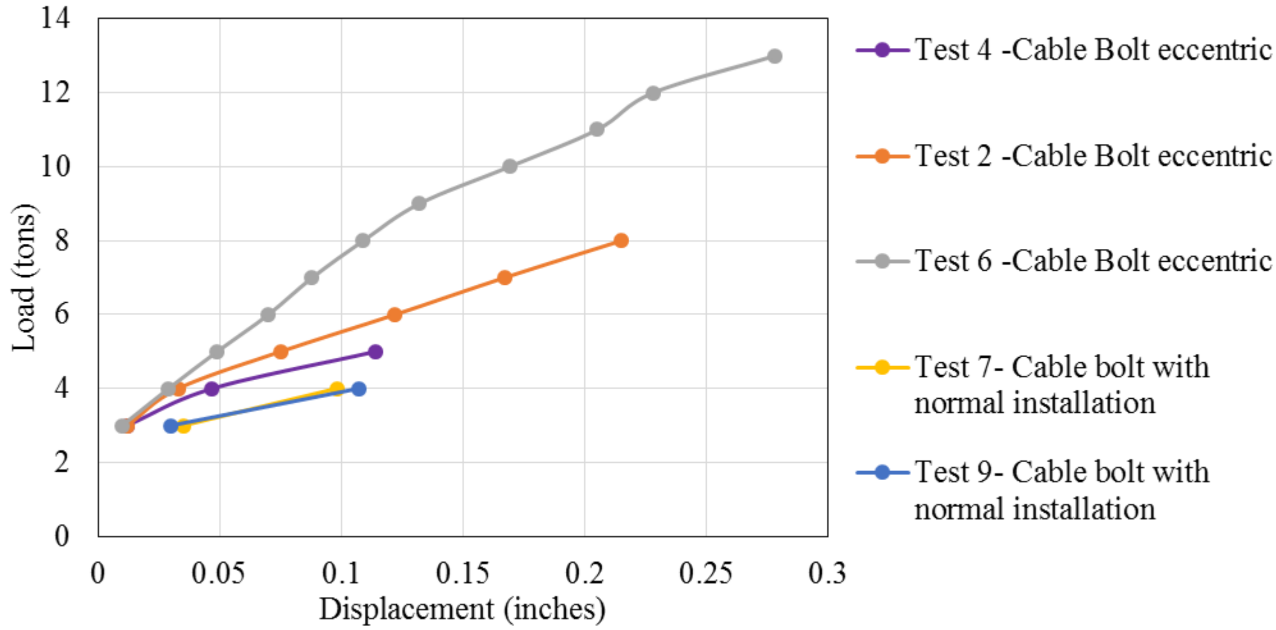

Figure 33: Pull tests results for cable bolts installed with eccentric wrench vs. standard wrench.

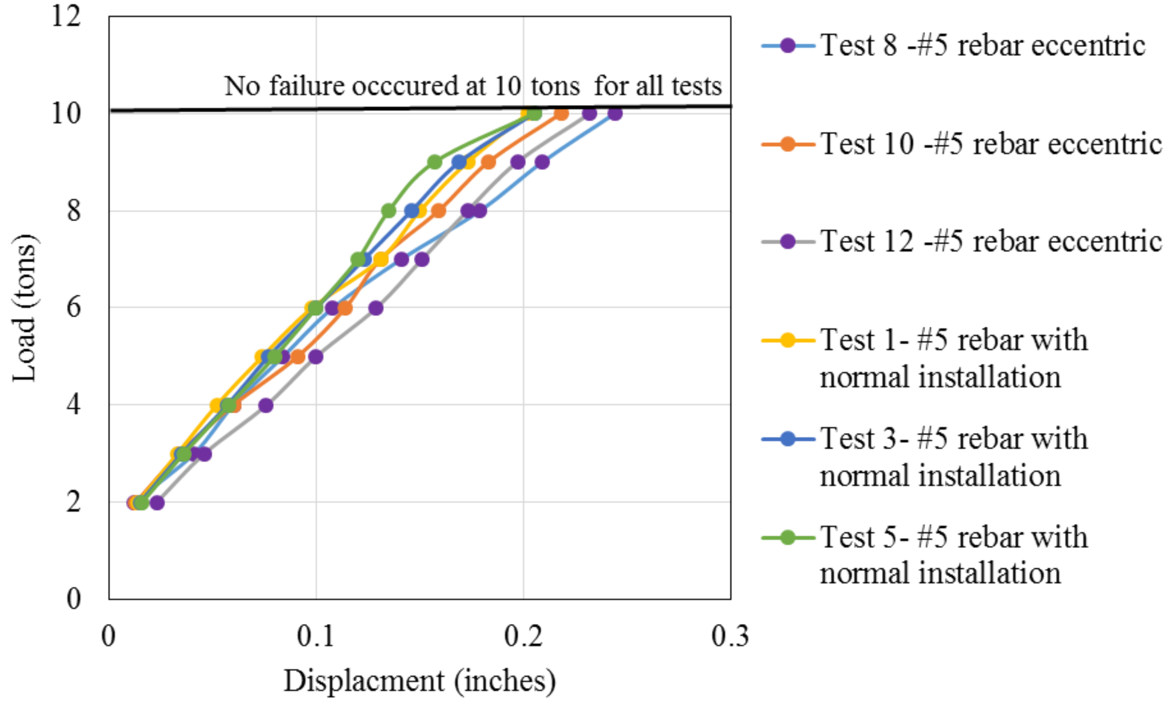

Figure 34: Pull tests results for #5 rebar installed with eccentric wrench vs. standard wrench.

SMALL-SCALE LABORATORY MODIFIED DRILL PRESS TESTS

Spearing and Mishra (2014) described the testing done for this step as follows:

"Large-scale laboratory tests might be easier, better controlled, and safer than in situ pull tests, but they are still time and resource consuming. For example, the four concrete block sets, which are used in the large-scale tests at Orica, must be poured and cured for about 28 days before use. Only 9 tests can be completed in each concrete block, and a maximum of 18 to 27 tests can be completed in a day, with a minimum of 3 people are needed. A small-scale laboratory test that can be done quickly, easily, and economically, is therefore, desirable to qualitatively represent short encapsulation tests.

"At Southern Illinois University Carbondale (SIUC) a drill press was modified, in order to test its suitability as a test rig that would give results similar to those obtained with large-scale short encapsulation tests. This method would not be used as a replacement test, but to initially establish trends, that could be later quantified with limited full-scale laboratory short encapsulation tests. The equipment selected for these tests comprised of a used drill press that was modified [Figure 35 and Figure 36] *and a steel block that had a 1" hole at the center. This hole could be split into two halves axially along the diameter of the hole [Figure 37].*

"A mechanical jack was fitted at the base of the drill press where the split steel block could be mounted. The jack was useful because it increased the vertical travel by 12," which helped in the insertion of rebar into the simulated drill hole in the steel block. The vertical travel of the stock drill press chuck was

only around 5." After resin mixing, the cured resin column was left to strengthen. It was then taken out of the steel block [Figure 37].

"Earlier unpublished tests by Spearing (2010) indicated that resin hardness appeared related to short encapsulation pull-out loads. The higher the average resin hardness of the resin column, the higher the pull-out load. The resin hardness can be measured using a durometer [Figure 38]. *A durometer is used to measure the hardness of mixed resins. The hardness is obtained by pushing a steel pin into the resin a fixed distance. Depending on the hardness of the resin, a smaller or larger force is needed, which is recorded on* the *graduated scale. Hardness measurements are taken along the entire length of the resin column at regular intervals."*

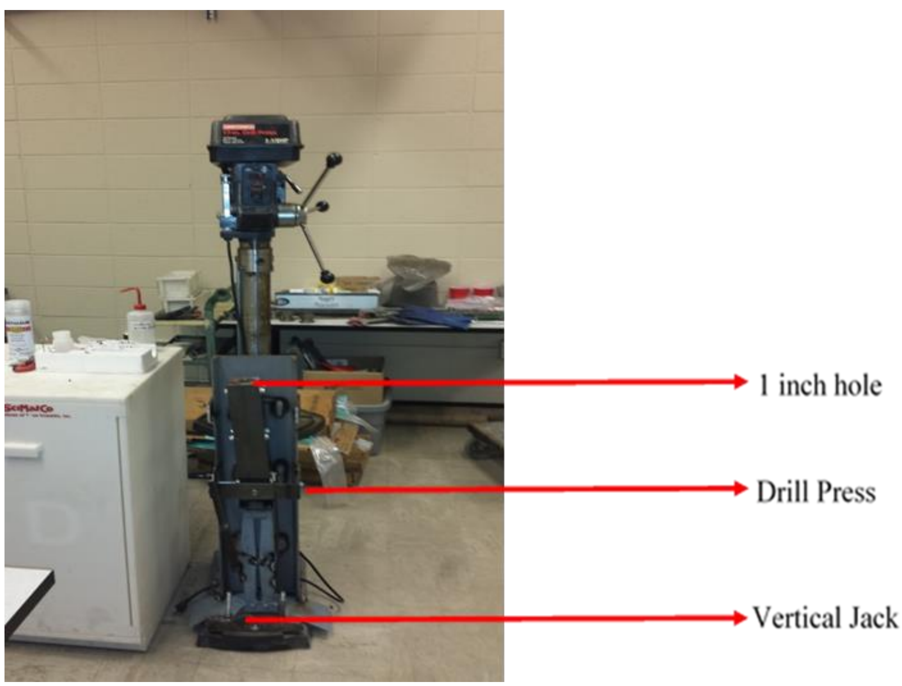

Figure 35: Modified drill press (front view).

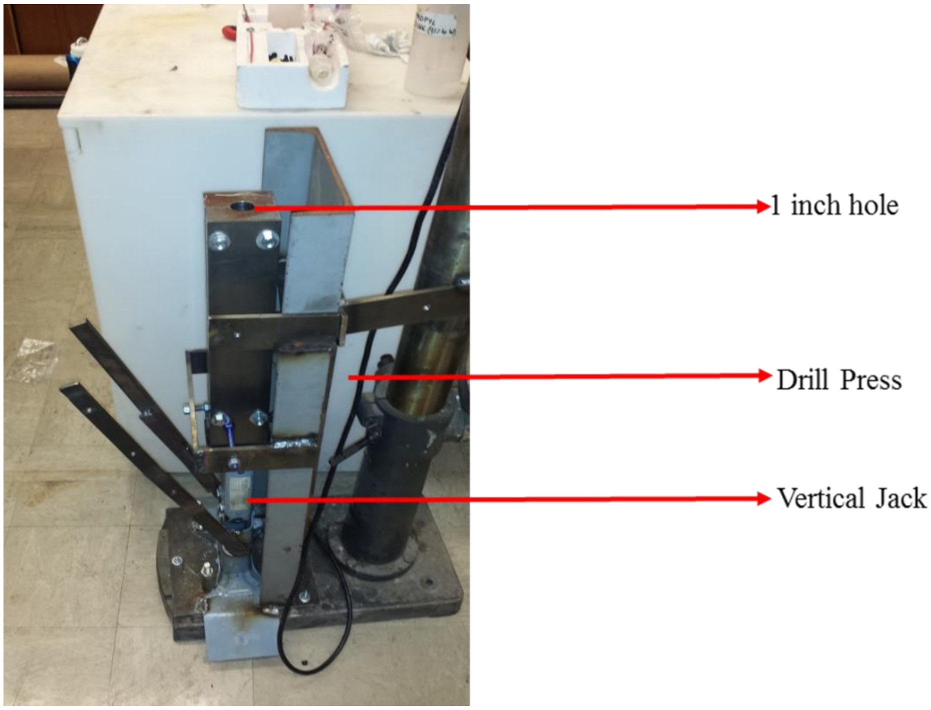

Figure 36: Modified drill press (side view).

Figure 37: Split steel block.

Figure 38: Durometer reading.

First Test Series

Since a new test method and equipment was being designed to evaluate resin mixing, it was required to undertake some initial test data for validation and to check on repeatability. Three tests were conducted in the laboratory with the use of #6 rebar grouted with H10 resin at an intended encapsulation length of 6” in a hole of 1” diameter. The rotation of the drill press was 580 rpm because the drill speed is belt controlled and 580 rpm is closest to the 540 rpm used at Bowerston. Spin time was set to 6 seconds including the delay for the drill to reach 580 rpm and a couple of residual turns when turned off. The hardness of each specimen was measured with a durometer after 1 hour. Results are shown in Table 13.

Table 13: Average results of all preliminary modified drill press tests.

Test	**Durometer Reading**	**Durometer Reading(90° apart)**
1	69.75	67.25
2	68.87	74.75
3	53.40	69.40

The following observations were made based on these preliminary tests:

- The steel block shook with the rebar during rotation of the drill press.
- There was a time delay of 1 second between insertion of bolt into the hole and starting the drill press motor.
- There was a drag of about 2-3 rotations after the drill press was switched off after instllation.

The amount of grease applied on the steel block varies with each test affecting resin mixing and hardness (Figure 39).

Figure 39: Involvement of grease affecting resin mixing.

Resin mixing during initial testing was found to be inconsistent (Figure 40) making it necessary to identify the sources of error and take appropriate steps to reduce them. Primarily there were two sources of error which can explain inconsistencies in resin mixing. The first was the shaking of the steel block with respect to the rebar during rotational action after insertion. This resulted in non-uniform mixing, giving an undesirable encapsulation length. The second was the amount of grease applied on the steel block which delays or stops the curing process of the mixed resin. Attempts were made to reduce these errors. To reduce shaking, a guide was placed on the front side and bolts on either side of the steel block, which hindered movement of the steel block in all directions with respect to the rebar. To reduce the effects of grease, a specified amount of grease was applied for every experiment to insure consistency of results.

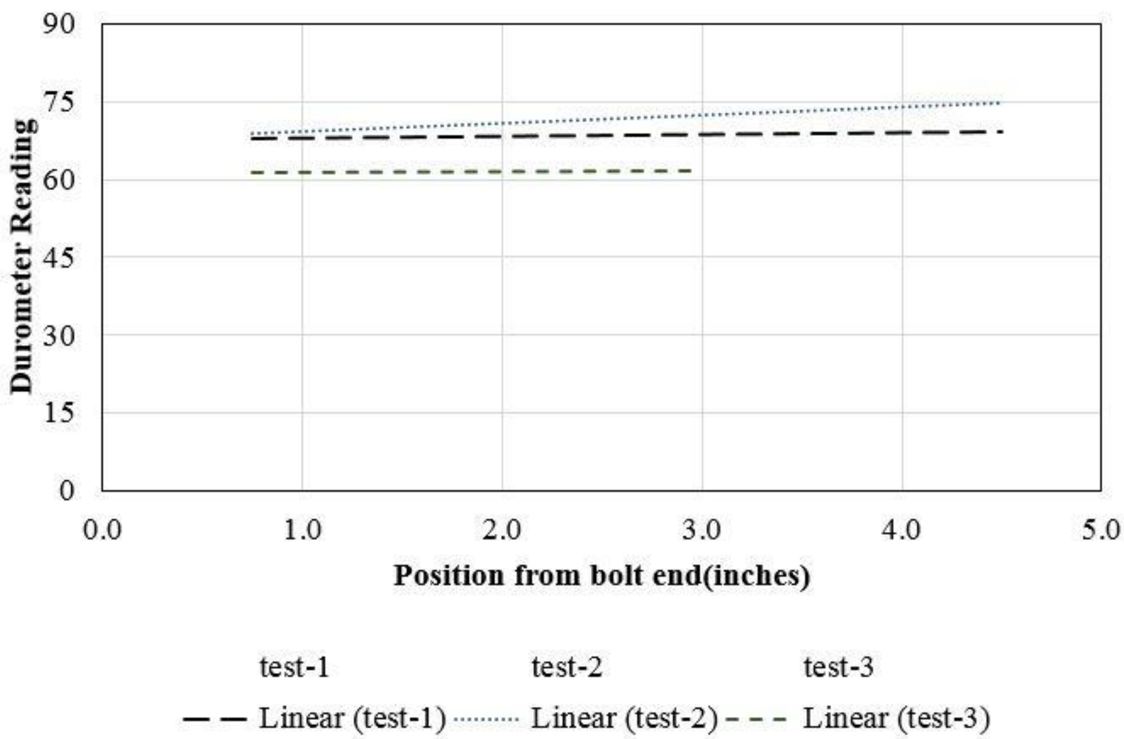

Figure 40: Durometer reading at different positions using best linear fit.

During initial tests with the drill rig and split steel block, not using a mold release proved to be a problem because resin adhered to the steel block. After several trials with different mold release systems, a small amount of grease was applied on the entire surface of both steel halves.

