

New Hampshire
DOT
Research Record



Barron Mountain
Rock Reinforcement Evaluation – Phase II
I-93, Woodstock, New Hampshire
Validation of NDT Results for Condition Assessment of
Rock Reinforcements

Prepared by McMahon & Mann Consulting Engineers, P.C., Buffalo, New York for the New Hampshire Department of Transportation, in cooperation with the U.S. Department of Transportation, Federal Highway Administration. Completed as Pooled Fund Study No. TPF-5(096)

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<p>16. Abstract Thirty-year old rock reinforcements at the Barron Mountain rock cut along I-93 near Woodstock, NH are the subject of condition assessment and estimation of remaining service-life. The New Hampshire Department of Transportation (NHDOT) and McMahon & Mann Consulting Engineers, P.C. (MMCE) performed the condition assessment in two phases. Phase I of the condition assessment was completed in the fall of 2003 and included an evaluation of site conditions, a review of installation details, estimation of remaining service life and condition assessment using nondestructive testing. MMCE submitted an interim report to NHDOT in February 2004 describing results from Phase I and recommendations for Phase II. The second phase of the project (Phase II) consists of invasive testing of selected reinforcements to verify results from Phase I.</p> <p>Phase II was conducted in the fall of 2004 as a pooled fund study (TPF-5(096)) with participation from the New Hampshire (lead agency), New York and Connecticut Departments of Transportation and the Federal Highway Administration. The invasive test program included lift-off tests and physical, chemical and metallurgical testing on steel and grout samples retrieved from exhumed reinforcements. Replacement reinforcements were installed prior to invasive testing at selected locations.</p> <p>Examination of exhumed rock bolt samples tended to confirm results from NDT that recognized the occurrence of corrosion. Tendon elements protected by Portland cement grout were in very good condition compared to the resin grouted rock bolts and this observation is also consistent with results from NDT. Only minor corrosion was observed along the tendon sample. We estimate that the Portland cement grout will continue to protect the steel reinforcements for at least another twenty-years.</p>			
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EXECUTIVE SUMMARY

Thirty-year old rock reinforcements at the Barron Mountain rock cut along I-93 near Woodstock, NH are the subject of condition assessment and estimation of remaining service-life. The New Hampshire Department of Transportation (NHDOT) and McMahon & Mann Consulting Engineers, P.C. (MMCE) performed the condition assessment in two phases. Phase I of the condition assessment was completed in the fall of 2003 and included an evaluation of site conditions, a review of installation details, estimation of remaining service life and condition assessment using nondestructive testing. MMCE submitted an interim report to NHDOT in February 2004 describing results from Phase I and recommendations for Phase II. The second phase of the project (Phase II) consists of invasive testing of selected reinforcements to verify results from Phase I.

Phase II was conducted in the fall of 2004 as a pooled fund study (TPF-5(096)) with participation from the New Hampshire (lead agency), New York and Connecticut Departments of Transportation and the Federal Highway Administration. This report presents (1) a summary of results from Phase I followed by (2) a detailed description of the approach used to conduct Phase II, (3) results from Phase II, (4) comparison of observations and results from NDT conducted in Phase I and invasive testing in Phase II, (5) conclusions, and (6) recommendations.

Phase II

The Phase II invasive test program includes lift-off tests and physical, chemical and metallurgical testing on steel and grout samples retrieved from exhumed reinforcements. Replacement reinforcements were installed prior to invasive testing at selected locations. Two types of rock reinforcements are installed at Barron Mountain including: (1) partially bonded, resin grouted, prestressed rock bolts, and (2) fully bonded, Portland cement grouted, passive tendons. Approximately forty feet of rock reinforcements were exhumed from the site including samples from four rock bolts and

one tendon element. One sample each of resin grout and Portland cement grout were also obtained.

Results

Examination of exhumed samples tended to confirm results from NDT that recognized the occurrence of corrosion. Tendon elements protected by Portland cement grout were in very good condition compared to the resin grouted rock bolts and this observation is also consistent with results from NDT. Only minor corrosion was observed along the tendon sample. The tendon elements do not include sacrificial steel and rely on the integrity of the Portland cement grout for corrosion protection. Thus, remaining service life depends on the durability of the Portland cement grout. Results from sampling and testing the grout indicate it is porous and some chloride intrusion has occurred to date; although measured chloride concentrations are not currently high enough to initiate corrosion. We estimate that the Portland cement grout will continue to protect the steel reinforcements for at least another twenty-years.

The free length of the resin grouted rock bolts is unprotected, and poor coverage was observed along the bonded length. Pits and craters were observed at a number of locations along the rock bolt samples, and craters appear to coalesce into areas of uniform corrosion. The most severe corrosion was observed near the backside of the anchor plate. Three of the rock bolt samples exhibited a loss of cross section of approximately 20% at this location. This observed section loss is consistent with expectations based on site conditions, installation details, and existing mathematical models of service life. We estimate that rock bolts will not become overstressed from metal loss due to corrosion for another fifteen to twenty years.

Lift-off tests were performed on seven rock bolts. Reasonable agreement was recognized between results from lift-off testing and NDT. In general, the results indicate that approximately one third of the rock bolts have suffered a loss of prestress. There appears to be a general trend of greater loss of prestress with respect to elevation for rock bolts. This observation correlates with the more frequent occurrence of open joints, corresponding to a poorer quality rock mass, observed at higher elevations (near the bench area) at the site. Compared to loss of service from corrosion, results from the condition assessment revealed that loss of prestress, and possibly loss of anchorage, have a greater effect on durability of rock bolts at this site.

Recommendations

General and “site specific” recommendations are included at the end of the report. General recommendations refer to application of condition assessment at other sites. “Site specific” recommendations refer to the rock bolt and tendon elements studied at Barron Mountain, and describe means to extend the estimated remaining service life by continued monitoring and remediation, or retrofit, of selected reinforcements.

General Recommendations

Condition assessment data provides a check of the veracity and/or calibration of existing metal-loss models that may then be applied to extrapolate metal loss estimates and predict remaining service life. Monitoring should be ongoing throughout the service life at selected sites to document performance and validate assumptions made in estimating metal loss and other factors effecting service life. Priority should be given to sites where reinforcements have been in service for more than 25 years. Sites with resin- grouted reinforcements should be assigned a higher priority, but others should also be studied in an effort to generate a database that will be useful for documenting the performance of different systems.

The potential for increased corrosion activity for reinforcements installed at lower elevations along highways where deicing salts are utilized should also be explored.

Site Specific Recommendations

Tendons. The tendons should be monitored at five-year intervals to evaluate the integrity of the Portland cement grout. The monitoring need only include measurement of half-cell potential and represents less effort than is recommended for future monitoring of rock bolts. These data may provide a basis for extending the remaining service life beyond the estimated twenty years.

Strain gages were installed along two replacement tendon elements, which may serve as transducers to monitor rock mass performance and indicate loss of service from nearby reinforcements. Thus, the gages serve to monitor performance of the rock tendons. These strain gages should be monitored annually for at least five years to establish a baseline. Subsequent readings may be taken at longer intervals. Access to the strain gages is facilitated by the placement of a junction box bear the base of the rock cut, and data may be acquired within approximately half an hour after arriving at the site. A special readout box is required for monitoring.

Rock Bolts. The remaining prestress and load carrying capacity of the rock bolts should be further evaluated and replacement bolts installed as necessary. Because the data indicates that currently 30% of the rock bolts at the site have not maintained their prestress, we recommend that NHDOT take action within the next five years (i.e. before 2010). Placement of corrosion protection along the free length of remaining bolts, and behind the anchor plate in particular, may be useful for extending service-life. The veracity of the remaining service life estimate should be confirmed by corrosion monitoring at approximately 10-year intervals. This monitoring should include electrochemical tests, mechanical wave propagation tests, lift-off tests, and visual observations of conditions behind the anchor plate. Monitoring may be useful for extending the estimated service life beyond fifteen to twenty years.

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PROJECT BACKGROUND AND HISTORY

Thirty-year old rock reinforcements at the Barron Mountain rock cut along I-93 near Woodstock, NH are the subject of condition assessment and estimation of remaining service-life. Two types of rock reinforcements are installed at Barron Mountain including: (1) partially bonded, resin grouted, prestressed rock bolts, and (2) fully bonded, Portland cement grouted, passive tendons. The two-year project includes nondestructive testing (NDT) of selected elements (Phase I), and invasive testing (Phase II) to verify results from Phase I.

An interim report describes results from Phase I. This final report describes results from Phase II, and compares results from invasive testing and those from nondestructive testing (NDT).

Background

In 1972, during the construction of Interstate 93 in Woodstock, NH, a rockslide occurred at the base of Barron Mountain. The slide, consisting of approximately 17,000 cubic yards of rock, buried the I-93 northbound barrel. A redesign of the roadway was immediately undertaken to include stabilization of the rock slope by installing extensive rock reinforcement and instrumentation. Fifty to sixty feet long rock tendons were installed to counteract sliding along the anticipated sliding failure plane. Shorter, 10 to 30 feet long, rock bolts were installed to keep the rock mass intact; to preserve the full gravity effect of the rock bench used to maintain global stability, and to prevent minor rock falls onto the highway. The rock reinforcement system has performed well to date and no major rock slides or rock falls have occurred in this area subsequent to construction.

The estimated design life of unprotected rock reinforcement systems is approximately 50 years (Kendorski, 2003). The New Hampshire Department of Transportation (NHDOT) is concerned with the longevity of the system given half the anticipated design life has passed. The NHDOT engaged McMahon & Mann Consulting Engineers, P.C. (MMCE) to perform a condition assessment, estimate the remaining service-life of the rock reinforcement, and make recommendations for future monitoring to document the performance of rock reinforcement at the site throughout the intended service-life.

Phase I and II Condition Assessments

MMCE performed the condition assessment in two phases. Phase I of the condition assessment was completed in the late summer and fall of 2003 and included an evaluation of site conditions, a review of installation details, estimation of remaining service life and condition assessment using nondestructive testing. An interim report "*Phase I: Condition Assessment and Evaluation of Rock Reinforcement Along I-93, Barron Mountain Rock Cut, Woodstock, New Hampshire,*" prepared by MMCE and submitted to NHDOT in February 2004, describes details from the Phase I condition assessment. Salient details are reviewed in this report.

The second phase of the project (Phase II) consists of invasive testing of selected rock bolts and tendons to verify results from Phase I. Invasive testing includes lift-off tests; and physical, chemical and metallurgical testing on steel and grout samples retrieved from exhumed reinforcements. Corrosion of reinforcements is observed in terms of surface distress and metal loss. Data from Phase II are compared to results and interpretations from NDT. The comparison is in terms of qualitative and quantitative condition assessment relative to the reinforcement population at the site, as well as features and attributes observed for specific reinforcements.

Phase II was conducted in the fall of 2004 as a pooled fund study with the New Hampshire, New York State and Connecticut Departments of Transportation and the Federal Highway Administration as participants. Appendix I is a detailed description of the scope of services provided to NHDOT by MMCE in support of Phase II.

The following sections present a summary of results from Phase I followed by a detailed description of the approach used to conduct Phase II, details of results from Phase II, and a comparison of observations and results from NDT conducted in Phase I and invasive testing in Phase II.

