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REPORT 477

NATIONAL
COOPERATIVE
HIGHWAY
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PROGRAM

Recommended Practice for Evaluation of Metal-Tensioned Systems in Geotechnical Applications

TRANSPORTATION RESEARCH BOARD

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Recommended Practice for Evaluation of Metal-Tensioned Systems in Geotechnical Applications

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FOREWORD

By Edward T. Harrigan
Staff Officer
Transportation Research Board

This report presents the findings of a research project to evaluate procedures for (1) estimating the design life of metal-tensioned systems in new geotechnical installations and (2) determining the condition and remaining service life of systems already in place. It presents a recommended practice for assessing the present condition and remaining service life of metal-tensioned systems with nondestructive testing techniques and an appropriate prediction model. The report will be of particular interest to geotechnical engineers with responsibility for design, construction, inspection, and maintenance of metal-tensioned systems.

Transportation agencies use metal-tensioned systems to solve geotechnical engineering problems associated with construction and repair of foundations, retaining walls, and excavated and natural soil and rock slopes. Metal-tensioned systems include, but are not limited to, rock bolts, ground anchors, tiebacks, and soil nails. These systems are anchored by various means, including mechanical systems, epoxy and polyester resins, and cement grout, and may have varying levels of active or passive corrosion protection.

Metal-tensioned systems have been widely used for almost 25 years, and some of the earliest examples have been in place for more than 35 years. If the predicted 50-year design life of these systems is reasonable, then the useful lives of the earliest examples are more than half over. Once installed, metal-tensioned systems are vulnerable to failure by corrosion of the metal elements, loss of anchorage, or both, but visual observations of the conditions at the element head assembly often do not indicate actual or potential problems, and cases of premature failure have already been documented. Any failure has the potential to cause injury or loss of life, substantial property damage, significant economic loss to the public, and large rehabilitation costs to transportation agencies.

Under NCHRP Project 24-13, "Evaluation of Metal-Tensioned Systems in Geotechnical Applications," the D'Appolonia Engineering Division of Ground Technology, Inc., undertook research to identify or refine tools to predict the remaining useful life of existing installations of metal-tensioned systems and the design life of new installations, with the goal of producing a recommended practice, suitable for adoption by AASHTO, for assessing their condition and estimating their remaining service life.

The research team surveyed the literature on the application of nondestructive testing (NDT) methods to the problem of detecting corrosion and loss of anchorage in buried metal-tensioned systems. It identified several electrochemical tests, including measurement of half-cell potential and polarization current, that can detect the presence of corrosion and gauge the integrity of any corrosion protection systems. However, it found that mechanical nondestructive tests, principally wave propagation methods such as impact and ultrasound techniques, must be used to determine whether corrosion has caused loss of element cross section in the metal-tensioned system. A suite of selected

NDT methods was first evaluated in the laboratory and in controlled field installations and then validated through a program of testing metal-tensioned systems installed at eight field sites in New York State; Washington, D.C.; North Carolina; and Texas.

Information obtained through NDT must be used in conjunction with an appropriate prediction model to estimate remaining service life. For this purpose, the team chose an existing model expressed in terms of simplifying equations and nomographs; the model uses input data describing the corrosivity of the groundwater and surrounding soil and rock mass at a metal-tensioned system installation.

Finally, the research team developed a recommended practice that permits transportation agencies to tailor the application of NDT and other tests assessing site conditions to the degree of hazard associated with possible failure of a specific metal-tensioned system installation. This practice, in turn, is incorporated in a suggested agency management plan for its metal-tensioned system installations. The plan addresses (1) development of a metal-tensioned system inventory, (2) prioritization of the installations for detailed evaluation of site and metal-tensioned system element conditions, and (3) what actions agencies may take in response to the estimate of remaining service life provided through the recommended practice. Possible actions include doing nothing, conducting further NDT to more closely assess in situ conditions, performing invasive (i.e., destructive) testing, or initiating rehabilitation or retrofitting of the existing metal-tensioned system.

The final report includes summaries of existing metal-tensioned system practice and of a critical literature review on NDT methods and prediction models, a detailed description of the NDT methods selected for use with the recommended practice (see also Appendixes C–F), a discussion of the development and features of the recommended practice (see also Appendix A) and the metal-tensioned system management plan, details of the field validation studies, and six supporting appendixes:

- Appendix A: Recommended Practice for Evaluating Metal-Tensioned Systems Used in Geotechnical Applications;
- Appendix B: Percentage Points of the *t*-Distribution;
- Appendix C: Recommended Test Method for Half-Cell Potential Measurement of Rock Bolts, Ground Anchors and Soil Nails (2002);
- Appendix D: Recommended Test Method for Measurement of Polarization Current for Rock Bolts, Ground Anchors and Soil Nails (2002);
- Appendix E: Recommended Test Method for Impact Echo Test of Bar-Type Rock Bolts, Ground Anchors and Soil Nails (2002); and
- Appendix F: Recommended Test Method for Ultrasonic Probe of Rock Bolts, Ground Anchors and Soil Nails (2002).

This published report includes the entire text of the final report and all appendixes. The Phase I interim report was previously published as *NCHRP Web Document 27*.

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Dr. J. L. Withiam, P.E., from D'Appolonia was the principal investigator. Dr. K. L. Fishman, P.E., from MMCE, and Professor M. P. Gaus, from UB, are co–principal investigators. Consultants to the project include Mr. M. Schupack, P.E. (Schupack Suarez Consulting Engineers); Mr. L. Pinto, P.E. (National Fuel, Inc.); and Professor D. Chung (UB). Ms. M. Bontea, Ph.D candidate at UB, conducted evaluation of the nondestructive testing (NDT) methods and assisted in the field studies. Dr. Yasser A. Hagazy, P.E., from D'Appolonia assisted in preparing the recommended practice.

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RECOMMENDED PRACTICE FOR EVALUATION OF METAL-TENSIONED SYSTEMS IN GEOTECHNICAL APPLICATIONS

SUMMARY

Buried metal-tensioned systems include prestressed ground anchors (i.e., strands and bars), soil nails, and rock bolts. These systems have been used with increasing frequency by transportation agencies for the construction and repair of foundations, of retaining walls, and of excavated and natural soil and rock slopes. Some early 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 often does not indicate potential problems, the overall condition of existing systems is uncertain. Transportation agencies, 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.

NCHRP Project 24-13 was to develop procedures to evaluate the condition and remaining useful service life of buried metal-tensioned systems. The specific objectives of this project were to (1) evaluate and select viable performance-monitoring systems, (2) identify viable mathematical models to estimate remaining service life, (3) evaluate new and existing metal-tensioned systems installed at selected field sites, (4) develop a recommended practice for assessing condition and estimating remaining service life of existing and new metal-tensioned systems, (5) develop the framework of a database for summarizing performance data, and (6) prepare a work plan for agencies to use for collecting and analyzing performance data.

In the project, corrosion was identified as a major source of distress for metal-tensioned systems. Four different nondestructive testing (NDT) methods are recommended for condition assessment of buried metal-tensioned systems. Electrochemical tests, including measurement of half-cell potential and polarization current, are used to assess whether corrosion is present and the integrity of installed corrosion protection systems. Results from these tests may indicate that corrosion is occurring or can occur, but mechanical tests are needed to determine whether the condition of the element has been compromised by loss of cross section. Wave propagation techniques, such as impact and ultrasonic tests, are used to assess the existing condition of elements (i.e., severity of corrosion). Impact tests are also useful for identifying elements with loss of prestress, which may be due to other factors affecting service life, including creep.

Equipment for performing the NDT is commercially available, and the NDT may be performed by people with limited specialized training. Knowledge of corrosion processes, wave mechanics, and signal processing are helpful for data processing and interpretation, and these tasks should be performed by a qualified engineer. Results from NDT must be supplemented with more certain, detailed information from invasive tests (e.g., lift-off tests). The value of NDT is to screen and identify element locations where more detailed invasive testing may be recommended.

The work plan proposed describes a rational approach to estimate future maintenance, rehabilitation, and retrofit needs for existing installations of metal-tensioned systems. The plan has four basic components: (1) develop an inventory of sites with installations of buried metal-tensioned systems within the agency's jurisdiction, (2) establish priorities regarding the need for detailed evaluation of site and element conditions, (3) formulate and implement a test protocol for condition assessment, and (4) formulate a recommended action plan. Recommended actions may include doing nothing, further NDT, invasive testing, or design of rehabilitation or retrofit of the existing metal-tensioned system.

The recommended practice describes a corrosion assessment model, a sampling strategy for element condition assessment, and parameters and input required for service-life prediction modeling. A simple decision tree is incorporated into the recommended practice to identify sites with a high risk of corrosion. Risk is the combined consideration of hazard at a site and vulnerability of the elements. A few parameters that describe the subsurface conditions are all that is required to describe site hazard. Element vulnerability depends on the age of the element, type of element, and level of corrosion protection afforded to the element.

A sampling strategy is needed because at many sites it is not feasible to test every element. A table, based on probability, is presented that permits for a simple decision on number of samples to test using the total number of elements at the site, the importance of the facility relative to the consequences of failure, and the anticipated level of performance as input.

Remaining service life is estimated using equations and nomographs, which relate rate of corrosion to factors associated with the corrosivity of the surrounding soil or rock mass. Service-life prediction models require results from testing soil, groundwater, and rock samples as input. Results from the service-life prediction and the condition assessment are compared in order to formulate a recommended action plan.

Eight sites were included in the field study to demonstrate application of the work plan and recommended practice for condition assessment and estimation of remaining service life for existing metal-tensioned systems. The field sites were located in the northeast, southeast, and southwest United States. Information was obtained for each site, including type of anchorage application (e.g., rock bolts, tieback, or wall anchors); type of element (i.e., bar or strand); date of installation; element vulnerability; subsurface conditions and site hazard; and prestress level.

The ages of the elements included in the field study range from 2 to 33 years old. Different anchorage types, including mechanical and cement- or resin-grouted anchorages within a variety of soil and rock types, are represented in the site inventory. Not all the elements were installed with corrosion protection systems that meet today's standards, and this fact is reflected in the different element vulnerabilities. A range of site conditions is also present, and the study includes sites corresponding to hazard conditions ranging between low and high. In addition to potential hazard due to corrosion, several of the sites have hazards related to distress from creep movement or poor drainage conditions.