This was then covered with a very thin plastic sheet completely eliminating any contact between steel and resin. This drastically and consistently reduced the adhesion effect.

Another practical constraint was that the 3-horsepower motor on the drill press was inadequate to insert rebar in the hole with a resin cartridge and achieve 1’ encapsulation length for any resin formulation. Therefore, 6” encapsulation length had to be used.

Second Test Series- Effect of Spin Time and Type of Resin

Pull tests of #6 and #5 rebar were conducted at Orica’s Innovation and Technology Center in Bowerston, OH. Some part of these test were conducted at 540 rpm with spin times of 2, 3, 4, and 7 seconds. The performance of these tests was evaluated by plotting the relation between load and spin time as shown in Figure 41 and Figure 42. Matching these results was the target of 20 tests (5 tests with each spin time) conducted in the laboratory grouting #6 rebar with H10, M35, and H20 resins at an intended 6” encapsulation length in a 1’ hole. However, 2 tests with 7 seconds as spin time were could not be analyzed due to the experimental error. Due to resin viscosity, the drill press required time to reach 580 rpm when started, then gave a couple of residual turns when turned off due to inertia caused by a sudden change in momentum. These errors are assumed constant in all tests, *i.e.*, they do not affect consistency of results. For each test #6 rebar was grouted with either H20, H10, and M35 resin at a particular spin time. The resin manufacturer recommends a spin time in the range of 2 to 3 seconds. Thus, four spin times were chosen, *i.e.*, 2, 3, 4, and 7 seconds to match the Bowerston tests.

Results from the second test series (Figures 41 and 43) were found to be consistent with that of Bowerston pull tests as well as the first drill press test series done with H10 resin cartridges. M35 resin performed better than H10 resin on the basis of hardness (*i.e.*, durometer reading). H20 resin was harder than H10 for 2- and 3-second spin times; then H10 resin was harder than H20 resin for 4- and 7-second spin times. However, the peak always occurred at 4 seconds for all resin types.

It was regularly observed that even if only 6” encapsulation length was planned, 0.5-2.0” of resin does not cure. The reason for this is thought to be admixing of grease, which can be considered a constant for each test. The causes for decrease in hardness from 2 seconds to 3 seconds are unknown (Figure 42 and Figure 43) and more tests were needed which were beyond the scope of the thesis.

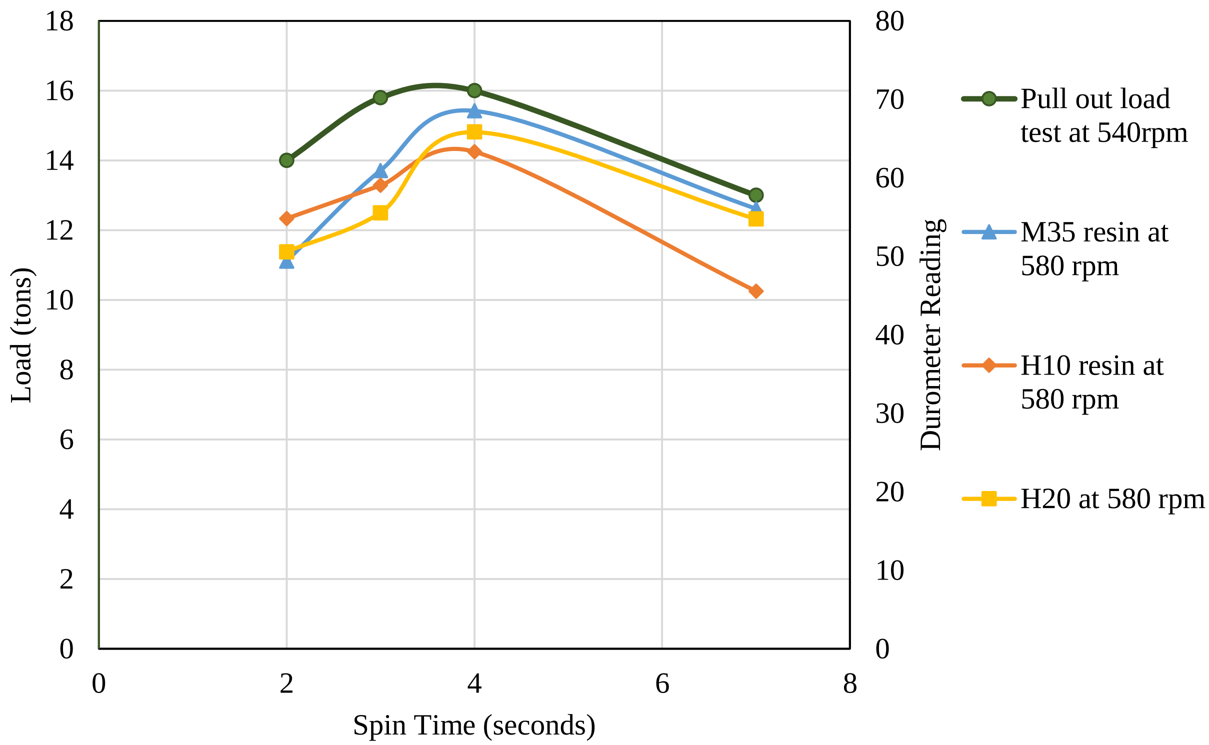

Figure 41: Hardness vs. pull-out load with respect to spin time and different resin types for #6 rebar.

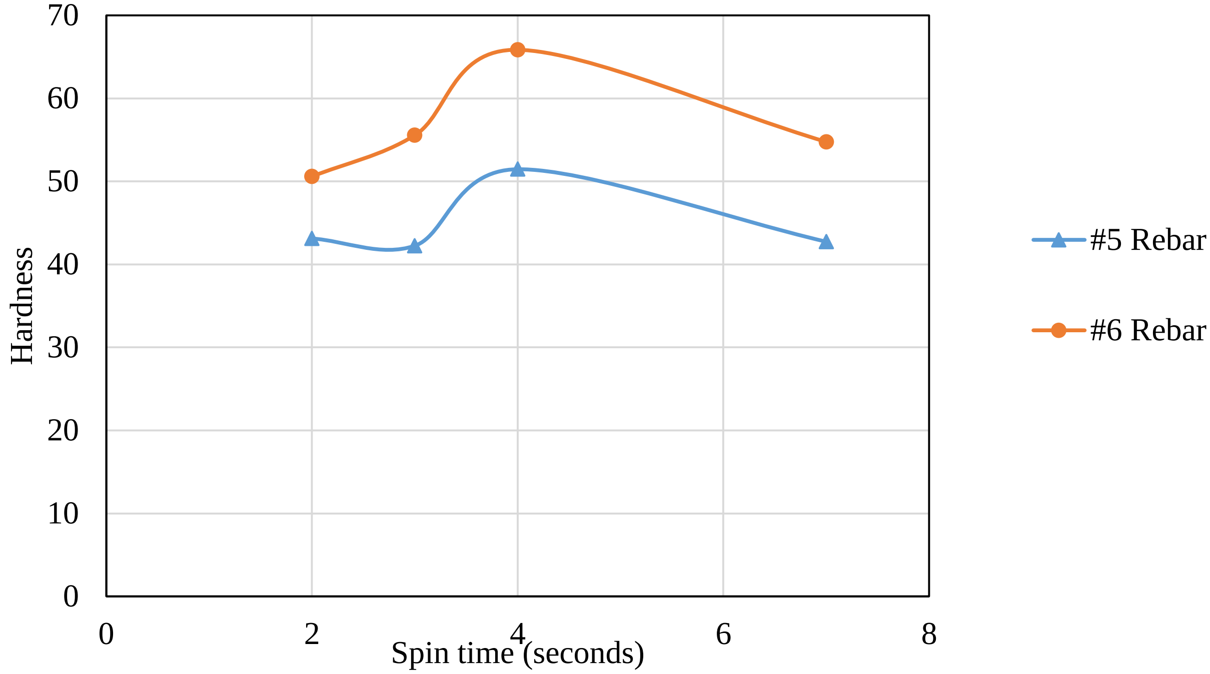

Figure 42: Hardness comparison for #5 and #6 rebar at 540 rpm.

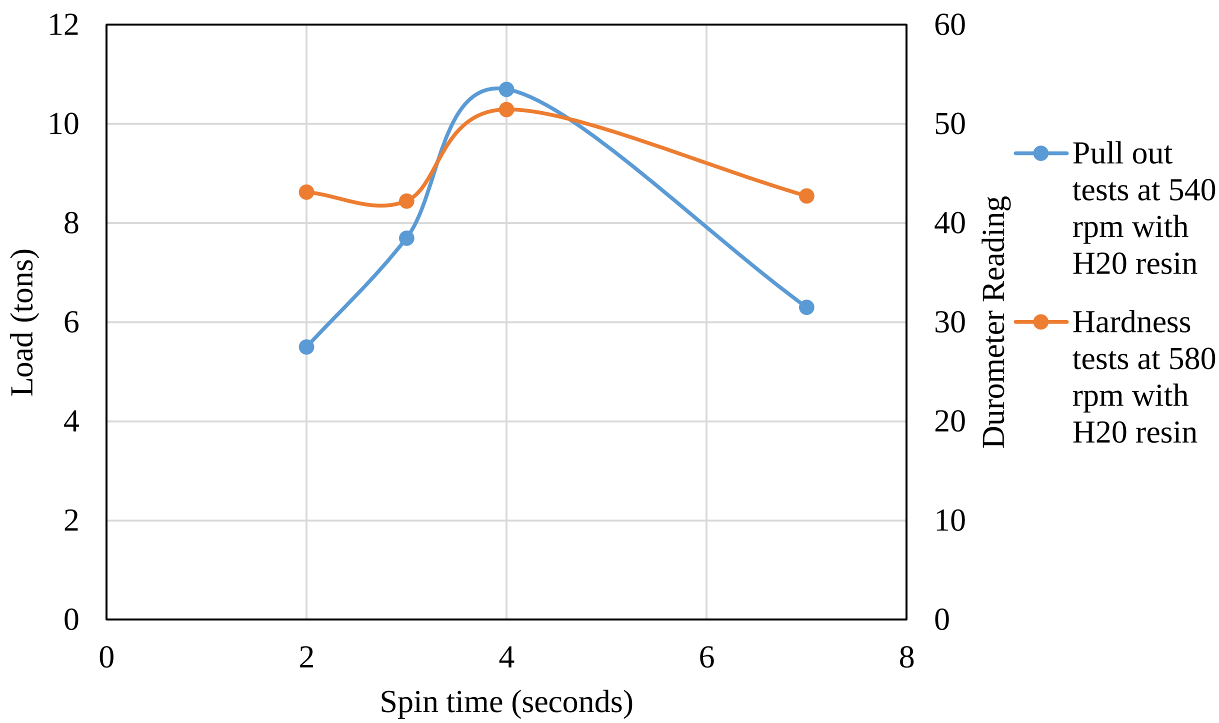

Figure 43: Hardness comparison between pull tests and drill press for #5 rebar.

Third Test Series – Effect of Rotational Speed with Constant Number of Rotations

Pull tests were conducted under similar conditions with H20 and LIF20 resin cartridges to understand and measure the effect of rotational speed. The objective of these tests was to compare resin hardness following pull tests conducted at a constant number of rotations, but with different rotational speeds and spin times. A total of fifteen tests were conducted at 290, 350, and 650 rpm with 7-, 6-, and 3-seconds spin times, respectively (Table 144). These rotational speeds provided the closest match to rotational speeds used for pull tests in Orica's Bowerston Laboratory. Spin times were chosen so as to be close to the number of rotations used in previous hardness tests at which the peak was attained at 580 rpm rotational speed and 4-second spin time. This rotational speed and spin time could not be exactly matched with the drill press and its belt and pulley driven motor due to a limited selection of pulley sizes. Thus, tests at 580 rpm were not conducted in this test series, but data from the previous tests series (for H20 resin cartridges) was included in comparisons made during this test series.

Table 14: Common parameters for all tests in third test series.

Bolt diameter	0.75" (#6 rebar)
Bolt length	25.25"
Encapsulation length intended	6"
Spin time (average of 4) with rotational speed	7 seconds @ 290rpm 6 seconds @ 350rpm 3 seconds @ 650rpm
Number of rotations (including an estimate of drill drag)	39 rotations + 3-4 residual turns
Bolt tangential speed	1.81, 2.18, and 4.06 inch/sec
Annulus thickness ([Hole diameter-Bolt diameter]/2)	0.125"

Some results from these tests were found to be consistent with pull test results from Bowerston (Table 15 and Figure 44). Results from these hardness tests compare favorably with pull test results even when different bolt installation parameters were tested. Again, it was observed that even if only 6” encapsulation length was planned, 0.5-2.0” of resin does not cure due to admixing of grease, which can be considered a constant for each test.

Table 15: Average results for rotational speed analysis.

Number of Tests	Rotational Speed (rpm)	Spin Time (seconds)	Average Durometer Reading
5	290	7	58.97
5	350	6	50.28
5	650	3	51.77
5	580	4	65.85

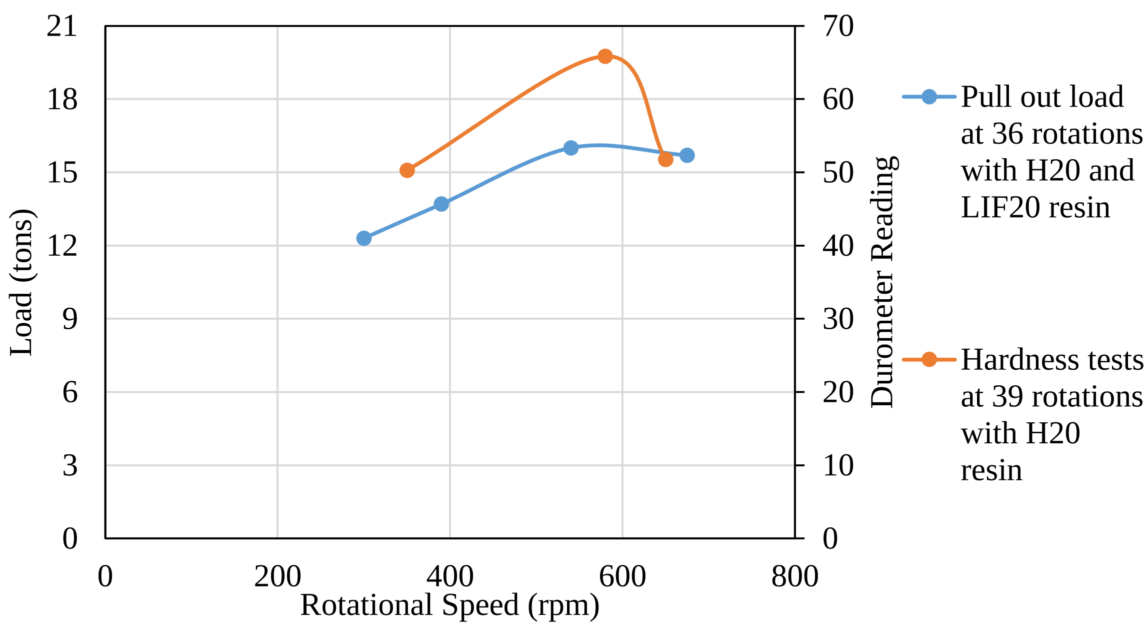

Figure 44: Pull-out load and hardness vs. rotational speed at constant number of rotations.

GLOVE FINGERING INVESTIGATION AT CARTERVILLE FACILITY

The resin used to grout rock bolts is packaged in plastic film in the form of a capsule. It has been found that sometimes during installation, the rotating bolt does not fully shred the cartridge packaging creating a plastic interface between the resin column and rock around the drill hole. This effect can reduce the mechanical key at the resin-rock interface – a phenomenon called glove fingering. This phenomenon is the subject of debate (Pettibone, January 1, 1987) and the exact reasons for its occurrence are still not fully known.

An insertion rig was built in the High Bay Laboratory at the Illinois Coal Development Park in Carterville, Illinois (Figure 45) to study the bolt installation process. It was used to install fully grouted rock bolts into steel pipes that were afterwards split open to investigate resin consistency. The test rig has the capability to install 4’ rock bolts (rebar or cable) into 4’ steel pipes simulating drill holes in coal mines. The installation is carried out by a hydraulically powered motor with an up-thrust capacity of 12,370 lbs.