SUMMARY OF RESULTS FROM PHASE I

Site Conditions

Details of the rock cut geometry and rock conditions are described in the interim project report (Fishman, 2004). The corrosiveness of the rockmass and the vulnerability of the reinforcements to metal loss were also evaluated. Generally, moisture content, chloride and sulfate ion concentration, resistivity and pH are identified as the factors that most affect corrosion potential of metals underground. Quantitative guidelines are available for assessing the potential aggression posed by an underground environment relative to corrosion (FHWA, 1993).

Samples of the weathered rock and groundwater were collected to evaluate the corrosiveness of the rockmass. Details are described in the interim report and additional sampling and testing performed during Phase II are described in Appendix II. Sample locations and a summary of the test results are shown on Figure 1. The measured pH (4.2 to 6.2), resistivity (min. $\approx 4200 \Omega\text{-cm}$), and moisture conditions within the weathered rock correspond to a corrosive environment. Measured sulfate and chloride ion concentrations (650 ppm and 720 ppm, respectively) are also at levels high enough to be conducive to a corrosive environment. In general, higher salt concentrations are observed near the base of the rock cut. The corrosiveness classification at the site is between II and III, on a scale where "I" is considered highly corrosive and "IV" is slightly corrosive (FHWA, 1993). This rating is used to estimate the rate of metal loss anticipated over the service life of the reinforcements.

Details of Rock Reinforcements

Figures 2 and 3 portray the rock bolt and tendon installations, respectively. Rock bolts and rock tendons include 1 inch and 1.25 inches diameter steel thread bars, respectively. Most of the reinforcements are Dywidag, Grade 150, high-strength prestressing steel thread bars. Some rock bolts are Grade 80 threaded steel rods supplied by Bethlehem Steel. Prestressed rock bolts are essentially end point anchorages, grouted at the distal end with polyester resin grout, and supported by an anchorage assembly consisting of a nut and a bearing plate at the rock face (proximal end). Rock bolts were initially prestressed to 20 or 40 kips depending on the steel grade. Tendon elements are fully grouted with Portland cement grout, and the proximal ends are recessed into the rock mass. The tendons are passive elements, i.e. they were not prestressed, and there is no anchorage assembly.

Due to the different installation details including grout type, method of grouting, anchor head details, drill hole diameter, and the lengths of the reinforcements, we considered rock bolt and tendon reinforcements separately for the purpose of condition assessment. Grout type is an especially important detail. Portland cement based grout is alkaline and protects the steel reinforcement by passivating the steel as well as providing a barrier to moisture and oxygen. However, passivation of the steel may be compromised by the presence of chlorides or acidic conditions. Polyester resin grouts are neutral and do not passivate the steel. They protect the steel by creating a barrier. However, the rock bolts include an unprotected free-length and the amount of cover associated with the resin grout within the bonded zone is uncertain. Also, prestressing tends to cause resin grout to crack. One of the goals of the condition assessment is to study the integrity of the grouts with respect to providing a barrier surrounding the reinforcements, and the degree to which Portland cement grout is passivating the steel.

Phase I NDT & Condition Assessment

NDT

Nondestructive test techniques are used to probe the reinforcements, and the results are analyzed for condition assessment. Four NDT's are employed including measurement of half-cell potential, polarization current, impact and ultrasonic testing. Details of NDT including test procedures are described in NCHRP Report 477 (2002).

Half-cell potential and polarization measurements are electrochemical tests and the impact, and ultrasonic techniques are mechanical tests involving observations of wave-propagation. In general, these NDT's are useful indicators of the following:

- Half-cell potential tests serve as an indicator of corrosion activity.
- Results from the polarization test are indicative of grout quality and degree of corrosion protection.

- Impact test results are useful to diagnose loss of prestress, assess grout quality and may indicate if the cross section is compromised from corrosion, or from a bend or kink in the bolt.
- Ultrasonic test results are useful for obtaining more detailed information about the condition of reinforcements within the first few feet from the proximal end of the reinforcement.

Results from NDT

Detailed description of the results from the NDT conducted during Phase I can be found in the interim report for the project (Fishman, 2004). Results from Phase I can be generally summarized as follows:

1. Site conditions are moderately corrosive, corresponding to an estimated remaining service life of approximately fifteen to twenty years due to metal loss from corrosion of the rock reinforcements,
2. Fully grouted rock tendons are apparently in better condition than resin grouted rock bolts,
3. Corrosion is occurring or has occurred along many of the rock bolts,
4. At least 30 percent the rock bolts have suffered loss of prestress,
5. The grouted length of the rock bolts is variable and grout quality is questionable along many of the rock bolts,
6. Some elements may have suffered loss of section of at least 20 percent due to metal loss, which is equivalent to a loss of approximately 0.1 inches in diameter,
7. More problems with loss of section and/or prestress were observed for rock bolts located within an identifiable, lower quality section of the rockmass (Lane et al., 2005) located in the vicinity Station of 1775+25, near the andesite dyke (see Figure 1).

PHASE II APPROACH

Phase II includes invasive testing of selected rock bolts and tendons to verify results from Phase I including NDT, condition assessment and service life estimates. Invasive testing includes lift-off tests; and physical, chemical and metallurgical testing on steel and grout samples retrieved from exhumed reinforcements. Replacement bolts are installed prior to invasive testing. Two of the replacement bolts are instrumented with strain gages to facilitate future monitoring. Some additional NDT was also performed in support of invasive testing, to collect data using alternative test techniques, and from areas of the site that were not accessible during Phase I.

MMCE subcontracted with JANOD, Inc., (JANOD), a specialty subcontractor in the area of rock fall hazard mitigation, to perform invasive testing and install replacement bolts. The following sections describe details of Phase II in chronological order including detailed descriptions of additional NDT, installation of replacement bolts, followed by invasive testing.

Additional NDT

Further NDT was performed by MMCE and by an independent consultant. MMCE checked the repeatability of results for elements selected for invasive testing, and conducted NDT on reinforcements at the north end of the site that could not be accessed during Phase I. MMCE used NDT techniques similar to those employed for Phase I. An independent consultant performed alternative NDT for comparison to results from MMCE and invasive testing.

GRANIT™ Test

AMEC Group Ltd. (AMEC) performed an alternative, impact type NDT on selected rock bolts using their patented GRANIT test. The GRANIT ground anchor integrity test is a sophisticated, rapid method for impact testing of rock bolts and ground anchorages. Analysis of GRANIT test data provides an estimate of rock bolt prestress, and unbonded length. Thus, results from GRANIT testing can be verified from prestress measured directly via lift-off testing, and serve as an alternative NDT from which results obtained with different equipment, operators and methods of data interpretation can be compared.

AMEC tested fifty-six rock bolts, identified as G-1 through G-56, using the GRANIT integrity test. Thirteen of the bolts tested by AMEC were also tested by MMCE during Phase I using different equipment for impact testing, and these bolts are identified in the interim report as bolts 1 through 10 and 15, 17 and 18. Throughout this report bolts tested in Phase I will be identified by "MMCE - #" and those tested by AMEC as "G-#". All the GRANIT tests are located in the center of the site near Station 1775+00 as shown in Figure 4. Details of these bolts in terms of total bolt length, estimated bond length, estimated free length and other tests performed on the same bolt including MMCE's NDT, and lift-off testing are summarized in Appendix III. Bolts selected for lift-off testing are also indicated as dark symbols on Figure 4.

A general description of the equipment and procedure for running the GRANIT Test and a report from the University of Aberdeen describing analysis of results are included in Appendix III. Apparently, there are factors present at the Barron Mountain site that present difficulties with respect to the analysis of data from the GRANIT test. These factors may be related to the stiffness of the anchor head assembly, or uncertainties with respect to construction details including free length and the use of couplings along the elements. Therefore, the analysis of the GRANIT test results could not render estimates of the remaining prestress. However, the GRANIT test results appear to be internally consistent in terms of repeatability and estimated free length, although the free lengths rendered by the analysis were not consistent with estimates based on construction records available from NHDOT. GRANIT test results could be used to distinguish bolts with high versus low levels of prestress similar to the results obtained by MMCE.

NDT by MMCE

MMCE performed additional NDT in Phase II to (1) check the consistency of measurements from Phase I, (2) include data from bolts selected for invasive testing during Phase II that were not included in Phase I, and (3) acquire data from testing bolts with difficult access at the north end of the site.

Details of the NDT performed by MMCE are included in Appendix IV. In general, results from NDT were consistent between tests performed in 2003 (Phase I) and 2004 (Phase II). One notable exception is a loss of prestress was observed for Bolt Sample #17 that was not observed in 2003. This indicates that MMCE #17 suffered a detectable loss of prestress over the course of the year.

Replacement Bolts

JANOD installed replacement bolts corresponding to locations where invasive testing and decommissioning of existing reinforcements by overcoring was anticipated. JANOD installed eight replacement reinforcements corresponding to rock bolts G-1, G-8 (MMCE #6), G-19 (MMCE #4), G-30 (MMCE #7), G-52 (MMCE #17), MMCE #11, and Tendons –MMCE #'s 1-4, and 2-4 (see Figures 1 and 4).

The installation of replacement bolts required advancement of the drill hole, placement and grouting of the reinforcing elements, and proof testing and prestressing in the case of active rock bolt elements. Data presented in Appendix V document installation of the replacement bolts.

Strain Gage Installation

MMCE attached strain gages along the replacement tendons 1-4/R and 2-4/R, installed near Stations 1774+50 and 1775+00 at approximate Elevation 755 ft. Instrumented tendons will serve as transducers to monitor rock mass performance, and strain accumulation may indicate loss of service from nearby reinforcements. Appendix V includes details related to the installation and initial monitoring of the strain gages.

Invasive Testing

The Phase II invasive Test Program includes some reinforcements with questionable condition, and some reinforcements considered to be in good condition, based on the results from NDT. Appendix VI includes a photo log of the fieldwork, reinforcement samples, and observations from sample reinforcements. Table 1 is a summary of the reinforcements included in the Phase II test program, and sample locations are also indicated in Figure 4. Seven rock bolts were selected for lift-off tests and three rock bolts and one tendon element were over cored and sampled. JANOD retrieved another rock bolt sample with a loose block of rock they removed from the face of the cut using their Boulder Buster™.