Results from the field studies contribute to a database documenting the performance of in-service, metal-tensioned systems. Performance data obtained so far are consistent with risk assessment models that identify sites where corrosion is likely and with mathematical models of service life, which estimate rate of corrosion. Although corrosion was observed at many of the sites, significant distress was not identified at sites with installations less than 20 years old and ground conditions that were not highly aggressive relative to corrosion.

CHAPTER 1

INTRODUCTION AND RESEARCH APPROACH

Buried metal-tensioned systems include prestressed ground anchors (i.e., strands and bars), soil nails, and rock bolts. These systems have been used with increasing frequency by transportation agencies for the construction and repair of foundations, retaining walls, and excavated and natural soil and rock slopes. Although soil nailing is a more recent innovation, rock bolts were first used in the mining industry and later adopted for use by the transportation industry in the early 1960s. The use of permanent ground anchors in public-sector projects became common in the United States in the late 1970s.

Thus, 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 often does not indicate potential problems, the condition of existing systems is uncertain. Transportation agencies, 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.

This report presents results from Phase II (Tasks 8–14) of NCHRP Project 24-13, "Evaluation of Metal-Tensioned Systems in Geotechnical Applications," which is a study to develop procedures to evaluate the condition and remaining useful service life of in-place, buried, metal-tensioned systems. The results of Phase I (Tasks 1–7) are published as *NCHRP Web Document 27*.

Project 24-13 consisted of the following tasks:

- Task 1—Review and evaluate existing practice, performance data, and research findings;
- Task 2—Evaluate and summarize technical information;
- Task 3—Evaluate and select viable performance-monitoring systems;
- Task 4—Identify viable mathematical models to estimate remaining service life;
- Task 5—Develop work plans for field evaluations to validate Tasks 3 and 4;
- Task 6—Develop work plans for installation of new systems and monitoring;
- Task 7—Prepare and submit an interim report;
- Task 8—Implement the Task 5 work plans;
- Task 9—Develop the recommended practice;
- Task 10—Implement the Task 6 work plans;
- Task 11—Tabulate relative cost-benefit data for metal-tensioned systems;
- Task 12—Develop a database for summarizing performance data;
- Task 13—Prepare a work plan for collecting and analyzing performance data; and
- Task 14—Prepare a final report.

A technical description of each task is presented in the following sections.

1.1 TASK 1—REVIEW AND EVALUATE EXISTING PRACTICE, PERFORMANCE DATA, AND RESEARCH

Survey, review, and evaluate relevant practice, performance data, case studies, research findings, and other information, from both public and private organizations, related to the useful life of existing and new installations of metal-tensioned systems, factors affecting their useful life, and test and analysis methods required for service-life estimation. Work presently being carried out for other applications—such as pretensioned concrete systems, bridge cables, buried pipelines, and other situations that involve environments having characteristics of interest—are studied for potential application to buried metal-tensioned elements.

1.2 TASK 2—EVALUATE AND SUMMARIZE TECHNICAL INFORMATION

Evaluate and summarize technical information on (1) types of metal-tensioned systems and anchoring techniques that are now installed or currently available for new installations, (2) corrosion of the metal elements in tensioned systems in soil and rock, and (3) other potential failure mechanisms for these systems.

1.3 TASK 3—EVALUATE AND SELECT VIABLE PERFORMANCE-MONITORING SYSTEMS

Several nondestructive tests are evaluated and selected for monitoring and condition assessment of buried metal-tensioned systems. Electrochemical tests, such as half-cell and polarization measurements, are used to explore the nature of the corrosion process. Mechanical tests, including impact and ultrasonic tests, based on the principal that the vibration characteristics of the element are affected by features encountered by waves traveling along its length, are used to distinguish locations of distress along the element.

1.4 TASK 4—IDENTIFY VIABLE MODELS TO ESTIMATE REMAINING SERVICE LIFE

Identify mathematical models for use in predicting the remaining useful service life of existing and newly installed metal-tensioned systems. Study the significance of parameters required as input to the model, and describe the test methods and procedures for determining the required parameters, including NDT methods selected in Task 3 for condition assessment.

1.5 TASK 5—DEVELOP A WORK PLAN FOR FIELD INVESTIGATION TO VALIDATE TASKS 3 AND 4

Prepare a detailed work plan for field investigation of existing metal-tensioned systems representing a range of types, subsurface conditions, and ages to validate the measurement methods selected in Task 3 and the models for estimation of remaining useful life identified in Task 4. Use information obtained in Tasks 1 and 2 to identify features of installations and conditions that may significantly affect the performance of existing systems. The field investigation should include sites that are potentially problematic, as well as those where significant deterioration or loss of performance is not anticipated. Contact selected individuals from state and federal transportation agencies and specialty contractors to compile a list of potential test sites, and pay particular attention to sites where demolition of the facility is planned and anchors, rock bolts, or soil nails may be exhumed.

1.6 TASK 6—DEVELOP A WORK PLAN FOR INSTALLATION OF NEW SYSTEMS AND MONITORING

Prepare a detailed work plan for field investigations of new metal-tensioned systems to permit measurement of their condition throughout their useful lives. Identify instruments and monitoring systems that may be installed with metal-tensioned anchor systems. Identify potential sites using information obtained from Task 1 and contacts with transportation agencies, designers, and specialty contractors.

1.7 TASK 7—PREPARE AND SUBMIT AN INTERIM REPORT

The interim report is a summary of the findings from Tasks 1 through 6, which constitute the first phase of the project. Chapter 2 is a summary of the findings described in the interim report. These findings provide the basis for Tasks 8 through 13, which were undertaken in the second phase of the project.

1.8 TASKS 8 AND 10—CONDUCT A FIELD STUDY TO EVALUATE EXISTING AND NEW METAL-TENSIONED SYSTEMS

Application of the test protocol and recommended practice is demonstrated at selected field sites. Preliminary evaluations are described at several sites, which are useful for evaluation and verification of the NDT methods. These preliminary evaluations are followed by presentation of detailed evaluations of metal-tensioned systems supporting retaining walls and rock slopes. Subsurface conditions at each site are described, and results from measurement of soil pH, soil resistivity, and sulfate and chloride ion concentrations are provided, followed by results from the NDT. Whenever possible, anchor element visual inspections that confirm results from the NDT are reported.

1.9 TASK 9—DEVELOP A RECOMMENDED PRACTICE

A recommended practice is proposed for element condition assessment and estimation of remaining service life of existing metal-tensioned systems. The practice includes recommendations for assessment of ground hazard and element vulnerability, including necessary test methods, sampling plans for condition assessment, new test methods for condition assessment, and selection of parameters for service-life prediction models.

A draft of the recommended practice was submitted to the project panel in June 2001, and comments were received from the panel and discussed at a meeting in Washington, D.C., on August 8, 2001. The comments received from the panel are incorporated into the current version of the recommended practice.

1.10 TASK 11—TABULATE RELATIVE COST-BENEFIT DATA FOR METAL-TENSIONED SYSTEMS

The costs of implementing the proposed work plan and maintaining a performance database are compared with the benefits (i.e., associated cost savings related to maintaining, rehabilitating, or retrofitting existing metal-tensioned system; and the risks and costs associated with element failure).

1.11 TASK 12—DEVELOP A DATABASE

Results from the field study are included within the framework of a database summarizing performance data of existing metal-tensioned systems. The database provides needed information for validation, calibration and improvement of risk assessment and service-life prediction models.

1.12 TASK 13—PREPARE A WORK PLAN

A recommended work plan is proposed for collecting and analyzing performance data to validate the test methods and service-life prediction models used for condition assessment of buried metal-tensioned systems. The first step in the work plan is to establish an inventory of sites where metal-tensioned systems are installed, followed by application of a screening exercise to decide which sites need detailed evaluation of remaining service life, and, finally, by development of a recommended action plan.

1.13 TASK 14—SUBMIT A FINAL REPORT

Chapters 2 thorough 5 of this final report describe findings from Tasks 8 through 13; Chapter 6 provides interpretation, appraisal, and application of the findings, including limitations; and Chapter 7 presents conclusions and recommendations for future work.

CHAPTER 2

SUMMARY OF FINDINGS FROM PHASE I OF THE PROJECT

This chapter summarizes the findings from Phase I of NCHRP Project 24-13. Phase I activities included a survey of existing practice, evaluation of NDT techniques, a study of service-life prediction models, and preparation of work plans for monitoring and condition assessment of new and existing systems. The work plans were implemented during the second phase of the project.

2.1 SURVEY OF EXISTING PRACTICE

The survey of existing practice included a literature search and solicitation of information from state agencies, industry specialists, and consultants involved in the design and installation of metal-tensioned systems. The survey covered types of metal-tensioned systems in use, factors affecting their service life, performance data (including case histories), recommended practice, mathematical models used for service-life prediction, and testing techniques that may be used to monitor the condition of the metal-tensioned system throughout its useful service life.

2.1.1 Types of Metal-Tensioned Systems

Geotechnical applications of metal-tensioned systems include ground anchors, rock bolts, and soil nails. Table 2-1 summarizes key features of the different types of metal-tensioned systems.

Tensioned elements of the system include bar and strand components. The steel grade and level of prestress employed in these systems are relevant to the type of corrosion problems that may occur and prediction of service life. Soil nail systems use bar elements, but ground anchors and rock bolts may be either bar or strand. Bar elements are available in a variety of steel grades, ranging from Grade 60 to Grade 160. Strand elements are manufactured from Grades 250 and 270 high-strength steel. Wire tension systems, using the button head anchorage of BBRV and Prescon, were used in some early applications, but are now obsolete. These systems are not discussed further in this report.

Current guidance documents (PTI, 1996; Sabatini et al., 1999) recommend incorporating corrosion protection measures into the design of metal-tensioned systems. Corrosion protection measures include the use of coatings, protective sheaths, passivation with grout, encapsulation, and electrical isolation. Passivity refers to the loss of chemical reactivity experienced by certain metals and alloys under particular environmental conditions.