The test procedure used two kinds of pipe – one thick-walled (1.375” internal diameter) and one thin-walled (1” internal diameter). The thick-wall pipe was pre-cut diametrically along its length. Bolts were inserting into the thin-walled pipe (Figure 46(a)), which was not cut until after bolt insertion. The thin-walled pipe was placed in the thick-walled pipe (Figure 46(b)), which was secured with U-clamps to prevent premature failure of the thin-walled pipe during bolt installation and pull testing. This arrangement was necessary because the thin-walled pipe was not robust enough to withstand thrust forces pull testing while it would have been cost prohibitive and time consuming to cut thick-walled pipe after bolt insertion. After resin curing, the thin-walled pipe was split into two halves to examine resin consistency. This was done by cutting it diametrically with a radial saw.

Figure 45: Test insertion rig.

(a) (b)

Figure 46: (a) Rebar with pull collar and plate to be installed in thin-walled pipe;

(b) Pre-split thick-walled pipe being fitted around thin-walled pipe for testing.

The testing procedure is summarized as follows:

1. Grout rebar in pipe using insertion rig. Bolts were inserted into the thin-walled pipe and rotated for a fixed length of the time (*i.e.*, spin time).
2. Conduct pull test to failure, then remove thick-walled pipe and split thin-walled pipe into two halves with a radial saw.
3. Measure hardness of exposed resin column with a durometer.

All resin used in this series of tests came from the same batch to insure consistency. Test conditions are described in Table 16 and as follows:

- Each thin-walled pipe had hole diameter of 1" and length of 48".
- The length of the forged head, plate, and test collar combined was 2.75", which left that same length of empty space at the back end of the pipe after bolt insertion.
- Bolts were loaded to 1 ton before starting the pull test.
- Pull tests were conducted about 75 minutes after bolt installation.
- For safety, pull tests were terminated at 16 tons for #6 rebar and 10 tons for #5 rebar.

Table 16: Test conditions used to examine glove fingering.

Bolt Diameter	**Spin Time (sec)**	**Temperature**	**Resin Type (Diameter)**	**Encapsulation Length**
#6 (0.625")	5	Not recorded	2-speed MLIF10-50 (0.90")	15", 30", and 48"
#5 (0.750")	5	Not recorded	M35 (0.90")	15" and 48"

Upon splitting thin-walled pipes, glove fingering was examined (Figure 47 and 48). Glove fingering lengths shown in Figure 48 were measured from the end of the bolt. A significant change in glove fingering was not observed for different encapsulation lengths, but it can be still deduced that at higher encapsulation length (30” and 48”), less glove fingering occurs as compared to lower encapsulation length (15”). It was also observed that the interference of shredded plastic is a major concern with regard to resin anchorage strength. It reduces the effective interlocking between bolt and cured resin as it was seen that a significant portion of the cured resin was damaged wherever interference had taken place. Damage to the cured resin in Test 5 was so severe that the durometer test could not be performed (Table 17).

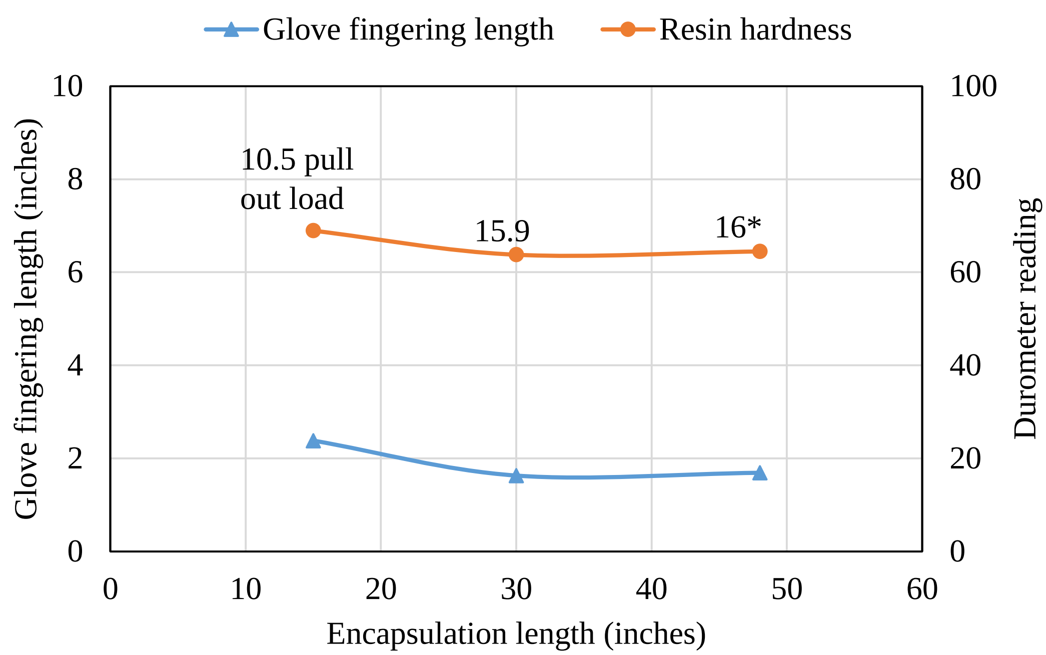

Figure 47: Hardness and glove fingering length vs. encapsulation length with #6 rebar.

All #6 bolts having 15” encapsulation failed in pull tests at an average load of 10.5 tons apparently from resin anchorage failure. Failure at higher encapsulation lengths was not seen confirming the increase in anchorage strength with increase in encapsulation length. However, none of the #5 bolts failed at 10 tons for either 15” or 48” encapsulation length with the reasons unknown. As with #6 rebar, tests with #5 rebar showed that glove fingering varies proportionally to hardness (Figure 48). However, more tests are required to get conclusive results.

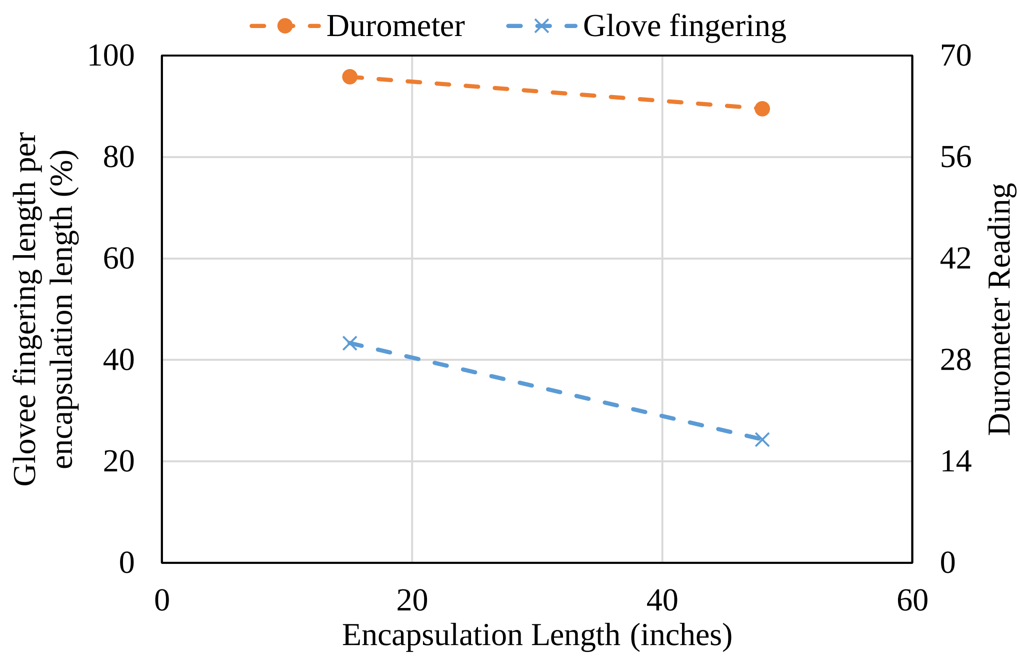

Figure 48: Glove fingering and hardness vs. encapsulation length with #5 rebar.

As described in paper from Spearing and Mishra (2014):

"Resin hardness was measured with a durometer on each bolt at 0.5" intervals from the end where the line of cured resin starts. Two sets of reading were taken, one at 90° from the other to check for consistency. Interestingly, hardness reading seemed to follow the same trend as glove fingering; however, this comparison is still in its initial stages. With hardness reading as the calibrated measure, it was evident that good mixing is obtained at lower encapsulation (15") compared to higher encapsulation (30" and 48"). Table 17 provides average results. Those marked with "" had not failed when terminated for safety reasons (i.e., before rebar tensile strength was exceeded)."* "N.A."

denotes that durometer readings could not be measured because some of the resin column was broken or had adhered to the steel pipe (Figure 49).

Table 17: Average results from glove fingering investigation with #6 rebar ('*' indicate average).

Test	Resin Cartridge Length (inches)	Nominal Encapsulation Length (inches)	Spin Time (seconds)	Average Peak Load (tons)	Average No. of Revolutions on Installation	Average Durometer Reading	Glove-fingering Length (inches)
1	12	15	5	16*	31.30	73.06	3.25
2	12	15	5	12	32.70	70.33	3.00
3	12	15	5	7	28.80	68.00	0.00
4	12	15	5	7	29.70	64.36	3.25
*1-4	12	15	5	10.5	30.60	68.94	2.38
5	24	30	5	16*	33.70	N.A.	3.00
6	24	30	5	16*	29.50	67.20	0.00
7	24	30	5	16*	29.70	66.26	1.25
8	24	30	5	15.5*	28.60	57.87	2.25
*5-8	24	30	5	15.9	30.40	63.78	1.63
9	33	48	5	16*	31.10	63.49	2.75
10	33	48	5	16*	30.60	61.02	0.00
11	33	48	5	16*	28.40	66.05	2.00
12	33	48	5	16*	30.80	67.40	2.00
*9-12	33	48	5	16*	30.20	64.49	1.69

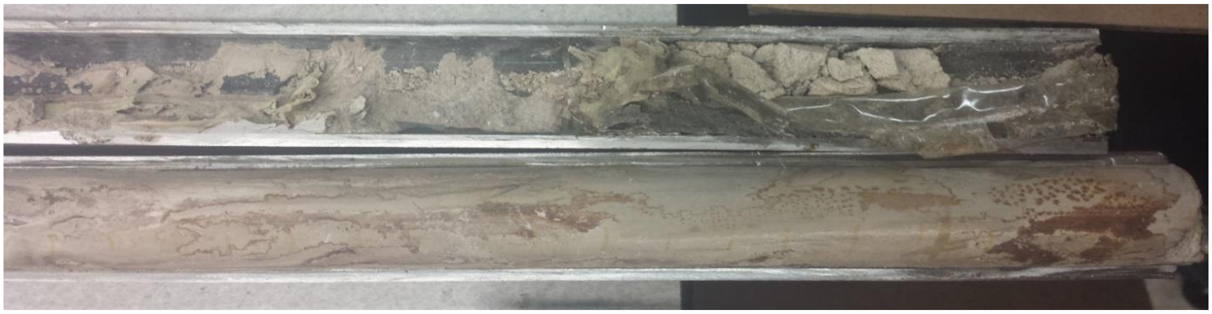

Figure 49: Resin adhering to pipe.

CREEP BEHAVIOR OF TENSIONED GROUTED REBAR

Active bolts are tensioned during installation whilst no (or limited) tension is applied to passive bolts, which need subsequent strata movement to generate reactive loads. During installation of passive bolts, some reactive pre-load may be generated by a significant up-thrust from the rock bolter boom. The selection of active or passive bolts is critical and mostly dependent on mine conditions and properties of the immediate roof (Unrug, 2009). This decision is frequently made based on personal experience and bias. Active bolts are generally used in cases of highly laminated roof rock. When tensioned, they develop compressive forces between these layers forming a beam, which helps to stabilize weak individual layers thereby increasing frictional forces between layers and decreasing effects of horizontal stresses. The fully grouted tensioned rebar system is high in stiffness as compared to mechanical bolts and provides higher surface area of contact along the length of the bolt due to the grout, which permits it to develop a strong anchorage mechanism.

Studies conducted at two room-and-pillar coal mines in the Illinois Basin and one longwall mine in Colorado to evaluate the performance of fully grouted tensioned rebar (FGTR), fully grouted passive rebar (FGPR), and resin-assisted mechanical rebar (RMAB) showed that under similar geologic and mining conditions, no significant pre-load was observed (Spearing *et al*., 2011). Initial readings at the two room-and-pillar mines were delayed as data loggers were not intrinsically safe. When data collection began, the active tensioned rebar showed loss of initial load (Figure 50). While exact reasons for such losses were not investigated in this study, it was postulated that creep could be the cause; however, this was not determined.

Creep is the tendency of a solid material to deform permanently under the influence of mechanical stress as a function of time. Shear stresses at the grout column-rock interface are generated by frictional forces along this interface as a result of ground movement or by application of torque during installation. Earlier studies have concluded that the failure of fully grouted passive rock bolts is caused when these shear forces exceed the shear strength of grout or rock, whichever is weaker, at the grout-rock interface (Signer, 1988). It is thought that loss of the initial load on grouted tensioned rebar is also due to failure of the mechanical key at this interface, which reduces frictional forces causing "stick-slip" behavior thereby reducing the effective rock bolt load. Failure is due to the weak rock rather than the resin column.

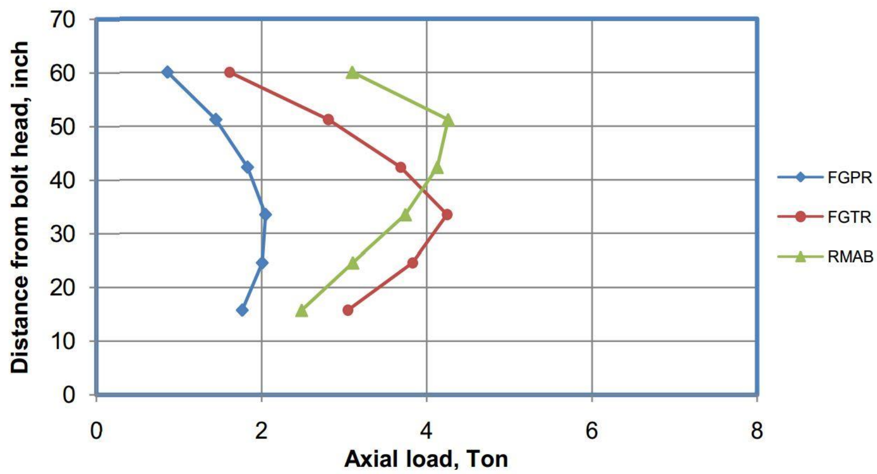

Figure 50: Average initial load for different rebar (Spearing et al., 2011).

Torque has been used in coal mines to determine the axial load on a rock bolt (Barry, 1954). Tests were conducted to measure the residual torque of about 120 randomly selected active bolts installed in a coal mine in the Kentucky No. 9 Seam of the Illinois Basin. Torque applied to these rock bolts on installation was not checked, but it was set to be 300 ft-lbs. Torque was checked 24 to 48 hours after installation. All active bolts were installed with steel plates. Results showed that almost one-third of tested bolts had significantly less torque than what is considered to be sufficient (Figure 51). It can be clearly seen that results from these studies indicate a similar problem, but the underlying cause of this problem was not specifically identified.

It was critical to understand the interaction between rock bolts and movement of rock to have a safe and cost effective underground excavation support design. Conventional load cells have been used to successfully predict loads developed between resin-grouted bolts and bearing plates in fully grouted passive bolts in a field investigation (Tadolini, 1986); however, in this study they have been used to determine the change of load with time (creep) occurring between the bearing plate and the grouted rock bolt after the application of torque. A novel U-Cell device was developed to measure the point load at the head of a rock bolt using a strain gauge (Mitri, 2012). This U-cell (Figure 52) is a load cell that can be coupled to the threaded (left-handed) end of a bolt prior to installation. It is interfaced with digital signal processing technology to provide measured axial strain output data at the rock bolt head.

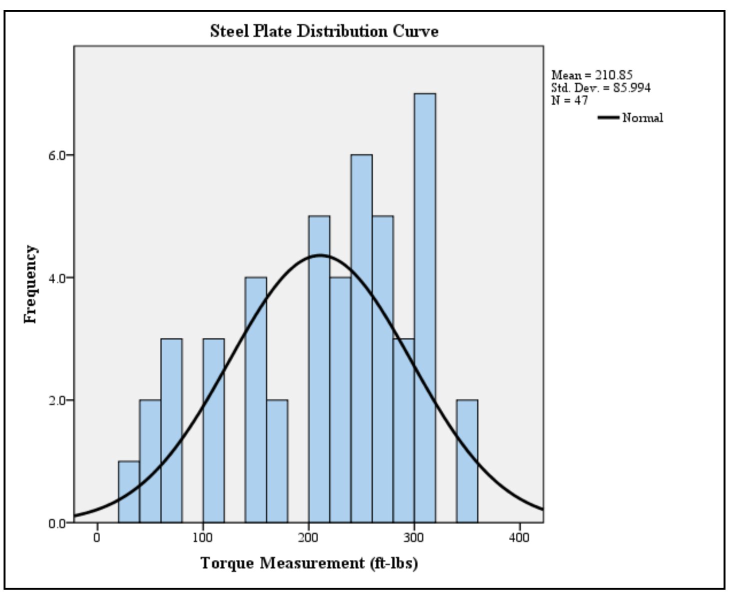

Figure 51: Retained torque after 24-48 hours using a steel plate.