Table 1. Reinforcements for Phase II Invasive Testing

Phase I NDT (MMCE#)	Phase II #	Lift-off	Exhumed	Condition Assessment (NDT)	Comments from NDT Testing
NA	G1	Y	Y ¹	Questionable	Apparent loss of prestress; relatively poor gout quality; likely corroded
NA	G3	Y	N	NA	NA
3	G18	Y	N	Good	No apparent loss of prestress; relatively good quality grout; likely corroded
4	G19	Y	Y	Good	No apparent loss of prestress; relatively good quality grout; not likely corroded
6	G8	N	Y	Questionable	Apparent loss of prestress; relatively poor quality grout; possible loss of cross section or kink in bolt; very likely corroded
7	G30	Y	N	Questionable	No apparent loss of prestress; relatively poor quality grout; not likely corroded
8	G31	Y	N	Questionable	No apparent loss of prestress; relatively poor grout quality; likely corroded
9	G36	Y	N	Questionable	Apparent loss of prestress; relatively poor quality grout; very likely corroded
16	NA	N	Y ²	Questionable	No apparent loss of prestress; relatively poor grout quality; very likely corroded.
2-4 ³	NA	NA	Y	Questionable	Relatively good grout condition; likely corroded

¹ Exhumed by NHDOT

² Sheared-off as loose block was removed

³ 2-4 is a tendon element, all others are rock bolts

Lift-off Testing

Lift-off tests provide a direct measure of the prestress sustained by the anchorages. In this study, they are useful to check the veracity of NDT results, which are an indirect measure of prestress. JANOD conducted seven lift-off tests on rock bolts G1, G3, G-18 (MMCE #3), G-19 (MMCE #4), G-30 (MMCE#7), G-31 (MMCE #8), and G-36 (MMCE #9).

MMCE observed the lift-off tests to document that they were performed using equipment and procedures recommended by PTI (1996). Load is applied to the end of the rock bolt with a center hole hydraulic jack. Figure 5 shows the load assembly including a chair to seat the hydraulic jack and facilitate load transfer at the bolt head. The preload in the rock bolt is observed when the anchor nut lifts off the bearing plate at the anchorage assembly.

MMCE also observed loose or slack bearing plates at the anchorage for bolts G-40, G46, G-47, G-52 (MMCE #17), and G-54 (see Figure 4). Because a slack bearing plate indicates that the anchorage cannot sustain prestress, these observations contribute to five additional direct observations. Thus, Phase II includes twelve direct observations of prestress; seven lift off tests and five slack plates.

Over Coring

Thirty-eight feet of rock reinforcements were exhumed from the site including samples from four rock bolts and one tendon element as shown in Figure 6. Approximately, three feet long samples of resin grout and Portland cement grout were also obtained.

Three rock bolts and one tendon were exhumed by overcoring. A fourth rock bolt was sampled when a large block of rock was loosened and removed with a Boulder Buster™. The over-coring was accomplished with two types of drill rigs both using water to flush drill cuttings. JANOD overcored Bolt #'s G8 and G19 (MMCE #'s 6, and #4) and Tendon - MMCE #2-4 using an air powered rotary drill and a four-inch inside diameter core barrel. Janod's drill head was mounted to a mobile wagon that could be hoisted and secured to the rock face. NHDOT overcored Bolt # G-1 with a track-mounted, diesel powered rig and a 4.6 inch diameter core barrel.

Lane et al. (2005) describe details, difficulties, and limitations of over coring. A copy of this paper is included in Appendix VI. In most cases, recovery was accomplished by over-coring along a segment of the reinforcement at an angle slightly different from the drill hole, until the diamond drill bit encountered and cut through the steel. Difficulties included no grout within the free stressing zone of the rock bolts, deviation of the drill holes for the existing rock reinforcement, steel couplings, maintaining constant down pressure, anchoring the drill rig, alignment of the drill and core barrel with respect to the rock bolt, jamming of the core barrel, cutting the reinforcement and access for the drill rig.

PHASE II RESULTS

Rock reinforcement and grout samples were studied by visual observations, measurement of geometry and laboratory testing. Detailed observations and laboratory test results for grout and steel samples are presented in Appendices VI, VII and VIII. Interpretation and discussion of results are presented in the following sections.

Condition of Reinforcements

Figure 6 shows the lengths and overview of the reinforcement samples obtained from the site. Appendix VI describes detailed observations obtained for each sample including general observations, measurements of remaining diameter, observations of pitting corrosion and hardness measurements where grout samples were available.

Bolt #G-8 (MMCE # 6) was retrieved in its entirety for a sample length of approximately fourteen feet. The bolt appears to be in relatively good condition although some pitting corrosion is evident. Grout was observed at intermittent locations beginning four feet from the distal end of the bolt. The resin grout appeared to provide poor coverage to the bar, and for most of the area that had traces of grout, the thickness was not sufficient to cover the bar deformations. The best coverage was observed in an area about 4.25 inches in length, covering one side within the last foot of the bar. This poor coverage probably accounted for the bond breaking and the bolt spinning as the contractor removed the nut and bearing plate prior to overcoring. Bolt #G8 (MMCE #6) was installed at an upwards angle and slid out of the hole after over coring to a depth of approximately eight feet. Bolts #G30 (MMCE #7) and #G19 (MMCE #4) were also loosened as the nut was turned. Bolt # G30 (MMCE #7) could not be pulled from the hole with 70 kips, and the bolt was not over cored.

An approximately four feet long sample of MMCE #G19 (MMCE #4) was retrieved. The sample was terminated after a coupling was encountered. This sample exhibited more corrosion compared to # G8 (MMCE #6) and loss of cross section was observed at a location near the backside of the bearing plate. A three feet long section of Bolt # G-1 was also retrieved and similar loss of cross section was observed near the bearing plate.

Bolt – MMCE #16 was not overcored, but was recovered when JANOD fractured and removed the loose block of rock it supported with the Boulder Buster™. The Boulder Buster™ is a small charge that usually fractures the rock surrounding a rock bolt when detonated, causing the loosened rock to slide toward the base of the rock cut. In this case the rock bolt was severed and removed with the block. The fracture surface at the end of the approximately four feet long sample of bolt – MMCE #16 was sent to a metallurgy lab for analysis as described in the “Results” Section. Loss of section was also observed near the backside of the anchor plate.

Tendon – MMCE #2-4 was over cored to a depth of approximately twelve feet. The proximal end of the sample included an approximately two feet long annulus of grout adhered to the reinforcement and surrounding rock core. Cement grout was exposed on one side of the sample as the core barrel wandered slightly off track. The exposed grout measured about 5 by 20 inches in plan. Recorded observations include hardness, absorption and bulk specific gravity. The steel reinforcement appeared to be in excellent condition and the surface did not appear to have been subject to corrosion. A coupling was encountered at a depth of approximately five feet from the rock face.

Consistency and Physical Properties of Grout Samples

Grout condition is evaluated in terms of the observed coverage of the reinforcement (discussed in the preceding section), and consistency, and physical properties of the grout mix. Consistency is observed via hardness measurements distributed along the Portland cement and resin grout samples. The distribution of results from consistency measurements is considered to reflect the relative quality of the grout mixtures. Physical properties include bulk specific gravity and absorption, which relate to the effectiveness

of the grout to act as a moisture barrier and mitigate the intrusion of harmful elements such as chlorides. Bulk specific gravity and absorption were only obtained for the Portland cement grout sample. Portland cement grout samples were also evaluated for chloride content and chemical analysis was performed on one sample of resin grout. Detailed measurements for hardness are presented in Appendix VI. Appendix VII includes test results from measurements of specific gravity and absorption, and chlorides content of Portland cement grout and chemical analysis of resin grout.

Hardness Measurements

Hardness measurements were obtained using a Type D durometer (Shore D scale) in general accordance with the procedure described in ASTM D 2240. The Shore D scale ranges from 0 to 100, and is considered a useful indicator of material type and consistency. A template was used to scribe a 0.5 square inch area at each measurement location. Five measurements were obtained at each measurement location and averaged to yield one data point. One hundred and sixty measurements were obtained along the Portland cement grout sample exhumed with Tendon – MMCE #2-4. About 100 measurements were obtained from 20 locations, where the coverage was sufficient, along the resin grout sample exhumed with Rock Bolt #G8 (MMCE #6).

Figure 7 compares histograms for the Portland cement and resin grout hardness measurements. Hardness measurements for the Portland cement grout ranged between 84 and 96, with an average of 93 and standard deviation 2.1. Hardness measurements for the polyester resin grout ranged between 83 and 90, with an average of 85 and standard deviation 1.8. The comparison shown in Figure 7 indicates that hardness testing may be a useful technique to identify grout type. Grout hardness measurements are very consistent along both samples. Based on this data, differences between Portland cement or resin grout condition appear to be more in terms of the amount of coverage of the reinforcement elements, rather than with respect to the consistency of the different grout mixtures.

Bulk Specific Gravity and Absorption

The Portland cement grout was removed from the bar following completion of hardness testing. After the grout was removed, a very slight amount of corrosion was evident on sample Tendon - MMCE # 2-4, within about 2 feet of the rock face. The close-up in Figure 8 depicts numerous pore spaces distributed throughout the grout sample. To evaluate the porosity, MMCE selected five specimens for measurement of specific gravity and water absorption.

Bulk (G_b) and maximum (G_m) specific gravities were determined corresponding to the volume including and not including the pore spaces, respectively. The porosity, n , of the specimens can then be computed as:

$$n = V_v/V_T \times 100 = (1 - G_b/G_m) \times 100 \quad (1)$$

where n is porosity, V_V is the volume of voids, and V_T is the total volume of the specimen.

MMCE determined the bulk specific gravity and absorption using the mass of the specimen in air and also submerged in water in general accordance with the procedure described by AASHTO T166. Measured bulk specific gravity averaged 1.58 (99 pcf) with a range from 1.57 to 1.59 (98 pcf to 99 pcf). The grout specimen was pulverized and the maximum theoretical specific gravity of 2.41 (150 pcf) was determined in general accordance with the procedure described in ASTM D 854. Thus, the average porosity of the Portland cement grout sample is computed as 34% with Equation (1).

For comparison, the absorption corresponding to the ratio of absorbed water to the total volume was also measured after soaking each specimen for fifteen hours. Measured absorption ranged between 36.3% and 33.7 % corresponding to an average of 35.2%, which compares well with the measured porosity of 34%. This comparison demonstrates that the high porosity of the grout contributes to a high capacity for absorption of water and possible intrusion of chlorides. Chlorides may be present along the rock face as a residue from salt spray produced from deicing of the highway. The possibility of chloride intrusion is considered as discussed in the following section.

Chlorides Content

Portland cement grout samples were tested for chloride ion content as described by AASHTO Test Method T260 "Sampling and Testing for Total Chloride Ion Content and Concrete Raw Materials," and for pH using SW846 Method 9045. Details of the test results from Geotechnics, Inc. and Clark laboratories, LLC are presented in Appendix VII. Five specimens were located approximately three to four inches apart along the length of the grout sample from the proximal end of the sample to a depth of approximately seventeen inches, and prepared for testing.

These data are useful to study chloride diffusion and the corresponding potential for depassivation of steel surrounded by Portland cement grout. Observed chloride contents ranged from less than 0.002 percent to 0.016 percent by weight of grout. Detected levels of chloride concentration are elevated, but below levels of 0.1 percent often associated with initiation of corrosion in reinforced concrete (Johnsen et al., 2003). These levels are expected to be higher for tendons located at elevations lower than sample Tendon – MMCE # 2-4, which is approximately twenty-five feet above the roadway, because the likely source of chlorides at this site is from deicing salts. Higher chloride concentrations were observed near the proximal end of the tendon, and near the intersection of a rock joint evident in the rock core retrieved with the tendon sample, approximately 12 inches from the proximal end. Measured pH levels ranged between 11.7 and 12.3, and are considered normal for Portland cement grout.