Ground anchors include an anchored or "bonded" zone and a free-length or "unbonded" zone. The bonded zone is anchored to the soil or rock with cement grout. Recent installations use Class I or Class II protection as recommended by PTI (1996). For Class I protection, the anchor is encapsulated (often referred to as double corrosion protection), and, for Class II, the anchor is protected by grout (often referred to as single corrosion protection). Double corrosion protection is recommended for ground anchors in aggressive ground conditions and permanent installations. Products on the market today all offer systems that comply with the current standards. However, many of the older installations (1) do not incorporate details that meet today's standards or (2) may have been installed without any corrosion protection beyond the passivation of the grouted portion of the tensioned elements.

Rock bolts either are installed with mechanical anchorages or are grouted into rock using cement grout or resin. Older-style rock bolts with mechanical anchorages may have no corrosion protection. Grouted or resin-grouted rock bolts are surrounded by grout, but the bolts heads are often not encapsulated. There is also the possibility of voids along the grouted length.

Soil nails are surrounded by grout, and both rock bolts and soil nails may be epoxy coated.

2.1.2 Performance

The main factors affecting the service life of metal-tensioned systems are corrosion, loss of prestress because of creep or loss of bond within the bonded zone, loading not considered in the design (such as stress from bending), cyclic loading, ice loads or hydrostatic pressures, and anchorage failure.

Particularly for the higher-strength steel, corrosion is often localized and evident in the form of pitting. Stress crack corrosion is aggravated by high tension from prestressing, which is often required for ground anchors and rock bolts.

Compared with failure from corrosion, less information is available in the literature describing the effect of creep on service life of metal-tensioned systems. However, some information is described relative to evaluating conditions for which

TABLE 2-1 Summary of types of metal-tensioned systems

System Type	Tendon Type	Anchorage Type	Corrosion Protection
Ground Anchors	Strands or bars	Cement grout in bond zone	More recent permanent installations use Class I or Class II Protection (PTI, 1996); older systems may have no protection other than grout cover
Rock Bolts	Usually bars, but could be strand	Mechanical, resin grout, or cement grout	Epoxy coating, galvanization, grout cover; older installations may have none
Soil Nails	Bars	Cement grout entire length	Grout cover; bars may be epoxy coated

creep may be a problem and the performance testing of anchors used to evaluate the potential for creep deformations during the service life of the structure.

In addition to geotechnical applications, the performance of metal-tensioned systems in other applications was reviewed. Other applications include prestressed containment structures built by the nuclear power industry, prestressed concrete pipe and tanks, and prestressed reinforced concrete for bridge and building construction. After a review of the performance of metal-tensioned systems, the research team has reached conclusions that are similar to those of Telford (1986):

- Most of the corrosion problems documented in the literature have been correlated with the presence of aggressive ground conditions or stray currents.
- The majority of corrosion problems tend to occur near the element head or within the free length of the tendon element.
- There have been only a few documented cases where corrosion problems were observed within the bonded zone. Cracking of the grout has been observed in the transition zone between the bonded zone and the free length. During prestressing, there is a concentration of strain in this area, which can lead to cracking of the grout. The cracks may compromise the ability of the grout to passivate the metal element and may provide electric conductivity between the element and soil electrolyte, which facilitates corrosion. This is a particular concern if groundwater is located at or near the transition zone.
- For systems with a properly installed and intact corrosion protection system, corrosion is not a problem, not even with aggressive ground conditions.

According to the above conclusions, the performance and service life of metal-tensioned systems depend on the details of the design, manufacture, and installation of corrosion protection systems, particularly with respect to encapsulation at the tensioned element head. If stray currents are present in the ground or if aggressive ground conditions exist, then the elements should be electrically isolated.

For strand tendons, the sheathing should be extruded onto the strand stressing length. Care must be taken during transportation and installation of tendon elements so as not to damage sheathing or disturb the grease or corrosion inhibitor compound surrounding the metal element. If grease is heated by the sun, it may lose viscosity and flow, leaving the upper portions of the tendon element exposed. The type of grease or corrosion inhibitor should be selected such that it does not have an affinity for water, does not promote microbacterial-induced corrosion, and contains an effective corrosion inhibitor.

2.1.3 Recommended Practice

Standards are available for assessment of aggressive ground conditions. If aggressive ground conditions are present, the condition of the existing anchor system is suspect. Further testing is needed to check whether corrosion protection systems are intact, whether corrosion is occurring, and the current condition of the metal-tensioned element. There is a European standard for electric resistance testing of grouted ground anchors, but this standard requires that each tendon element be electrically isolated from the rest of the system. In practice, this isolation is rare or may be difficult to achieve. Results from the electrical resistance test indicate whether the corrosion protection system has been compromised, but do not indicate whether corrosion is occurring or the existing condition of the metal element. Thus, NDT techniques are needed to obtain information about the condition of the system.

Some NDT techniques have been employed for monitoring the condition of other types of metal elements, including buried pipe, concrete reinforcement, and prestressing steels. Standards for these tests either are available or are currently under development. Existing NDT techniques were evaluated in Phase I of this study relative to their potential application to monitor the condition of metal-tensioned systems.

2.2 EVALUATION OF NDT TECHNIQUES

NDT techniques with the potential for application to metal-tensioned systems were reviewed. On the basis of the review, a number of techniques were further evaluated in the laboratory to study their application to condition assessment of ground anchors, rock bolts, and soil nails.

TABLE 2-2 Summary of NDT methods considered for condition assessment

Method	Previous Application	Equipment	Suitability for Testing MTS in Geotechnical Uses
Impact Echo	Evaluation of plate-type elements, honey combing in concrete, voids in ducts of bonded prestressed systems.	Commercially available at reasonable cost, but instrumented and modally tuned impact hammers offered as part of the system are expensive.	Need to apply impact and measure response at same end of element; will use impact with lower frequency compared with previous applications; therefore, same style hammer employed by previous researchers may not be necessary.
Impulse Response	Evaluation of drilled-shaft foundations; evaluation of tension levels in prestressed rods.	Similar to impact echo except response is measured w/ velocity transducer and larger impact w/ lower frequency content applied.	Similar to impact echo.
Parallel Seismic	Evaluation of drilled-shaft foundations.	Similar to impulse response method.	A hole needs to be advanced adjacent to the tendon element; usually not practical.
Continuous Acoustic Emission	Used by mining industry to detect rock instability, monitoring of post-tensioned concrete structures, cables of suspension and cable-stayed bridges, and prestressed concrete pipe.	Commercial equipment is available. Sensors are relatively inexpensive, but signal conditioning and data acquisition equipment is expensive. Specialized software is required, and signals must be processed and interpreted by expert.	Has potential for application to MTS, but capability of system is limited. Current technology is used to listen and indicate when failures occur. Does not indicate condition prior to failure.
Ultrasonic	Used to evaluate plate-type elements, weld quality, anchor bolts, anchor rods, bridge cables.	Commercial equipment is available. Cost of the equipment is moderate.	Can be readily adapted to MTS. Need access to element head, but can perform the test with access to only one end. Much of signal may be lost due to dispersion, and there is a limit to the length of element that can be tested.
Ground-Penetrating Radar	Evaluation of pavements, bridge decks, subsurface investigations.	Commercial equipment is available. Costs are moderate, although many agencies may already own this equipment	May be useful for locating MTS, but not sensitive enough to detect defects in elements that are not relatively close to the ground surface.
Reflective Impulse Measurement Technique	Evaluation of unbonded posttensioned cables.	Commercial equipment is available at moderate cost.	Implementation is possible as it may be applied to only one end of an element. Technique is not proven to be effective.
Time Domain Reflectometry	Used to locate discontinuities in electrical transmission lines. Recently studied for application to study cables in cable-stayed bridges.	Commercial equipment is available at moderate cost. Data require an expert for interpretation.	Not useful for existing systems. A silver monitoring wire running parallel to the element is required. Test method is interesting due to claims that it may detect pitting corrosion.
Magnetic Flux Leakage	Condition assessment of reinforced concrete structures.	Commercial equipment is available at moderate cost.	Not directly applicable as full length of element must be accessed. Technique is interesting because of claim that it may measure loss of cross section as low as 3%.
Fiber-Optic Corrosion Sensing	Evaluated condition of concrete reinforcement. Most of the research using this system is conducted in the laboratory. Some buildings have been instrumented.	Commercial equipment is available. Fiber-optic cable, which acts as a sensor, is relatively inexpensive. Equipment required to analyze the signal is very expensive.	Not suitable for existing systems since fiber-optic cable must be placed along the length of the element.
Half-Cell Potential	Monitor condition of steel reinforcement in concrete and rock bolts.	Equipment is readily available and relatively inexpensive.	May be applied. Electrical connection to one end of element is required. Half-cell must make electrical connection through electrolyte. May be difficult to get meaningful readings from elements that are not electrically isolated.
Polarization Measurements	Steel soil reinforcements, steel piles, steel reinforcing in concrete.	Similar to half-cell measurements with additional equipment to impress current on system.	Similar to half-cell measurement.

TABLE 2-2 ***(Continued)***

Method	Previous Application	Equipment	Suitability for Testing MTS in Geotechnical Uses
Electromagnetic Impedance Spectrometry	Similar to polarization measurement.	Similar to polarization resistance, but need source of AC current with variable frequency and more sophisticated data acquisition equipment.	Similar to polarization measurement.
Electrochemical Noise Technique	Concrete reinforcing steel. Test technique and data interpretation is still under development.	Similar to other electrochemical techniques. Need expert to interpret data.	Similar to half-cell potential and polarization measurements. May be useful to identify severity and type of corrosion.
Contact Resistance	Concrete reinforcing steel. Previous experience is with laboratory specimens.	Equipment is readily available and relatively inexpensive.	Need access to end of bar. Need to know surface area of element in order to interpret data.
Corrosion Potential Method	Unbonded prestressed tendons.	Commercially available at reasonable cost.	Difficult to implement. Need to access two points along the length of the tendon element. Need tendon sheath with space for air.

Note: MTS = metal-tensioned systems.