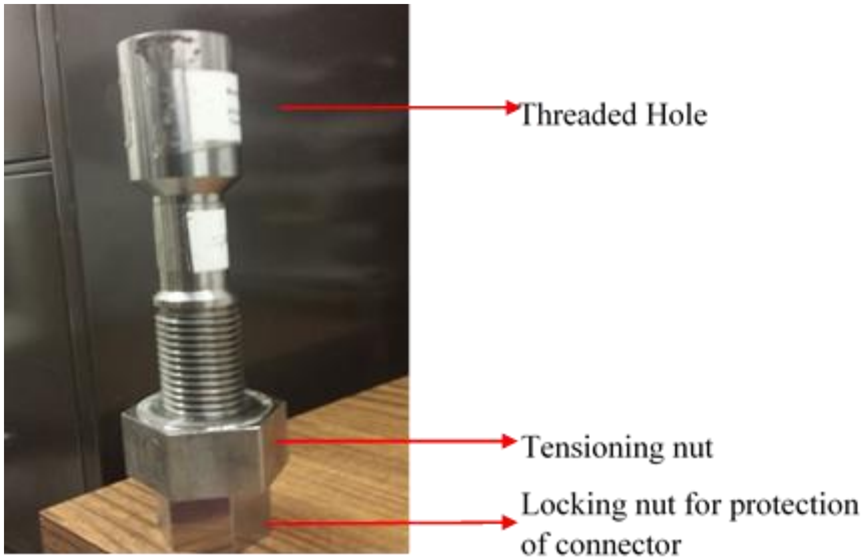

Figure 52: Parts of a U-cell.

U-cells as well as conventional hollow load cells were used for the creep investigation to quantify and understand creep behavior of fully grouted rebar just after the rock bolt is tensioned using a torque wrench. Tests were undertaken to calibrate the U-cell for a specific rock bolt and test its potential for accurately monitoring deformation in the rock bolt due to tensioning and ground movement between the rock bolt anchor and the bolt head and plate. These small-scale laboratory tests were conducted to understand and measure the creep behavior of fully grouted tensioned rebar and they found that U-cells provide a reliable method for determining the axial loading of rock bolts at the exposed head and plate by measuring axial strain.

To set up these tests, flanges were welded on a pipe having a larger diameter than the U-cell to provide a suitable column for it to be able to fasten to the bolt head and still provide considerable surface area for the nut to tension the bolt. The load cell was also placed between the bearing plate and the flange (Figure 53 and Figure 54). A #6 Grade 60 rebar (the most commonly used rock bolt in coal mines) was encapsulated (using pourable resin) in 9×9×9” and

9×9×18” concrete blocks at SIUC’s ground control laboratory. Concrete blocks represented a relatively stronger rock mass (6,000 psi) than those in situ. Pourable resin was used mainly to remove any effects of installation variability when using conventional resin cartridges. A torque wrench was used to provide tension to the resin-encapsulated bolt. Applied torque ranged from 100 to 400 ft-lbs. in 100 ft-lb. increments (300 ft-lbs. is common practice) and three tests were conducted at each torque value.

Figure 53: Welded flange.

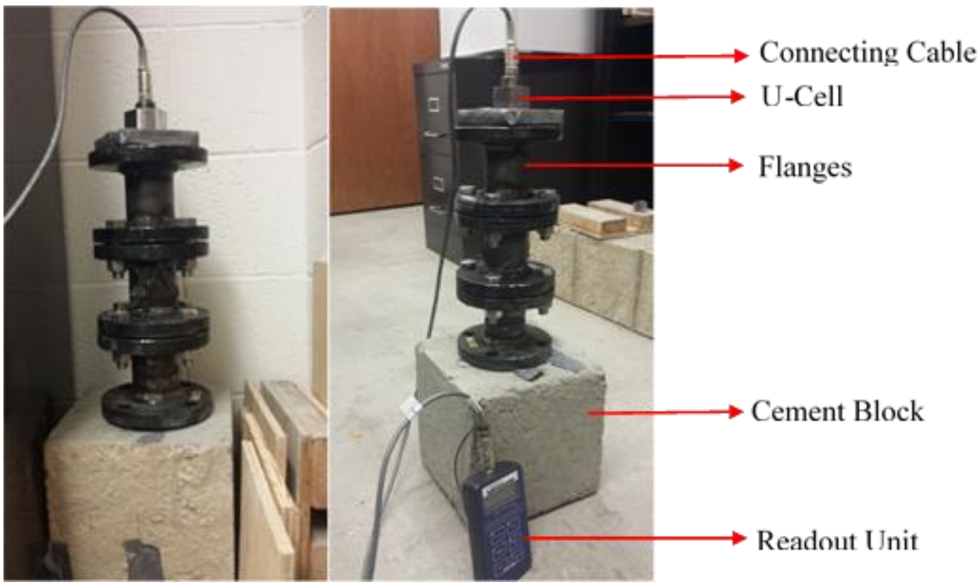

Figure 54: Test set-ups.

Initial readings were taken from the U-cell read-out unit prior to tensioning each bolt. Then, following bolt tensioning, readings were taken as soon as the read-out unit was connected to the U-cell and allowed to stabilize. This caused a few seconds of delay in each test; however, it was relatively constant throughout the test series. The load on the conventional hollow load cell was recorded as soon as desired torque was applied. All tests were conducted within a temperature range of 75.2 to 82.4^{o}F. For each test, axial strain measurements from the read-out unit as well as the load cell were recorded for two hours.

Load is then calculated using Equations 6 and 7 as follows:

$$F = k \times (R - R_0) \qquad (6)$$

where R is the strain reading at the read-out unit, R_0 is the initial strain reading on the read-out unit, and k is a calibration factor for a particular U-cell made for a particular bolt diameter.

$$F = C * P \qquad (7)$$

where C is a calibration constant and P is the pressure reading on the load cell.

Tests were first conducted with U-cells to quantify the creep behavior of resin-grouted rock bolts installed in 9×9×9” concrete blocks. Figure 55 shows load creeping with time in #6 rebar tensioned to 200 ft-lbs. Load was measured and recorded every ten seconds for the first ten minutes. Close observation showed that load loss was typical “stick-slip” behavior.

Initial test results also showed the effect of torque on creep behavior for #6 rebar. A significant increase in initial load was observed when applied torque increased from 200 to 300 ft-lbs. (Figure 56). The reason for this significant increase is unknown, but it could be the result of being high enough to reduce frictional losses mainly between the bolt head (threads) and the plate. More tests are required to explain this non-linear behavior of torque-tension relationship.

Subsequent tests were conducted with U-cells and load cells to understand the effect of encapsulation length on creep (Figure 57 and Figure 58). It was interesting to note that as tension on the bolt increased with an increase in applied torque, the time to occurrence of first slip decreased with increasing load, indicating failure at the concrete-grout interface. Results also showed a decrease in loss of load with increases in torque and encapsulation length.

Table 18 shows that the rate of creep decreased with time; however, it is significant that when tested for 2 hours, almost 50% of total load loss occurred within 1000 seconds in all tests. On comparing U-cell and load cell results (Appendix: Figures 81-86), both showed uniform creep trends. The load cell, being an analog device, showed a step function while the U-cell exhibited more accurate results as indicated by smooth curves.

Table 18: Percentage loss of load with varying torque and encapsulation length.

Torque (ft-lbs.)	**Encapsulation Length**	
	9”	**18”**
	% Load Loss	**% Load Loss**
200	9.8%	7.1%
300	7.2%	5.1%
400	6.3%	4.4%

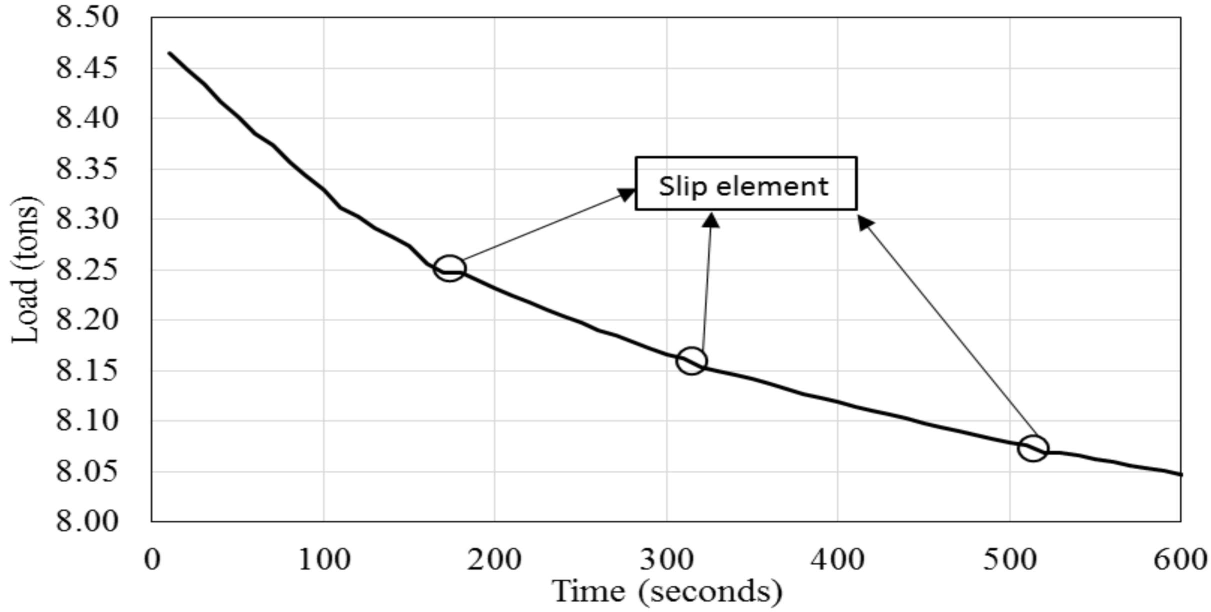

Figure 55: Occurrence of slip as creep progresses.

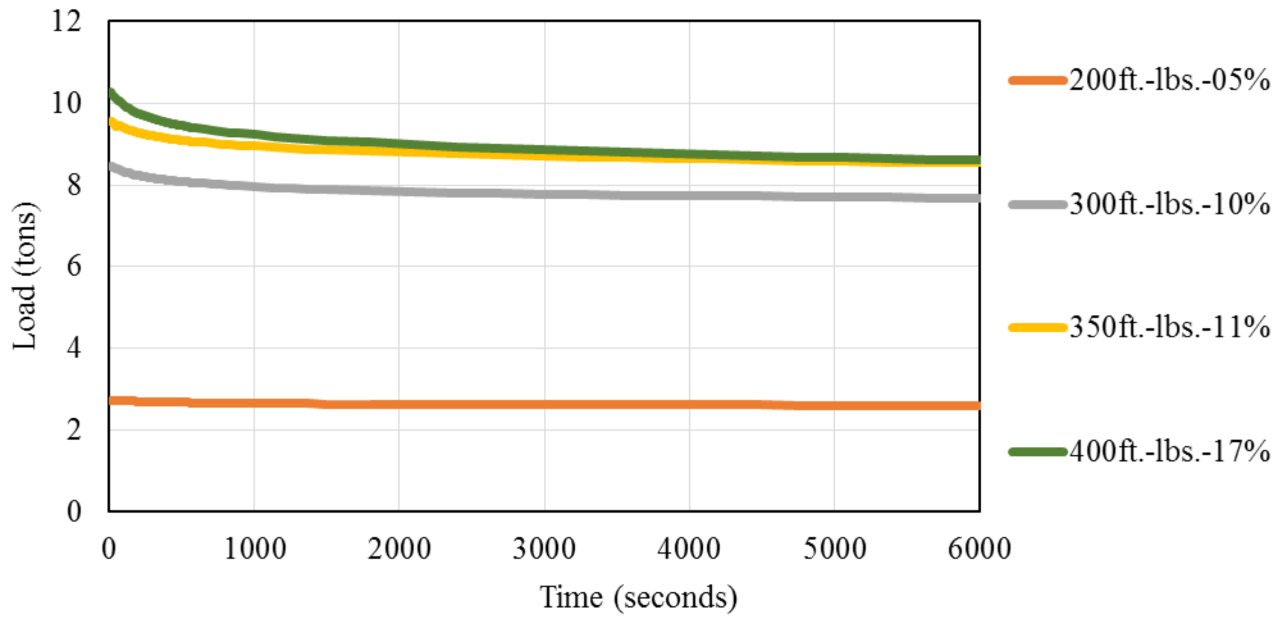

Figure 56: Initial test results when varying torque with respective load loss.

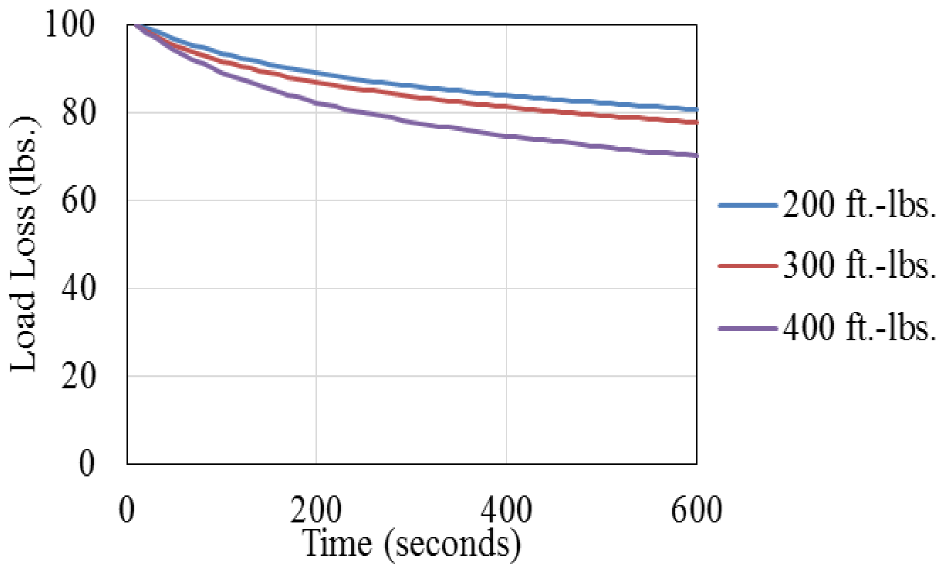

Figure 57: Tests with 9” encapsulation length.

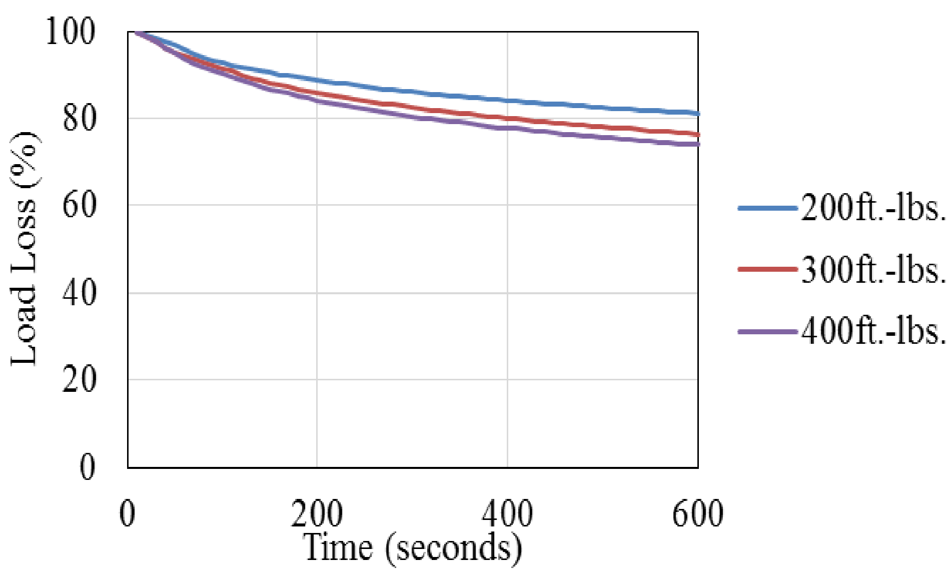

Figure 58: Tests with 18” encapsulation length.

