A Recommended Practice for Condition Assessment of Ground Anchors and Soil Nails

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A Recommended Practice for Condition Assessment of Ground Anchors and Soil Nails

K.L. Fishman[1], J.L. Withiam[2], and B. H. Jackson[3]

Abstract

This paper describes a recommended practice for condition assessment and evaluation of ground anchors (including soil and rock anchors) and soil nails. Although the recommended practice mainly applies to evaluation of remaining service life for existing systems, some aspects of the protocol can be applied to construction QA/QC to verify details of element installation. The protocol includes nondestructive testing (NDT) supplemented by invasive testing of selected elements for condition assessment. General concepts of NDT are described including the GRANIT^{TM} ground anchor integrity test, which is a commercially available system for impact testing of rock bolts and ground anchorages.

The paper presents results from case studies used to verify results from the NDT and demonstrate application of the recommended practice. At one site, selected rock reinforcements were tested by lift-off testing and then exhumed for comparison with results from a variety of NDT including the GRANIT integrity test.

Introduction

Prestressed ground anchors (strands and bars), soil nails, and rock bolts have been used with increasing frequency for the construction and repair of foundations, retaining walls, and excavated and natural soil and rock slopes. Some of the earlier rock bolt and ground anchor installations are approaching a service life of approximately 30 to 40 years. Because visual observation of conditions at the element head assembly is often not indicative of potential problems, the condition of existing systems is uncertain. Owners, faced with the task of allocating budgets to rehabilitate aging facilities, need a protocol for performing condition assessment and estimating the remaining useful service life.

Performance monitoring, and condition assessment of existing installations of ground anchor systems, will enhance our ability to (Fishman and Withiam, 2002):

1. define service life (i.e., can a service life greater than 50 years be achieved?)
2. quantify the impact of adverse conditions on service life,
3. evaluate the reliability of corrosion protection systems, and
4. manage existing facilities and optimize use of scarce resources for infrastructure.

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This paper describes results from recent research for condition assessment, estimation of remaining useful service life, and application of these results to a variety of ground anchor systems. The original research was sponsored by the National Cooperative Highway Research Program (NCHRP) under the auspices of NCHRP Project 24-13, “Evaluation of Metal-Tensioned Systems in Geotechnical Application,” (Withiam et al., 2002). The protocol and recommended practice were applied to a number of field studies as part of NCHRP 24-13, and recently to evaluate the condition of 30-year old rock reinforcements at the Barron Mountain Rock Cut near Woodstock, New Hampshire (Fishman, 2004; Fishman et al., 2005; Lane et al., 2005).

Recommended Practice

Details of the recommended practice for condition assessment are described in Withiam et al. (2002). In general the protocol requires:

- Collect preliminary information including installation details and site conditions.
- Identify appropriate mathematical models of service life and use these models to estimate metal loss from corrosion and remaining service life.
- Probe the elements with nondestructive tests, supplemented with invasive testing as appropriate, to assess the existing condition of selected elements comprising the metal-tensioned system.
- Compare results of the condition assessment to expectations based on site conditions and estimated metal loss.
- Recommend an action plan based results from the condition assessment.

Installation details have an effect on the vulnerability of the system to corrosion and on our ability to probe the elements and interpret data from NDT. Relevant details include steel type, corrosion protection measures, drill hole dimensions, bond length, free length, total length, date of installation, level of prestress, grout type and use of couplings. If the system is protected with an adequate, well constructed, corrosion protection system (e.g., meeting the requirements of PTI Class I [PTI, 1996]), then corrosion has not been found to be a problem. However, construction details, element durability, and workmanship associated with the corrosion protection system may affect the service life.

The subsurface environment surrounding the elements must be characterized in terms of soil or rock types, moisture conditions, presence of organics, and electrochemical parameters known to contribute to corrosiveness. Quantitative guidelines are available for assessing the potential aggression posed by an underground environment relative to corrosion (FHWA, 1993). Generally, moisture content, chloride and sulfate ion concentration, resistivity and pH are identified as the factors that most affect corrosion potential of metals underground.

Mathematical models are available for estimating metal loss and remaining service life (Romanoff, 1957). These models consider the corrosiveness of the subsurface environment, metal type, age of the system, and type of corrosion (e.g., uniform, localized, pitting). Calculated metal losses serve as a datum to assess system performance. The models do not directly consider the presence of corrosion inhibitors or other corrosion protection measures.

