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ABSTRACT

Corrosion affects the performance of rock support and reinforcement at various underground mines, including the George Fisher Mine, where recent studies showed that this risk to mining operations is generally not well understood. Corrosion reduces the capacity and life expectancy of ground support and generally is undetected unless the traditional destructive pull out test is employed to detect defect bolts. Where it is still acknowledged that the pull out test has still an important role to play in determining critical bond lengths for static and quasi static ground support designs, it does not provide an underground operation with any reassurance regarding its bolt's integrity, which has been compromised by in-situ aggressive conditions. Non-destructive rock reinforcement calibration testing on single and twin strand cable bolts, rebar bolts and friction bolts at the George Fisher Mine enabled the testing team to detect bolts with compromised integrity in one area of the mine. The non-destructive rock reinforcement integrity testing conducted uses a complex "Stress Wave Analysis" package based on the processing of clear seismic signals imparted into the *rock reinforcement element that* is being tested. The seismic signals are processed by *"Fourier Transform"* into various criteria which can be used to produce models of the element such as mechanical admittance, frequency spectra and velocity which are all being used in the final modelling of the rock reinforcement element under analysis. This paper highlights the enormous potential to effectively manage the geotechnical risk that corrosion presents for an underground mining operation.

INTRODUCTION

It has been found through previous studies that corrosion reduces the capacity and life expectancy of ground support (Villaescusa, Hassle and Thompson, 2007) and generally is undetected unless the traditional destructive pull out test (see Figure 1) is employed to detect defect bolts.

If the ground support elements do not fail after employing the pull testing device the uncertainty in regard the ground support system integrity remain. It was also apparent that the hydraulic pull testing device cannot provide a good indication of the effectiveness of the fully grouted bolt following a recent review of non-destructive testing (Buys, 2008). It is however acknowledged that the pull out test has still an important role to play in determining critical bond lengths for static and quasi static ground support designs but does not provide an underground operation with any reassurance regarding its bolt's integrity, which could have been compromised by in-situ aggressive conditions. Thus in order to determine whether ground support elements have been compromised

through corrosion an operation would need to undergo destructive over coring of their rock reinforcement system. Not only is it an extremely time consuming exercise but quite expensive too.

The George Fisher Mine, which is located 20km north of Mt. Isa and over 950 km west of Townsville in North Queensland (See Figure 2), embarked on a program of Non-Destructive rock reinforcement calibration testing on single and twin strand cable bolts, rebar bolts and friction bolts which enabled the testing team to detect bolts with compromised integrity in one area of the mine. This testing program highlighted the enormous potential to effectively manage the geotechnical risk that corrosion presents for underground mining operations.

BACKGROUND TO NON-DESTRUCTIVE TESTING

Investigations into premature rock bolt failures in the Australian Coal Mining industry identified the problem in 9 different collieries throughout NSW and Queensland. Samples of broken bolts have been collected from at least 5 of these mines for detailed metallurgical testing. Because of the sporadic occurrence of premature bolt failure and the low level of record keeping within the mines, only a limited sample was available for the investigator's database. The investigation also highlighted that only bolts which have fallen to the ground are normally recognised as premature failures and to date there has been no method of identifying broken bolts which have failed within the encapsulated portion of the bolt or are restrained in the roof by strata shear. The investigation recognised from the outset that the database obtained from broken bolts found on the floor of mine roadways may only be a small part of the problem. The type of failures being identified could just as easily be occurring within the grouted horizon. This would leave partially or totally failed bolts grouted into the roof, providing the potential for an extremely hazardous situation of roof instability developing, with little or no warning to operators. It was recognised that there was a need for a non-destructive device to be available to the industry to be able to check bolt integrity on a routine basis (Hebblewhite, Fabjanczyk and Gray, 2003).

Buys (2008) found that studies to develop non-destructive testing methods to determine a rock bolt's integrity have been conducted since 1977. He further critically reviewed current developed systems and highlighted their respective limitations.

(i) Boltometer

Instrument can be used on cemented grouted bolts and also on polyester and some other grouts. Buys found that the Boltometer can indicate bad ground, but if the impedance between the grout and surrounding rock are the same; wave energy will dissipate into the rock before it could reach a major defect, reporting good grouting.

(ii) JK rockbolt tester

Device developed by the JKMRC at the University of Queensland and is based on the measurement and analysis of the complete dynamic response of the bolt. The essence of the method is to measure the frequency response function of the bolt. This function can be considered as the signature of the bolt in situ. Resonant frequencies and associated damping ratios are primarily determined by the mechanical characteristics of the bolt in situ, resin grout and the rock.

(iii) GRANIT

The ground anchorage integrity testing (GRANIT) system was developed at the University of Aberdeen in Scotland. This testing system applies an impulse of known force by means of an impact device that is attached to the tendons. The vibration signals that arise from this impulse are complex in nature and require analysis to be performed. Novel artificial intelligence techniques are used in order to learn the complicated relationships that exist between anchorage and its response to an impulse. A single axial impulse load is generated at the head of the tendon. A set of datum responses signatures for different post-tension levels is recorded for the anchorage immediately after installation. The condition of the tendon at any later date can be determined by comparing the datum with the characteristics of the subsequent response signatures. A change in characteristics indicates a potential change in the integrity of the anchorage. It is difficult determining the relationship between the dynamic response of the anchorage and its post tension level. Therefore, neural networks are implemented, which suit the task of learning such complicated non-linear relationships. The neural network, trained on a single tendon, may be used to diagnose the post tension level of a number of tendons. The disadvantage of this system is that it can only work on rock bolts that have been charaterised at installation.

(iv) Ultrasonic Guide Wave Testing

A technique developed by Imperial College in England and it's based on an ultrasonic pulse echo test carried out from the free end of the bolt. Beard and Lowe (2003) showed that the approach could determine the bolt length and identify major defects such as loss of resin encapsulation, necking and deformation. A short duration Gaussian windowed sine burst is used to excite a guide wave in the bolt, which is then reflected from the bolt end and from any major defects. The reason for using a Gaussian windowed sine burst signal is that the excitation frequency can be tightly controlled and thereby excites specific modes. From the reflection arrival time and knowledge of the wave velocity dispersion curves the positions of the defects or the bolt length can be calculated. The maximum test range is limited by the amount of attenuation that the wave experiences as it propagates. The major cause of attenuation is the fact that ultrasonic energy tends to leak from the bolt into the surrounding rock. The study concluded that bolts should be tested in its low frequency range (30-70 kHz) as well as in its high frequency range (2-5 MHz). The low frequency test can be used to identify defects such as partial bolt encapsulation and possibly corrosion patches near the bolt surface. The high frequency test is not sensitive to surface defects, but can give a reliable indication of the bolt length. It was not possible with their modeling tools to determine the dispersion relationships for bars with 3 dimensional features, as the rock bolt end and certain defects are not always axial symmetric it may be neccesary to model the bolt with three-dimensional finite element methods.

The MODSHOCK system used within the testing program at the George Fischer mine compared to the above methods is relatively simple in operation. In simple terms, the modified shock test as described by Higgs and Tongue (1999) is a seismic test using a hammer blow as the force and a transducer to pick up the resultant vibrations With the application of digital filtering techniques an accurate mechanical admittance vs. frequency plot is obtained which can then be interpreted using the concepts developed by Davis & Dunn (1974).

This non-destructive method by vibration has its origins from Davis and Dunn where they carried out various types of non-destructive pile tests on sites in Western Europe and other French speaking countries for *"The Centre Experimental de Recherches et d'Etudes du Batiment et des Travaux Publics"* (CEBTP) of France. This vibration method had also been used and described by Gardner and Moses (1973), but this technique had not been exploited by British engineers to the extent that

could have been useful to them because of a lack of knowledge of the technique and a degree of mysticism associated with the interpretation of the results.

Since vibration testing of piles was first started by the CEBTP a considerable amount of theoretical work had been done which shed light on interpretation, and the experience of testing many thousands of piles led to the technique being applied with more confidence to piles on sites which lend themselves to being tested in this way. The main function of the test was to detect any major defect such as an open fracture or an important strangulation of the concrete, particularly in the upper portion of the pile.

The method is based on measuring the frequency and amplitude response of a rock reinforcement element known as impulse. This response, known as Mechanical Admittance (or mobility), contains all the information necessary to confirm rock reinforcement integrity and to analyse the surrounding influences. At higher frequencies the resonating harmonics of the rock reinforcement element are detected, whereas at low frequency the response is generally linear allowing measurement of the element-head stiffness.

The non-destructive rock reinforcement integrity testing analysis is conducted using a complex "Stress Wave Analysis" package based on the processing of clear seismic signals imparted into the *rock reinforcement element that* is being tested. The seismic signals are processed by *"Fourier Transform"* into various criteria which can be used to produce models of the element such as mechanical admittance, frequency spectra and velocity which are all being used in the final modelling of the rock reinforcement element under analysis.

In research and laboratory applications of modal analysis, particularly of complex machinery, dynamic excitation was often provided by a linear hydraulic or eccentric mass shaker. Experience gained in testing over 140 bridges indicated that simpler means of excitation are suitable for 90% of all bridges where attaching shakers to bridges were seen as a complex and costly method and is only practical for research purposes or for extremely complex structures [4]. Similarly the application for rock reinforcement integrity testing it was found that a simpler method to excite bolts is adequate for the detection of defects.

The development of the Australian based testing method started in the late 80's and has been used for the correct assessment on a large variety of elements, which now exceeds well over 1,000,000

tests for more than 20 years [5]. Integrity Testing PTY LTD (i.e. developers of the Modshock system) has for over 15 years carried out testing of long length steel rods, either as strand or solid steel bars. The most notable project was for BHP, when they owned the Whyalla steel works where they tested the tie rods holding back the crucial steel pile wall of the coal handling jetty.

The rods were tested and not only were the defective rods identified but it was indicated at what point the rods had lost a large cross section. This was located at a point where the rods came close to the base of the coal handling pit and water was seeping onto the rods causing corrosion. Thus a large successful background in the testing of steel embedded elements, generally with the lengths in excess of 5 meters.

CALIBRATION TESTING PROGRAM AT GEORGE FISHER MINE

The bolts tested at George Fisher Lead-Zinc Mine consisted of:

i) Cable Bolts

ii) Resin Bolts and

iii) Friction Stabiliser Bolts.

i) Cable Bolts (Figure 3 – Table 1)

The cable bolts tested were:

- 6m and 8m Single Strand Cable Bolts – One bulb per meter.
- 6m and 8m Twin Strand Cable Bolts - One bulb per meter.

ii) Resin Bolts (Figure 4 and Table 2)

The resin bolts tested were:

- 2.2m Thread Bar Posimix.

iii) Friction Bolts (Figure 5 and Table 3)

The friction bolts tested were:

- 2.4m Friction Bolt, 47mm diameter.