The grouted rebar system starts to exhibit “stick-slip” behavior on application of torque with time indicating onset of creep. On the application of tension, the load is transferred from the bolt to the concrete via the stronger bolt-grout interface to the weaker grout-concrete interface. This causes mechanical key failure at the weaker (grout-concrete) interface, which is less in magnitude but continuous in nature in order to achieve equilibrium. This causes a loss of axial load in the bolt system with time confirming the creep behavior to be significant.

It was found that loss of load decreases as applied toque is increased. The loss of load also decreased when encapsulation length increased from 9” to 18”. Load losses were all less than 10% of the initial load; however, it should be noted that all tests were terminated after two hours. If tests were allowed to continue, creep may have been sustained resulting in more loss of initial load and further testing was beyond the scope of the thesis. Thus, creep in grouted tensioned rebar is found to be significant, which makes it no more superior than fully grouted passive rebar when bolt performance is measured and compared over the life of a mine.

The percentage load losses seem to reduce at higher initial torques (Table 18) but this may be as a result of experimental constraints rather than actual creep behavior, i.e., the rock bolts were torqued using a torque wrench, so the time to torque the bolts was significantly longer than in situ where a rock bolter was used. Also, same torque wrench was used during the entire experiment which makes the variability of the applied load by the wrench consistent. In the lab, the higher the torque - the longer the time to obtain the required torque. Results indicate that the bulk of the creep (load shedding) occurs immediately after tensioning, so the initial creep may have been missed and more testing is needed.

In this study, grouted tensioned rebar was installed in concrete blocks. The failure of the mechanical key at the grout-rock interface is decided by the shear strength of the grout and the rock, whichever is lower. The shear strength of typical grout is found to be 3,000 psi (Martín *et al.*, 2013). The shear strength of concrete lies between 2,000-2,500 psi (4,300 psi UCS) (Brooks, 2014). The immediate roof in a coal mine consists of beds of shale. The shear strength of shale varies from 1,160 to 1,700 psi, under very low compressive stress (Heng *et al.*, 2015). It can be said that the creep found in the grouted tensioned rebar would be more significant under *in situ* conditions compared to the concrete blocks used in the laboratory. However, further testing would be needed to deduce any conclusions.

REPRESENTATIVE NUMERICAL MODEL OF A LABORATORY PULL TEST

To develop a clearer understanding of pull tests conducted in the laboratory and in mines, a basic representative numerical model was designed to predict pull-out load. Rock bolts are used for support in a variety of mining situations. The pull test (ASTM F432) is used to provide a quantitative measure of the relative performance of different anchor systems. .

Around 200 pull tests have been conducted in the laboratory by the author of this thesis. Laboratory pull tests consist of testing rock bolts anchored in homogenous and isotropic material (concrete block) while varying key installation parameters. Based upon laboratory and *in situ* test results, reliable values for load vs. deformation of resin-grouted bolts have been obtained; however, these results do not always capture the complete failure mechanism. Additionally, performing further laboratory and *in situ* tests is difficult because of the time and costs associated with these tests and/or limited access to mines. Therefore, the most logical and cost effective method to further understand the failure mechanism of resin-grouted bolts, both *in situ* and in the laboratory, would be numerical modelling studies, if they can be calibrated.

To generate relevant data required to build such a model, a typical pull test was conducted in SIUC's laboratory to determine the failure mechanism of grouted rebar. The pull test was conducted with #6 rebar grouted in a 9x9x9 inch^3 concrete block. Pull-out load was less than the bolt's tensile strength, but was applied at a high enough load to cause system failure. It was observed that the interface between grout and concrete block failed first, *i.e.*, the bolt with grout was partially pulled out with no damage seen on the grout (Figure 59). The concrete block was then split to check whether the cause of failure at the interface was due to movement of the complete grout column inside the hole or whether movement was caused as a result of tensile strength failure of the grout or some other mechanism.

Figure 59: Failure of resin-grout interface.

This laboratory test data was incorporated into a FLAC3D numerical model of a rock bolt installed in a homogenous isotropic material (concrete). Using this model, two different approaches are available to simulate pull tests using the structural elements in FLAC3D (FLAC3D Manual, 2014):

1. **Cable elements**, which assume the grout behaves as an elastic-perfectly plastic material with confining stress dependence, but no loss of strength after the failure.
2. **Modified pile elements**, which account for changes in confining stress, strain-softening behavior of the grout, and rupture of the reinforcement

The pull-out load is however, defined as the load at which the anchor starts to slip. The load deformation curve is linear initially, showing the elastic property of the system. But as the applied load increases and exceeds anchor capacity, the load deformation curve starts to deviate from straight line, depicting anchor failure (Mark, 2001). FLAC3D provide various models for pull tests, considering both the bolt and the grout a single element, the properties of which can be defined. Using both approaches, it has generated models to determine pull-out strength under confining and non-confining stress conditions. These models assume the grout to depict a strain-softening behavior, which is a post-yield characteristic of a material and should not be considered to be important, according to the definition of pull-out load (

Figure 60). This model also showed that pull-out strength varies linearly with respect to encapsulation length, which may not be the case as the shear stress distribution in response to applied load is non-linear in behavior (Li, 2000). Also, the sample model of the rock bolt by the Itasca in FLAC3D is represented by a rock bolt model consisting of inter-connected nodes, which the author postulates to be inappropriate for a pull test.

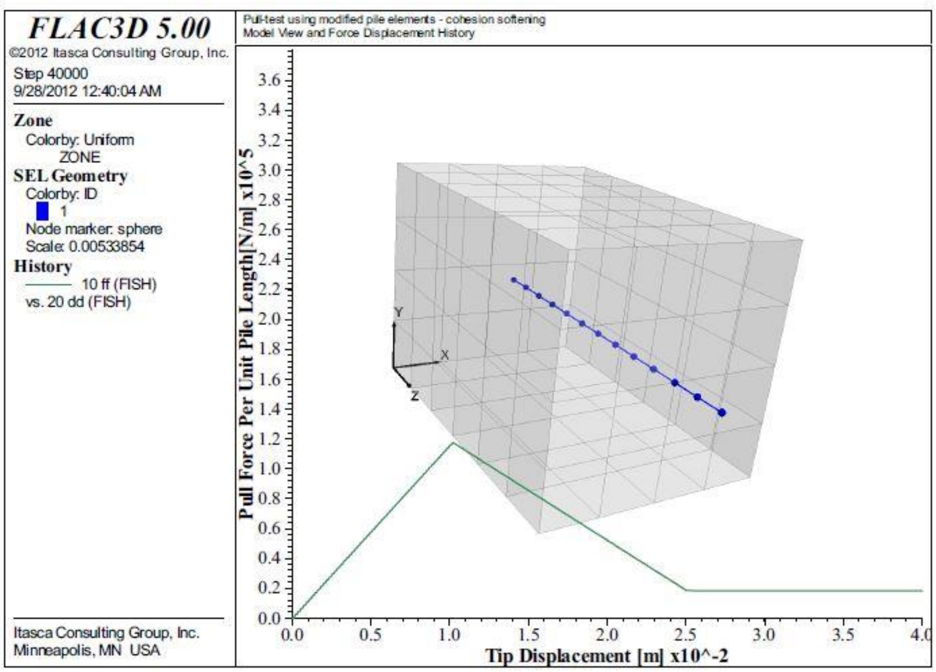

Figure 60: A sample pull test model by FLAC3D.

A model was designed by the Itasca in FLAC3D where the bolt and grout were modeled as two different elements installed in a concrete block. The load deformation curve acquired from laboratory pull tests conducted at Orica's Bowerston Research Facility was plotted and used as a reference curve to validate the representativeness of the model. These laboratory tests with #6 rebar and 12" of encapsulation length were then simulated. Grout and concrete properties were taken from the study conducted by Martín *et al.*(2013). Since it was found that the rock bolt system fails at the grout-concrete interface, an interface element was created between grout and concrete block. It was assumed that the bolt and the grout follow the Mohr-Coulomb model and concrete was to have an elastic model since it has been postulated by the author that the failure at the pull test occurs at the resin-rock interface and not the rock (concrete) itself. This was done so that stresses generated on the bolt due to application of load can be effectively transferred to the grout and then to the concrete at the grout-concrete interface.

In the numerical simulation, pull-out load was determined by applying small incremental displacements to the bolt head. The simulation was stopped when the load displacement curve started to show non-linear behavior. Results from the study were then compared with results from laboratory test results at Bowerston (Figure 61). It is clear from the figure that the representative model is comparable to the laboratory pull test. The model indicated that if an extensive analysis be made on the properties of resin and the mechanical interlock behavior of the bolt-resin interface, a numerical model is enough to describe the physical outputs of the pull tests.

It is important to note that mechanical properties of the grout assumed in the model are not exactly the same as those of the grout used in the laboratory because it has been not provided by the manufacturer and extensive tests needed to be conducted to find the properties of the particular resin used for pull tests under this thesis which was beyond the scope of the thesis. Thus, additional tests and simulations need to be conducted to determine the mechanical properties of each resin used and modeled, which is beyond the scope of this thesis. Nevertheless, it has been shown that with FLAC3D, a representative model has been developed and validated with data obtained from pull tests and this model can be used to predict the pull-out load of a grouted rebar. Furthermore, the model can be used to analyze resin rock bolt systems and improve them by changing properties of the system that are affected by key installation parameters involved in resin rock bolt installation. Concrete properties in the model can be replaced by properties for immediate roof layers of a coal mine to determine the pull-out load under *in situ* conditions.

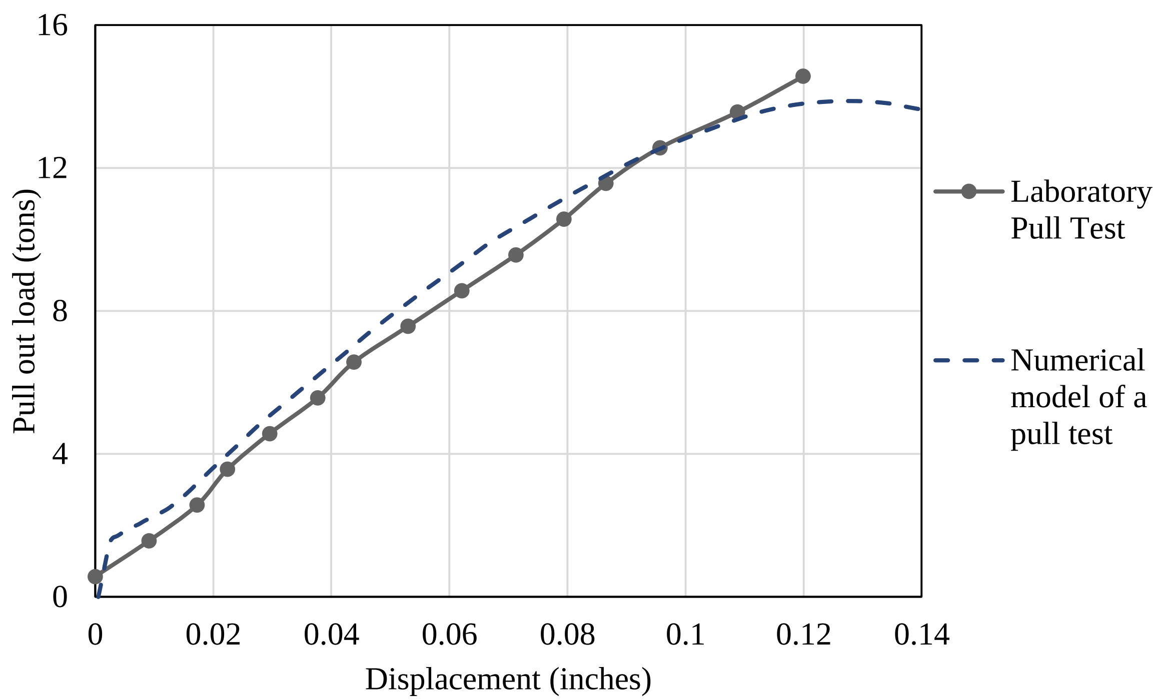

Figure 61: Comparison of laboratory pull tests vs. numerical model of a pull test.

COST ANALYSIS

Profit and cash-flow are the two most important factors in determining the success of a business. The aim to optimize the production is followed by a underground mine without sacrificing safety. .The basic cost benefit analysis is a widely used technique to evaluate solutions to a problem and decide whether to make a change. Bowerston results indicate that #6 rebar performs better than #5 rebar for the same encapsulation length. Since 68 million bolts are installed in underground coal mines, better bolt performance cannot be the only reason for a company to use it in their mines. The cost analysis was done to correlate performance benefits with economic benefits.

Results from tests conducted at Orica's Bowerston Research Facility suggested that if a linear relationship is assumed between tensile strength and rebar length, the length #6 rebar that can hold its ultimate tensile load of 19.8 tons is 15". Using the same approach, the length of the #5 rebar that can hold its ultimate tensile load of 13.8 tons is approximate 15". Though the material cost for #5 rebar would be less than #6 rebar, the load capacity of #6 rebar is 143% higher than that of #5 rebar.

It is also important to note that rock bolts in underground coal mines are installed with only estimated spin times. The roof bolter has not the ability to provide accurate data on spin time during installation. Thus, spin time is purely based on the roof bolter operator and significant variations of mixing are obtained. However, if the optimized spin time is known and miners are trained in the practical application of that knowledge, both safety and productivity can be increased, thereby improving revenue and profitability. Table 19 shows a simple cost analysis on spin time. It can be seen that there is as little as a 14% rise in cost per ton per foot of support when sub-optimum spin time is used. This becomes significant when one considers the volume of bolts installed in one mine over the course of a year.

Table 19: Cost analysis on spin time optimization

Spin Time (sec)	Pull-out Load (12" encapsulation)	Cost per Ton (12" encapsulation)
#6 Rebar		
2	14 tons	114%
4	16 tons	100%
#5 Rebar		
2	5.8 tons	190%
4	11 tons	100%

Table 20: Cost analysis on #5 and #6 rebar

Rebar	Pull-out Load (12" encapsulation)	Cost per Ton (12" encapsulation)
2 seconds		
#6	14 tons	100%
#5	5.8 tons	179%
4 seconds		
#6	16 tons	100%
#5	11 tons	142%

The numbers in Table 19 and Table 20 were generated based on the following information gathered from the manufacturer:

- The cost for a 4', #6 bolt fully grouted rebar system with face plate is $3.75.

  The cost for a 4', #5 bolt fully grouted rebar system with face plate is $2.80.

The cost invested by a company to support 1 ton of load for each #6 bolt if installed with spin times of 2 and 4 seconds will be $0.268 ($3.75/14) and $0.175 ($2.80/16), respectively. The difference in cost is $0.093 for each bolt per each ton of load supported. Similarly, for each #5 bolt installed with spin times of 2 and 4 seconds will be $0.48 ($2.80/5.8)) and $0.25 ($2.80/11), respectively. The difference in cost is $0.23. Thus, for 500,000 4' bolts, the savings will be $46,500 and $115,000 million for #6 and #5 bolts, respectively. Furthermore, the #5 rebar installed with 2 seconds and 4 seconds spin time cost more than the #6 rebar to the company by 79% and 42% respectively. Since, more than 68 million bolts are installed every year and the roof the bolting being the major bottleneck found during the production cycle, an optimized and a constant usage of a spin time would definitely increase the productivity.

CONCLUSIONS

The following conclusions can be made based on this research:

- It was confirmed in laboratory pull tests and thus, concluded that #6 rebar performs better than #5 rebar by almost 50% on the basis of pull out load and the key installation parameters do not play any significant role when both bolts are compared on basis of pull-out load.
- *In situ* pull tests using eccentric wrench as opposed to standard wrench play no significant role if standard installation procedures are followed.
- Hardness of cured resin is found to be a significant parameter to estimate the anchorage behavior of fully grouted rebar. It is concluded that a reliable and repeatable test method has been devised such that a pull test can be done followed by an examination to determine resin column hardness and the extent of glove fingering.
- More significant glove fingering was seen with #5 rebar than with #6 rebar. This can be explained as the effective shredding of cartridges may depend on the effective annulus.
- In simulated weaker rock (standard concrete), resin-grouted tensioned rock bolts exhibit creep (load shedding) caused by the mechanical key failure (“stick-slip”) of the resin column against the rock surface around the hole.
- Pull-out loads can be predicted by conducting numerical model studies using FLAC3D. It was indicated that a representative numerical model of pull test can be designed by actually designing the interfaces and causing it to fail by pull force rather than using the nodal design as recommended by Itasca in FLAC3D. Further study has to be conducted to investigate the full mechanism of the rock bolt system when pulled axially and assess the reliability of model in detail which is beyond the scope of this thesis.