Chemical Analysis

A specimen of pulverized (minus No. 50 sieve) resin grout was submitted to Clark Laboratories, LLC for chemical analysis with Fourier transform infrared (FTIR) spectroscopic analysis and non-quantitative analysis. Results from Clark Laboratories are presented in Appendix VII.

The FTIR spectra obtained for the submitted specimen matched the polyester resin grout signature available within the library of available spectra. These results confirm that the resin grout is a polyester resin grout consistent with the information obtained by NHDOT documenting the installation of the rock bolts in 1974.

Analysis of Steel Thread Bar Samples

Samples of rock reinforcement elements were evaluated for metal loss from corrosion, metallurgy, and tensile strength. MMCE measured pit depths and loss of cross section along the samples and sent selected specimens to Westmoreland Mechanical Testing and Research, Inc (WMT&R) for metallographic analysis and tension testing. Appendix VI includes detailed measurements of pit depth and loss of section (diameter). Appendix VIII presents results from metallographic analysis and tension testing from rock bolt sample MMCE #16.

Corrosion Measurements

Pits and craters were observed at a number of locations along the rock bolt samples, and craters appear to coalesce into areas of uniform corrosion extending for lengths of approximately four inches. One hundred and seventy-eight pit depth measurements were obtained from the surface of the rock bolt samples. Pit depths were measured with a pit depth gage having a sensitivity of 0.0001 inches. The average measured pit depth was approximately 0.015 inches with a standard deviation of 0.014. Figure 9 shows the cumulative distribution of pit depth measurements indicating that the maximum measured pit depth was 0.1 inches and 10 percent of the measured pit depths were greater than 0.031 inches. We observed that deeper pits are often associated with larger pit diameters, supporting the notion that pitted areas coalesce into areas of uniform corrosion.

Three of the rock bolt specimens exhibited a maximum loss of section corresponding to approximately 0.1 inches in diameter. This loss is consistent with existing mathematical models of service-life and with the observation from NDT that 70% of the rock bolts have experienced significant corrosion. Considering the initial diameter, level of prestress, and rate of metal-loss, we estimate that rock bolts will not become overstressed from loss of section due to corrosion for another fifteen to twenty years.

Metallurgical Analysis

The fracture surface from MMCE #16, which was dislodged as JANOD removed a block of rock with their Boulder Buster™, was submitted to WMT&R for metallographic analysis. The fracture face was ultrasonically cleaned and viewed in the scanning electron microscope using energy dispersive spectroscopy, and a longitudinal section of the fracture surface was also prepared.

The fracture initiation site is identified near the perimeter of the cross section. Inclusions, one high in chlorine, were found near the fracture initiation site; and, in another area of the fracture surface, intergranular separation cracks were found. Inclusions with higher concentrations of silicon, aluminum, with lesser amounts of sodium, potassium and calcium were also observed in areas away from the fracture initiation site.

A corrosive environment, e.g. one high in chlorine may have been the cause of the intergranular separation cracks. We postulate that fracture initiated at the inclusions, and propagated by fast fracture through the structure, which was already weakened by intergranular corrosion.

Tension Test

MMCE sent six specimens to WMT&R for tension testing (ASTM E 8). The six specimens included areas with loss of cross section, pitting and no distress as follows:

- Two (2) specimens exhibiting no distress, i.e. no pitting or loss of section, were obtained from Rock Bolt Sample #G8 (MMCE #6).
- Two (2) specimens that exhibited significant pitting were obtained from Rock Bolt Sample #G8 (MMCE #6).
- Two (2) specimens where significant loss of cross section (diameter) was observed were obtained from Rock Bolt Samples #G19 (MMCE #4) and MMCE #16.

WMT&R tested one machined-section and one full section from each pair of specimens. The purpose of testing the full sections was to determine if the ultimate tensile stress (UTS) was affected by a change in geometry due to non-uniform loss of section, or localized corrosion such as pitting. The machined sections have a controlled geometry and are similar to the test specimens described in ASTM A 370, "Standard Test Methods and Definitions for Mechanical Testing of Steel Products." Machined sections had an initial diameter of 0.5 inches and a gage length of 2 inches. Full sections were approximately 18 inches long with a gage length of 2 inches. Typical specimens are pictured in Figures 10 (a) and (b). Results from tension testing are summarized in Table 2.

Table 2. Summary of Tension Test Data

Sample	Location ¹	Condition	Section	UTS (ksi)	σ_y (ksi)	ϵ_{break} %
6B	24" to 28"	no distress	machined	158	151	7.5
6A ²	7.5" to 24"	no distress	full	151	146	0.5
6C	68.5" to 74.5"	pitted	machined	159	156	7.5
6D ²	92" to 111.25"	pitted	full	150	149	1.5
16A	13" to 18"	section loss	machined	158	108	14
4A ³	6.5" to 22.5"	section loss	full	154	153	1.0

¹ location w.r.t end of sample

² stress computed using nominal area = 0.85 in²

³ stress computed for specimen 4A considers remaining area = 0.735 in²

UTS- ultimate tensile stress

σ_y - yield stress, 0.2% offset

Full specimens had a lower strain at break compared to the machined specimens, and the break occurred near the grips used to secure the ends of the specimen. This may have affected the measurement of strain for the full sections because the break did not occur between the gage points. The machined specimens broke near the middle of the specimens. Otherwise, similar results were obtained from the full and machined sections. With one exception, the strength data are consistent with the specifications for Dywidag Grade 150 thread bar (ASTM A722 steel). Specimen 16A, had a larger strain at break compared to the other specimens and a yield stress below expectations. The ultimate tensile stress for Specimen 16A was consistent with the other samples. A possible explanation for the discrepancy is that specimen 16A was weakened from intergranular corrosion as evidenced from the SEM examination of the fracture surface for this bolt and described in the preceding section. Figures 11 (a) and (b) depicts the stress strain curve for specimen 16A compared to a typical stress-strain curve obtained for the thread bar.

Comparison with Results from Phase I NDT

Corrosion

Electrochemical measurements (half-cell potential and polarization resistance) are only able to assess the portion of the element in electrical contact with the surrounding electrolyte (rock mass). However, the ability of NDT to correctly identify the presence of corrosion along the grouted length was verified by invasive testing.

Bolt #G8 (MMCE #6) and Tendon - MMCE # 2-4 were the only samples that included grout that could be compared with electrochemical measurements included in the Phase I NDT. The half-cell potential and polarization measurements for Bolt #G8 (MMCE #6) indicate that the element is likely corroded and the grout condition is questionable. The distal end of Bolt #G8 (MMCE #6) did not appear to be completely surrounded with grout and corrosion was evident, which is consistent with the results from NDT. Half-cell potential and polarization measurements for Tendon – MMCE #2-4 also indicate

that grout condition is questionable and that corrosion is likely. The high porosity observed for the exhumed grout sample may confirm the interpretation of grout quality from the results of NDT. In general, the exhumed tendon element (Tendon – MMCE #2-4), which was protected by Portland cement grout, was in very good condition compared to the resin grouted rock bolt (Rock bolt G8- MMCE #6) and this is also consistent with observations from NDT.

Prestress

Table 3 is a summary of lift off test results and comparison with the interpretation from NDT. Damping, or the rate of decay, of acceleration amplitude response observed from an impact test has been shown to increase with respect to level of prestress for rock bolts (Rodger et al., 1997). Loss of prestress is diagnosed from NDT by comparing the rate of decay observed for the sample population and identifying rock bolts associated with relatively low rates of decay as having an apparent loss of prestress. Thus, NDT results are described qualitatively in terms of “Good” or no apparent loss of prestress, or no good, “NG” corresponding to an apparent loss of prestress.

Table 3. Lift-Off Test Results

Bolt #	Lift-Off (Kips)	NDT Result	Correct NDT
G-18 (MMCE #3)	36	Good	Y
G-19 (MMCE #4)	38	Good	Y
G-30 (MMCE #7)	17	Good (?)	N(?)
G-31 (MMCE #8)	22	Good	N
G-36 (MMCE #9)	20	NG	Y
G-1	7	NG	Y
G-8 (MMCE #6)	Loose	NG	Y
G-52 (MMCE #17)	Loose	G/NG	Y(?)

Reasonable agreement was recognized between results of lift-off tests and NDT. In general, the results indicate that a high percentage of the rock bolts have suffered loss of prestress. The comparison between NDT and lift-off test results is favorable for between 63% (5 of 8) and 88% (7 of 8) of the measurements. Some ambiguity exists with respect to interpretation of NDT results when an intermediate level of prestress remains, and this is apparent in the interpretation of results for Bolt G-30 (MMCE #7). Large losses of prestress, or, at the other extreme, rock bolts with the majority of prestress remaining were correctly identified from the results of NDT. Results from GRANIT testing (see Appendix III) also identified bolts with low versus high levels of prestress. However, diagnosis of the magnitude of prestress was not possible at this site due to ambiguity with respect to the “known” free lengths of the rock bolts and/or the relatively high stiffness of the anchor head.

As indicated on Figure 4 a general trend of increasing loss of prestress with respect to elevation appears to prevail for rock bolts installed in the vicinity of Station 1775+00. This observation correlates with the more frequent occurrence of open joints, corresponding to a poorer quality rock mass, as the elevation of the rock bench (\approx El. 820 ft.) is approached at this station.

Specific Reinforcement Conditions

Generally, condition assessment of rock reinforcements does not benefit from analysis of data to identify a specific feature along an element. Rather, the data are compared to one another to identify groups of responses that may be separated into either “good” or “questionable” condition. The interpretation is performed in terms of the character of the observed waveform including the initial rate of decay and the attenuation of the wave reflections. However, for the purpose of describing the measured response, interpretation of data from NDT is compared to physical observation of features observed along the lengths of exhumed reinforcement samples.

Table 4 describes the locations of reflectors observed from the results of impact tests conducted during Phase I. The location of the reflector, L_r , is computed using compression wave velocity, V_p , and observed reflection time, t_r , as:

$$L_r = (V_p \times t_r) / 2 \quad (2)$$

The compression wave velocity of steel is taken as 16,000 ft/sec and 12,000 ft/sec for Portland cement grout. Reflections were observed from relatively proximal locations denoted as L_1 , and from a more distal location, L_2 , often corresponding to the length of the bolts (L_T). Direct observations are described in the comments column including loss of section from corrosion, the presence of couplings and rock conditions observed during drilling of replacement bolts as noted on the drillers logs (Appendix V). In most cases L_1 and L_2 are either correlated with direct physical observations, or with the known lengths of the bolts; within approximately three feet, i.e., corresponding to the wavelength inherent to the impact test. Figures 12 (a), (b) (c) and (d) depict reflections observed along Rock Bolts G-19, G-8 (MMCE #4 and #6) and MMCE #16, and Tendon - MMCE # 2-4.

The presence of couplings makes interpretation of reflections from impact testing difficult. Couplings appear to cause reflections in impact test data from Bolt G-19 (MMCE #4) and Tendon – MMCE # 2-4. Although this is useful from the standpoint of verifying the meaning of reflections observed in the test data, this could be misinterpreted as a loss of cross section or other distress in the absence of prior knowledge of the coupling locations.