Test Site Locations

The site testing was conducted on the 8th September 2009 and 16th October 2009. The following sites were selected by Mr. Fred Harvey (Rock Mechanics Superintendent):

- 6 Level 5053 EXC – Calibration Tests for Cable Bolts, Resin bolts and Friction Bolts.
- 16 Level – Tests for grouted and resin solid bars (Test results not part of this paper).

6 Level 5053 East Crosscut Calibration Tests Set-up

Table 4 and Figure 6 provide information regarding the Single Strand Cable bolt calibration set-up.

Table 5 and Figure 7 provide information regarding the Twin Strand Cable bolt calibration set-up.

Table 6 and Figure 8 provide information regarding the Resin Bolt calibration set-up.

Table 7 and Figure 9 provide information regarding the Friction (Splitset) Bolts calibration set-up.

16 Level Tests

Four tests areas were selected:

- Test Area 1 - Northern shaft bypass excavation
- Test Area 2 - Old Crib Room
- Test Area 3 – Opposite Sub Station
- Test Area 4 – Opposite temporary crib room

16 Level were chosen in particular as the area is subject to very aggressive corrosive conditions. Test Area 2 (Old Crib Room) rock bolts exhibits a high level of corrosion and evidence of this was found in a rock bolt that dislodged from the backs (see Figure 10 - corroded rockbolt).

TESTS

A total of 56 bolts were tested between the 8th September and 16th October 2009 (19 bolts on the 8th September '09 and 37 bolts on the 16th October '09). The data obtained were transferred from the field Notebook PC of the Mod-Shock System and analysed with the IntegxD7 Programme (Integrity Testing Pty Ltd). Slight modifications (i.e. Graph presentation) to the programme's output graphs were made during the analysis process. These changes were necessary to provide George Fisher Lead-Zinc Mine with clear results.

TEST RESULTS AND DISCUSSIONS

Calibration Bolts – 6 Level 5053

Single Strand Cable Bolts

During our investigation (calibration testing) on the cable bolts we noticed an interesting phenomenon in the 2D mechanical admittance plot for the cable bolts which were previously noticed when testing cable bolts at another mine. This relates to the first part of the mechanical admittance plot (see below Figure 11 – Single Strand Cable Bolt Test No.4).

The two opposing curved black lines on the graph represent structural stiffness through good embedment or load transfer. The top (blue) and bottom (green) horizontal lines in the graph collectively represent the element's full diameter. The structural stiffness presented in the two dimensional plot together with the element's diameter are used to indicate whether any disturbance

(i.e. bolt necking, bolt volume reduction through corrosion, bolt shearing (Hartman, 2003) and/or ineffective grout or resin embedment) or reflection point can be detected during testing.

Figure 11 above shows some similarity to the load vs dissipation rate graph of a fairly recent (2004) collaborative investigation into the behaviour of cable bolts between the University of Saskatchewan, Saskatoon; Itasca Consulting Group, Inc., Minneapolis; University of British Columbia, Vancouver (BC) and the US National Institute of Safety and Health (NIOSH), which provided valuable information regarding their loading and strain behaviour.

An article by Martin et al (2004) showed that a critical load is required before the cable bolt, at a given location, sense any load. This was done through instrumented cable bolts loaded at the collar and plotted against recorded microstrain at individual gauge locations (see below in Figure 12 the load profile along the length of the cable at different collar loads). This implied that a gauge positioned 25.4 cm from the collar will sense load only when the collar load exceeds 25.4 × 2,043 N/cm.

The above phenomenon however would need to be confirmed with some instrumented bolts.

The test results obtained from the calibration tests are provided in Table 8 below and discussed further.

One of the vital pieces of information obtained from the non-destructive test is the "Head Stiffness" as this is the basis of all the load predictions and it also indicates the serviceability of the total bolt system. The head stiffness is the "E" prime of the bolt, measured as a direct measurement of the first part of the "structural stiffness plot", and is similar to a load/displacement graph for a pull out test.

The "bolt head stiffness (tonnes/mm)" is compared to the two model stiffness values "E" min and "E" max. "E" min is a bolt model with the bolt pinned at its toe (end anchored) but with no clamping (no resin or grouting) along its length. "E" max is a bolt model with an infinite rigid base and "*clamped*" (full column grouted / resin) along its length. These models are based on the work carried out by Davis & Dunn (1974).

The "Stiffness" value of the bolt is a good indicator of the serviceability of the bolt, but cannot be used in its entirety to give a serviceability rating for the bolt, as a number of factors come into affect when measuring the stiffness. The measurement of the stiffness can be affected by the fixity of the end of the bolt, the bonding effect of the resin/grout around the bolt and the bond from the rock to the resin/grout to ensure a fully encapsulated scenario of the bolt.

The following comments were made regarding the calibration tests for the single strand cable tests data interpretation:

- Tests 4 and 8 were done on the same bolt. The test results showed that the grouting of the toe and collar with a centre free length was clearly detected using the Modshock system. The anticipated grout location of 1.5m away from the toe and collar could not be accurately picked as the location of the grout appears to be slightly different that what has been proposed. This could very well be a sign of the lack of control during the grout installation.
- Test 3 results showed that the grouting of the toe (1.9m) and free length towards the collar was clearly detected using the Modshock system. The 6.0m long cable had a 0.692m tail and 5.31m length inside the hole. The anticipated grout embedment length of 1.9m has been detected (i.e. an area of load transfer) and appear to be located at 3.6m to 5.5m. The test result also suggests a gap towards the end of the hole (see Figure 13).
- Test 51 showed that the grout location appear to be between 2.7m and 4.4m an embedment length of 1.7m and not the anticipated 1.3m grouted length. The cable bolt had a tail length 0.402m and 5.598m length inside the 6.2m long hole. This would be one of the cable bolts required to be checked for correct embedment length.
- Test No 34 showed that the grouting of the cable bolt toe (0.9m) and free length towards the collar was clearly detected using the Modshock system. The 8.0m long cable had a 0.402m tail and 7.42m length inside the hole. The anticipated grout embedment length of 0.9m has been detected (i.e. an area of load transfer) and appear to be located at 6.35m to 7.35m. The test result also suggests a gap towards the end of the hole (see Figure 14).
- Test No 57 – Very poor signal – Test discarded.
- Test No 15 showed that the grouting of the cable bolt toe (2.95m) and free length towards the collar was clearly detected using the Modshock system. The 6.0m long cable had a 0.500m tail and 5.5m length inside the hole. The anticipated grout embedment length of 2.95m has been detected (i.e. an area of load transfer) and appear to be located at 3.4m to 5.85m. The test result also suggests a gap towards the end of the hole.

- Test No 13 and 9 showed that full column grouting of the hole did not occur as load transfer for both tests appear to be towards the latter part of the hole and a gap towards the earlier parts of the hole.

Twin Strand Cable Bolts

The test results obtained from the calibration tests are discussed below. A summary of the test results are provided below in Table 9.

The following comments are made regarding the calibration tests for the twin strand cable tests data interpretation:

- Tests 1 results showed that the grouting of the toe and collar with a centre free length was clearly detected using the Modshock system. The anticipated grout location of 1.5m away from the toe and collar could not be accurately picked as the location of the grout appears to be slightly different that what has been proposed. The toe area only shows a 0.7m area embedment length with bolt end at 6.0m.

- Test 2 results showed that the grouting of the toe and free length towards the collar was clearly detected using the Modshock system. The 6.0m long cable had a 0.550m tail and 5.45m length inside the hole. The anticipated grout embedment length of 1.5m appears to cover a much larger area (i.e. 2.0m – 2.4 to 4.4m) than what was proposed (i.e. an area of load transfer). The test result also suggests a gap towards the end of the hole (see Figure 15).

- Test 41 results showed that the grouting towards the toe of the hole was clearly detected using the Modshock system. The 6.0m long cable had a 0.580m tail and 5.42m length inside the hole. The anticipated grout embedment length of 1.5m appears to covers a slightly larger area (i.e. 3.0m – 4.7m) than what was proposed (i.e. an area of load transfer). The test result also suggests a gap towards the end of the hole (See Figure 16).

- Test 52 results clearly shows the end of the bolt at 7.7m, but indicates that the grouting towards the toe of the hole only covered about 1m (i.e. between 6-7m). The 8.0m long cable had a 0.30m tail and 7.7m length inside the hole. The anticipated grout embedment length of 1.5m appears to covers a slightly smaller area (i.e. 6.0m – 7.0m) than what was proposed (i.e. an area of load transfer). The test result also suggests a gap towards the end of the hole.

- A test 58 result does not clearly show the end of the bolt at 7.36m and indicates that the grouting towards the toe of the hole covers about 1m (i.e. between 6.5-7.36m). The 8.0m long cable had a 0.640m tail and 7.36m length inside the hole. The anticipated grout embedment length of 3.0m appears to cover only a smaller area (i.e. 6.5m – 7.36m) than what was proposed (i.e. an area of load transfer). The test results indicate that grouting was done for a 1.7m distance to the back of the hole (i.e. 8.2m). The result also suggests remnant grout lower down the hole as an area of high load transfer is shown.

- A test 62 result does not clearly show the proposed end of the bolt at 5.7m. The results indicate that the grouting towards the toe of the hole covers about 1m (i.e. between 5.2m – 6.2m). The 6.0m long cable had a 0.30m tail and 5.7m length inside the hole. The anticipated grout embedment length of 3.0m appears to cover only a smaller area (i.e. 5.2m – 6.2m) than what was proposed (i.e. an area of load transfer). The result, similar to the previous test 58m, suggests remnant grout lower down the hole as an area of high load transfer is shown.

- Test 21 results clearly shows a full column grouted bolt with only one major reflection point within in the total length at 1.25m. The 2D mechanical admittance plot clearly indicates a full column grouted bolt with also similar stress / strain patterns closer to the collar of the hole. When testing cable bolts this is typical what the signature should show.

Resin Bolts

The test results obtained from the calibration tests are discussed below. A summary of the test results are provided below in Table 10.

The following comments are made regarding the calibration tests for the resin bolt tests data interpretation:

- Test 27 was a shorter bolt installed by George Fisher and following a detailed analysis of the bolt signal and admittance plot it shows clearly areas of load transfer and / or mechanical admittance. It is recognised that through the initial review the bolt appeared to be the full 2.2m length. However two significant reflection points showed a no load transfer or discontinued admittance in the 2D plot (see figure 17).

- Test 29 was a full 2.2m long bolt with a 1m encapsulation showed in the 2D mechanical admittance plot. The free length showed two spikes, possibly indicating some remnant resin providing some admittance.