- When installed with spin time of 4 seconds, both #5 and #6 rebar cost cheaper to support the immediate roof by 90 % and 14% respectively. Though #5 seems to be cheaper than #6, the cost per ton supported by the #5 bolt is significantly higher than #6 by 79% and 42% when installed with 2 seconds and 4 seconds respectively.

RECOMMENDATIONS

The following recommendations can be made based on this research to the mine operator:

- In underground coal mines with weak roof conditions, #6 rebar should be preferred over #5 rebar in a 1” hole. A system for controlling rotational speed and spin time should be added to roof bolters to achieve higher performance of fully grouted resin bolts. For resin with gel times of 10-20 seconds, 450-600 rpm is optimum, but a higher rotational speed is required for fast-settling resin. The optimized spin time for #5 rebar and #6 rebar is 2.5-5.0 seconds and 3.5-4.5 seconds, respectively. Such ranges are hard for a roof bolter operator to achieve with regularity, but are possible if an automated system is provided on the roof bolter. Additionally, this would also minimize the effects of human error caused by under-spinning or over-spinning while installing the bolt. Knowing the fact that 68 million (Tadolini and Mazzoni, 2006) fully grouted bolts are installed every year in underground coal mines, applying this knowledge could have significant results.
- The modified drill press has proven to be useful, quick, cheap, repeatable, and easy test for screening products or procedures. The test method is useful for parametric testing when only a single parameter associated with resin-grouted rock bolts is changed. The current drill press used for this thesis was underpowered and at least a 5HP or more motor may be better to test longer and more typical encapsulation lengths (*i.e.*, 12” as opposed to the current length of about 5”).

FUTURE RESEARCH WORK

- Poor results obtained from pull tests with #7 rebar were probably due to using low diameter resin cartridges. Thus, tests with a larger resin cartridge diameter should be performed. Once definitive results are obtained for #7 rebar, they can be effectively compared with results obtained from #5 and #6 rebar.
- A slight change in bolt installation procedures may make use of the eccentric wrench a means of obtaining better anchorage performance. Since these tests were few in number, it is suggested to conduct more tests with different test parameters to have conclusive results.
- Tests with the modified drilled press successfully proved its reliability for pre-screen prior to rigorous and time consuming pull tests, but more tests should be conducted with higher encapsulation lengths to check for consistency of the mixed resin. This can be accomplished by using a higher powered motor in the modified drill press.
- It is recommended to conduct more tests with the insertion rig at the High Bay Research Facility to investigate glove fingering. These tests can be performed with transparent high-strength polycarbonate pipes by which shredding of the cartridge can be studied during rebar rotation. Hardness can be recorded from these tests along the encapsulated length to analyze the grout.
- Creep tests conducted with tensioned rebar depicted significant loss of initial load; however, more tests are required to analyze creep with varying encapsulation length. *In situ* tests should also be conducted to understand creep behavior of fully grouted tensioned bolts installed in weak and strong immediate roof.
- Grout properties and the interaction behavior at rock-grout and grout-rock interfaces should be studied in detail to effectively predict pull tests results with FLAC3D.

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APPENDIX

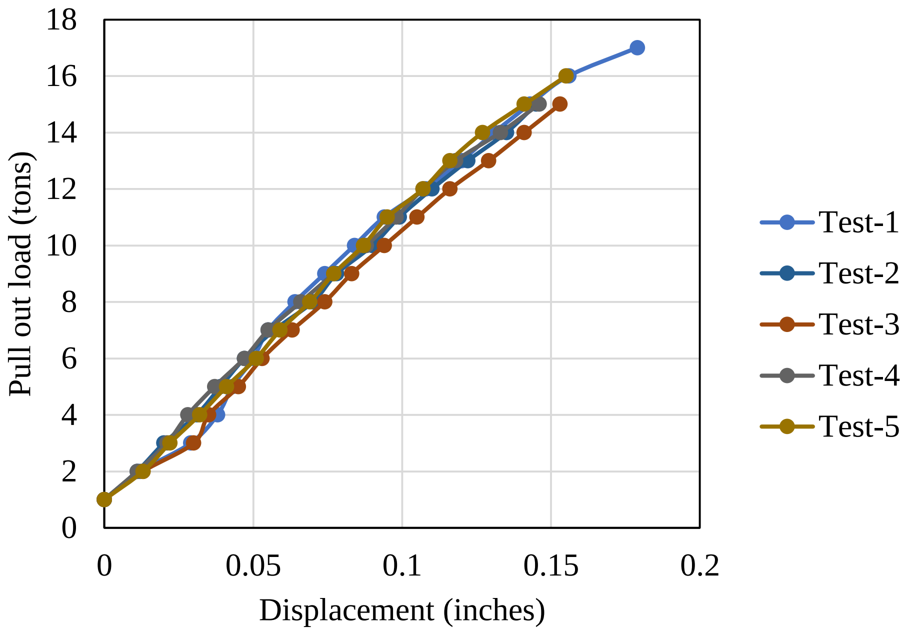

Figure 62: #6 bolt installed at 540 rpm with 3 seconds of spin time.

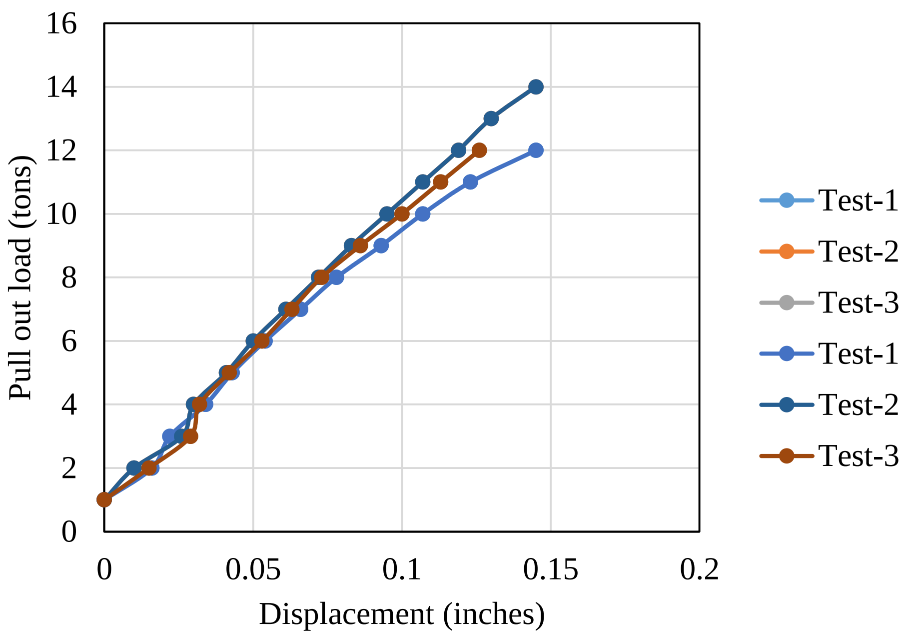

Figure 63: #6 bolt installed at 540 rpm with 2 seconds of spin time.

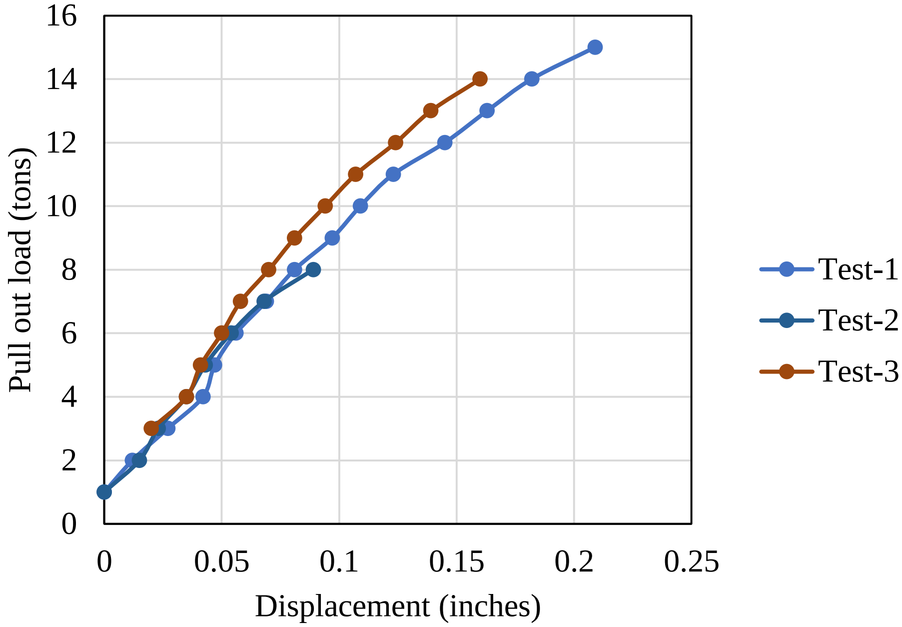

Figure 64: #6 bolt installed at 300 rpm with 9 seconds of spin time.

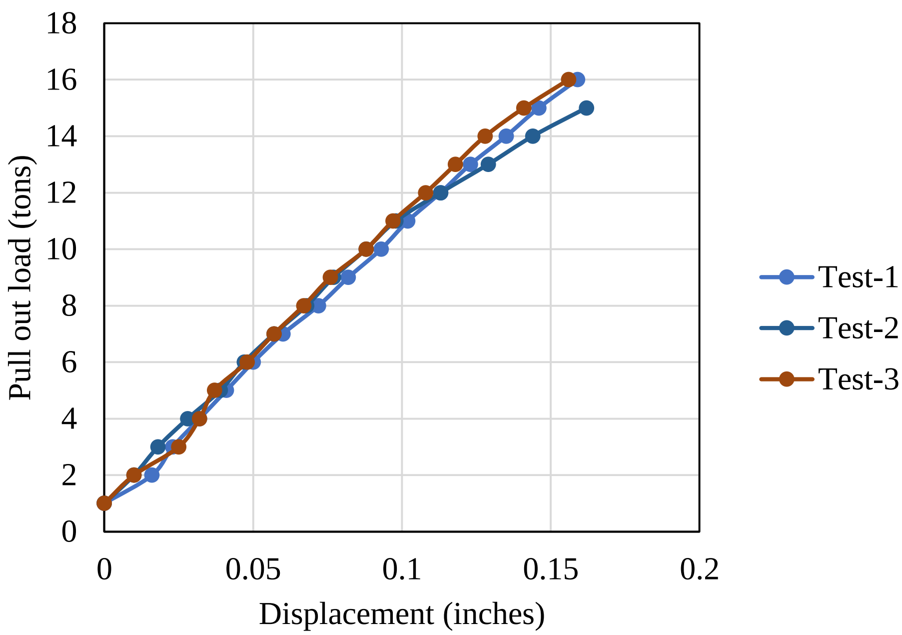

Figure 65: #6 bolt installed at 675 rpm with 4 seconds of spin time.

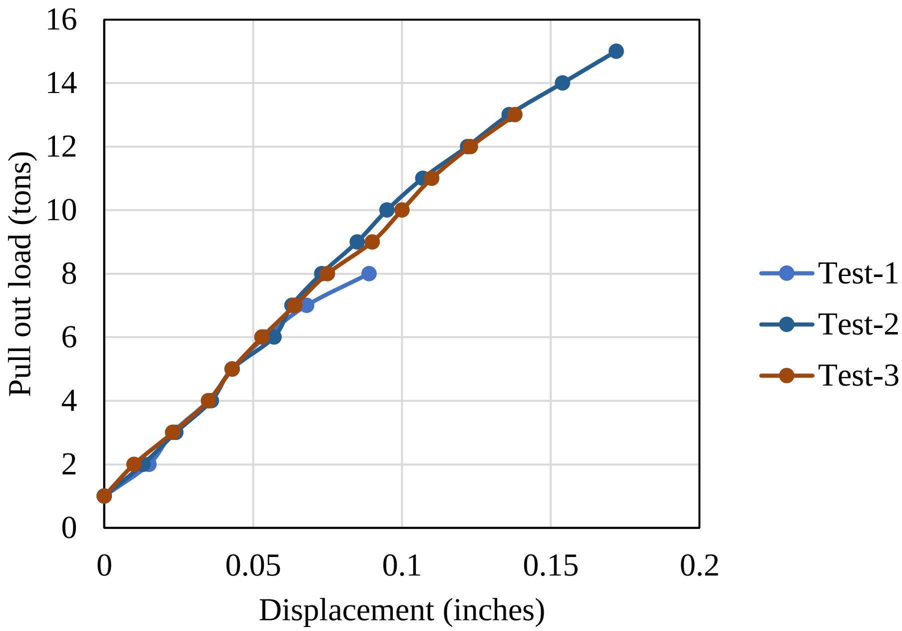

Figure 66: #6 bolt installed at 390 rpm with 7 seconds of spin time.

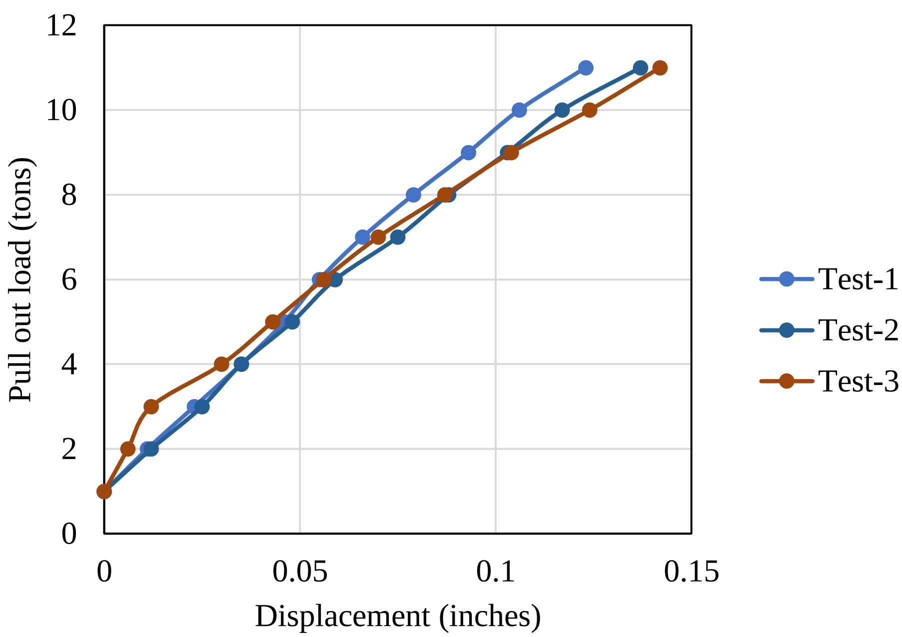

Figure 67: #5 bolt of 39" length with 12" of encapsulation.

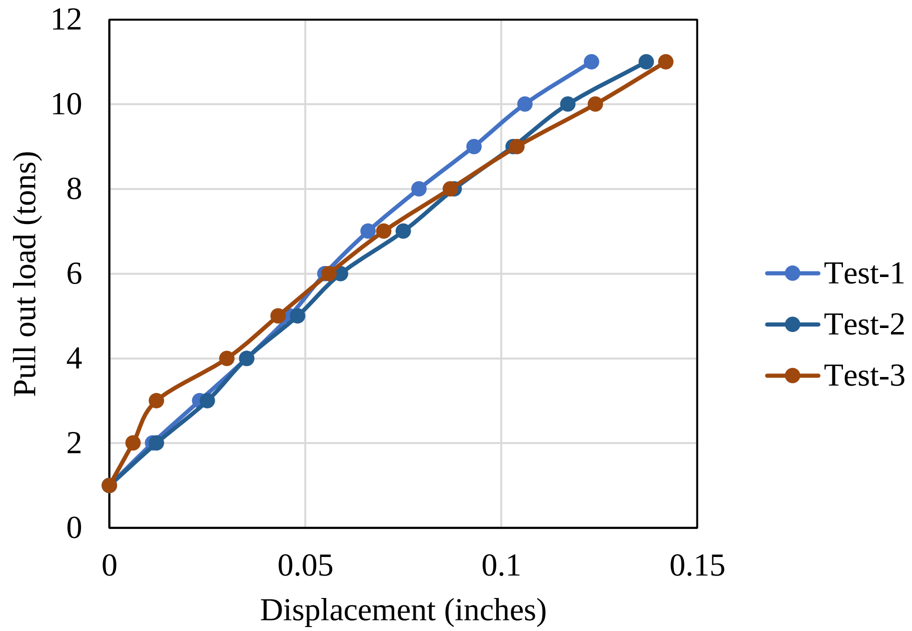

Figure 68: #5 bolt of 18" length with 12" of encapsulation.