2.2.1 Literature Search

Literature was searched to collect information on NDT techniques that could potentially be implemented for condition assessment of buried metal-tensioned systems. Mechanical and electromagnetic wave propagation techniques and electrochemical-type tests were studied. Tests were evaluated on the basis of their potential for success, ease of application, cost of instrumentation, and availability of needed equipment. Table 2-2 summarizes the test methods considered. The table identifies each method, describes previous research or application, and indicates (1) the level of expense and training required for operation of the equipment and (2) the relative ease by which the technique may be implemented with metal-tensioned systems.

2.2.2 Laboratory Evaluation of NDT for Implementation with Metal-Tensioned Systems

Using results of the literature survey, the research team identified the impact echo test, ultrasonic test, half-cell potential measurement, and polarization measurement as tests that had the potential for successful implementation for condition assessment of rock bolts, soil nails, or ground anchors. Details of these test methods are described in Chapter 3.

Test methods were evaluated in the laboratory using bench-scale and in situ specimens. The objective of the laboratory evaluations was to study implementation of the test methods with metal-tensioned systems, the sensitivity of the methods to changing parameters, the range of performance for a given test method, and the ability of a test method to detect defects along the length of an element. On the basis of the results of the literature search and laboratory evaluation, the research team recommends the impact echo test, ultrasonic test, half-cell potential, and polarization measurements for implementation at selected field sites. Implementation of the testing techniques at field sites, as well as study of the test results and data collected, was included in Phase II of this research.

2.3 MATHEMATICAL MODELS FOR SERVICE-LIFE PREDICTION

This study adopted (1) the power law similar to that applied by Elias (1990) for service-life prediction of buried steel soil reinforcements and (2) the approach resulting from NCHRP Project 10-46 for estimating the service life of steel pile foundations. Details of the service-life prediction model are provided in Chapter 4.

2.4 FIELD SITES FOR PHASE II

Eight field sites were identified for Phase II of the investigation. Details of the sites and results from the field studies are included in Chapter 5. Pertinent information for each site includes the application as rock bolts, as tiebacks (i.e., grouted anchorage), or as anchors (i.e., “deadman” anchorage) for a retaining wall system; the type of element (either bar or strand); the date of installation; the existence of a corrosion protection system; the availability of soil data; whether or not the elements are prestressed; and site-specific comments.

The ages of the elements planned for the condition assessment range from 3 years old to 40 years old. Not all the tendons at the sites considered were installed with corrosion protection systems that meet today’s standards.

CHAPTER 3

CONDITION ASSESSMENT AND DESCRIPTION OF NDT

NDT techniques are used to probe the elements, and the results are analyzed for condition assessment. In Phase I of the project, four NDT techniques were investigated for application to buried metal-tensioned systems. This chapter describes the test techniques and their application to metal-tensioned systems. Chapter 4 describes how the NDT techniques are incorporated into the Recommended Practice (Appendix A) for performance evaluation of buried metal-tensioned systems.

Electrochemical tests, including measurement of half-cell potential and polarization current, are used to assess (1) whether corrosion is present and (2) the element surface area vulnerable to corrosion. Results from these tests may indicate that corrosion is occurring or can occur, but mechanical tests are needed to determine whether the condition of the element has been compromised by loss of cross section. Wave propagation techniques, such as impact and ultrasonic tests, are used to assess the existing condition of elements (i.e., severity of corrosion).

While most of the equipment considered for NDT can be obtained commercially, the equipment's specific application to buried metal-tensioned elements is described in this chapter. A general description of each test method and necessary equipment is presented, followed by a general description of data acquisition and processing. Detailed recommended testing procedures are presented in Appendixes C through F.

3.1 ELECTROCHEMICAL TESTS

3.1.1 Half-Cell Potential

The half-cell potential, E_{corr}, is the difference in potential between the metal element and a reference electrode, as shown in Figure 3-1. Equipment required for measuring half-cell potential includes a half cell, a high-impedance voltmeter, and a set of lead wires. Lead wires are attached to the end of the test element and the half cell. The lead from the half cell is connected to the negative terminal of the voltmeter, and the element lead is connected to the positive terminal. A copper/copper sulfate reference electrode (CSE) was used in this study.

For a given material in a given environment, the potential is an indicator of corrosion activity. Interpretation of the data must consider whether the element under testing is electrically isolated.

In general, as element corrosion becomes greater, the half-cell potential becomes increasingly positive. This trend is useful if the element is electrically isolated. The possibility that relatively greater corrosion has occurred along the surface of an element may be identified if its half-cell potential is more positive relative to the potentials observed for other elements at the same site. As a guide, the half-cell potential (with respect to CSE) of clean, shiny, low-carbon steel in neutral soils and water ranges from −500 mV to −800 mV. The half-cell potential of rusted, low-carbon steel in neutral soils and water is generally between −200 mV and −500 mV. Although the test results may be useful to identify where corrosion has occurred, they do not indicate whether the corrosion process is still taking place.

More negative half-cell potentials indicate a greater potential for corrosion at that element. This trend is useful if electrical connectivity exists between elements. Here, the element with more positive half-cell potential acts as cathode and the element with the lower potential is the anode, where corrosion can occur. As a guide, considering reinforcing steel bars embedded in concrete, limits recommended by ASTM C876 (ASTM, 2001) suggest that half-cell potentials more positive than −200 mV indicate a low likelihood that corrosion is occurring, while values more negative than −300 mV indicate a high likelihood that corrosion is occurring. Although the potential for corrosion may be indicated by this condition, it does not necessarily mean that corrosion has occurred.

Half-cell potentials are affected by a number of environmental factors, and in some instances, the trends described above may be different. For this reason, the environmental conditions of elements surrounded by resin grout need to be evaluated to establish the range of half-cell potential typically encountered for noncorroding and corroding elements, respectively.

Further details and a recommended test method for measuring half-cell potential of rock bolt, ground anchor, and soil nail installations are described in Appendix C.

3.1.2 Polarization Measurements

The polarization measurement method, shown in Figure 3-2, involves installing a common ground at some distance from the measurement location, applying a known voltage

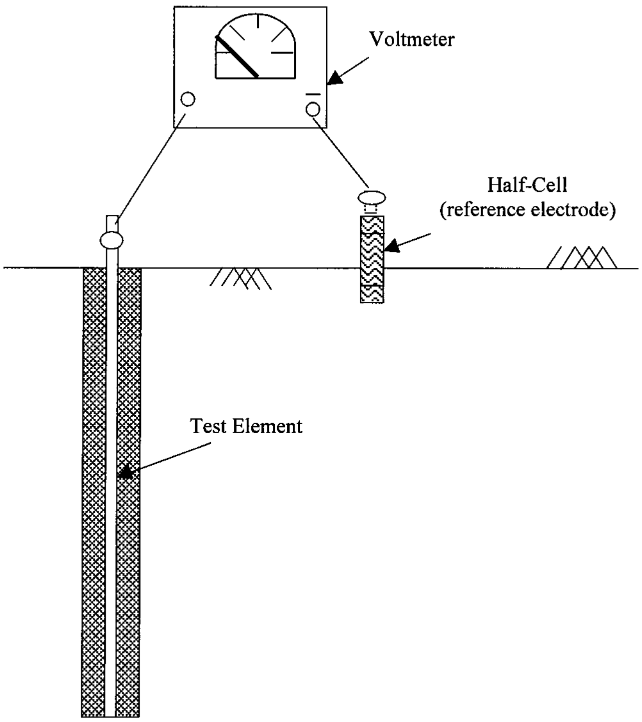


Figure 3-1.   Half-cell potential measurement.

between the metal element and the ground bed, and observing the relationship between surface potential and impressed current (E versus log I). Impressed current flows through the soil/water electrolyte from the element to the ground bed. Negatively charged ions within the soil/water electrolyte migrate toward the positively charged element. Current is increased in increments, and the change in potential of the element surface is observed. The basic premise of the test is that a level of current is reached for which the surface of the element is polarized and saturated with negatively charged ions. For the test data to be meaningful, the tested elements must be electrically isolated from the remainder of the system.

The equipment needed for the test includes a power supply with a rheostat, an ammeter, a high-impedance voltmeter, and a reference electrode (i.e., half cell). This equipment is standard and relatively inexpensive, and the components are readily available. Test components were assembled into a special portable unit, which was convenient for measurements made in the field. Three separate bus bars were arranged in the unit such that only three external connections (i.e., test bar, half cell, and ground bed) were required to set up the test.

The "E versus log I" curve (see Figure 3-3) is developed by applying increasing amounts of current for equal periods of time and plotting the polarized potentials versus the logarithm of the applied current until a definite break in the curve is obtained. The plotted data should result in a curve having an initial straight-line section curving into a second straight-line section (at a different slope). If this shape is not obtained, it is probable that a wide enough range of current was not used in the test. The second straight-line portion of the curve is known as the Tafel slope and should not have a slope greater than 0.1 volt per decade. The first point on the curve corresponding to the Tafel slope gives the polarization current.

Polarization measurements may be correlated with surface area of bare metal in contact with the ground. According to unpublished data compiled by the pipeline industry, for bare metal in contact with soil, approximately 21 milliamperes (mA) is required to polarize each square meter of surface area. Using this constant, the surface area of steel in contact with the ground can be computed using the measured current requirement, I_p.