- Tests 2-8 were conducted on the same bolt. The multiple tests were done to check whether there would be any correlation between the actual load applied and the load detected through the Modshock system within the elastic range. The bolt had an encapsulation of 1m which was clearly shown in the 2D mechanical admittance plot. The plot does however indicate that the bolt was not fully covered in resin with a reflection (no load transfer) between 1.3m to 1.6m. The results from the load vs deformation plot shows that no correlation could be established between applied load and the load detected through the Modshock system. What is interesting to note is the increase in bar stiffness from test 2 through to test 4 and then stabilises at 2.2 tonnes/mm.

- Tests 8-15 were conducted on the same bolt. The multiple tests were done to check whether there would be any correlation between the actual load applied and the load detected through the Modshock system within the elastic range. The bolt had an encapsulation of 0.5m which was not clearly shown in the 2D mechanical admittance plot. The results obtained indicate no resin encapsulation towards the toe of the hole and rather an encapsulation between 0.7m and 1.6m. Overall the resin installation of this bolt appears to not resemble the proposed 0.5m toe encapsulation. The results from the load vs deformation plot showed some correlation between applied load and the load detected through the Modshock system. What is interesting to notice is that there is a gradual increase in the bar stiffness from test 8 through to test 15 (i.e. 0.6 tonnes/mm to 3.9 tonnes/mm – see Figure 18).

- Test 24 was a 2.2m long bolt with a proposed 0.5m encapsulation at the toe of the hole. The 2D mechanical admittance plot however suggests the resin encapsulation and load transfer to be between 1.1m and 1.7m. We also noticed a reflection point in the early part of the mechanical admittance plot which suggests an increase in load due to the pre-loading during the installation. This however would require further confirmation through instrumented bolts.

- Test 22 was a 2.2m long bolt with a proposed full column encapsulation. The 2D mechanical admittance plot clearly shows this to be the case with load transfer

throughout the length of the bolt. This bolt will thus be classified as a serviceable bolt with good anchorage (see Figure 19).

Friction Bolts

A summary of the test results are provided below in Table 11.

The test results for the Friction bolts are still under scrutiny as an appropriate diameter is still under investigation for the friction bolts. Hence the absolute values provided above may not be the accurate representation for the friction bolts as the stiffness principal value is shown with much higher values than the actual friction bolt tensile strength. With more tests done on friction bolts a more accurate stiffness value will be determined. The mechanical admittance plot however has provided the much required confidence with regard to signal continuity or admittance.

The following comments are made regarding the calibration tests for the friction stabiliser tests data interpretation:

- Test 30 was a 2.4m long friction stabiliser with an area that was damaged (grind) at round 1.2m. The 2D mechanical admittance plot clearly shows mechanical admittance from the collar to about 1.2m and then no load transfer detected from about 1.7m to the toe of the friction stabiliser (see Figure 20).

- Test 31 appear to have very similar mechanical admittance as test 30 with the the 2D mechanical admittance plot clearly showing mechanical admittance from the collar to about 1.7m and then no load transfer detected from about 1.7m to the toe of the friction stabiliser.

- Test 35 appear to have a different mechanical admittance as the two previous tests (i.e. 30 and 31) with the 2D mechanical admittance plot showing an area of no load transfer between 0.6m to 1.1m and no load transfer towards the toe of the bolt. The no load transfer towards the toe agrees well with the friction bolt’s tapered end.

- Test 34 was quite an interesting test. The friction bolt was damaged (i.e. cut slots) by George Fisher personnel at 1.2m from the collar. During the installation the friction bolt collapsed and was driven into itself. The test results or mechanical admittance plot shows that no load transfer occurred after 1.2m which agrees well with the actual bolt collapse (see Figure 21).

CONCLUSIONS

The non-destructive calibration testing conducted using the Modshock System will provide George Fisher rock mechanics personnel with some confidence in detecting rock reinforcement subject to corrosion and/or poor quality installation. The rock reinforcement installation for calibration purposes was done in a semi controlled environment where bolts were installed to simulate either poor installation techniques or damaged / corroded elements.

The test work on the cable bolts was slightly affected by gaps behind the plates (i.e. plates not tight with excavation surface – this is caused by the uneven nature of the rock surface). This had a distorting affect on the signals, some of which could not be relied on for analysis purposes and thus required several attempts to obtain a proper / valid signal.

During our investigation (calibration testing) on the cable bolts we noticed an interesting disturbance phenomenon in the 2D mechanical admittance plot for all the cable bolts tests and which were previously noticed when testing cable bolts at another mine. This relates to the first part of the mechanical admittance plot. Following an investigation into cable bolt behaviour an article was found describing load / strain dissipating from the collar, which could be an explanation for the disturbance in the 2D mechanical admittance plot. This phenomenon however would need to be confirmed with some instrumented bolts.

The test results for the calibration bolts showed that the grout / resin embedded and free length of the cable bolts and resin bolts were clearly detected using the Modshock System. However the anticipated grout / resin locations for a few of the test specimens could not be accurately picked, as the location of the grout / resin appeared to be in a slightly different location than what has been proposed. Discussions surrounding this let us to believe that the lack of proper control (i.e. not laboratory environment) during the grout / resin installation could very well be the reason for detecting some of the resin / grout installations in another location.

As the main use for the Modshock system is mainly to determine the integrity of rock reinforcement elements, the calibration testing provided us with enough confidence to use the Modshock system as a quality control tool. The two loading calibration tests (i.e. cumulative load pull tests compared with load stiffness calculations from the Modshock Tests for the elastic deformation range) conducted on resin bolts are inconclusive at this stage as only one of the two tests had positive results. Tests 8-15 (i.e. resin bolt encapsulation of 0.5m) showed a gradual increase in calculated bar stiffness (i.e. 0.6 - 3.9 tonnes/mm) from test 8 through to test 15, which indicates that the Modshock system do capture the bolt load response. There was also noticeable correlation between

the load pull tests and Modshock calculated load values for these tests. Hence it is acknowledge as indicated in our paper published at the international tunnelling conference in Vancouver (Hartman, et al., 2010b) that the following work is required to increase confidence in other data interpretation:

- Calibration testing to confirm the elastic load increase in tendons and solid rebars as referred to in this paper and
- Confirmation of two dimensional graph amplitude variance and descriptive analysis.

The test results for the friction bolts are still under scrutiny as an appropriate diameter is still under investigation for the friction bolts. Hence the absolute values provided in this report may not be an accurate representation for the friction bolts as the stiffness principal values obtained are much higher than the actual friction bolt tensile strength. But with more tests done on friction bolts a more accurate stiffness value will be determined. The mechanical admittance plot however has provided the much required confidence with regard to signal continuity or admittance as these results are in agreement with friction bolt behaviour (i.e. low or no load transfer towards the toe of the hole).

The non destructive testing can be employed to improve understanding of rock reinforcement elements loading conditions and verify the following important aspects of ground support (Hartman et al, 2010b):

- Verification of current design – this relates to cable bolt anchorage e.g. if the design or selection is for 10m cable bolts and the tests indicates poor anchorage (i.e. a section of around 2m – critical embedment length) or poor load transfer in the 2D mechanical admittance plot as a result of poor grouting and inefficient bonding, it would indicate that the design have been compromised.
- Integrity check of rock reinforcement in main access ways e.g. decline where the bolts need to be intact throughout the life of the excavation – this would then be a check for corrosion (significant volume loss) and/or overstressing where the calculated bolt stiffness is high.
- The third but very important check is for the general quality of ground support installation and this would then become part of the mine’s or underground construction’s ground support system frequent quality integrity check.

ACKNOWLEDGEMENTS

The authors would like to thank the George Fisher Mine Management for the opportunity to conduct the non-destructive rock reinforcement testing and publishing the results. A special thanks to Mr. Fred Harvey (Rock Mechanics Superintendent) for his arrangements and Mr. Graham Browne (Technical Rock Mechanics Officer) who was part of the testing team.

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FIGURE CAPTIONS

Fig 1 – Hydraulic Pull testing device used at the George Fisher Mine.

Fig 2 – George Fisher Lead-Zinc Mine (2009 – XSTRATA Operations).

Fig 3 – Cable Bolt Diagram.

Fig 4 – Thread Bar Posimix.

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Fig 11 - 2D Mechanical Admittance Plot for Single Strand Cable Test No.4.

Fig 12 – Collar load plotted against A, microstrain (load profile curve) and B, distance from head of cable (load correlation curve, Martin et al (2004)).

Fig 13 - 2D Plot for Single Strand Cable Test No.3.

Fig 14 - 2D Plot for Single Strand Cable Test No.34.

TABLE CAPTIONS

FIGURES

Fig 1 – Hydraulic Pull testing device used at the George Fisher Mine.

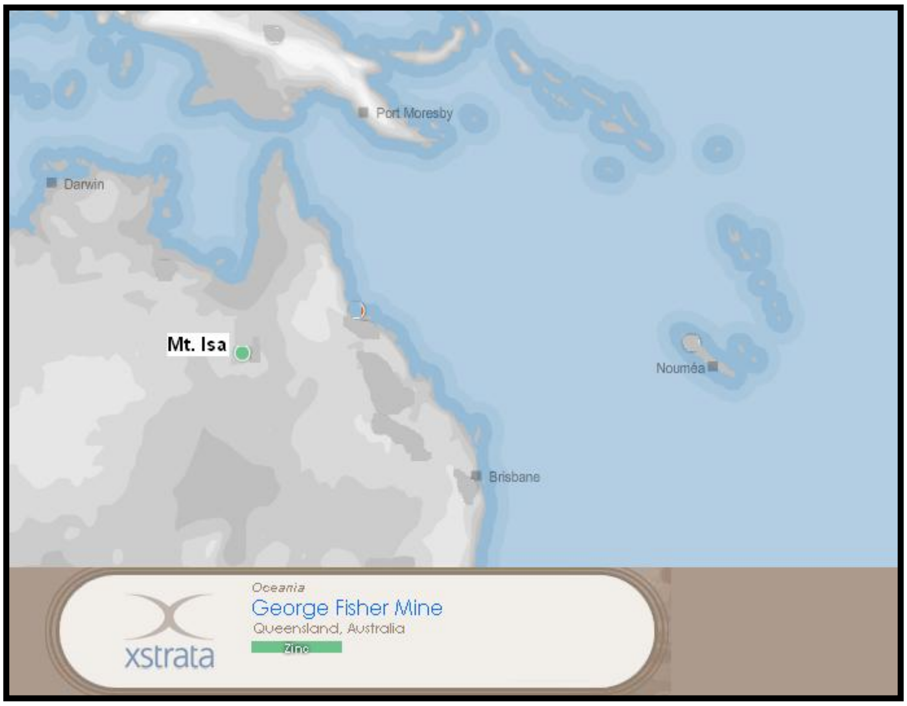

Fig 2 – George Fisher Lead-Zinc Mine (2009 – XSTRATA Operations).

Fig 3 – Cable Bolt Diagram.