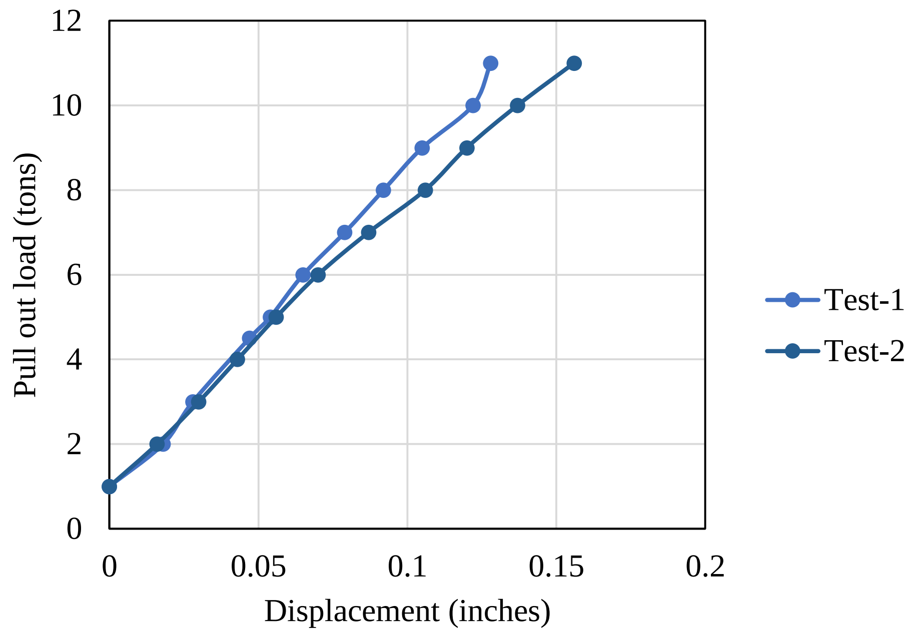

Figure 69: #5 bolt of 69" length with 48" of encapsulation.

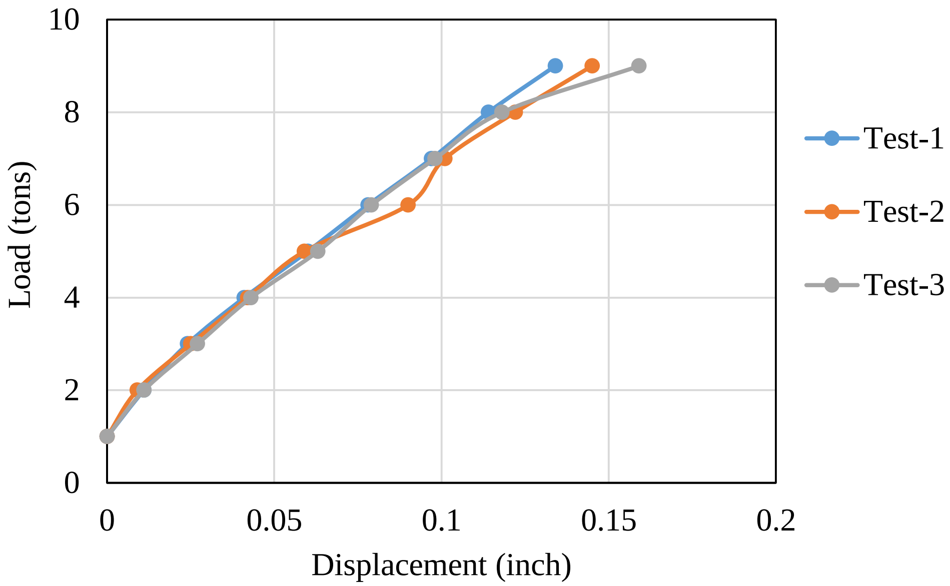

Figure 70: Pull tests with standard bolt (normal installation) at Mine A.

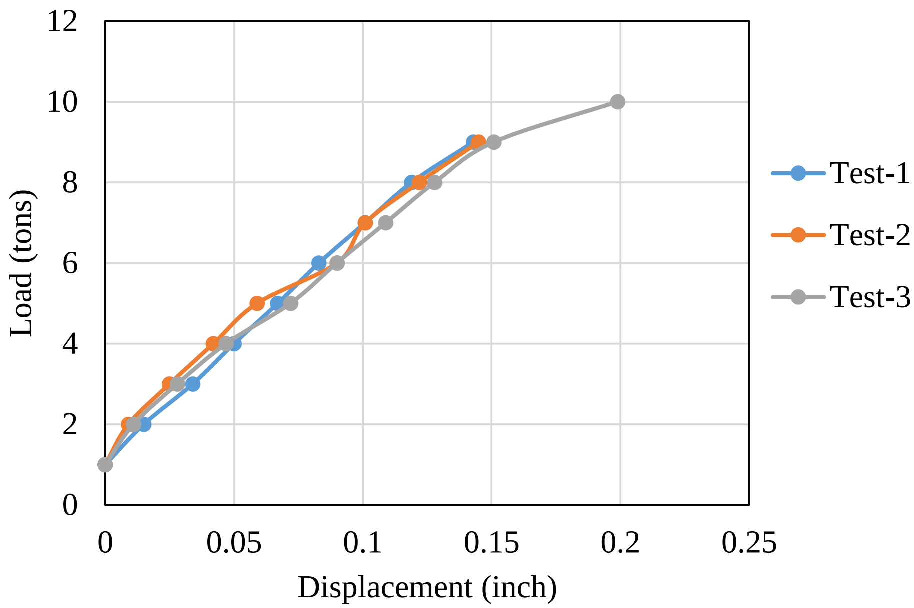

Figure 71: Pull tests with standard bolt (eccentric installation) at Mine A.

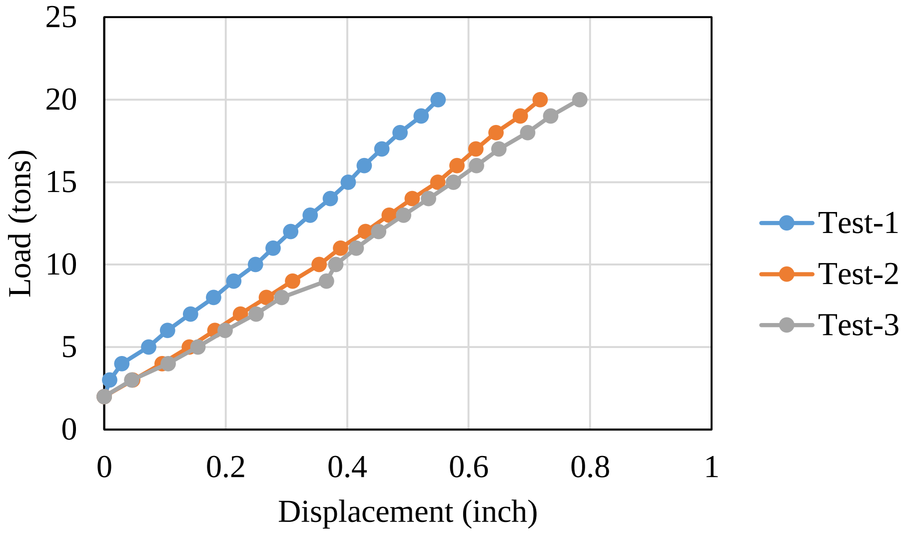

Figure 72: Pull tests with indented cable bolts at Mine B.

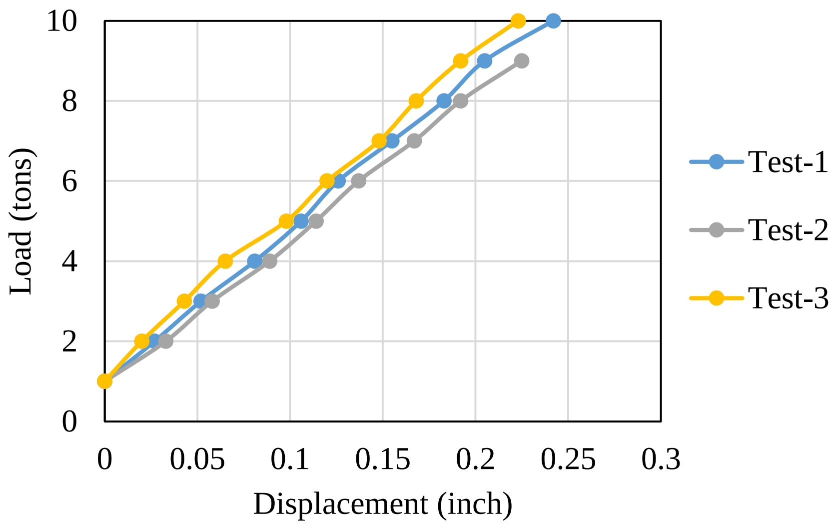

Figure 73: Pull tests with eclipse bolt at Mine A.

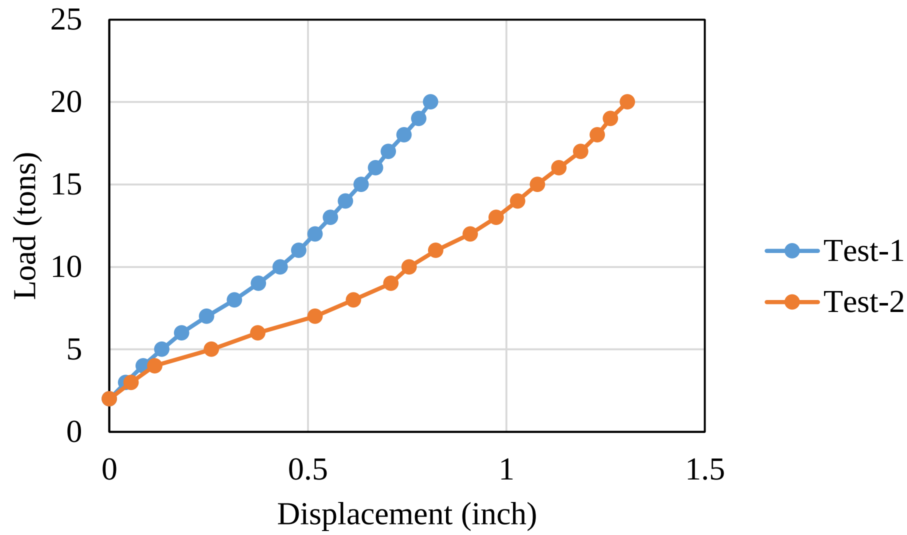

Figure 74: Indented cable bolts installed at Mine B (Test 1).

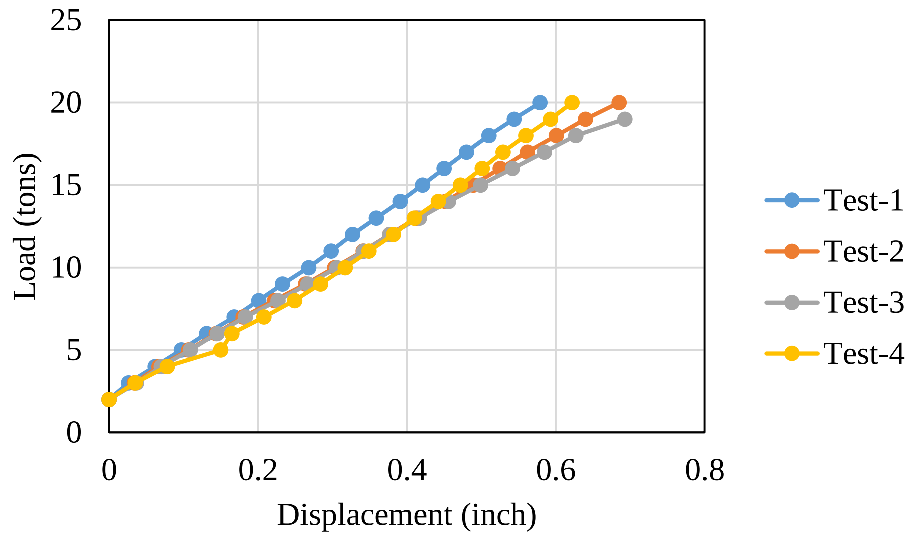

Figure 75: Bulbed cable bolt installed at Mine B (Test 1).

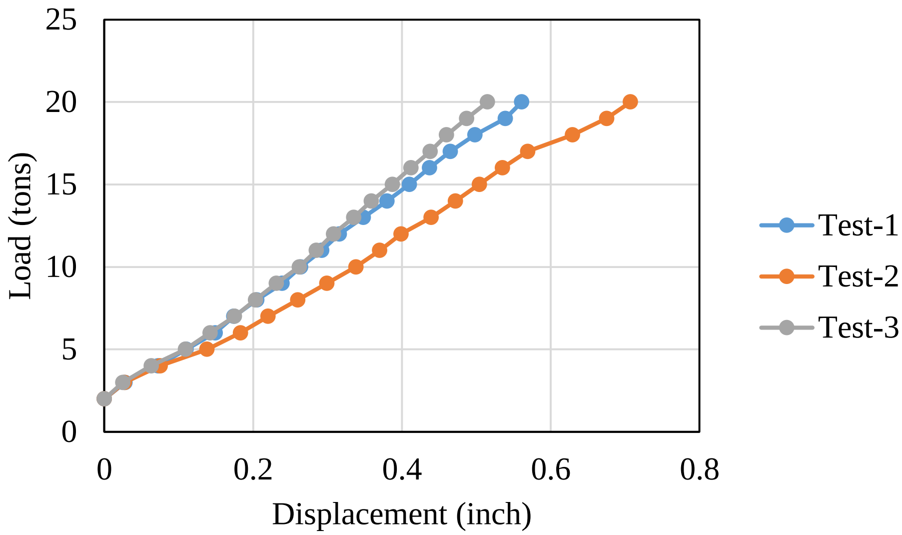

Figure 76: Bulbed cable bolt (eccentric wrench) installed at Mine B (Test 1).

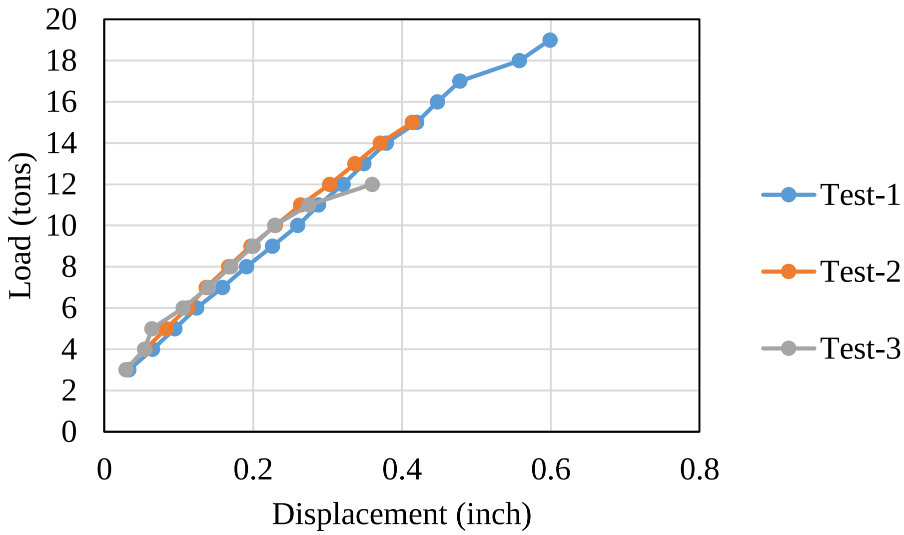

Figure 77: Indented cable bolt with welded tip installed at Mine B (Test 2).

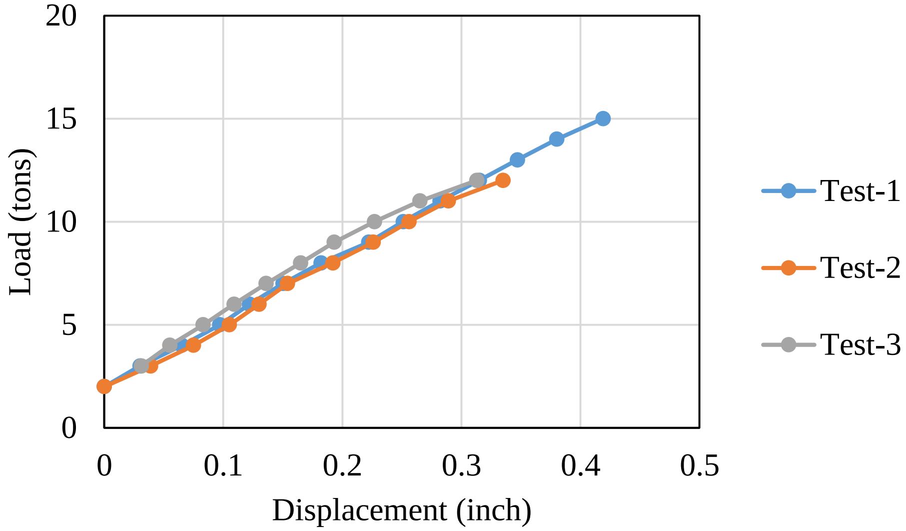

Figure 78: Indented cable bolt with button installed at Mine B (Test 2).