If rock joints or seams intercept the grout body this may also cause a reflection as evidenced in the data from Bolts G-19, G-8 (MMCE #'s 4, and 6) and MMCE #16. These reflections are likely caused by a change in the geometry of the cross section of the grout body in the vicinity of the joint.

Table 4. Comparison of Results from NDT and Direct Observations

Test #	L ₁ (ft)	L ₂ (ft)	L _T (ft)	Comment
G-1	~	14	15	Loss of cross section near anchor plate; observed soft rock between 9 and 11 feet during drilling for replacement bolt
G-19 (MMCE #4)	5	15	30	Loss of cross section near anchor plate; coupling approximately 4 ft. from end; observed rock joint at depth of approximately 14 feet during drilling for replacement bolt
G-8 (MMCE #6)	7	15	15	Poor grout quality; grout not observed until depth of nine feet
MMCE #16	8	17	20	Loss of section near anchor plate; preexisting fracture approximately 4 ft. from proximal end; rock joint observed at depth of 10 feet during drilling for nearby MMCE #17 replacement
MMCE #2-4	7	~	60	Good grout condition; coupling observed approximately five feet from end

Evaluation of test data from MMCE #16, as shown in Figure 12 (c), indicates that a preexisting fracture surface at four feet from the proximal end of the bolts may be evident in the impact test data. However, these reflections are very subtle and could easily be overlooked without knowledge of the existence of this fracture surface.

Given the details of the anchor head assembly, reflections from a loss of section directly behind the anchor plate are difficult to identify from results of ultrasonic testing. This is because the data are masked by a strong reflection from the anchor head location. Results from the ultrasonic test on Bolt G-19 (MMCE #4) are shown in Figure 13 and a relatively subtle reflection approximately two feet from the end of the element is evident corresponding to the area where significant loss of cross section ($\approx 20\%$) is observed.

CONCLUSIONS

Reinforcement Condition

Tendons appear to be in better condition compared to rock bolts. Tendon elements do not appear to have corroded, but some corrosion is evident for rock bolts, and many rock bolts have suffered a loss of prestress. Tendons are fully grouted, passive elements and their useful life depends on the durability and integrity of the surrounding grout. The rock bolts are prestressed and are essentially end point anchorages. The useful life of the rock bolts depends on the durability of steel, grout and conditions at the anchorage. Thus, with respect to impacts on service-life, the rock bolts at this site are more vulnerable than the tendon reinforcements.

The conclusions describing the condition of resin grouted rock bolts are applicable to rock bolts that are only grouted along the bonded zone using a single grout type similar to those observed at Barron Mountain. Modern resin grouted rock bolt systems use two

grout types including a quick set grout in the bonded zone and a slow set grout along the free length. A larger volume of grout is inserted into the drill hole with the two-grout system, which likely results in better coverage along the length of the bolt. Rock bolts grouted with the two-grout system may perform better than those observed at Barron Mountain.

With respect to the rock bolts, corrosion is present, but the rate of metal loss appears to be close to expectations, and was apparently considered in the original design and corresponding selection of reinforcements and levels of prestress. The most severe corrosion of rock bolts was observed within two feet of the anchor plate and this observation is consistent with those reported by FIP (1986). The possibility of stress-corrosion cracking exists, which may be correlated with the presence of chlorides and overstressing from bending as blocks of rock become loosened and transfer shear to the bolt head.

Tendon elements appear to be passivated by the alkaline conditions provided by the Portland cement grout. In spite of the apparently high porosity, the Portland cement grout appears to have protected the steel from significant corrosion to date. The alkaline environment of the grout is apparently sufficient to protect the steel, but some corrosion may be possible due an ample supply of oxygen near the rock face, and the possibility of moisture and chloride intrusion. However, given the high porosity of the grout observed from the samples, chloride intrusion is a concern.

Loss of prestress was observed for five out of seven elements examined with lift-off testing. Three of the elements have lost significant amounts of prestress and two others have lost an intermediate amount of tension, but still sustained at least 20 kips. This is consistent with results from NDT that identified at least thirty percent of the elements have lost significant prestress. There appears to be a general trend of greater loss of prestress with respect to elevation for rock bolts located near Station 1775+00. In one instance, insufficient bond was observed during over coring, however, we do not know the extent to which this contributes to loss of prestress throughout the rock bolt population.

Utility of NDT

Results from NDT serve as useful indicators of overall reinforcement condition at the site. However, specific features along the lengths of the reinforcements are difficult to identify. Detailed knowledge of installation details including the location of couplings and joints, seams and fissures within the rock mass can be helpful for interpretation of results, but in general this information is not readily available. The interpretation of NDT data should be in terms of the character of the waveform obtained from impact testing, which can provide useful indications of stress conditions and grout quality inherent to the reinforcements. Electrochemical tests can also provide useful data relative to the occurrence of corrosion and integrity of corrosion protection. At this time, we strongly recommend that conclusions and assessments made on the basis of results from NDT be verified by more invasive testing.

Invasive Testing

Retrieval of rock reinforcement samples by overcoring is feasible, but the operation needs to include flexibility with respect to aligning and maintaining alignment of the core barrel, core barrel size, down pressure used to advance the core barrel, applied torque and speed of rotation. Adjustments may be needed due to unexpected rock conditions or installation details. Detailed information regarding the existing rock reinforcement is critical to a successful outcome. Information should include the type of reinforcement, type and extent (full or partial) of grout, diameter of the drill hole and of the steel reinforcement, depth and orientation of the reinforcement, existence and location of couplings, location of seams and joints, and a detailed sketch/map of the slope face.

Lift-off tests are good indicators for rock bolt integrity and are simple to perform. Lift-off tests are recommended for assessing the loss of prestress for particular bolts, whereby the results from NDT are useful to indicate the extent and general location problem areas.

Benefits of Condition Assessment

Compared to loss of service from corrosion, results from the condition assessment revealed that loss of prestress is the bigger concern relative to remaining service-life at the Barron Mountain site. The condition assessment also revealed locations of increased corrosion activity. A sound technical basis is established for planning future maintenance and rehabilitation activities at the site. Thus, results from condition assessment will ultimately result in a cost savings to the NHDOT.

RECOMMENDATIONS

General and “site specific” recommendations are described in the following sections. General recommendations refer to application of condition assessment at other sites. “Site specific” recommendations refer to the rock bolt and tendon elements studied at Barron Mountain, and describe means to extend the estimated remaining service life by continued monitoring and remediation, or retrofit, of selected reinforcements.

General Recommendations

1. Nondestructive testing combined with invasive observations should be applied to condition assessment of rock reinforcements as described in the recommended practice developed as part of NCHRP 24-13 (Withiam et al., 2002).
2. Conclusions and assessments made on the basis of results from NDT must be verified by more invasive testing. Lift-off tests and visual observation of exhumed reinforcements are recommended candidates for invasive testing.

3. Condition assessment should be performed at sites where resin grouted elements are installed, or where there is uncertainty with respect to remaining service life. Uncertainty may be due to known corrosive conditions or questionable installation details and/or workmanship. Older resin grouted installations using only a single grout type in the bonded zone are more vulnerable than modern installations that use a two-grout system. Therefore, these installations should be assigned a higher priority, but others should also be studied in an effort to generate a database that will be useful for documenting the performance of different systems.
4. Condition assessment should be performed at all sites where reinforcements have been in service for more than 25 years. Corrosion problems are not anticipated where reinforcements include single or double corrosion protection as described by PTI (1996). However, it is useful to document performance and establish a baseline from which estimates may be made for remaining service life and for future reference.
5. A special study should evaluate the performance of reinforcements at lower elevations in proximity to highways that receive deicing salts. It may be possible to demarcate areas that are likely to be contaminated by intrusion of deicing salt, and focus condition assessment mainly in these areas. Due to the relative ease of access at lower elevations, this would result in a significant cost savings for condition assessment.
6. Condition assessment data should be used to check the veracity and/or calibration of existing metal-loss models that may then be applied to extrapolate metal loss estimates and predict remaining service life. Monitoring should be ongoing throughout the service life at selected sites to document performance and validate assumptions made in estimating metal loss and other factors effecting service life.

Site Specific Recommendations

Passive Elements - Tendons

Results from NDT and invasive testing indicate that, generally, tendon elements are in good condition, and have a useful remaining service life of approximately 20 years. Apparently, the tendon elements do not include sacrificial steel and rely on the integrity of the Portland cement grout for corrosion protection. Thus, the twenty-year estimated service life is with respect to the durability of the Portland cement grout. The service life estimate assumes that the elements remain passivated by the Portland cement grout, and that the passive film is not compromised by chloride intrusion into the grout. Results from sampling and testing the grout indicate it is porous and some chloride intrusion has occurred to date; although measured chloride concentrations are not currently high enough to initiate corrosion.

We recommend that the performance of the tendon elements continue to be monitored throughout their remaining service life as follows:

1. Considering the possibility of chloride intrusion, we recommend that these elements be monitored at approximately 5-year intervals to check the veracity of the remaining service life estimate. Monitoring should include measurement of half-cell potential to document that these are still in the range associated with passivated steel elements. If the measured half-cell potentials are below negative 350 mV, then further evaluation and remediation, or replacement, of these elements may become necessary.
2. Strain gages installed along replacement Tendons MMCE #'s 1-4 and 2-4 should be monitored annually to detect load changes in the reinforcements, which may indicate that the integrity of the elements has been compromised, or loads prevail that were not considered in the design of the reinforcements. Access to the strain gages is facilitated by the placement of a junction box bear the base of the rock cut, and data may be acquired within approximately half an hour after arriving at the site. A special readout box is required for monitoring.

Additionally, results from Phase I testing indicate that the half-cell potential of Tendon – MMCE #2-2 is well below –350 mV and this should be considered with respect to the redundancy of the rock reinforcement system, and the need to further evaluate or replace this particular element. A whitish colored material is observed along the rock face near this element, which probably leached from the Portland cement grout.

Active Elements – Rock Bolts

Rock bolt elements appear to have suffered significant loss of prestress, and corresponding lack of bond strength was evident at some locations as the anchor head assemblies were loosened. The lack of bond strength may be due to installation conditions and/or degradation of the grout. Rock bolts have an estimated remaining service life of approximately 20 years due to corrosion. Presently the service-life is limited due to metal loss by the existence of an unprotected free length.

We recommend that rock bolts be evaluated and identified for remediation (restressing), or replacement, as follows:

1. Rock bolts should be load tested with proof and performance tests as described by PTI (1996) to demonstrate that adequate bond strength exists. Bolts passing the load test should be re-stressed to the design load, and those not passing the load test should be replaced. Rock bolts are apparently losing prestress over time and some differences were noted within the course of a year. We recommend that remedial activities commence within five years (i.e. before 2010). In the interim, the rock face should be closely monitored for movement and additional rock bolts installed in response to excessive movements of rock blocks or opening of joints observed along the rock face.