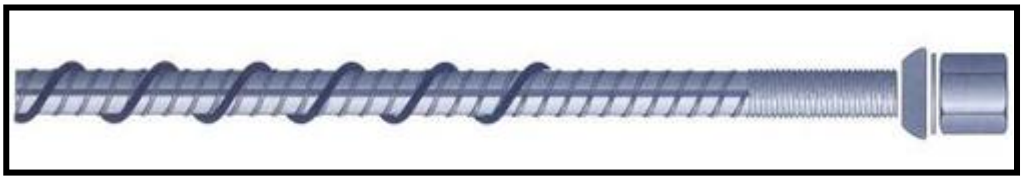

Fig 4 – Thread Bar Posimix.

Fig 5 – Friction Bolt 47mm diameter diagram.

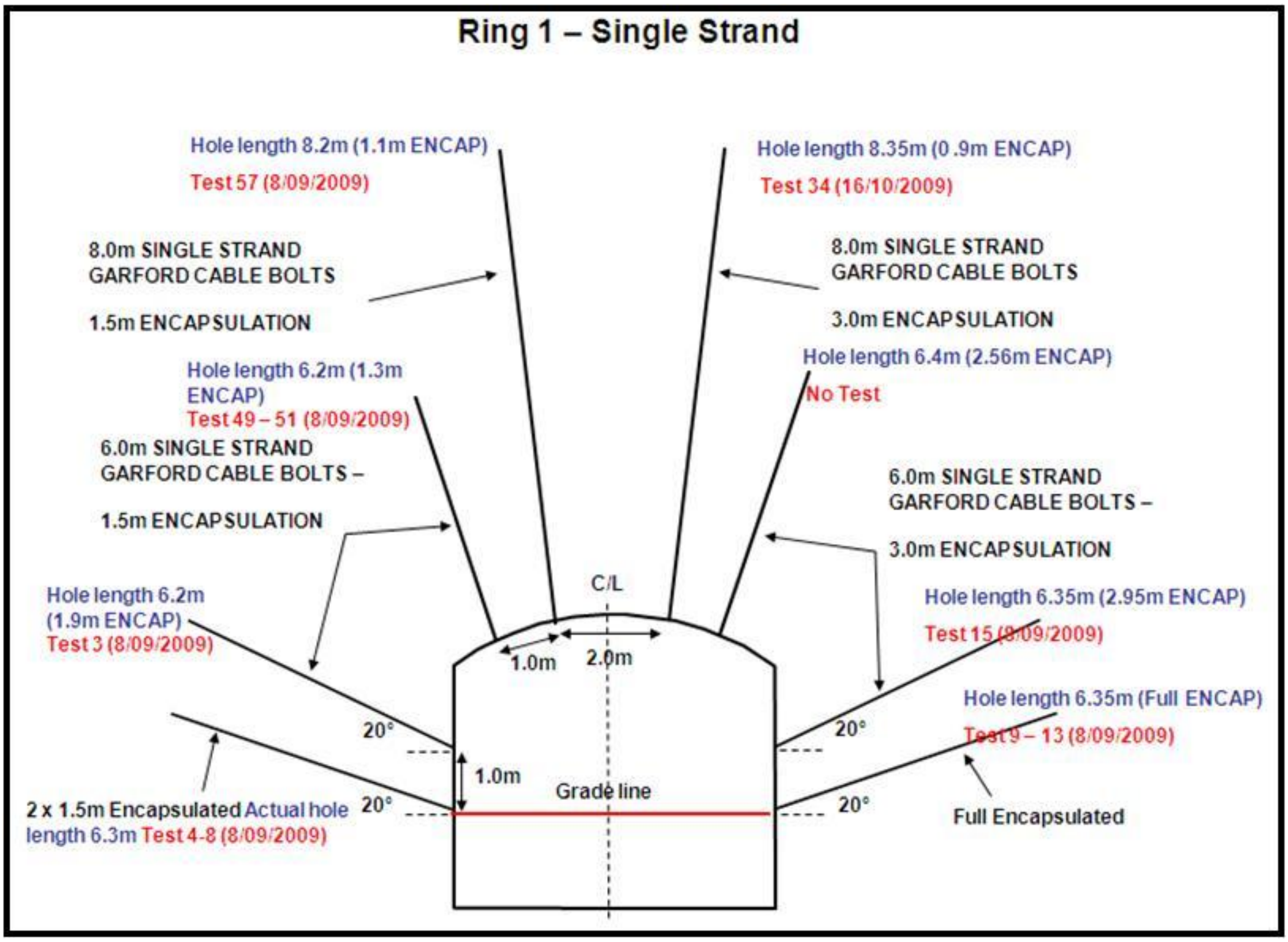

Fig 6 – Schematic illustrating single strand cable bolt calibration set-up.

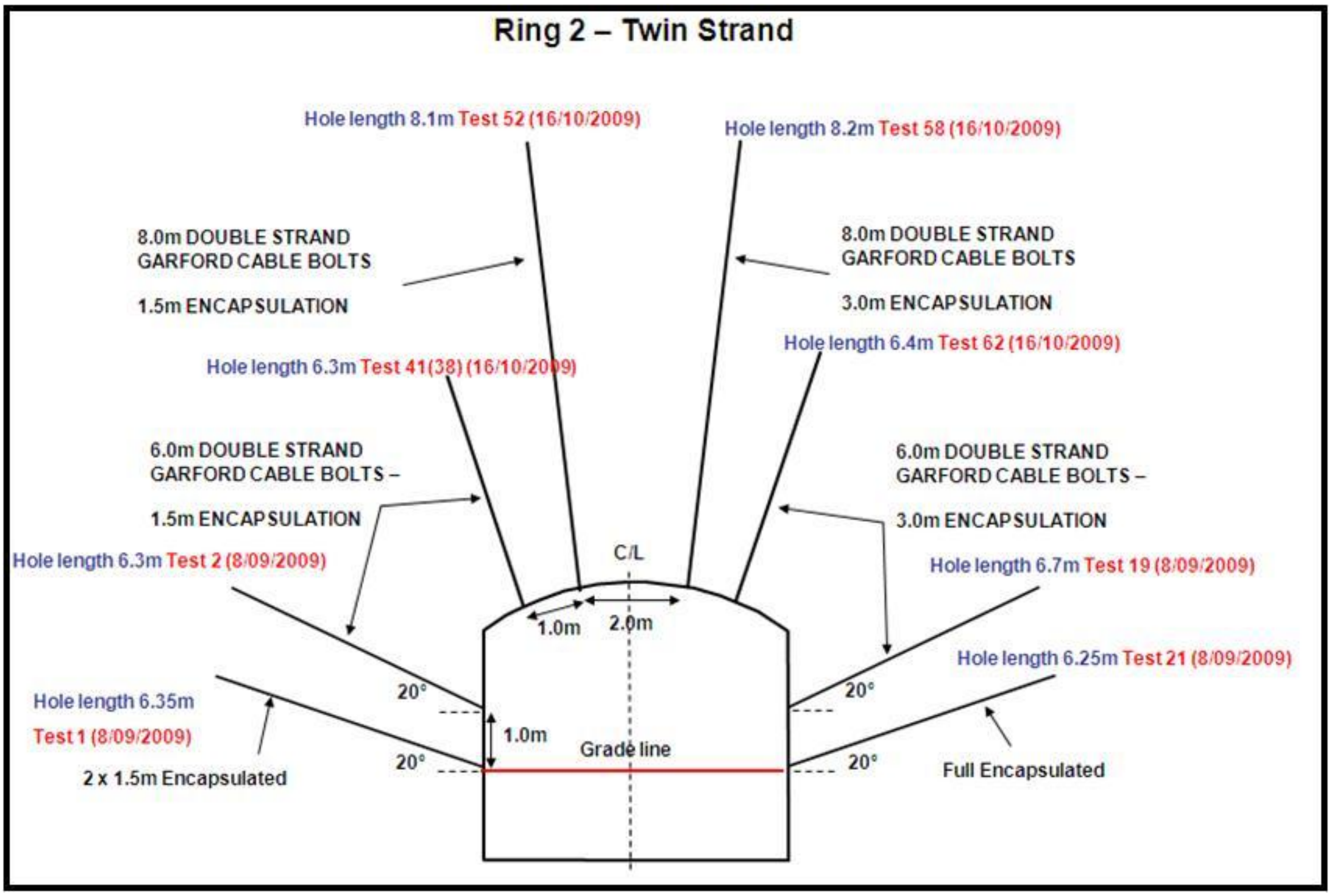

Fig 7 – Schematic illustrating twin strand cable bolt calibration set-up.

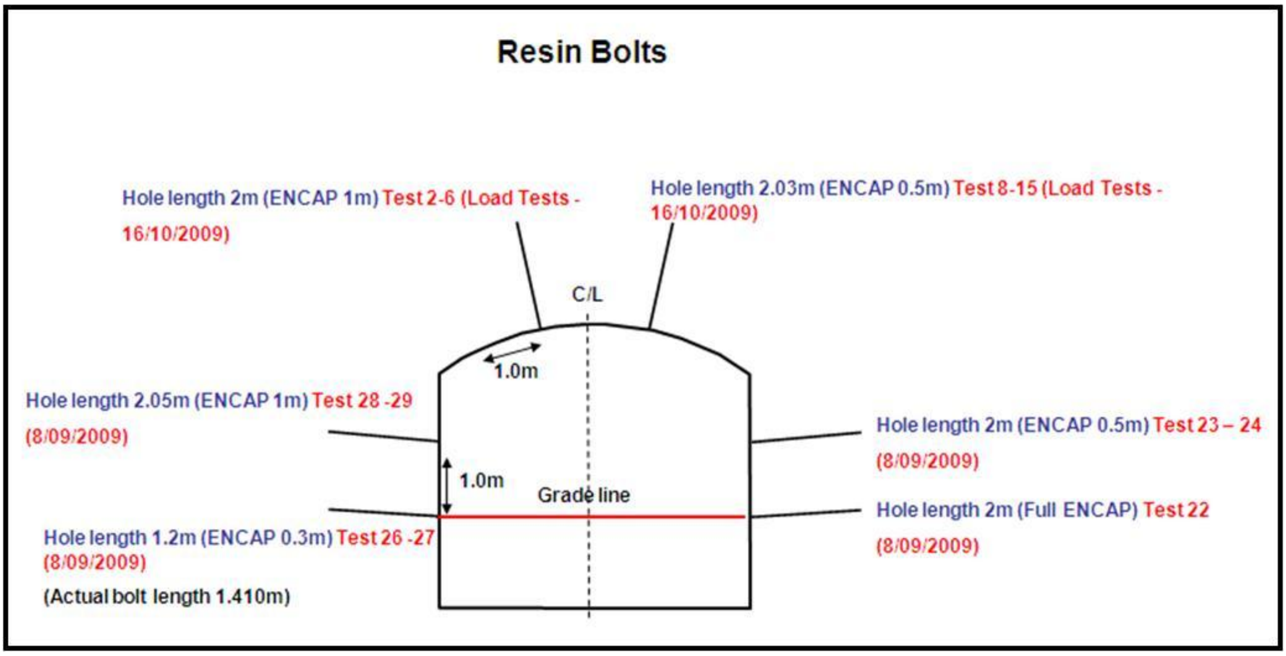

Fig 8 – Schematic illustrating resin bolts calibration set-up.