Nondestructive testing and condition assessment requires a sampling strategy whereby the appropriate sample size is selected to provide a statistical basis for the test results. Withiam et al. (2002) and Hegazy, et al. (2002) describe a simplified sampling criteria based on the probability that the sampled population will represent conditions throughout the site. The recommended sample size is based upon the total number of elements at the site, the importance of the facility relative to the consequences of failure, and a reference, or baseline condition, for comparison to observations. Generally, for a population consisting of 10 to 200 metal-tensioned elements, between 10 and 40 randomly distributed samples are required.

Description of NDT

Nondestructive test techniques are used to probe the elements, and the results are analyzed for condition assessment. Four NDT's are commonly applied for condition assessment including measurement of half-cell potential, polarization current, impact and ultrasonic testing. 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 aspects of the condition assessment:

- Half-cell potential tests serve as an indicator of corrosion activity.
- Results from the polarization test are correlated with the surface area of steel that may be in contact with the surrounding rock mass (i.e., indicator of grout quality and degree of corrosion protection).
- Impact test results are useful to diagnose loss of prestress, assess grout quality and indicate if the cross section is compromised from corrosion, or from a bend or kink in the element.
- Ultrasonic test results are useful for obtaining more detailed information about the condition of elements within the first meter from the proximal end of the element.

Withiam et al. (2002) and Fishman, et al. (2002) describe details of NDT including test procedures.

<u>Half-Cell Potential</u>

The half-cell potential, E_{corr}, is the difference in potential between the metal element and a reference electrode. A copper/copper sulfate (CSE) half-cell is used as the reference electrode and all potential measurements are with reference to the CSE potential. Half-cell potential measurements serve as an indicator of corrosion activity. However, in addition to corrosion, half-cell potentials are affected by a number of environmental factors including pH. Interpretation of half-cell potential must consider whether the environment is neutral or alkaline. In a neutral soil/water environment, half-cell potential becomes increasingly positive as elements corrode. The possibility that some elements have undergone greater corrosion may be identified if their half-cell potentials are more positive relative to the potentials observed for other elements at the same site, under similar environmental conditions. In general, the half-cell potential of noncorroded steel is between -800 mV to -500 mV, and that of corroded steel is between -500 mV to -200 mV.

Alkaline environments, such as Portland cement grout, tend to promote passivation of steel elements, wherein the surface is protected subsequent to development of a thin passive film layer, leading to relatively low metal loss from corrosion. Passivated steel is associated with a relatively high half-cell potential (i.e. less negative), which is lowered as the thin film responsible for passivation becomes compromised. If the half-cell potential is higher than -200 mV, the steel is passivated and corrosion is not likely. For half-cell potentials lower than -350 mV, passivation is compromised, and corrosion of the steel is likely. For half-cell potentials between -200 mV and -350 mV, corrosion is possible but uncertain.

Polarization Measurement

Polarization measurement involves installing a common ground at some distance from the measurement location, impressing a known current between the metal element and the ground bed, and observing the relationship between surface potential and impressed current (E vs. log I). Withiam et al. (2002) and Fishman, et al. (2002) describe how polarization curves are prepared and the polarization current, I_P, is observed. The polarization current is correlated with the surface area of steel that may be in contact with the surrounding rock mass (i.e., not surrounded by impervious grout or plastic sheath).

Impact Testing

The specimen is impacted, which generates elastic compression waves with relatively low frequency content. The traveling waves are reflected whenever a change in material or geometry is encountered along the length of the element, and the resulting waveform is observed with a transducer attached to the end of the element. Generally, condition assessment 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.

Specialized testing equipment is commercially available for impact testing of ground anchors including the GRANIT™ ground anchor integrity test (Ivanovic et al., 2002). The GRANIT test is useful for assessing rock bolt prestress and the free length of prestressed rock anchors. The GRANIT test includes the use of a patented air hammer that applies a measurable and repeatable impact to the anchorage, and proprietary software to interpret the signal in terms of experience and results from numerical analysis of the anchor system.