2. The veracity of the remaining service life estimate for the restressed bolts, as described in Item #1, should be confirmed by corrosion monitoring at approximately 10-year intervals. Future corrosion monitoring should include electrochemical tests (half-cell potential and polarization current) to check for the presence of corrosion, and wave propagation techniques (impact and ultrasonic tests) to check for loss of cross section.
3. Placement of corrosion protection along the free length of existing rock bolts should be considered and may allow a greater remaining service life for the rock bolts subsequent to remediation as described in Item #1. Provisions need to be included within the corrosion protection system so as not to restrict movement along the free length, because rock bolts may need to be restressed in the future. These provisions need to consider the possibility that couplings exist along the free length. Benefits include protection of the rock bolt surface area behind the anchor plate where the most severe corrosion is observed. It may also be possible to monitor corrosion activity separately along the grouted free length, and along the bond length if they are electrically isolated from one another. These data will be useful to justify extending the expected service life of the system as performance data is collected and analyzed. Thus, benefits from remediating and retrofitting the reinforcement system may be realized in terms of increased service life.

REFERENCES

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FIGURES

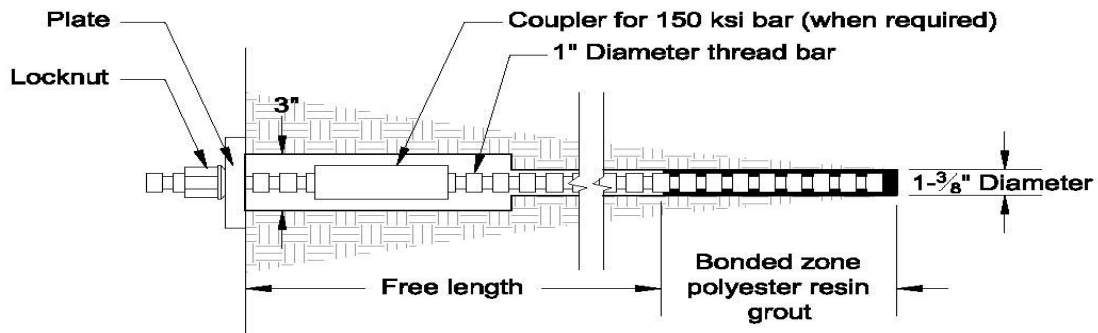


Figure 2. Rock Bolt Details

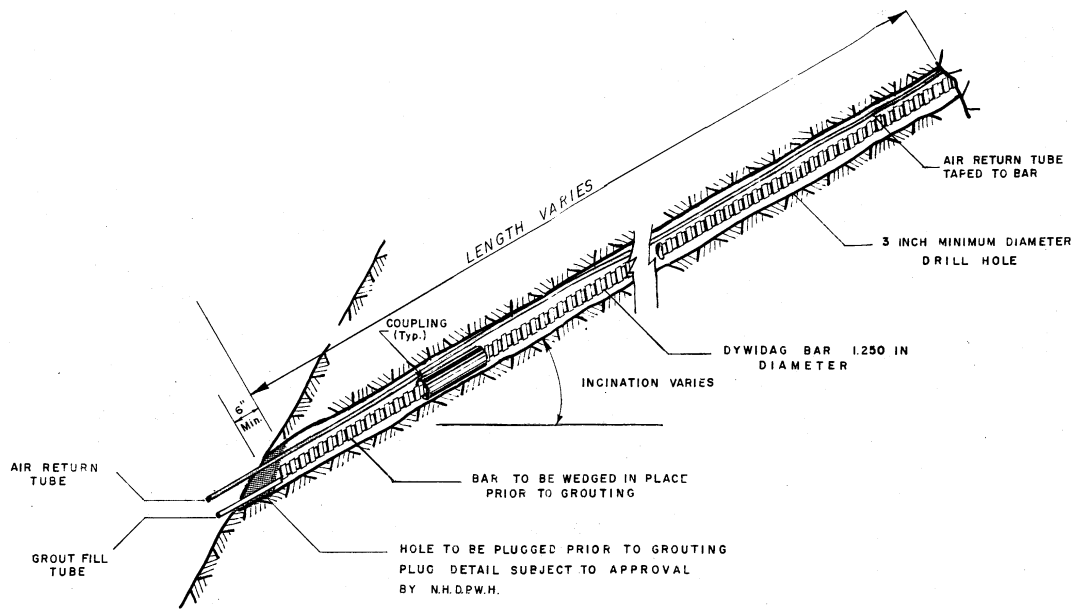


Figure 3. Details of Rock Tendons (Haley and Aldrich, 1973)

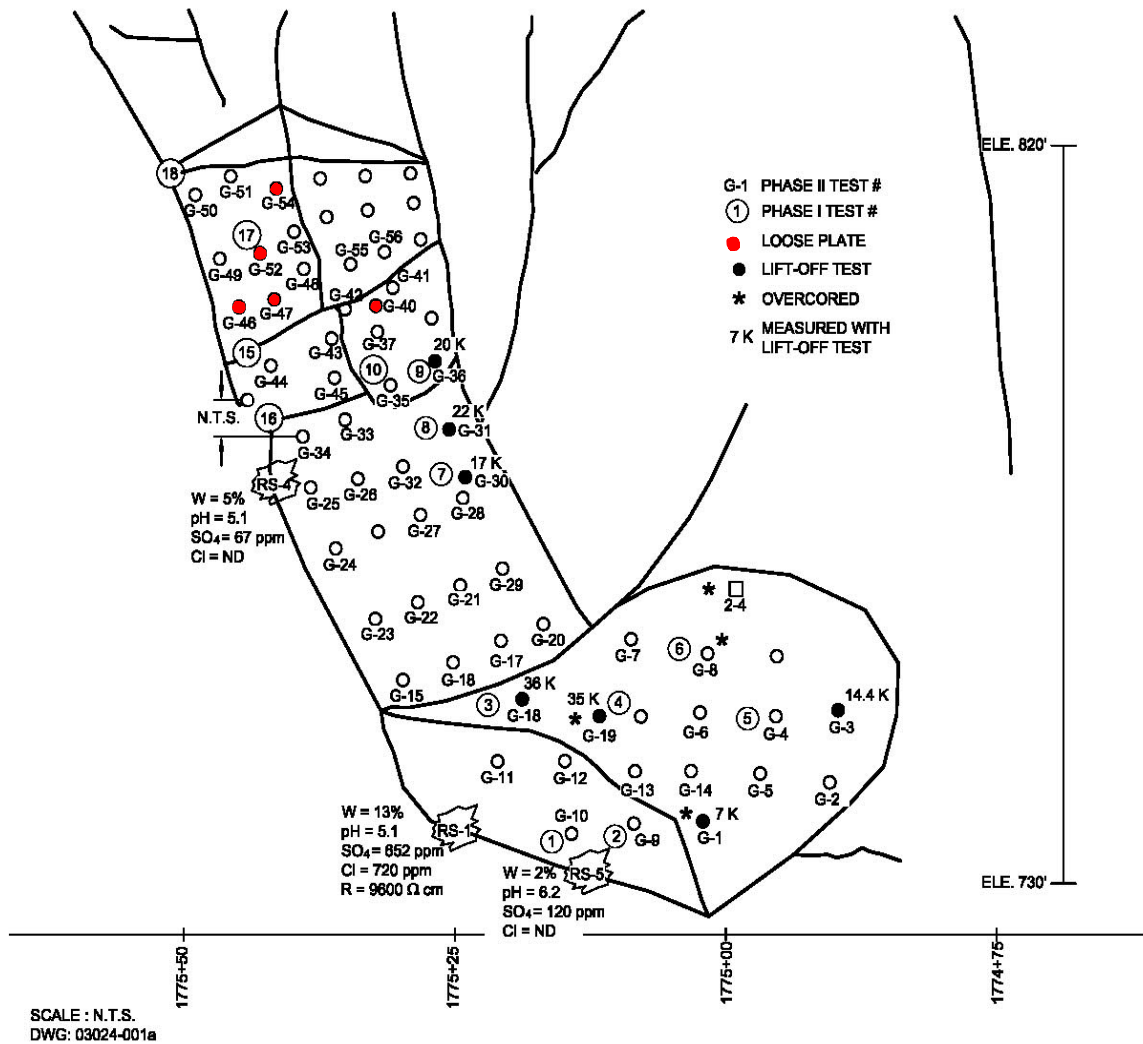


Figure 4. Rock Face Elevation View With Stationing, Reinforcement Locations, and Test Numbers for Phase I and II.