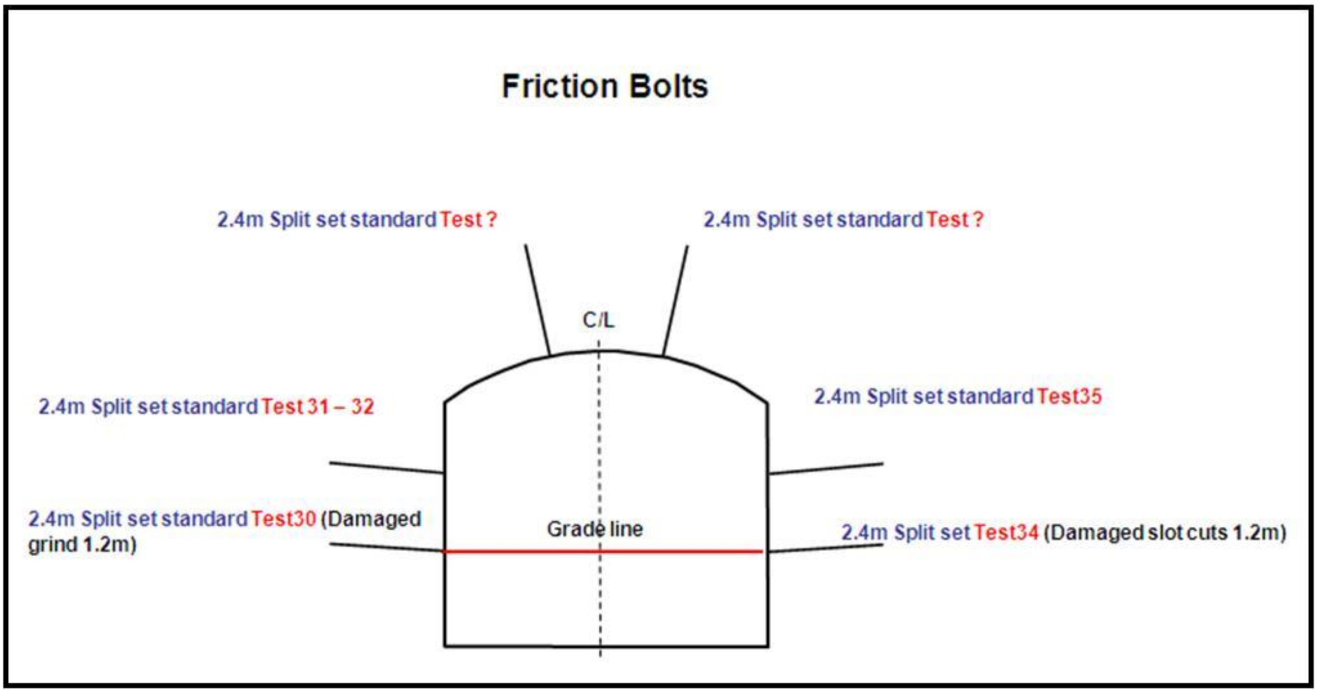

Fig 9 – Schematic illustrating resin bolts calibration set-up.

Fig 10 – Corroded bolt, 16 level crib room.

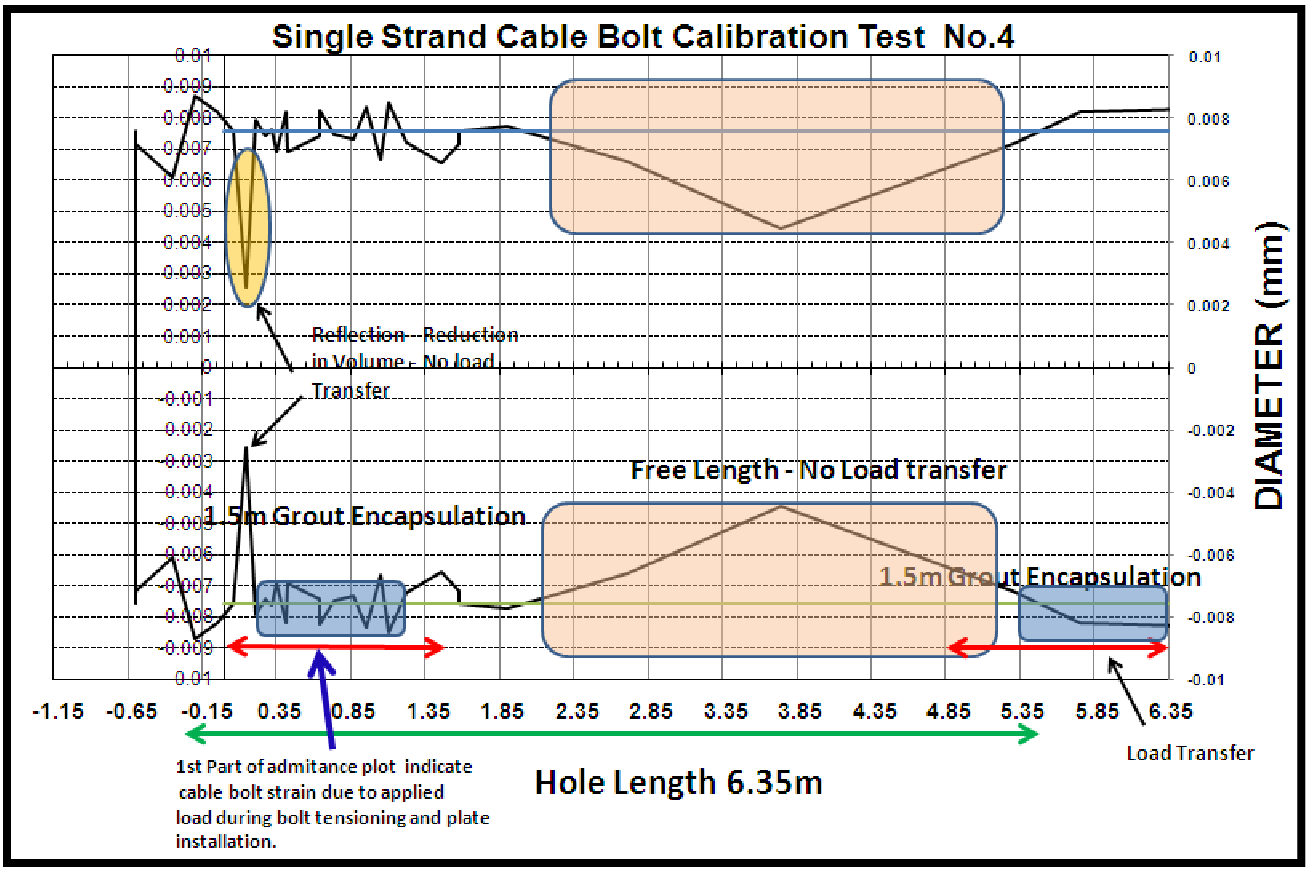

Fig 11 - 2D Plot for Single Strand Cable Test No.4.

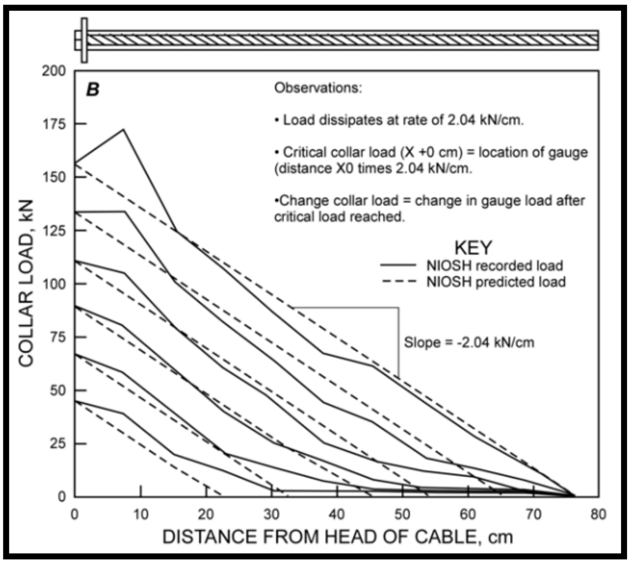

Fig 12 – Collar load plotted against A, microstrain (load profile curve) and B, distance from head of cable (load correlation curve, Martin et al (2004)).

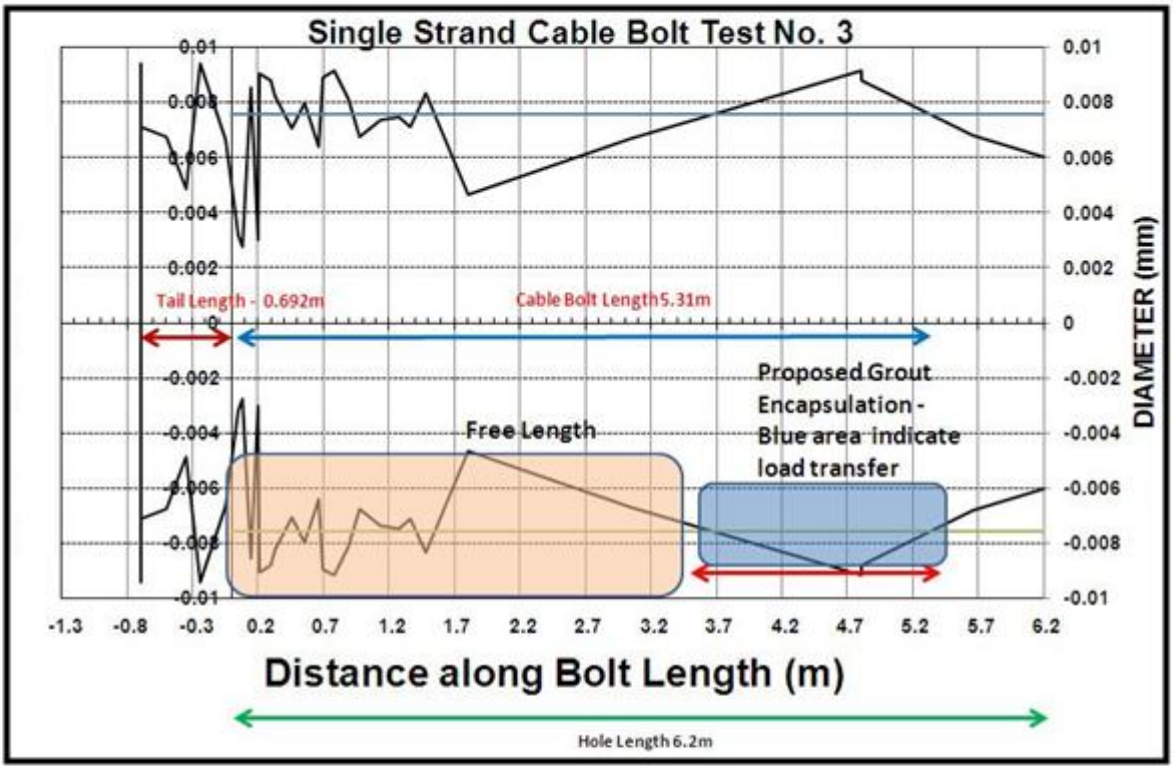

Fig 13 - 2D Plot for Single Strand Cable Test No.3.