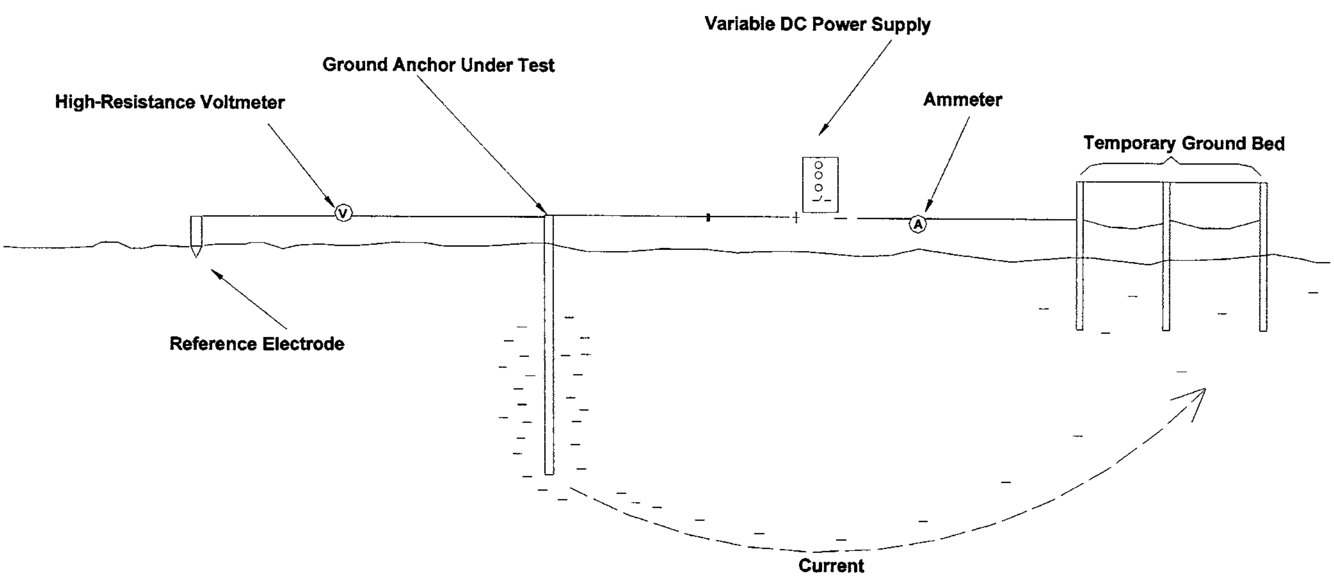


Figure 3-2.   Schematic of E versus log I measurement.

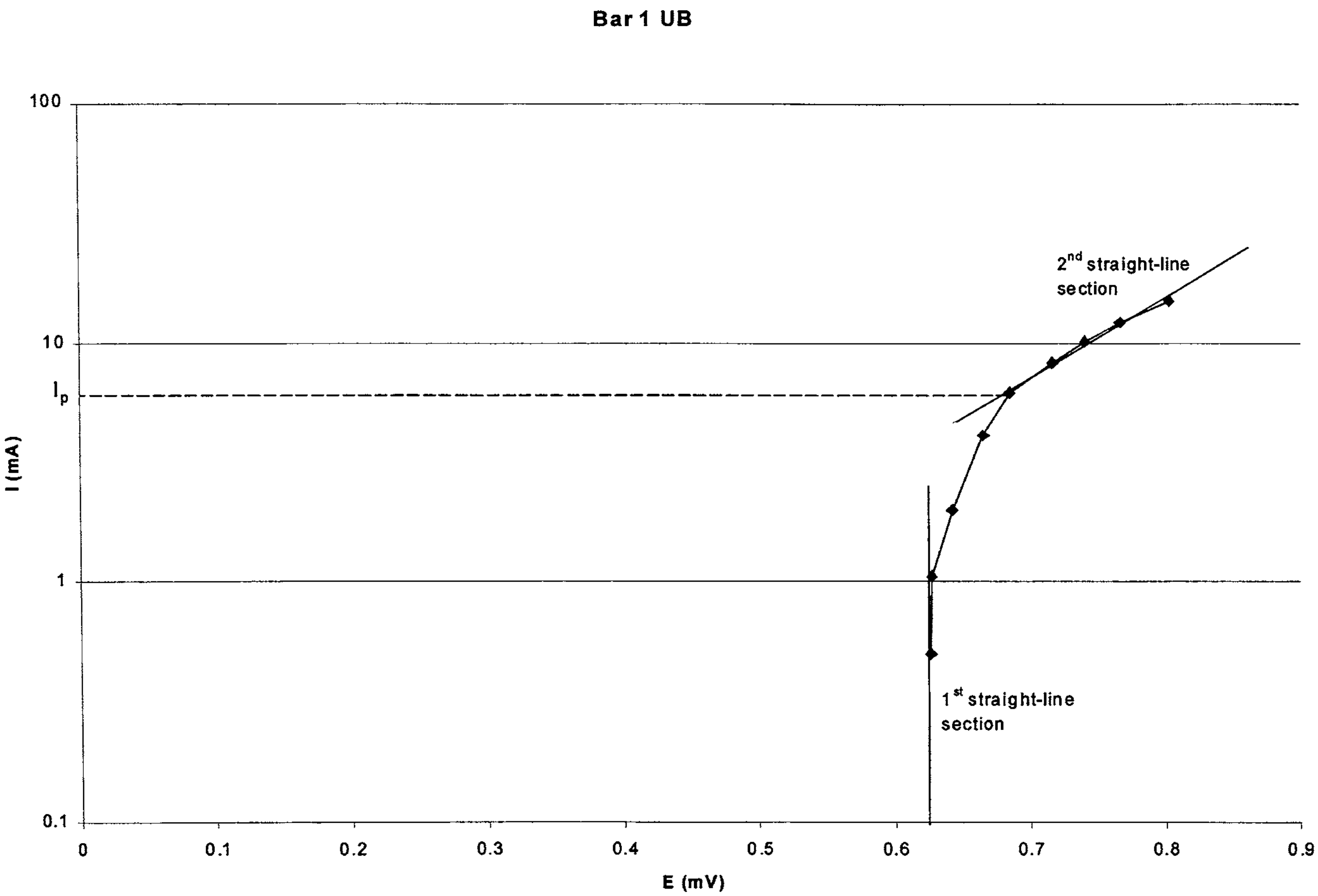


Figure 3-3. Typical polarization measurement showing characteristic curve.

Using the surface area of the steel element (A_s), the theoretical polarization current may be computed as I_{theory} (measured in milliamperes) = $21 \times A_s$ (measured in square meters). The estimated current requirement, I_{theory}, can be compared with measured current requirement, I_p, with three possible outcomes:

- If $I_p \ll I_{\text{theory}}$, the element is probably electrically well insulated and well protected.
- If $I_p < I_{\text{theory}}$, the element is probably coated or protected over just some part of its surface. Using the measured protection current (I_p), the unprotected length of the element can be estimated.
- If $I_p > I_{\text{theory}}$, more surface area is probably involved than was initially assumed, and electrical contacts with other elements having surface areas in contact with the ground may not have been considered.

This information can be used to assess the integrity of existing corrosion protection systems, which may involve plastic sheathing or other dielectric material surrounding or coating the element.

Further details and a recommended test method for measuring polarization current of rock bolt, ground anchor, and soil nail installations are provided in Appendix D.

3.2 MECHANICAL TESTS

For both the impact and ultrasonic tests, vibrations measured at the head of the element are recorded. Characteristics of the reflected waves are compared for different elements. One simple way to study the data is to compare the arrival times of the reflected waves. Other data-processing techniques include observing characteristics of the frequency response after transforming the signal into the frequency domain. Anomalies are located by comparing results with known installation details and comparing results of different elements at the same site. Also, measurements at the same site may be archived and results from testing at different times compared.

Using currently available equipment, loss of cross section less than approximately 25 percent is not detectable. As described by Briaud et al. (1998), the critical loss of cross section for a ground anchor corresponding to the end of useful service life can be computed assuming, initially, that the anchor is subjected to 60 percent of its yield strength under constant load. If the useful service life of the anchor is assumed to extend until the yield stress is reached, there is a corresponding section loss of 40 percent. Therefore, the results from NDT may indicate that substantial loss of cross section has occurred with some warning before the end of the service life is reached. It must be noted, however, that this correlation applies to uniform corrosion and does not address loss of tensile strength from pitting and the possibility of hydrogen embrittlement and stress crack corrosion.

3.2.1 Impact Echo Test

The impact echo test, as shown in Figure 3-4, may be used to evaluate cracking of grouts, fracture of tendons, and loss of element section. The specimen is impacted using a hammer or ball device, 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. Equipment required for the impact echo test method includes an impact device, an accelerometer, velocity or displacement transducer for measuring the specimen response, and a data acquisition system. The signal is processed with a signal conditioner that also includes a power supply with necessary excitation. As shown in Figure 3-4, tests may be conducted with the impact and receiver placed at the same end of the bar.

The accelerometer used in this study is a high-shock sensor (PCB Model U350A14) that has a sensitivity of 9 mV/*g*, a frequency range of 1–7,000 Hz, a resonant frequency of 56 kHz, and a measurement range of ±5,000 *g*. This accelerometer can measure the response of high accelerations associated with metal-to-metal impacts. The accelerometer may be fixed on a mounting base, which is attached to the specimen by special adhesives or magnets, or threaded directly to a drilled and tapped specimen face.

The signal conditioner (PCB Model 480E09) is battery powered and portable for applications in the field. The device supplies a DC excitation of 5 V, and the output has a range of selectable gain from 1 to 100.

A number of impact devices were evaluated for introducing the stress wave into the metal-tensioned systems. Small and medium-sized instrumented hammers and light hammers, such as tack hammers, were used with and without a centering punch.

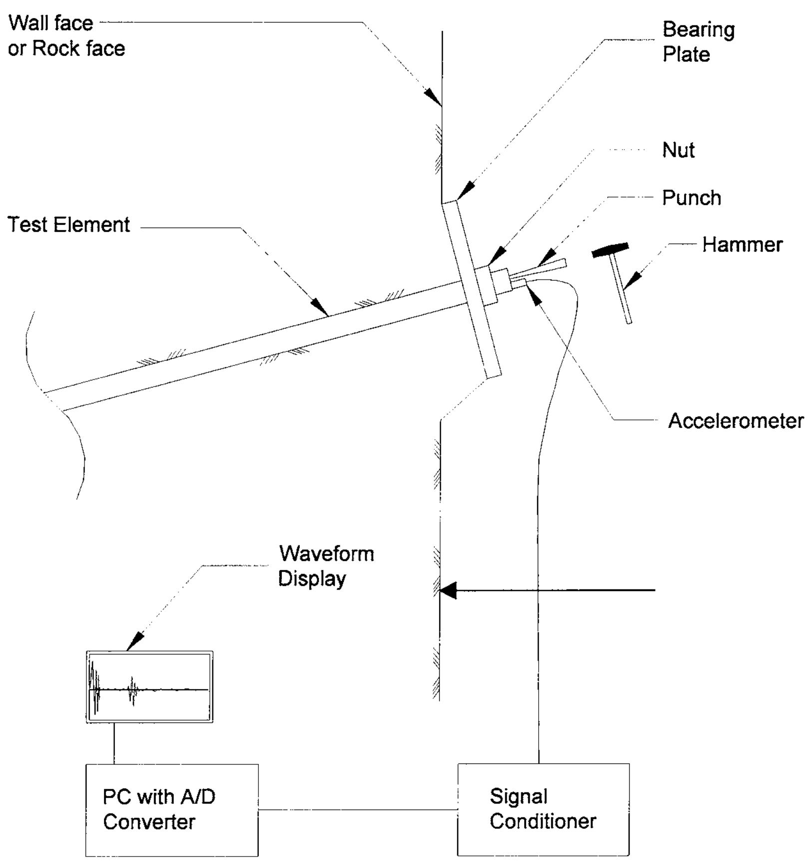


Figure 3-4. Schematic of impact echo test.