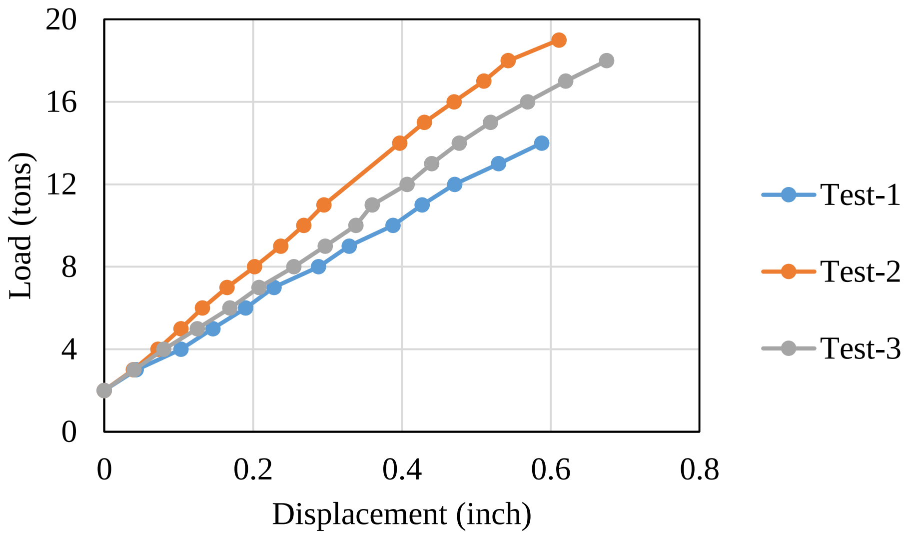

Figure 79: Indented cable double bulbed button with welded tip installed at Mine B (Test 2).

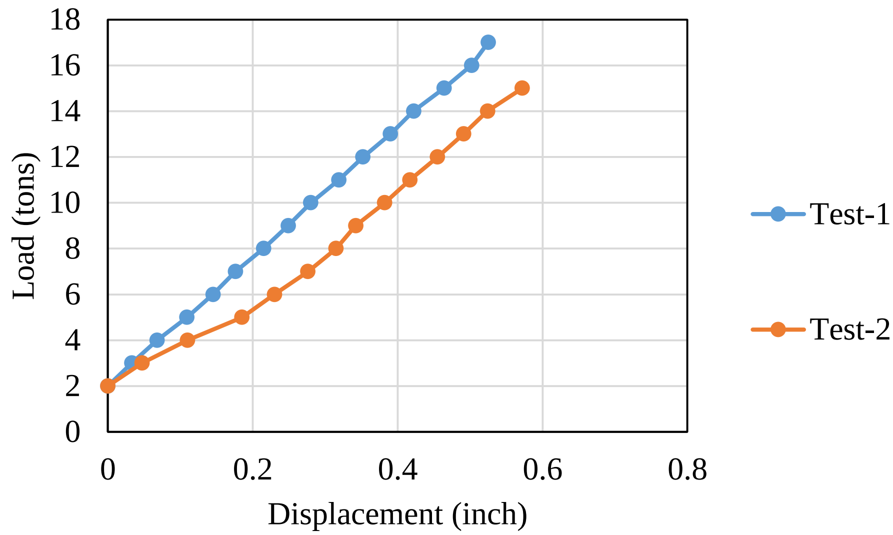

Figure 80: Indented cable bolt (eccentric wrench) installed at Mine B (Test 2).

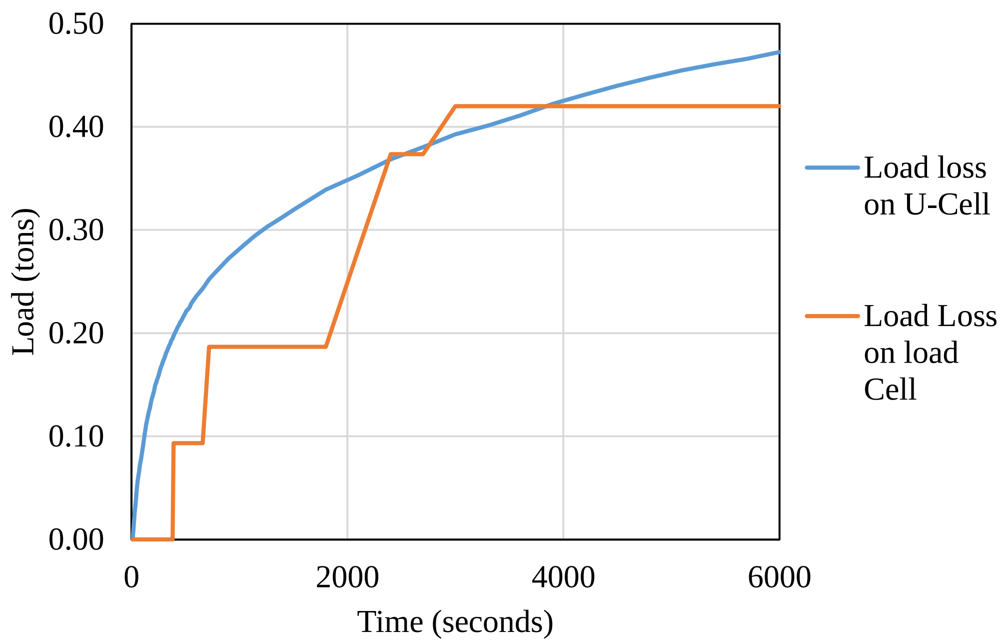

Figure 81: Tests with 18” encapsulation length at 200 ft-lbs.

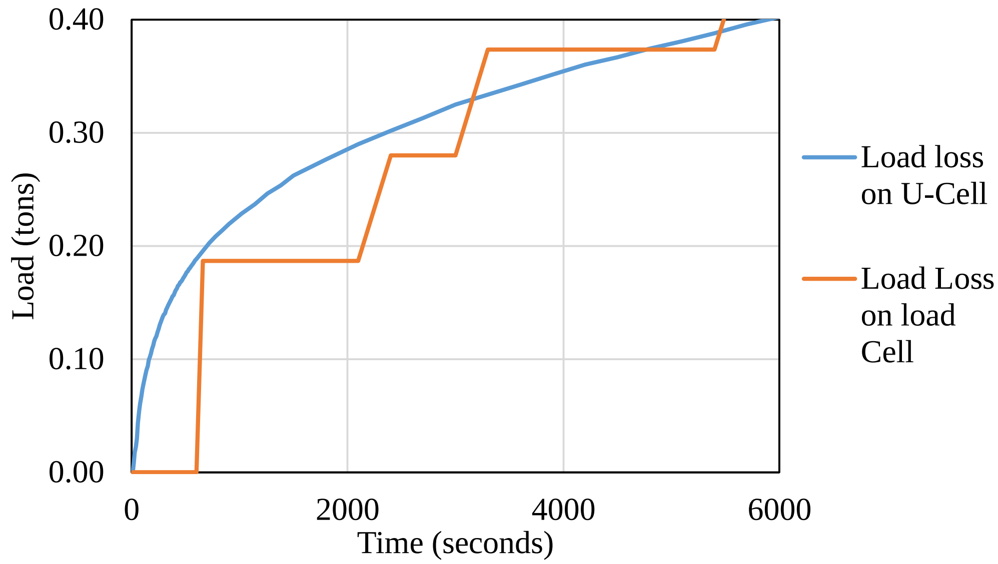

Figure 82: Tests with 18” encapsulation length at 300 ft-lbs.

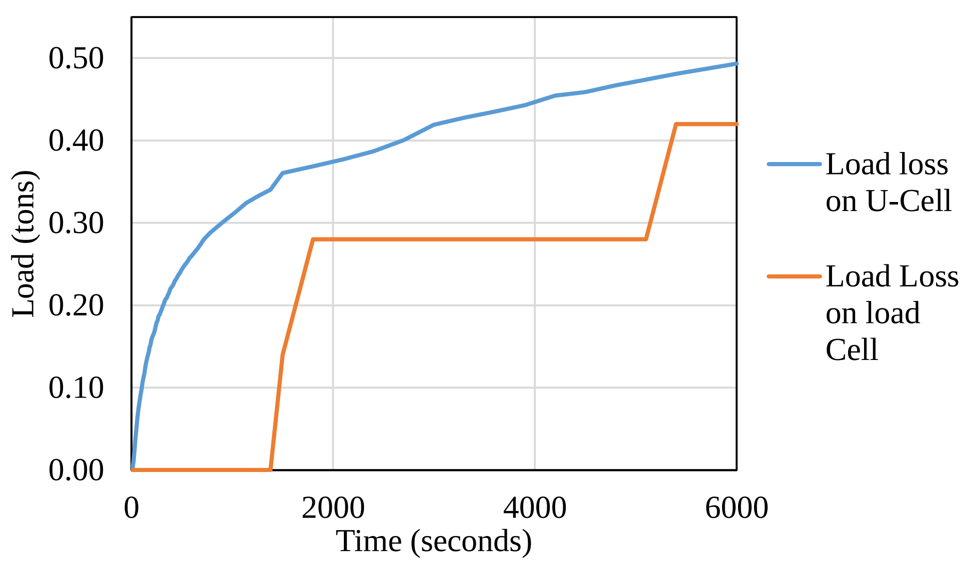

Figure 83: Tests with 18” encapsulation length at 400 ft-lbs.

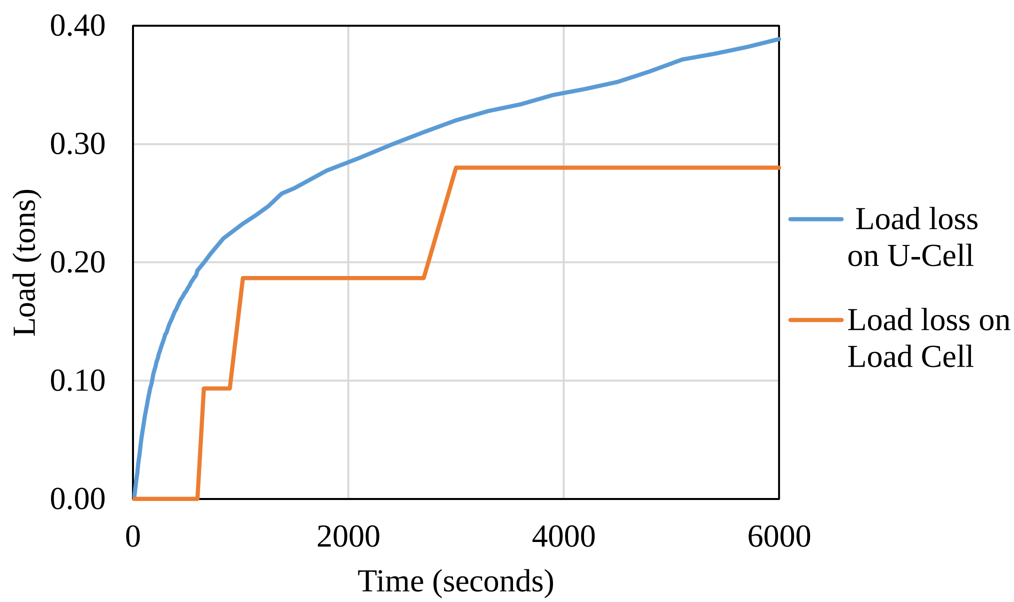

Figure 84: Tests with 9” encapsulation length at 200 ft-lbs.

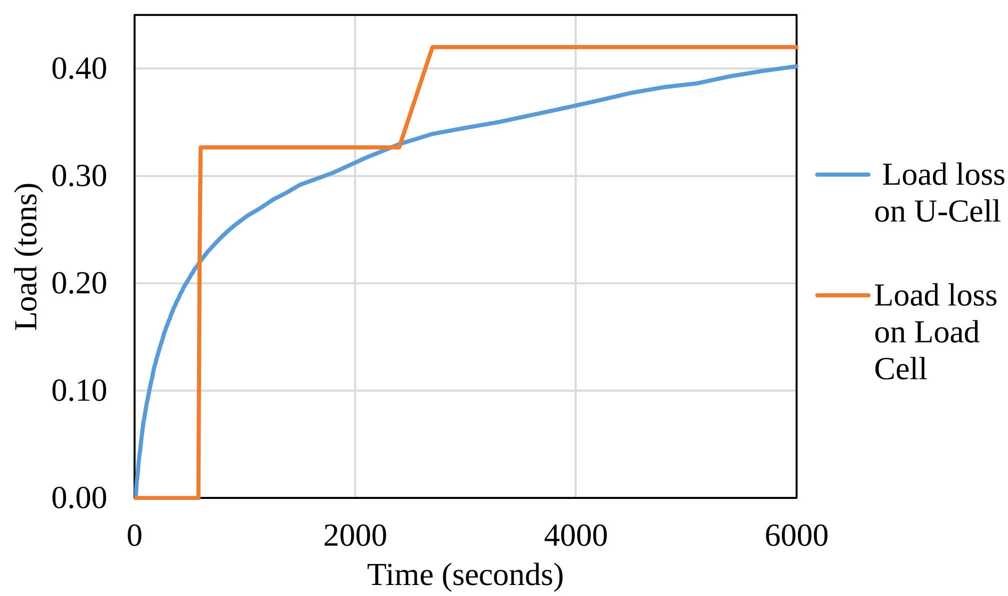

Figure 85: Tests with 9” encapsulation length at 300 ft-lbs.

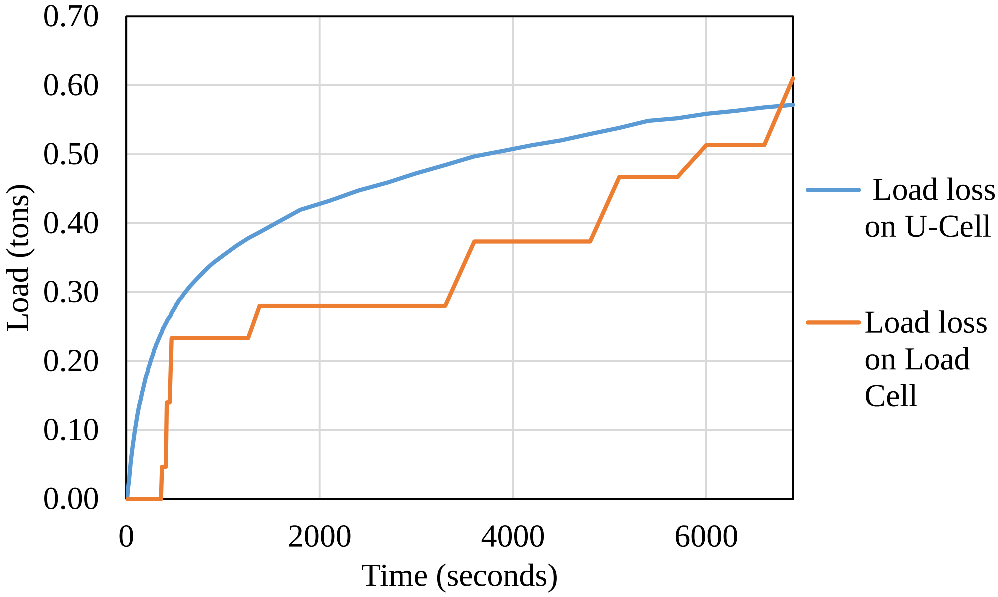

Figure 86: Tests with 9” encapsulation length at 400 ft-lbs.

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Thesis Paper Title:

Measuring, Understanding, and Improving the Performance of Fully Grouted Resin Bolts in Underground Coal Mines

Major Professor: Dr. A.J.S. Spearing

Publications:

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- Spearing, A.J.S., Mishra, A., Tirumalaraju, N.S.Y., Hirschi, J., 2014. "Understanding and Optimizing the Performance of Passive Primary Support on Coal Mines." 33rd International Conference on Ground Control in Mining.