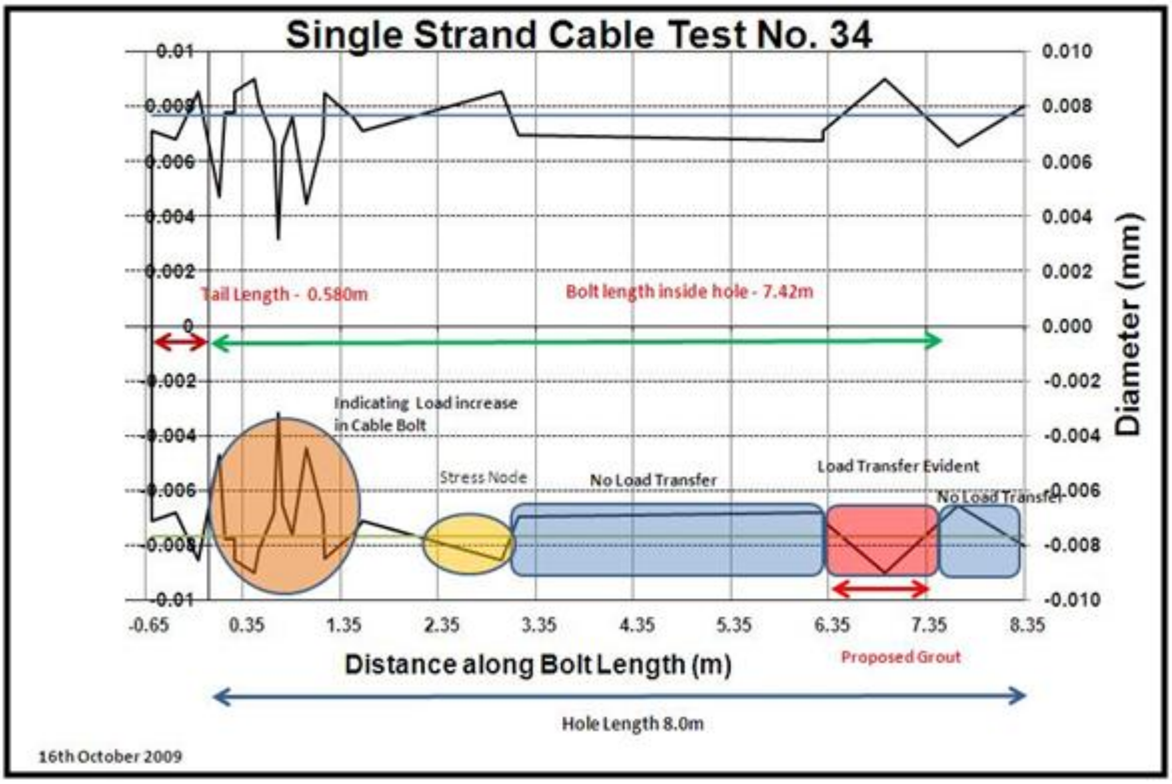

Fig 14 - 2D Plot for Single Strand Cable Test No.34.

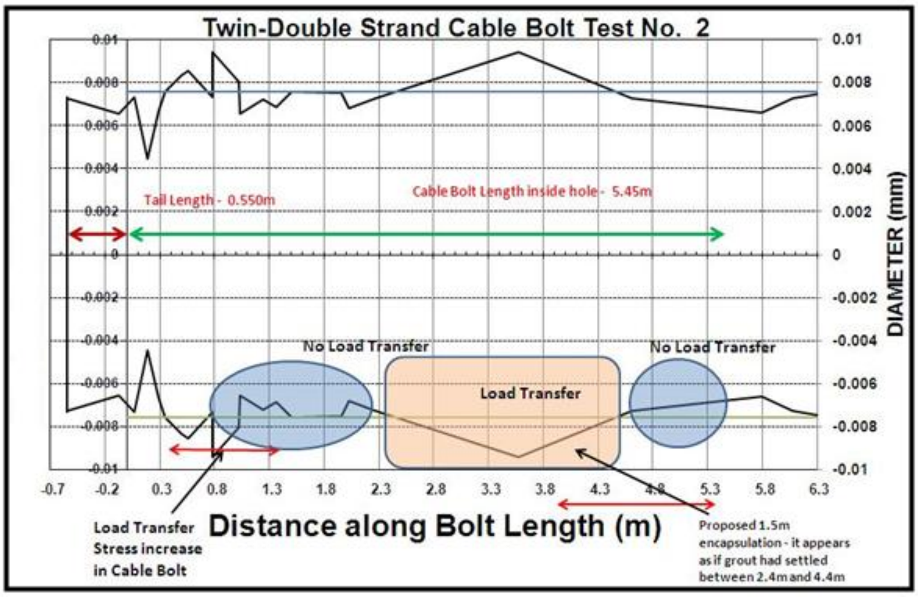

Fig 15 - 2D Plot for Twin Strand Cable Test No.2.

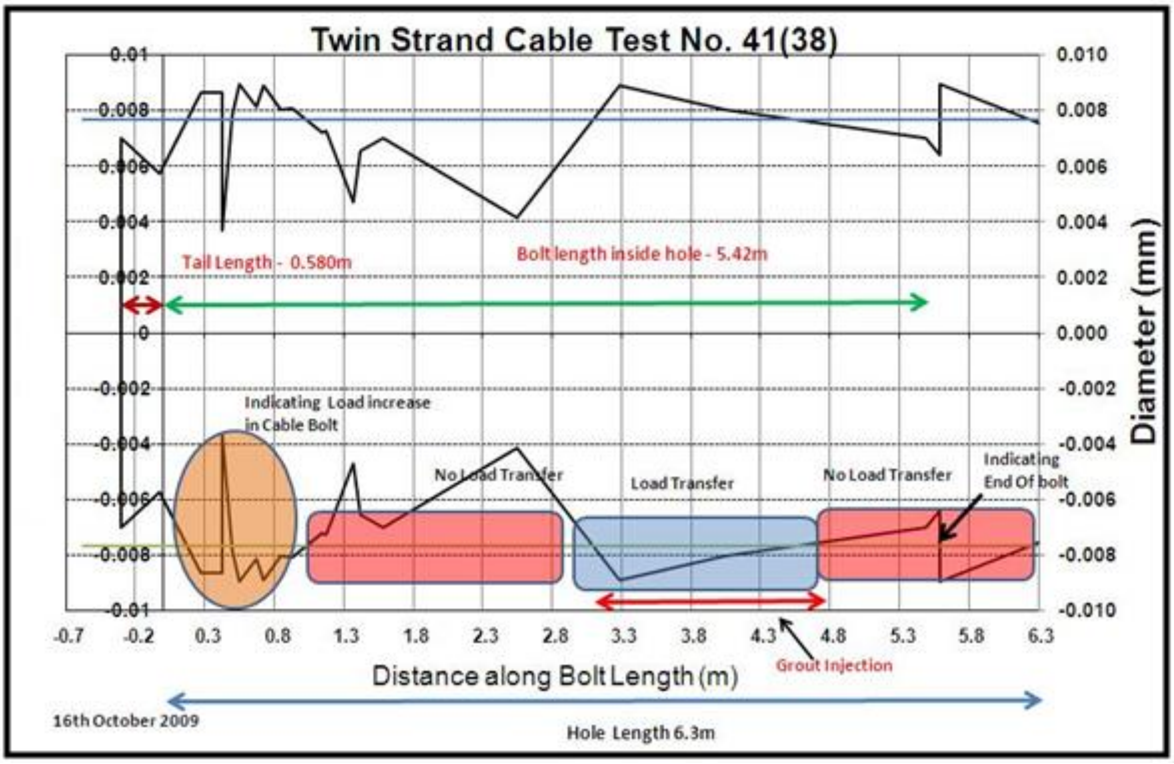

Fig 16 - 2D Plot for Twin Strand Cable Test No.41.

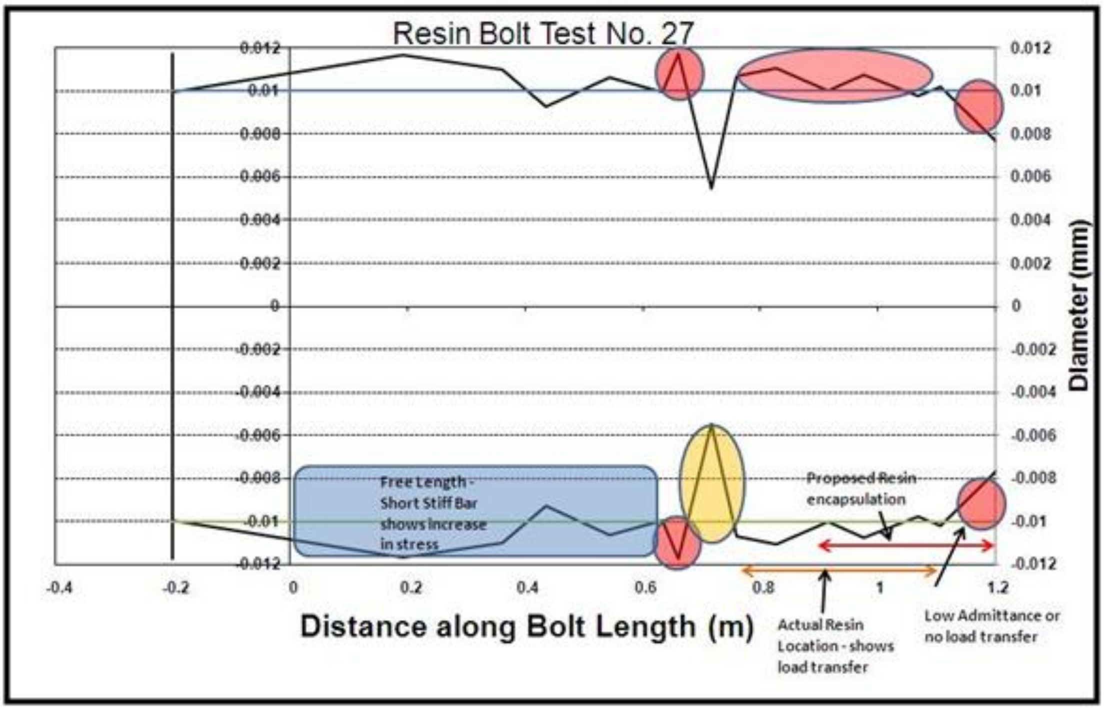

Fig 17 - 2D Plot for Resin Bolt Test No.27.