Figure 5. Lift-off Test for Prestressed Rock Bolts

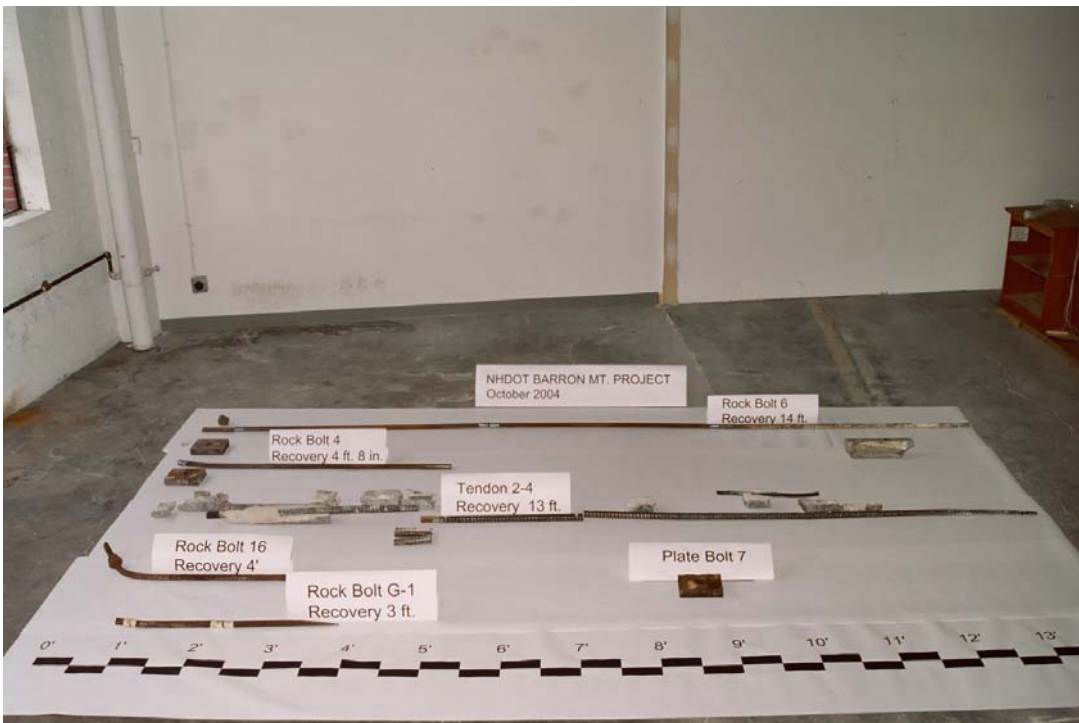


Figure 6. Recovered Rock Reinforcement

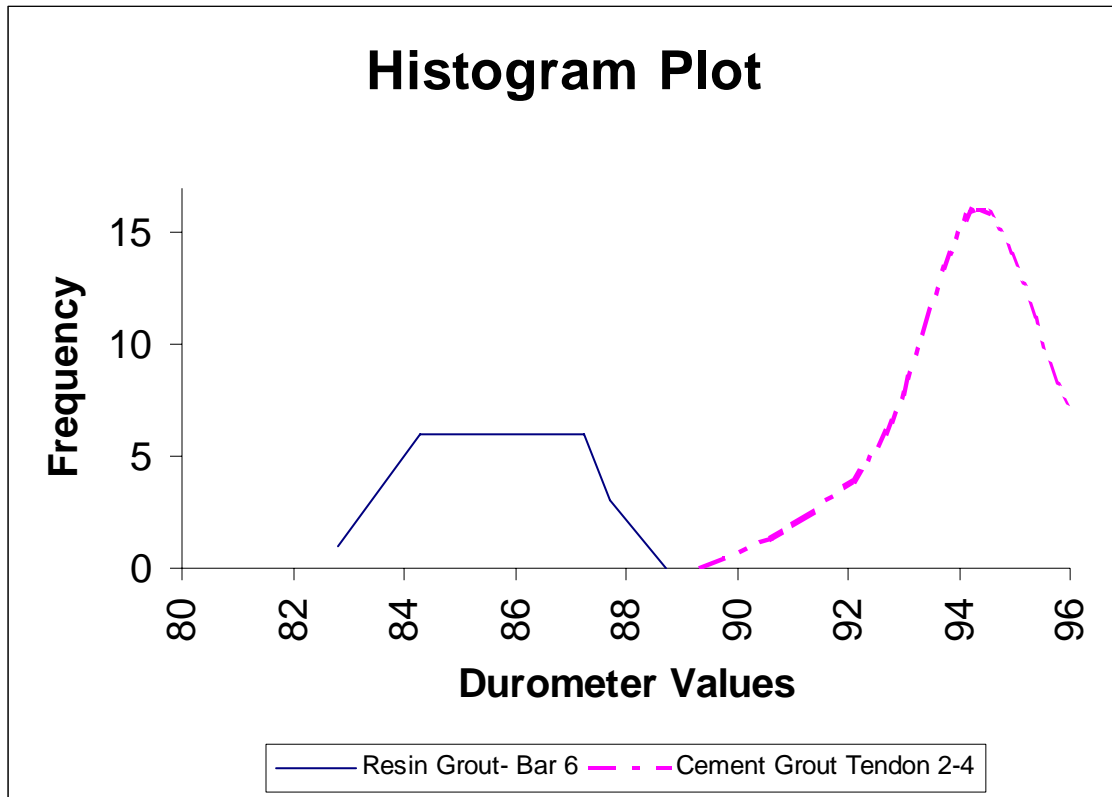


Figure 7. Histogram of Resin Grout and Portland Cement Grout Hardness Measurements.



Figure 8. Pores Distributed Throughout Portland Cement Grout Sample.