Ultrasonic Testing

The ultrasonic test method has many of the features of the impact test, except that the transmitted signal contains relatively higher frequencies. Ultrasonic waves are radiated when an ultrasonic transducer applies periodic strains on the surface of the test object that propagate as stress waves. The times for sound pulses, generated at regular intervals, to pass through the specimen and return, are measured. Good acoustic coupling between the transducer and the face of the element is a requirement for ultrasonic testing, and the face of the element must be flat and smooth. Due to the higher frequency content of the sound waves, reflections from sources within one meter of the proximal end of the rock bolts are more apparent in the results from ultrasonic testing compared to the impact test results.

Verification of NDT

In-Situ Test Facility

Eight, 3-m long elements were installed vertically in 150-mm diameter, 2.75-m deep, auger holes at an in-situ test facility established at the University at Buffalo, (Fishman, et al., 2002). All of the elements had a 0.3 m long grout bulb at their lower end to simulate anchorage in soil. Four types of elements were installed at the UB Test Facility including:

1) 32 mm diameter plain Dywidag bars
2) 32 mm diameter epoxy coated Dywidag bars
3) 32 mm diameter plain Dywidag bars surrounded by grout encased in a 2.7m long, 10 mm diameter plastic pipe
4) Polystrand 15-mm diameter, seven-wire strand coated with grease, and surrounded by an extruded HDPE sheath

Soil at the site is fine grained with varying amounts of gravel and is classified as CL according to the Unified Soil Classification System. Measured moisture contents range from 8 to 13.5 percent and a degree of saturation between 60 and 90 percent.

Two specimens were installed for each element type: one intact without a defect, and the other with a defect. Bar element defects were constructed as a notch placed about 1 m from the far end of the bar. The notch removed approximately 25 percent of the bar cross-section over a length of 75 mm. For grouted specimens, the notch extended through the grout and into the bar. For strand elements, a 75 mm length of the HDPE sheath was stripped to expose the strand to the subsurface environment.

TABLE 1. Summary of Electrochemical Test Results from UB Test-bed Specimens

Element #	Description	E_{corr} (mV)	I_p (mA)	A_s (m^2)	L_e (mm)
1	Plain bar (d = 32 mm)	-620	6.0	0.2860	2857
4	Epoxy bar w/defect (d=32 mm)	-752	0.2	0.0095	95
5	Strand (d = 15 mm)	-263	≈ 0	≈ 0	≈ 0
6	Strand w/defect (d = 15 mm)	-678	0.1	0.0048	101
8	Grouted bar w/defect (d = 32 mm)	-308	0.8	0.0380	379

E_{corr} is the free corrosion potential with reference to the half-cell
I_p is the polarization current observed from the E vs. Log I relationship
$A_s = I_p/21$ is the estimated surface area of the element in contact with the ground
$L_e = A_s/\pi d$ is the length of the element corresponding to A_s, and d is the element diameter.

Table 1 summarizes the results from the electrochemical tests performed approximately two months after installation. Specimens 5 and 8 had higher half-cell potentials (greater than –350 mV) and reflect the fact that these specimens are passivated by the alkalinity of the Portland cement grout (Specimen 5 was completely isolated along its free length by a dielectric plastic sheath). Other elements had half-cell potentials more negative than –500 mV, suggesting that these elements were in a neutral soil environment, and had not yet corroded.

The lengths of exposed element (L_e) estimated from the polarization test, are consistent with the known conditions of the elements. Element 1 was in direct contact with the ground for the majority of its length. By contrast, Element 5 was insulated with grease and plastic sheathing. This is reflected in the relatively large and small values of L_e for Elements 1 and 5. Elements 4 and 6 had L_e roughly corresponding to the 75-mm long defect. The larger L_e of Bar 8 includes some exposed metal near the top of the element in contact with the ground.

TAMU Wall

In 1991, a tied-back soldier pile wall with wood lagging (pictured in Figure 1) was constructed at the Texas A&M University-National Geotechnical Engineering Experimentation Site (TAMU-NGES). Soils at the site consist primarily of alluvial sand deposits. Based on the results from the chemical analysis, including pH close to neutral, and a relatively high resistivity, the soil at this site is not considered aggressive relative to corrosion.

Briaud et al. (1998) report details of the wall construction and performance monitoring during and after construction. Tiebacks are 32 mm diameter, Grade 1035, Dywidag bars which were installed in 89 mm diameter holes. Bonded lengths are 7.3 m, and unbonded lengths are either 4.6 m or 4.9 m, depending on location. All anchors were installed 30° to the horizontal, and pressure grouted in the bonded zone under pressures ranging between 1.4 MPa and 4.1 MPa. Grease and a plastic sheath surrounded the elements in the unbonded zone. The wall was instrumented with strain gauges, load cells, and inclinometers. Generally, load cells indicated relatively constant anchor loads over the five-year period during which the wall behavior was studied.