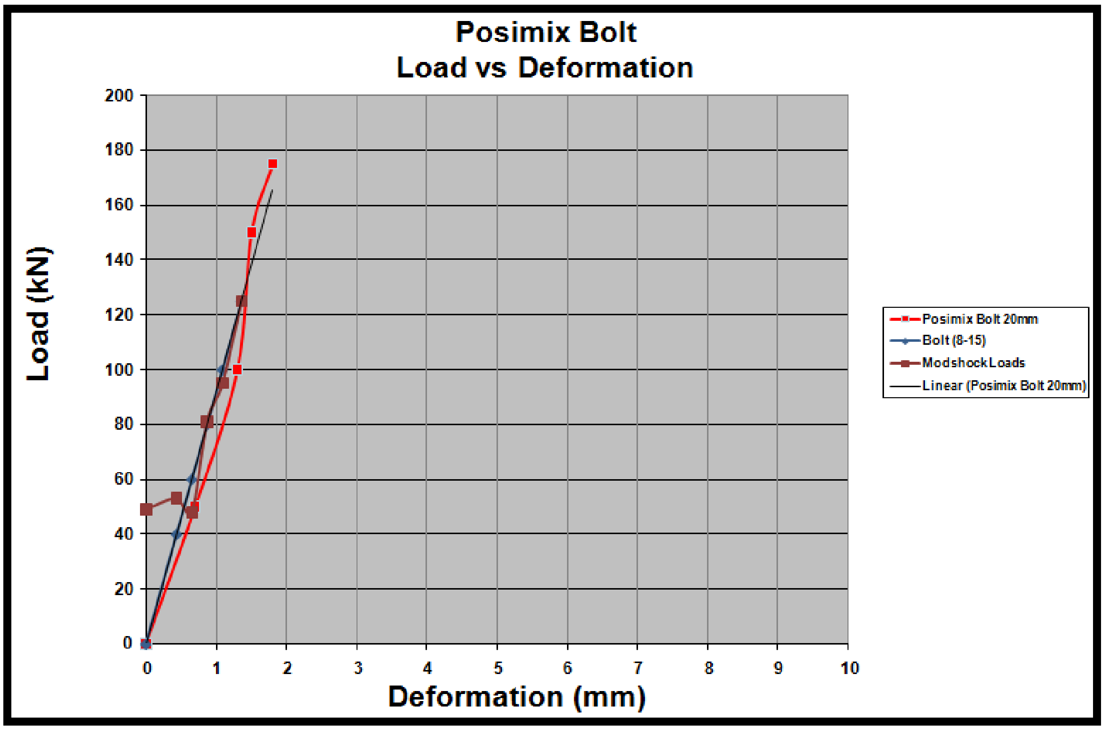

Fig 18 – Load vs Deformation graph for bolt test 8 -15 (Modshock vs Pull Test Results).

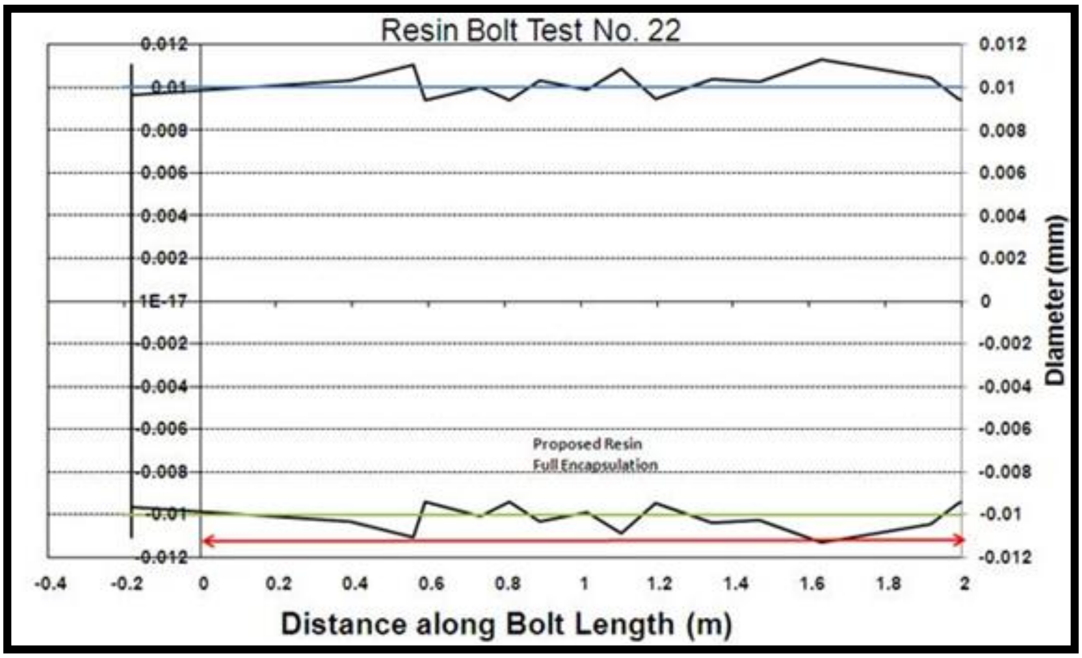

Fig 19 - 2D Plot for Resin Bolt Test No.22.

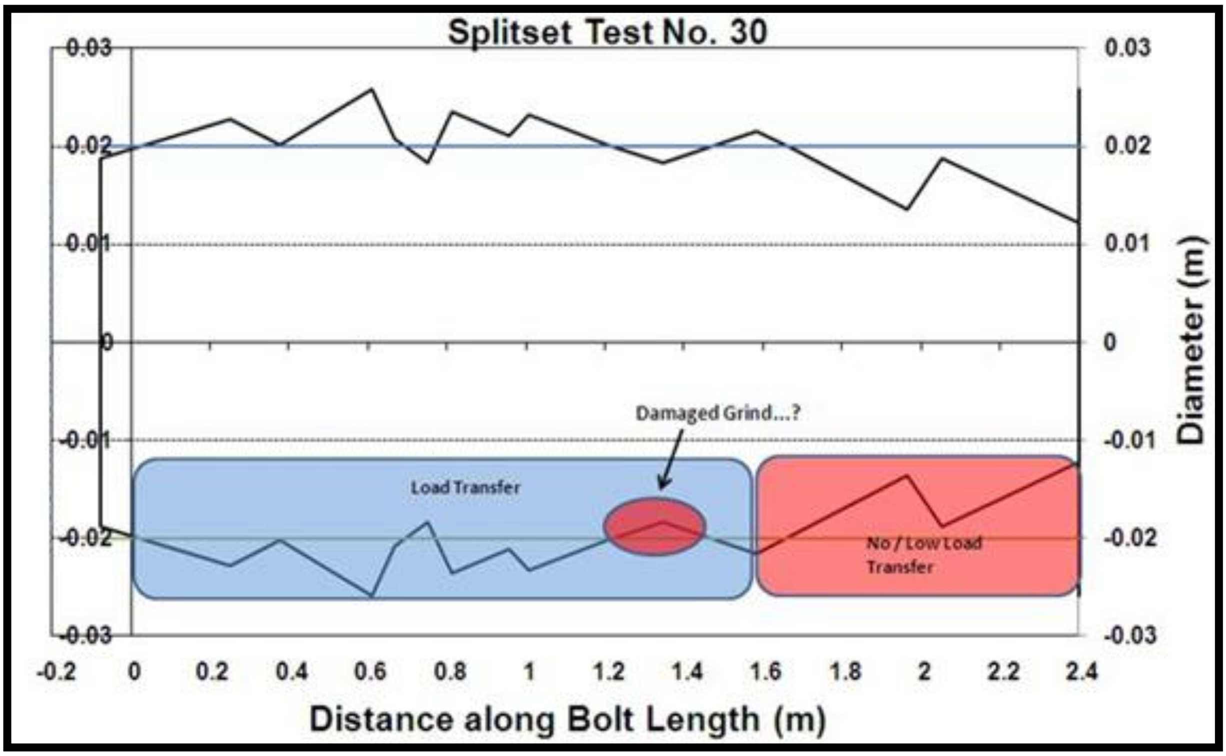

Fig 20 - 2D Plot for Splitset Test No.30.

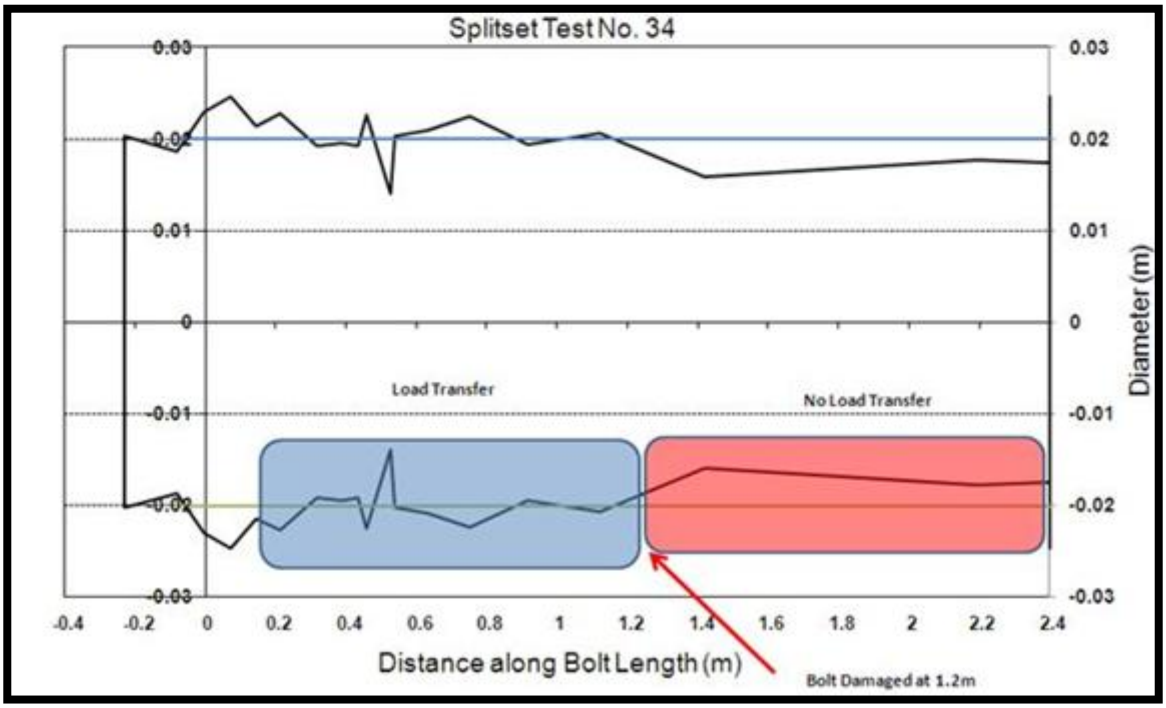

Fig 21 - 2D Plot for Splitset Test No.34.