The medium-sized hammer is a modally tuned device (PCB Model 086C05) that has a frequency range of 1–5 kHz, a sensitivity of 1 mV/lbf, a resonant frequency of 28 kHz, a mass of 454 g, and head and tip diameters of 25 mm and 6.3 mm, respectively. The hammer is equipped with an impact force sensor and a built-in amplifier and is designed to reduce double hits during impulse testing. The small hammer is equipped with an impact force sensor (PCB Model 208A03). Catalog cuts with details of the instrumentation and hardware for the impact echo test, and calibration certificates for each accelerometer, are presented in *NCHRP Web Document 27*.

The impact test procedure involves (1) striking the element with the impact device to generate compression waves along the specimen and (2) detecting reflected waveforms with an accelerometer attached to one end of the element. Test results are processed in both time and frequency domains.

Further details and a recommended test method for impact echo testing of rock bolt, ground anchor, and soil nail installations are described in Appendix E.

3.2.2 Ultrasonic Test

The ultrasonic testing (UT) method is another technique for evaluating grout condition, fracture of elements, and abrupt changes in the element cross section. The method has many of the features of the impact echo technique, 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.

With the pulse echo method (a single-probe operation) shown in Figure 3-5, the times for sound pulses, generated at regular intervals, to pass through the specimen and return are measured. The transducer, which is acoustically coupled to the exposed end of the element, receives a shock excitation and generates a short ultrasonic pulse. The transducer receives echoes of the pulses after reflection. The return of the leading edge of the first echo can be easily detected by visual means from the time history of transducer output. Good acoustic

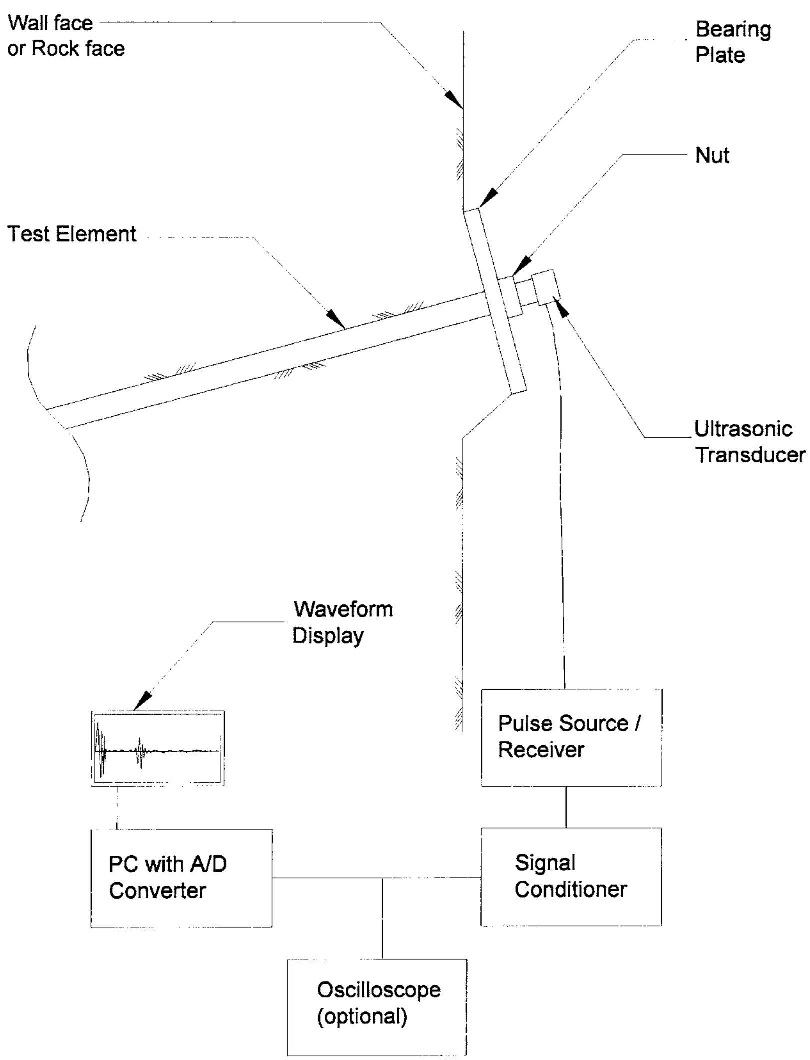


Figure 3-5. Schematic of ultrasonic test.

coupling between the transducer and the face of the element is a requirement for UT, and the face of each element must be flat and smooth. Care must be taken to ensure that the element faces are properly prepared for testing.

The equipment used included a Panametrics high-voltage pulser-receiver, Model 5058PR, and a V1011 piezoelectric transducer. The pulser-receiver unit can generate pulses with selected pulse repetition frequency rates of 20 Hz, 50 Hz, 100 Hz, 200 Hz, 500 Hz, 1,000 Hz, and 2,000 Hz, and damping and amplitude of the pulse can be varied. The signal may be attenuated (0–80 dB) or filtered by providing low-frequency cutoff points at 0.03 MHz, 0.1 MHz, 0.3 MHz, and 1 MHz or high-frequency cutoff points at 0.5 MHz, 1.0 MHz, 3 MHz, and 5 MHz. The equipment features a shock-excitation voltage of approximately 900 V, which was necessary to meet the high-energy requirements for generating acoustic waves in the bar system.

The Model V1011 is a low-frequency, broadband transducer that generates and receives compression waves. The V1011 sensor operates at a frequency of approximately 100 kHz. Although this frequency is relatively low for steel evaluation, it was necessitated by the high attenuation of the ultrasound in a metal specimen. The 38-mm diameter of the V1011, while a bit oversized compared with most element cross sections, was justified by the ability of the unit to generate sharper, less diverging beams for a given frequency; have better penetration properties; and handle high-pulse energy without saturation. Further details and specifications of the instrumentation and hardware used for the ultrasonic test evaluation are provided in *NCHRP Web Document 27*. Appendix F is a recommended test method for ultrasonic probe of rock bolt, ground anchor, and soil nail installations.

3.3 MONITORING OF NEW INSTALLATIONS

The electrochemical and mechanical tests described in Sections 3.1 and 3.2 apply to new installations. Special provisions may be included during installation to allow easy access to the anchor heads of selected elements. If necessary, selected elements may be electrically isolated to facilitate performance of electrochemical tests.

Nondestructive tests of existing elements are limited by the placement of a single instrument at or near the head of the element. New installations provide an opportunity to refine the detail of NDT measurements by placing additional instrumentation along element lengths prior to installation. For example, more powerful electrochemical measurement techniques are possible if reference electrodes are placed along the element. Additional instruments provide the ability to track compression wave propagation, thus enhancing results from impact testing. Several techniques for instrumenting the elements and the enhancements offered to the NDT techniques are available.

Improvements in electrochemical test results are possible if reference electrodes are placed closer to steel working elements. Better electrode placement reduces the contribution to electrical resistance from grout or soil electrolytes. A commercially available system called the Vetek V2000™ Corrosion Monitoring System offers a method for installing the reference electrode during the manufacture of encapsulated anchors. The V2000 monitoring system consists of a silver/silver-chloride wire that surrounds the metal element and that serves as a reference electrode. The reference electrode wire exits the system through the anchor head assembly for connection to electrochemical test equipment. Using this system and instrumentation available from Vetek, measurements of half-cell potential, polarization resistance, and electrochemical noise can be made. Thus, the metal-tensioned element can be monitored over time for the onset of corrosion, the intensity of the corrosion, the area involved in the corrosion, and whether or not pitting corrosion is present. The system is very simple and similar in many details to electrochemical measurements being evaluated for existing systems. Therefore, much of the hardware and data acquisition equipment needed for the tests is common to that described in Section 3.1.

For mechanical tests, such as the impact echo technique, sensitive strain gauges placed along bar elements may be used to track compression wave propagation. Multiple instruments allow easy identification of (1) reflections from different sources and (2) changes in compression wave velocity (i.e., material properties). Changes in compression wave velocity along the lengths of elements can be correlated with element distress, including changes in cross section, or voids in grout surrounding an element. In the future, data from multiple instruments could serve as a useful tool to calibrate analyses and improve test techniques applied to existing installations, which must rely on data from a sole transducer placed at the element head.

New installations offer the benefit of obtaining readings before installation, immediately after installation, and at subsequent intervals throughout the service life of the facility. Documentation should be obtained during installation, including the free and bonded lengths of the element, corrosion protection afforded to the system, anchor head protection, and splice locations. This information is useful for interpreting data obtained after installation. For some systems, where the test results are not significantly affected by the surrounding soil medium (e.g., encapsulated anchors), the effects of installation on the system integrity may be observed by comparing results obtained before and after installation. Readings taken immediately after installation are a useful reference for comparison with future readings.

3.4 DATA PROCESSING

The data acquisition (DAQ) equipment used for this project is manufactured by National Instruments and Compaq Computer Corporation. The system uses a multifunction data acquisition board (National Instruments Model DAQCard-1200) with eight differential analog inputs, each of which has a 12-bit precision. The board has the capability of performing analog or digital triggering and is equipped with an A/D converter having a maximum sampling rate of 100 kHz (100k samples per second). The DAQ board was incorporated into a Compaq Presario laptop computer, which has a 500-MHz processing speed, a 5.58-GB hard disk, and 64 megabytes of random access memory (RAM).