Ten out of 19 anchors available at this site were tested and results from impact testing are described. Several interesting features of the bar installation are apparent in the amplitude spectra. Peak frequencies corresponding to higher frequency contents of approximately 10,000 Hz were apparent in test result numbers 1, 3, 4, 5, 8, 9, and 10, but were not apparent in test result numbers 2, 6 and 7. This observation is consistent with the concept that higher prestress is correlated with higher natural frequencies of vibration (Rodger et al., 1997). Measured loads in the elements corresponding to test numbers 2, 6, and 7 range from 85 to 186 kN, which is lower than the range of 204 to 378 kN observed for the remainder of the elements.

Figure 2 is a typical time history of the response of an element to impact. Reflections in the signal are apparent at times of 0.0019 and 0.0043 seconds corresponding to the free length and total length of the element, respectively (assuming a compression wave velocity of 5500 m/s for steel). Table 2 shows a comparison between observations from impact tests and as-built details of the wall system for all of the elements tested at TAMU-NGES. The table presents information on the unbonded lengths, $L_{unbonded}$, bonded lengths, L_{bonded}, and the total lengths, L_{total}, of the tieback elements. The unbonded and total lengths include an approximately 1 meter extension of the tiebacks beyond the wall face. The bonded lengths are computed as differences between the total and unbonded lengths. Based on the data presented in Table 2, the comparison between observations and as-built details is considered satisfactory. All of the measurement errors are within ± 20 percent, and two-thirds of the measurement errors are within ± 10 percent.

Figure 1. Tieback Wall at TAMU-NGES

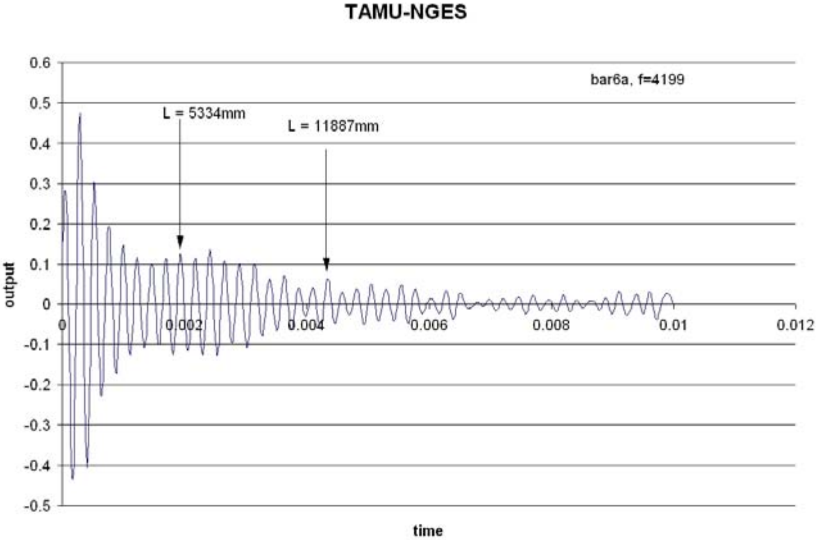

Figure 2. Impact Test on Bar 6 at TAMU-NGES

Table 2. Reflections Observed from Impact Test Results at TAMU-NGES

Test No.	Observed			As- Built Details		
	$L_{unbonded}$ (m)	L_{bonded} (m)	L_{total} (m)	$L_{unbonded}$ (m)	L_{bonded} (m)	L_{total} (m)
1	5.2	8.3	13.4	5.9	7.3	13.2
2	5.5	8.5	14.0	5.9	7.3	13.2
3	5.2	7.3	12.5	5.9	7.3	13.2
4	4.9	7.0	11.9	5.9	7.3	13.2
5	5.2	N.O.	N.O.	5.9	7.3	13.2
6	5.3	6.6	11.9	5.9	7.3	13.2
7	5.9	6.0	11.9	5.9	7.3	13.2
8	5.2	7.0	12.2	5.9	7.3	13.2
9	6.4	7.6	14.0	5.9	7.3	13.2
10	5.3	9.0	14.3	6.2	7.3	13.5

N.O. – not observed

Case Study

NDT was used for condition assessment and estimation of remaining service life for 30-year old rock reinforcements at the Barron Mountain rock cut along I-93 near Woodstock, NH. Two types of rock reinforcements are installed at Barron Mountain: (1) partially-bonded, resin-grouted, prestressed rock bolts, and (2) fully-bonded, Portland-cement grouted, passive tendons.