TABLES

Table 1 – Mechanical Properties for 15.2mm diameter Cable Bolt.

Mechanical Properties		
	Minimum	**Typical**
Core Diameter – unbulbed section (mm)	-	15.2
Cross Sectional Area – unbulbed section (sq.mm)	-	143
Yield Force – 0.2% (kN)	212	235
Tensile Force (kN)	250	265
Elongation on 600mm length (%)	3.5	6.5
Mass per metre – Bar (kg/m)	-	1.125

Table 2 – Mechanical Properties for Thread bar Posimix.

Thread Bar Posimix Physical Properties	**Minimum**		**Typical**	
Yield Strength	500 MPa	160 kN	550 MPa	170 kN
Ultimate Tensile Strength of Tube	600 MPa	185 kN	650 MPa	200 kN
Calculated Shear Strength	120 kN		130 kN	
Standard Elongation	16%		19%	
Uniform Elongation			12%	
Mass Per Metre			2.47kg	
Bar Core Diameter			20.0mm	
Cross Sectional Area of Bar			310mm^2	
Major Bar Diameter			22.1mm max	
Bar Straightness to AS 1442-1991				

Table 3 – Mechanical Properties for the Friction Bolt.

47mm Friction Bolt Physical Properties	Minimum		Typical	
Yield Strength	345 MPa	120 kN	445 MPa	160 kN
Ultimate Tensile Strength of Tube	460 MPa	165 kN	510 MPa	180 kN
Friction Bolt Diameter	47mm			
Recommended Hole Diameter Range	43mm min / 45.5mm max			
Mass Per Metre			2.79 kg	

Table 4 – Single Strand Cable Bolts Calibration Set-up.

Bolt Test No.	Test Date	Hole Length	Hole Angle	Comments
4 (8)	8th Sept. '09	6.3m	20 deg (Sidewall)	1.5m encapsulation at the toe and collar
3	8th Sept. '09	6.2m	20 deg (Sidewall)	1.9m encapsulation at the toe
49 (51)	8th Sept. '09	6.2m	75 deg (Backs)	1.9m (1.5m) encapsulation at the toe
57	8th Sept. '09	8.2m	75 deg (Backs)	1.1m encapsulation at the toe
34	16th Oct. '09	8.35m	75 deg (Backs)	0.9m encapsulation at the toe
15	8th Sept. '09	6.35m	20 deg (Sidewall)	2.95m encapsulation at the toe
9 (13)	8th Sept. '09	6.35m	20 deg (Sidewall)	Full encapsulation

Table 5 – Twin Strand Cable Bolts Calibration Set-up.

Bolt Test No.	Test Date	Hole Length	Hole Angle	Comments
1	8th Sept. '09	6.35m	20 deg (Sidewall)	1.5m encapsulation at the toe and collar
2	8th Sept. '09	6.3m	20 deg (Sidewall)	1.5m encapsulation at the toe
41 (38)	16th Oct. '09	6.3m	75 deg (Backs)	1.5m encapsulation at the toe
52	16th Oct. '09	8.1m	75 deg (Backs)	1.5m encapsulation at the toe
58	16th Oct. '09	8.2m	75 deg (Backs)	3.0m encapsulation at the toe
62	16th Oct. '09	6.4m	75 deg (Backs)	3.0m encapsulation at the toe
19	8th Sept. '09	6.7m	20 deg (Sidewall)	3.0m encapsulation at the toe
21	8th Sept. '09	6.25m	20 deg (Sidewall)	Full encapsulation

Table 6 – Resin Bolt Calibration Set-up.

Bolt Test No.	Test Date	Hole Length	Hole Angle	Comments
26(27)	8th Sept. '09	1.2m (Actual bolt Length 1.4m)	20 deg (Sidewall)	0.3m encapsulation at the toe
28(29)	8th Sept. '09	2.05m	20 deg (Sidewall)	1.0m encapsulation at the toe
2-6	16th Oct. '09	2.0m	75 deg (Backs)	1.0m encapsulation at the toe – Pull Testing conducted at 4, 6, 10 and 12.5 tonnes
9 - 15	16th Oct. '09	2.03m	75 deg (Backs)	0.5m encapsulation at the toe - Pull Testing conducted at 4, 6, 8, 10 and 12.5 tonnes
23(24)	8th Sept. '09	2.0m	20 deg (Sidewall)	0.5m encapsulation at the toe
22	8th Sept. '09	2.0m	20 deg (Sidewall)	Full encapsulation

Table 7 – Friction Bolts Calibration Set-up.

Bolt Test No.	Test Date	Hole Length	Hole Angle	Comments
30	8th Sept. '09	2.4m	20 deg (Sidewall)	Standard installation (Damaged grind at 1.2m)
31 (32)	8th Sept. '09	2.4m	20 deg (Sidewall)	Standard installation
35	8th Sept. '09	2.4m	20 deg (Sidewall)	Standard installation
34	8th Sept. '09	2.4m	20 deg (Sidewall)	Standard installation (Damaged slots cut at 1.2m)

Table 8 – Single Strand Cable Bolts test result summary.

Test	Element Test Number	Bolt Type	Location	Nominal Length of Element (m)	Bar Stiffness (t/mm)	"E" max (t/mm)	"E" min (t/mm)
MCAB3	3	Single strand Cable Bolt	6 Level 5053	7.0	2.2	1.5	0.9
MCAB 4	4	Single strand Cable Bolt	6 Level 5053	7.0	1.3	1.6	1.1
MCAB 8	8	Single strand Cable Bolt	6 Level 5053	7.0	0.8	2.2	1.0
MCAB 9	9	Single strand Cable Bolt	6 Level 5053	7.0	0.9	2.1	1.2
MCAB 10	10	Single strand Cable Bolt	6 Level 5053	7.0	1.5	2.6	0.8
MCAB 13	13	Single strand Cable Bolt	6 Level 5053	7.0	0.9	1.8	1.0
MCAB 15	15	Single strand Cable Bolt	6 Level 5053	7.0	0.5	2.0	1.1
MCAB34	34	Single strand Cable Bolt	6 Level 5053	8.6	0.4	1.8	0.9
MCAB 49	49	Single strand Cable Bolt	6 Level 5053	7.0	0.5	1.8	0.8
MCAB 51	51	Single strand Cable Bolt	6 Level 5053	7.0	0.2	2.5	1.0
MCAB 57	57	Single strand Cable Bolt	6 Level 5053	7.0	6.4	2.3	1.0

Table 9 – Twin Strand Cable Bolts test result summary.

Test	Element Test Number	Bolt Type	Location	Nominal Length of Element (m)	Bar Stiffness (t/mm)	"E" max (t/mm)	"E" min (t/mm)
MCAB 1	1	Twin Strand Cable Bolt	6 Level 5053	7.0	0.5	2.0	1.1
MCAB 2	2	Twin Strand Cable Bolt	6 Level 5053	7.0	2.2	1.9	0.8
MCAB 19	19	Twin Strand Cable Bolt	6 Level 5053	7.0	0.4	1.6	0.7
MCAB 21	21	Twin Strand Cable Bolt	6 Level 5053	6.3	1.4	1.6	0.7
MCAB 52	52	Twin Strand Cable Bolt	6 Level 5053	8.6	1.1	1.6	0.6
MCAB 58	58	Twin Strand Cable Bolt	6 Level 5053	8.6	0.5	2.0	0.3
MCAB41	41	Twin Strand Cable Bolt	6 Level 5053	8.6	0.7	2.2	1.0
MCAB62	62	Twin Strand Cable Bolt	6 Level 5053	8.6	0.7	1.9	0.8

Table 10 – Resin Bolts test result summary.

Test	Element Test Number	Bolt Type	Location	Nominal Length of Element (m)	Bar Stiffness (t/mm)	"E" max (t/mm)	"E" min (t/mm)
MRES27	27	Resin Bar	6 Level 5053	2.2	3.3	2.5	1.4
MRES29	29	Resin Bar	6 Level 5053	2.2	2.3	3.4	1.8
MRES2	2	Resin Bar	6 Level 5053	2.2	1.1	2.7	1.1
MRES3	3	Resin Bar	6 Level 5053	2.2	3.3	3.2	1.6
MRES4	4	Resin Bar	6 Level 5053	2.2	3.1	3.4	1.8
MRES5	5	Resin Bar	6 Level 5053	2.2	2.1	3.9	1.6
MRES6	6	Resin Bar	6 Level 5053	2.2	2.1	3.1	1.5
MRES 8	8	Resin Bar	6 Level 5053	2.2	0.6	2.5	0.6
MRES9	9	Resin Bar	6 Level 5053	2.2	1.0	3.3	1.7
MRES11	11	Resin Bar	6 Level 5053	2.2	0.8	3.3	1.9
MRES12	12	Resin Bar	6 Level 5053	2.2	2.3	4.1	2.4
MRES13	13	Resin Bar	6 Level 5053	2.2	2.9	3.6	1.8
MRES15	15	Resin Bar	6 Level 5053	2.2	3.9	3.6	1.6
MRES24	24	Resin Bar	6 Level 5053	2.2	0.8	3.8	1.8
MRES22	22	Resin Bar	6 Level 5053	2.2	1.9	3.3	1.7
MRES 1	1 (22)	Resin Bar	6 Level 5053	2.2	1.4	3.0	1.8

Table 11 – Friction Bolts test result summary.

Test	Element Test Number	Bolt Type	Location	Nominal Length of Element (m)	Bar Stiffness (t/mm)	"E" max (t/mm)	"E" min (t/mm)
MSSS30	30	Friction Bolt	6 Level 5053	2.4	6.2	15.6	6.7
MSSS31	31	Friction Bolt	6 Level 5053	2.4	19.1	13.2	6.8
MSSS32	32	Friction Bolt	6 Level 5053	2.4	16.1	13.2	7.5
MSSS34	34	Friction Bolt	6 Level 5053	2.4	17.1	13.8	8.5
MSSS 35	35	Friction Bolt	6 Level 5053	2.4	8.0	18.2	9.0