The VirtualBench software package was used for data acquisition and data processing. VirtualBench is a product of the National Instruments Corporation. VirtualBench acquires, displays, processes, and stores data. The software can emulate a digital oscilloscope (“Scope”), or can function as a dynamic signal analyzer (DSA) to perform dynamic signal processing in real time. The Scope program allows eight channels of input to be accessed with real-time selection of sampling rate, with record lengths ranging from 550 to 660,000 points, with time base ranging from 10 ns per division to 100 ms per division, and with a sensitivity range of 2 mV per division to 10 V per division. The program has real-time waveform analysis capabilities that permit the calculation of statistical parameters such as mean, root mean square, and peak-to-peak value. The DSA program can monitor two channels of input and permits data to be viewed, simultaneously, in time and frequency domains with the use of various built-in windowing functions.

Most of the data for this project were acquired using the Scope program, and data were processed manually after the test. Data include scatter, noise, and oversaturation at particular frequencies. Important signal characteristics—including frequency content, signal attenuation, damping, and arrival times of reflected waves—are enhanced by signal processing, which facilitates data interpretation. Raw data are processed with a moving average to reduce noise and scatter from the real-time signal. Weaker signals appearing at longer time intervals are enhanced by application of a scaling function. An autocorrelation function is used to help identify periods corresponding to arrivals of reflected waves.

The time history is transformed to the frequency domain by means of the Fast Fourier Transform (FFT), and then the amplitude spectra are computed from the Fourier coefficients. Peak and predominant frequencies in the amplitude spectra are used to identify physical features of test elements, including length, geometry, and level of prestress.

The element response corresponding to a particular range of frequency is studied in the time domain using a band-pass filter. Band-pass filters are applied to the amplitude spectra and the corresponding window inverted back to the time domain.

Processed data are presented graphically and interpreted visually to determine characteristics of the waveforms, which can be correlated with element condition. The following sections describe salient details of the data-processing techniques used to analyze the data. Further details of the data-processing techniques are described by Santamarina and Fratta (1998). These techniques are used to process the data presented in Chapter 5. Chapter 5 presents comparisons that demonstrate how signals from elements with defects or loss of tension look different than signals from conforming or intact elements.

3.4.1 Moving Average

Application of a moving average tends to smooth data and reduce scatter. To obtain a three-period moving average for a set of data, three consecutive data points are averaged and the value is placed in the middle bin location of three data points. For example, if the values 3, 4, and 5 reside in bins 1, 2, and 3, the values are averaged and the average, 4, is placed in the second bin. The next average is taken of numbers in bins 4, 5, and 6 with the average placed in bin 5. This process is repeated similarly until the end of the data set. A five-period moving average would take five values at a time and average them. The higher the period, the greater the smoothing that occurs. Too much smoothing is counterproductive because information within the averaged interval may be lost. Figure 3-6 is an example of the raw data plot versus data processed with a three-period moving average.

The vertical axis in Figure 3-6 is called the “volt ratio.” The volt ratio is the voltage output at each time step divided by the maximum voltage reading obtained during the test. All data presented in this report that include measurements of voltage output from an accelerometer or ultrasonic transducer are normalized in this fashion for plotting.

Microsoft Excel™ may be used to apply a moving average to the raw data. After the raw data are plotted, a trend line is added. Plotting the raw data and the moving average on the same plot portrays the relative amount of smoothing that occurs.

3.4.2 Scaling in the Time Domain

The time histories from the ultrasonic test begin with the trace of the transducer excitation, called the “bang,” followed by wave arrivals corresponding to reflections from features encountered along the element length. Because of the high attenuation of sound waves traveling through steel, wave reflections from distant features are often overshadowed by the bang. A scaling function can be used to enhance the signal such that reflected signals appear more distinct. Equation 3-1 is an exponential function multiplied by a constant that is well suited to this purpose.

$$A'(t) = a(A(t))e^{(bt)} \tag{3-1}$$

where

$A'(t)$ = modified amplitude of the signal at time t (s),
$A(t)$ = recorded amplitude of the signal at time t (s), and
a and b = constants.

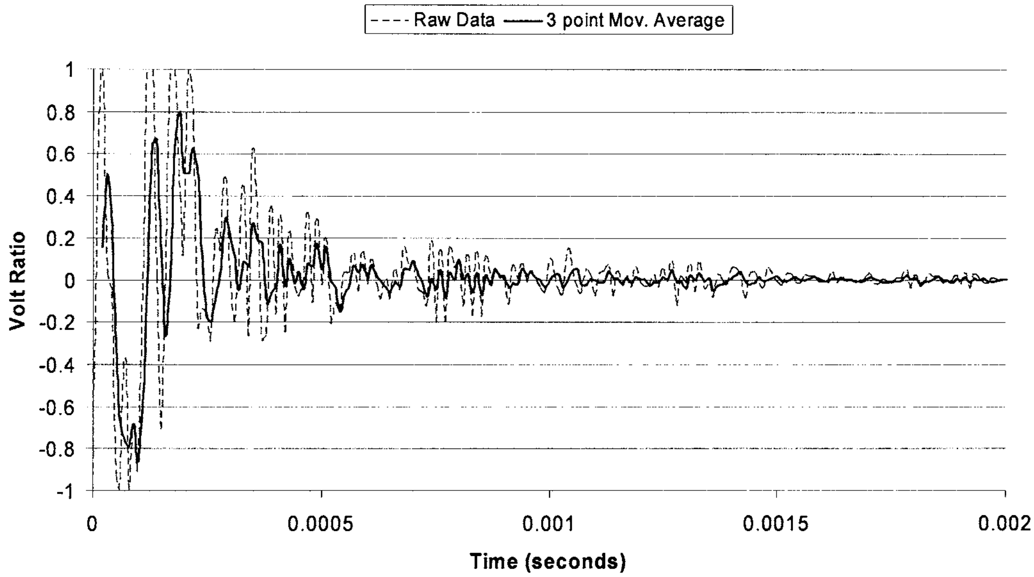


Figure 3-6. Ultrasonic test data and corresponding moving average plots.

The a parameter serves to attenuate the amplitude of the bang, and b amplifies the reflections from distant features. If b is too high, small errors at the end of the signal will appear to dominate the record. If b is decreased to reduce this tendency, then the value of a should be reduced so that the "bang" does not overshadow the results. The best values of a and b for use in Equation 3-1 may be found by trial and error.

Figures 3-7 and 3-8 compare the original record from an ultrasonic test to the same signal enhanced with the scaling function. Figure 3.7 shows a reflection received at approximately 1.25 ms that is enhanced by application of Equation 3-1 in Figure 3-8 ($a = 0.1$ and $b = 3{,}000$).

3.4.3 Autocorrelation in the Time Domain

Signals are embedded in output data that relate to information specific to each element. The information corresponds to lengths and other features of the element geometry. Because the signal is correlated to itself, the signal can be amplified where prominent waveforms exist and the return period of wave reflections can be identified.

Table 3-1 is a spreadsheet from Santamarina and Fratta (1998) describing the algorithm for autocorrelation. Each column in the central blocks shows the shifted signal (x_{i+k}) for increasing values of k; each k value corresponds to a shift in

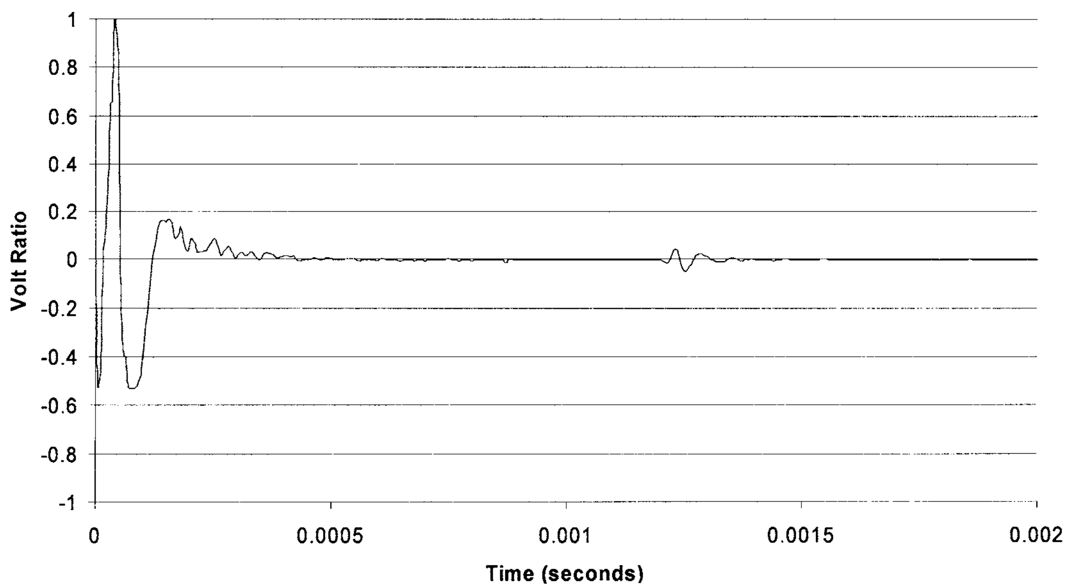


Figure 3-7. Example raw data record from ultrasonic test.