Rock bolts and rock tendons include 25 mm or 32 mm diameter steel threadbars. Most of the reinforcements are Dywidag, Grade 1035, high-strength prestressing steel threadbars. Some rock bolts are Grade 550 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). The rock bolts were initially prestressed to 90 kN or 180 kN depending on the steel grade. Tendon elements were fully grouted with Portland cement grout, and the proximal ends were recessed into the rock mass. The tendons are passive elements (i.e., they were not prestressed) and there is no anchorage assembly.

Detailed description of the site conditions and results from the NDT can be found in the interim report for the project (Fishman, 2004). Results from NDT can be generally summarized as follows:

1. Site conditions were moderately corrosive, corresponding to an estimated remaining service life of approximately 15 to 20 years due to metal loss from corrosion of the rock reinforcements.
2. Fully grouted rock tendons were apparently in better condition than resin grouted rock bolts in terms of grout quality and likelihood of corrosion.
3. Corrosion is occurring or has occurred along many of the rock bolts as indicated by half-cell potential.
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 based on impact test results.
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 2.5 mm 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 rock mass.

Both simple impact testing and GRANIT testing were performed on prestressed rock bolts as depicted in Figures 3 and 4. Damping, or the rate of decay, of the acceleration amplitude response has been shown to increase with respect to level of prestress for rock bolts (Rodger, et al., 1997). The envelop of the positive peaks of the amplitude response over the initial portion of the acceleration time history (damping envelop) portrays the rate of decay as shown in Figures 5 (a) and (b). Loss of prestress is diagnosed from NDT by comparing the rates 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 is a summary of lift off test results and comparison with the interpretation from NDT.

Table 3. Lift-Off Test Results

Bolt #	Lift-Off (kN)	NDT Result	Correct NDT
3	160	Good	Y
4	170	Good	Y
7	76	Good (?)	N(?)
8	98	Good	N
9	90	NG	Y
G-1	30	NG	Y
6	Loose	NG	Y
17	Loose	G/NG	Y(?)

Figure 3. Simple Impact Test

Figure 4. GRANIT Test

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 #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 also identified bolts with low versus high levels of prestress. However, diagnosis of the magnitude of prestress from the results of GRANIT tests 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.

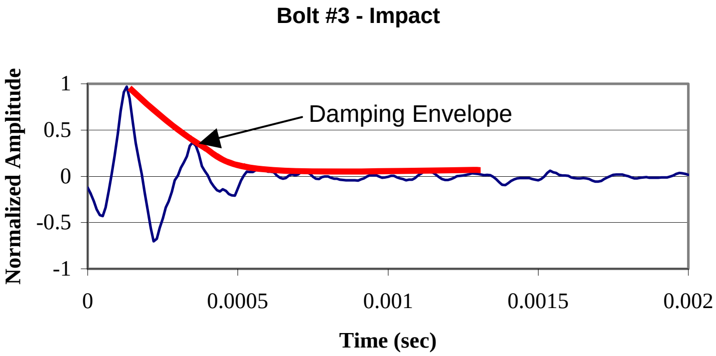

Figure 5 (a) High rate of decay indicative of relatively high bolt load.

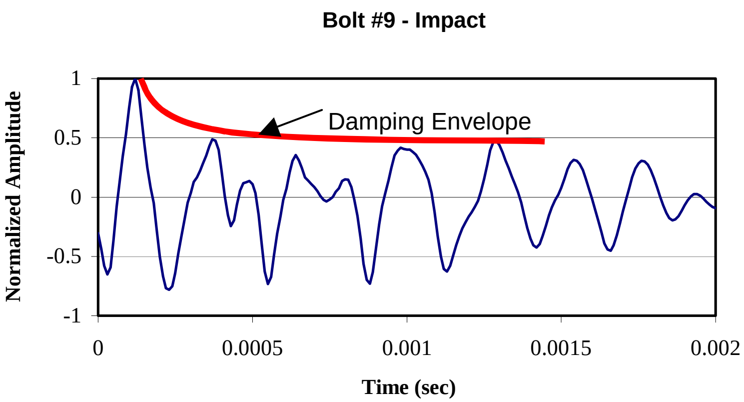

Figure 5 (b) Low rate of decay indicative of prestress loss.