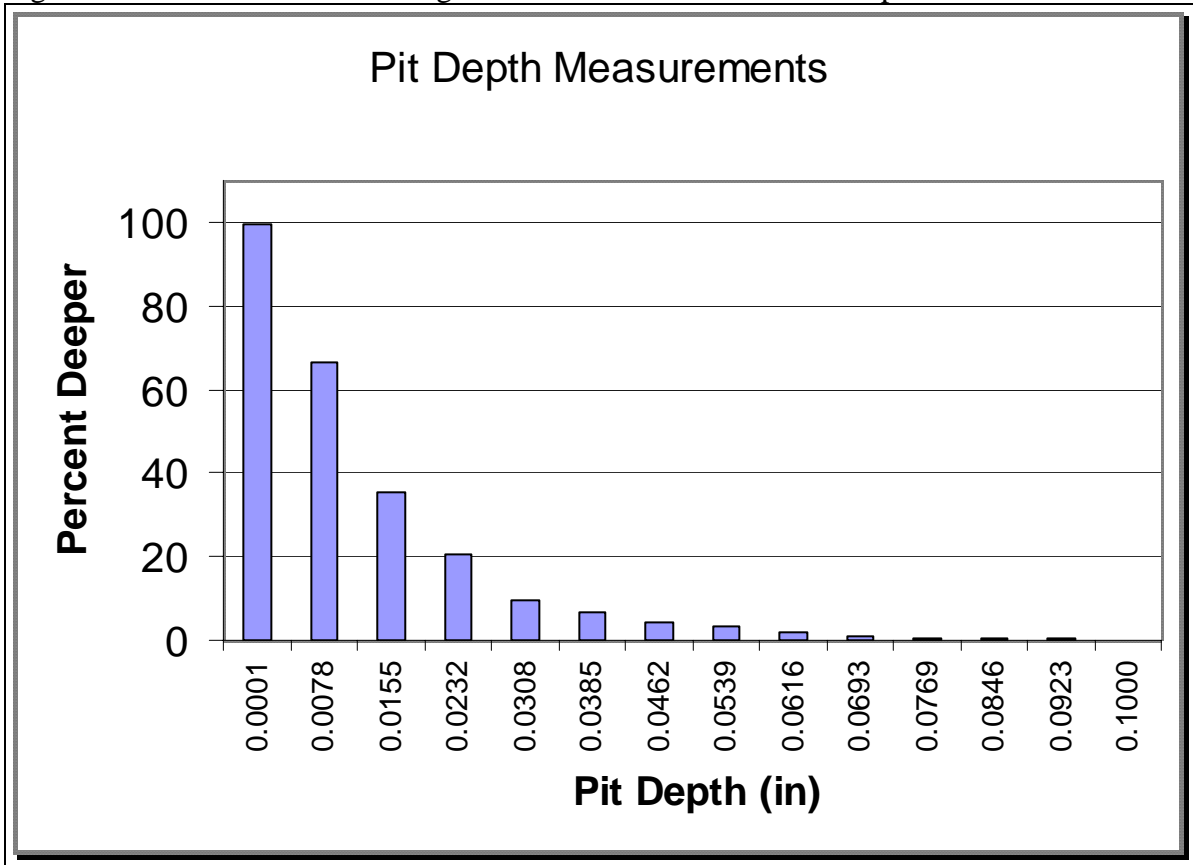


Figure 9. Cumulative Distribution of Pit Depth Measurements

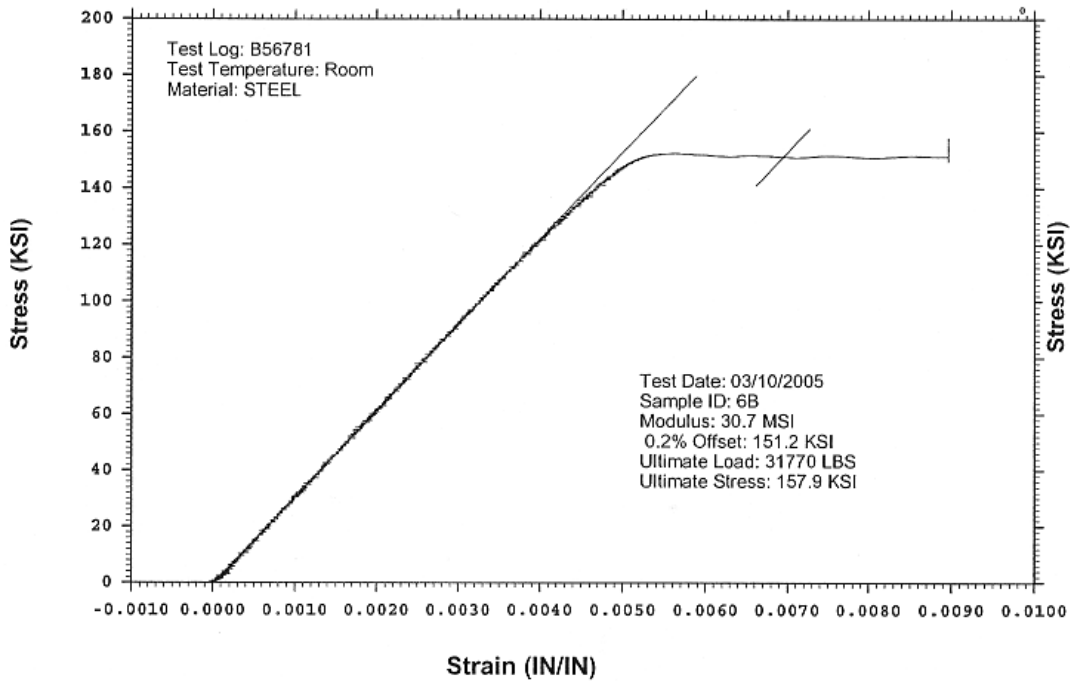


a) Full Section

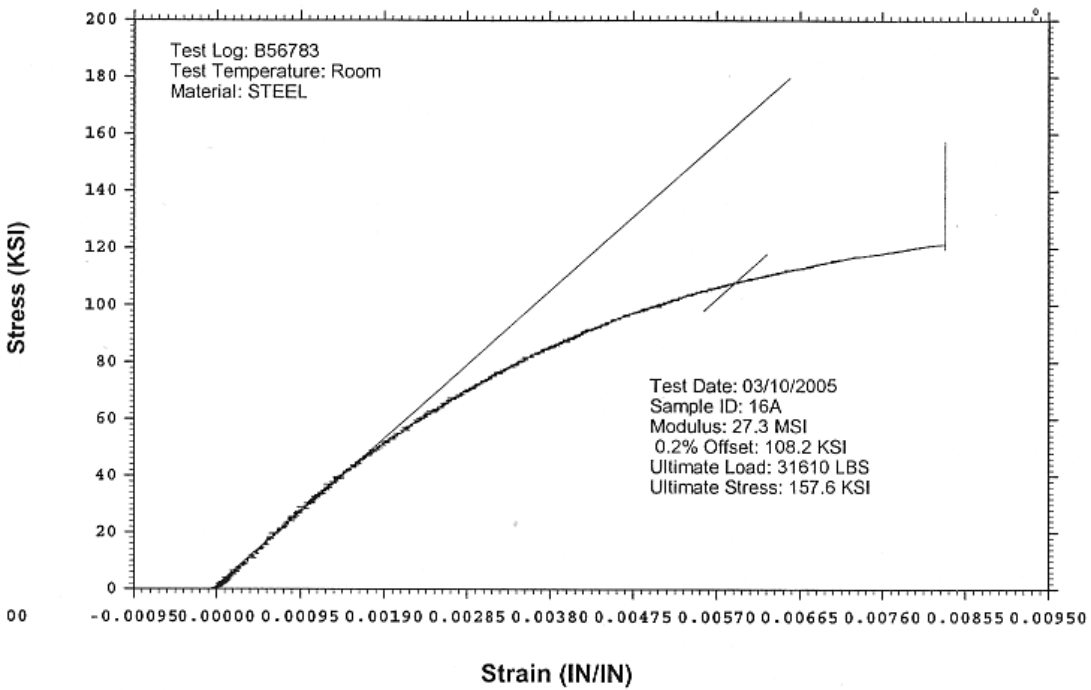


b) Machined Section

Figure 10. Tension Test Specimens

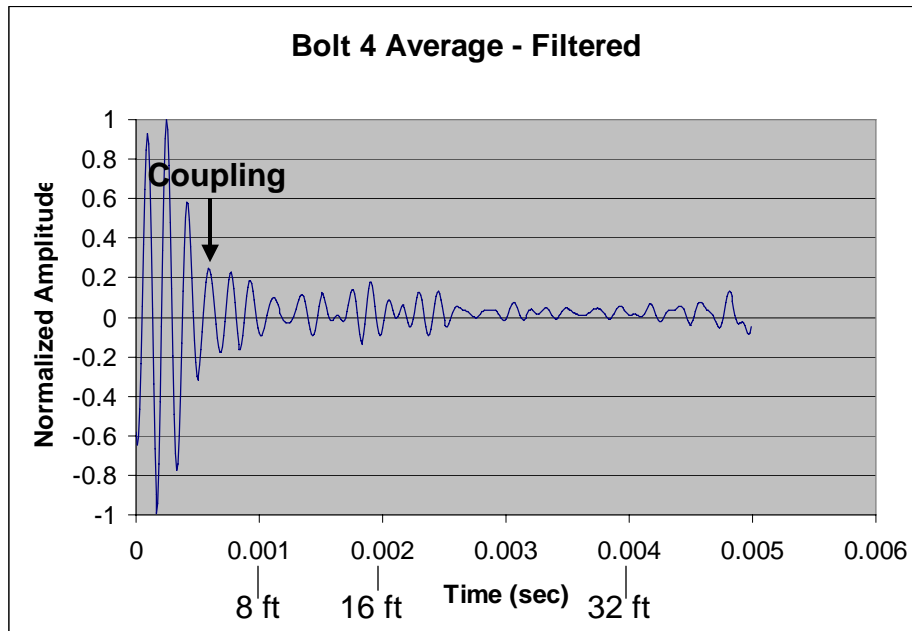


(a) Typical Response

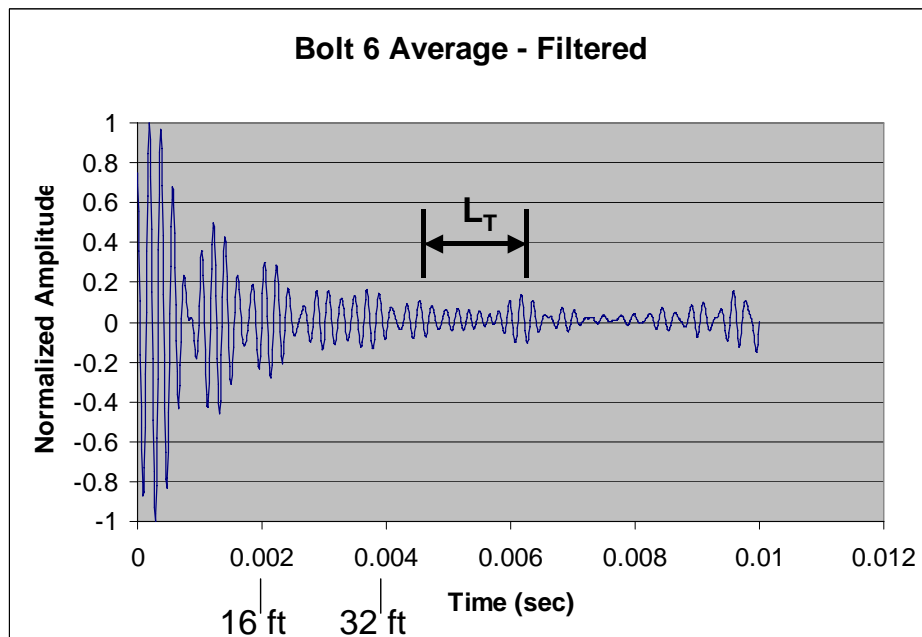


(b) Response of Specimen from Sample MMCE #16

Figure 11. Stress-Strain Curves from Tension Testing

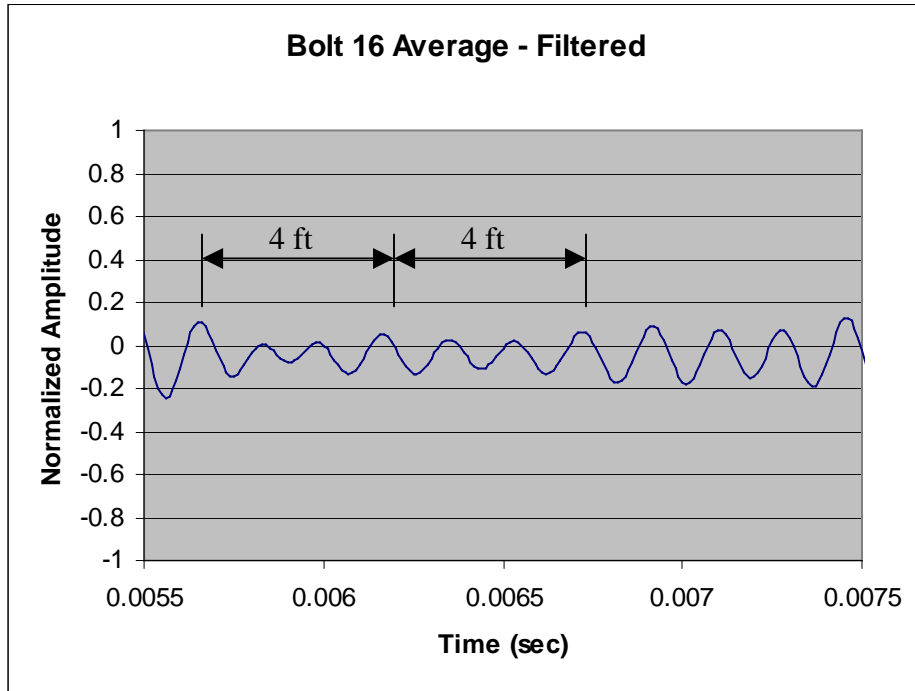


(a) Rock Bolt – G-19 (MMCE #4)

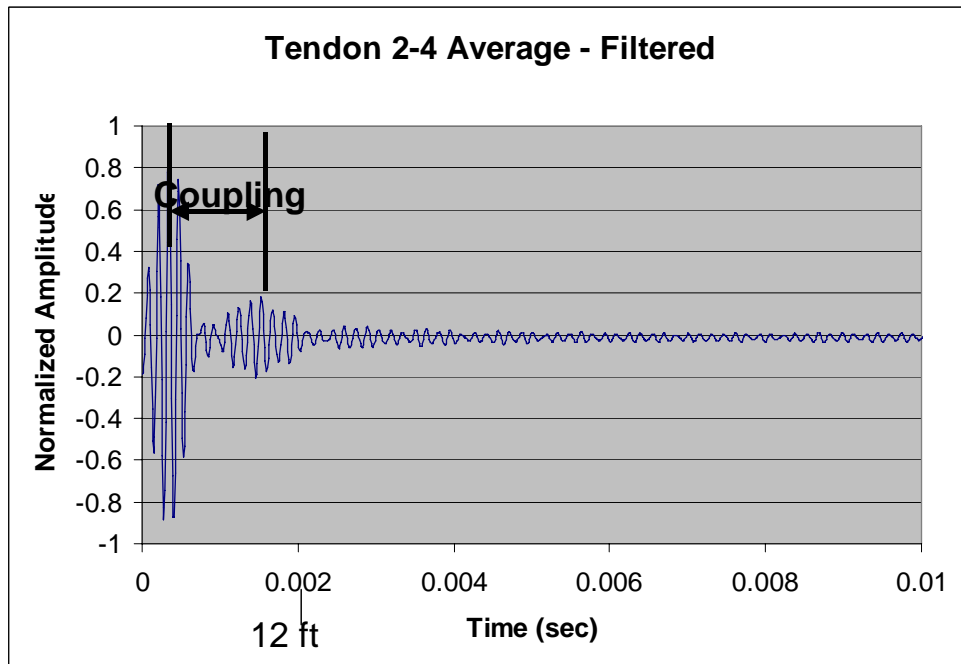


(b) Rock Bolt – G-8 (MMCE #6)

Figure 12. Wave Forms From Impact Tests



(c) Rock Bolt – MMCE #16



(d) Tendon MMCE # 2-4

Figure 12. Wave Forms from Impact Tests (Cont.)

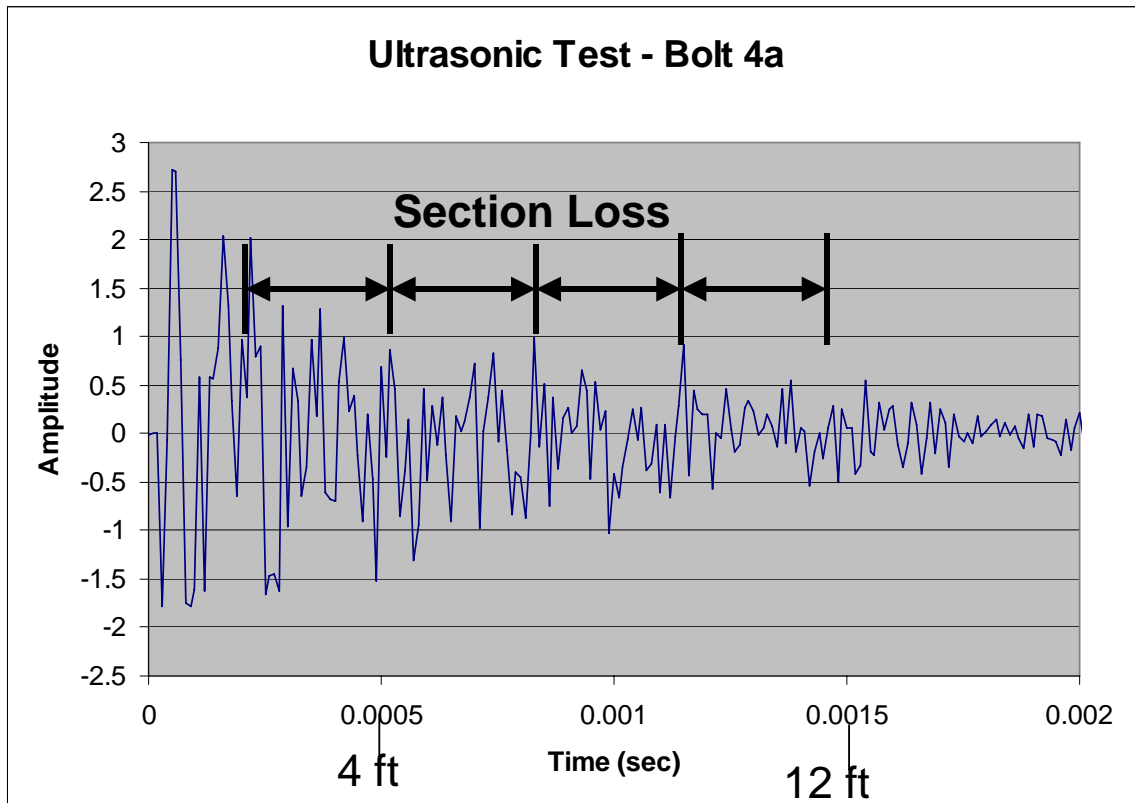


Figure 13. Wave Form From Ultrasonic Test for G-19 (MMCE #4).

A second document (Appendices I - VIII) accompanies this report, and is available on request from the Research Section of the NHDOT Materials & Research Bureau.

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