Conclusions

A comprehensive practice and protocol for the evaluation of in-service buried metal-tensioned systems is presented. The protocol includes NDT for condition assessment, but invasive testing of selected elements is also required to confirm element performance and the reliability of the NDT methods and test interpretation. Results from NDT are verified with respect to measurements under known conditions at a specially constructed in situ test facility, and for tiebacks installed at a NGES.

A case study is presented describing application of the recommended practice to 30-year old rock reinforcements at the Barron Mountain rock cut near Woodstock, NH. The condition assessment revealed that a large number of prestressed rock bolts have suffered loss of prestress. Loss of prestress at the Barron Mountain site was measured with the

GRANIT rock anchor integrity test, lift-off tests, visual observations of anchor conditions at the site, and from an alternative, but relatively crude, impact test.

Results from NDT serve as useful indicators of overall reinforcement condition. Detailed knowledge of installation details including the location of couplings and joints, seams and fissures within the subsurface 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.

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References

Briaud, J.L., R. Griffin, A. Young, A. Soto, A. Suroor, and H. Park. (1998). "Long Term Behavior of Ground Anchors and Tieback Walls," *Report No. FHWA/TX–99/1391–1*, NITS, Springfield, VA.

Federal Highway Administration, (FHWA). (1993). *Recommendations Clouterre 1991* (English Translation), report on the French National Project Clouterre, FHWA Report No. FHWA-SA-93-026, Washington, D.C., 321 p.

Fishman, K.L. and Withiam, J.L. (2002). "Condition Assessment, Performance Evaluation and Estimated Service Life of Buried Metal-Tensioned Systems," *Foundation Drilling*, September/October, 2002, 8p.

Fishman, K.L., Gaus, M.P., Withiam, J.L. and Bontea, M. (2002). "Condition Assessment of Buried Metal-Tensioned Elements," *Journal of the Transportation Research Board*, No. 1786, TRB, Washington, D.C.

Fishman, K. L. (2004). *Phase I: Condition Assessment and Evaluation of Rock Reinforcement Along I-93*, Report No. FHWA-NH-RD-13733L, National Technical Information Service, Springfield, VA., 21 p.

Fishman, K.L., Lane, R.M. and Bojarski, R.J. (2005). "Condition Assessment of Thirty-Year Old Rock Reinforcements," *Proceedings of the 56th Highway Geology Symposium*, North Carolina Department of Transportation, Wilmington, NC.

Hegazy, Y.A., Withiam J.L. and Fishman, K.L. (2003). "A Practical Semi-Objective Sampling Criterion – How Many Samples Do We Need?", *Soil and Rock America 2003*, Proceedings, 12th Pan American Conference on Soil Mechanics and Geotechnical Engineering, P.J. Culligan, H.H. Einstein and A.J. Whittle, eds., Vol. 2, pp. 2771 – 2776.

Ivanovic, A, Neilson, R. and Rodger, A.A. (2002). "Influence of Prestress on the Dynamic Response of Ground Anchorages," *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 128(3), 237-249.

Withiam, J.L., Fishman, K.L. and Gaus, M.P. (2002). "Recommended Practice for Evaluation of Metal-Tensioned Systems in Geotechnical Applications," *NCHRP Report 477*, National Cooperative Highway Research Program, Transportation Research Program, Washington, DC, 92p.

Lane, R., Fishman, K., and Salmaso, A. (2005) "Exhuming Rock Reinforcement," *Proceedings of the 56th Highway Geology Symposium*, North Carolina Department of Transportation, Wilmington, NC.

Post-Tensioning Institute (PTI). (1996). *Recommendations for Prestressed Rock and Soil Anchors*, Phoenix, AZ.

Rodger, A.A., Milne, G.D., and Littlejohn, G.S. (1997). "Condition Monitoring & Integrity Assessment of Rock Anchorages," *Ground Anchorages and Anchored Structures*, Ed. S. Littlejohn, Thomas Telford, London, England, pp.343-352.

Romanoff, M. (1957). Underground Corrosion, National Bureau of Standards, Circular 579, U.S. Department of Commerce.