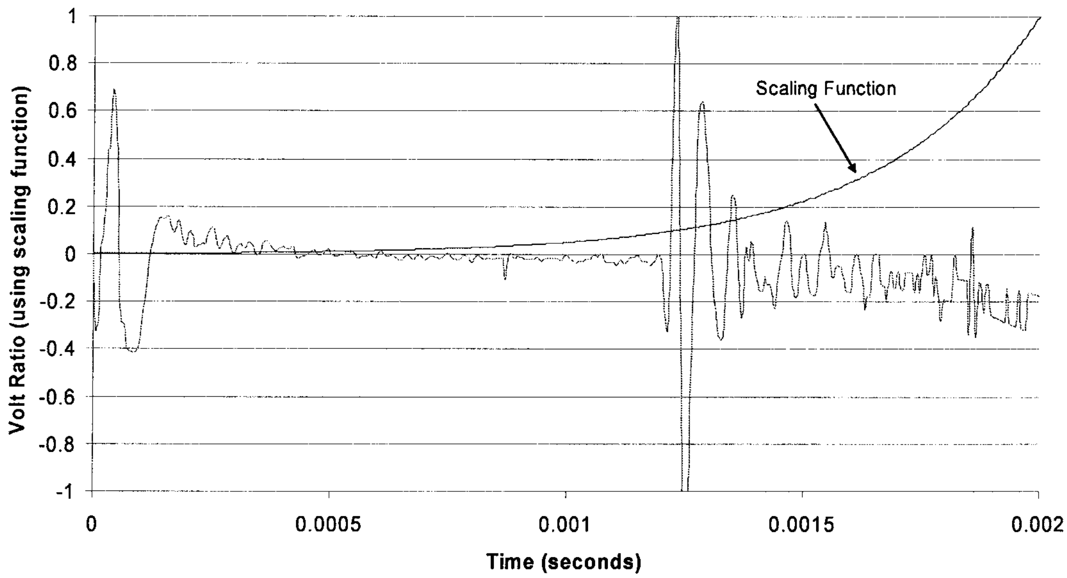


Figure 3-8. Example scaled data record from ultrasonic test.

time. The unshifted signal (x_i) is multiplied by the shifted signal in each column. The sum of each column is equal to the autocorrelation of x for each shift k: the first column corresponds to zero shift, $k = 0$, the second column for a shift of one time interval, $k = 1$, etc. The autocorrelations are plotted versus k, and points where the autocorrelation is strongest appear as peaks in the autocorrelation function. The k values correspond to reflected wave travel times.

3.4.4 Amplitude Spectra

Data were recorded in real time, rendering the output signal in the time domain. To observe the amplitude spectrum and corresponding peak and predominant frequency contents, the signal is transformed into the frequency domain. This is achieved by application of the Fast Fourier Transform (FFT). Howell (2001) provides an in-depth explanation of the FFT.

Microsoft's Excel data analysis package was used to perform FFT. Raw data in plain text were imported into an Excel workbook. To properly perform FFT, the total number of samples in the set needs to be a power of two (i.e., 2; 4; 6; 8; . . . 1,024; 2,048; 4,096). Excel can perform FFT for a maximum of 4,096 data points. For data files with more than 4,096 samples, the signal outputs are truncated. This truncating has no adverse effect because all the signal periods have fewer than 4,096 bins. For data sets with fewer than 4,096 samples, the ends of the data sets are padded to provide a total sample number to the nearest power of two (e.g., if the number of data points is 2,000, the end of the data set is padded with 48 zeros). Figure 3-9 is a typical amplitude spectrum from an impact test performed on a bar element. The "amplitude ratio" on the vertical axis of the spectrum shown in Figure 3-9 is a normalized scale similar to the "volt ratio" used in the time domain as described in Section 3.4.1. Each amplitude in the frequency domain is divided by the maximum amplitude computed for the spectrum. All of the amplitude spectra

TABLE 3-1 Sample spreadsheet showing autocorrelation process

k / i	k = 0	k = 1	k = 2	k = 3	k = 4
i	$x_i \cdot x_i$	$x_i \cdot x_{i+1}$	$x_i \cdot x_{i+2}$	$x_i \cdot x_{i+3}$	$x_i \cdot x_{i+4}$
0	$x_0 \cdot x_0$	$x_0 \cdot x_1$	$x_0 \cdot x_2$	$x_0 \cdot x_3$	$x_0 \cdot x_4$
1	$x_1 \cdot x_1$	$x_1 \cdot x_2$	$x_1 \cdot x_3$	$x_1 \cdot x_4$	$x_1 \cdot x_5$
2	$x_2 \cdot x_2$	$x_2 \cdot x_3$	$x_2 \cdot x_4$	$x_2 \cdot x_5$	$x_2 \cdot x_6$
3	$x_3 \cdot x_3$	$x_3 \cdot x_4$	$x_3 \cdot x_5$	$x_3 \cdot x_6$	$x_3 \cdot x_7$
4	$x_4 \cdot x_4$	$x_4 \cdot x_5$	$x_4 \cdot x_6$	$x_4 \cdot x_7$	$x_4 \cdot x_8$
5	$x_5 \cdot x_5$	$x_5 \cdot x_6$	$x_5 \cdot x_7$	$x_5 \cdot x_8$	$x_5 \cdot x_9$
6	$x_6 \cdot x_6$	$x_6 \cdot x_7$	$x_6 \cdot x_8$	$x_6 \cdot x_9$	$x_6 \cdot x_{10}$
7	$x_7 \cdot x_7$	$x_7 \cdot x_8$	$x_7 \cdot x_9$	$x_7 \cdot x_{10}$	$x_7 \cdot x_{11}$
Σ	$\Downarrow$	$\Downarrow$	$\Downarrow$	$\Downarrow$	$\Downarrow$
cc_k	$\sum_i x_i \cdot x_i$	$\sum_i x_i \cdot x_{i+1}$	$\sum_i x_i \cdot x_{i+2}$	$\sum_i x_i \cdot x_{i+3}$	$\sum_i x_i \cdot x_{i+4}$

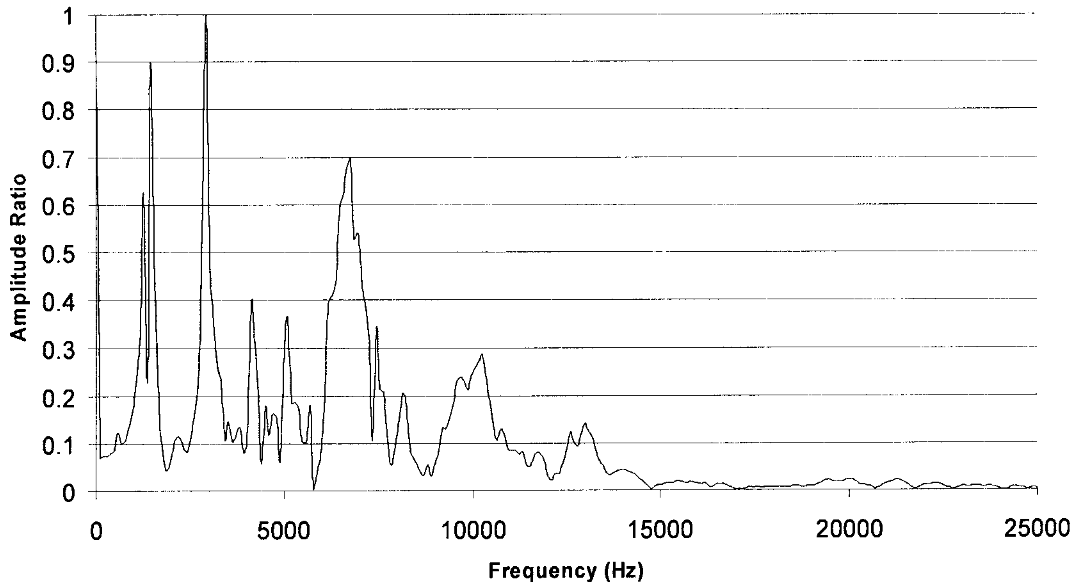


Figure 3-9. Typical amplitude spectrum for impact test.

included in this report have an amplitude scale normalized in this fashion.

The observed dynamic response can be compared with the response predicted with equations based on one-dimensional wave propagation along a slender bar. These equations can be used to relate element length, compression wave velocity, and fundamental frequency.

For a slender bar that is fixed at each end (e.g., bolted at the anchor head with a grouted anchorage at the other end),

$$f_n = \frac{mV_c}{2L} \quad \text{(3-2a)}$$

where

f_n = natural frequency of vibration (Hz),
m = an integer V_c = compression wave velocity ≈ 5,500 m/s for steel, and
L = length (m).

For a slender bar that is free at one end and fixed at the other (e.g., the dynamic response of a tendon extension beyond the anchor head assembly),

$$f_n = \frac{mV_c}{4L} \quad \text{(3-2b)}$$

In Chapter 5, Equations 3-2(a) and 3-2(b) are used to relate frequency content to the location of reflection sources along the lengths of test elements. This use assumes that wave propagation is one-dimensional and does not account for the effects of dispersion (i.e., the velocity of wave propagation is assumed to be independent of frequency).

3.4.5 Band-Pass Filters and Windowing in the Time Domain

Filtering is a method of processing data where specific frequencies inherent to the signal are identified and isolated. The amplitude responses of waveforms having specific frequency contents are studied in the time domain. Using this technique, vibration characteristics, otherwise obscured by dominant frequencies in the signal, are made more prominent. A number of specialized data analysis software packages are available that apply different types of data filters. Excel was used on this project; however, this use required many steps in the analysis to be performed manually.

Peak frequencies are identified from the amplitude spectrum of the signal. Frequency bands are isolated and centered with respect to a particular peak frequency.

Example: Consider an amplitude spectrum that exhibits three peak frequencies. The spectrum ranges from 0 Hz to 50,000 Hz, and peak frequencies occur at 5,000 Hz, 10,000 Hz, and 15,000 Hz. Frequency bands are identified surrounding each peak. If minimum amplitudes occur between the peaks at 0 Hz, 7,500 Hz, 12,500 Hz, and 17,500 Hz, this would describe frequency bands in ranges 0–7,500 Hz, 7,500–12,500 Hz, and 12,500–17,500 Hz.

Once the frequency bands are established, an inverse FFT is performed on each frequency band to transform the data back into the time domain. This results in signal time histories corresponding to particular frequency contents. Different frequency bands are identified and graphed separately in the time domain.

To perform the inverse FFT, data need to be prepared in the same manner explained earlier in the section on amplitude spectra (Section 3.4.4). An inverse FFT selected from the

Excel data analysis software is performed on a specific frequency band. The null bins, before and after the frequency band, are padded with zeroes such that the inverse FFT is performed on the same number of samples, and the frequency band has the same bin position, as the original transformation. Figure 3-10 is an example of the total time history from an impact test performed on a bar element. Figure 3-11 is the filtered time history corresponding to the same data. A reflection at approximately 2.5 ms is apparent in the filtered data, but is not as clear in the original time history.

The band-pass filter must be selected carefully because a narrow band-pass filter deforms the signal, creating phantoms of the signal before the true signal appears in time.

3.5 LIMITATIONS OF NDT TECHNIQUES

For the electrochemical tests, the following limitations apply:

- Stray currents can negatively affect results from electrochemical tests. Stray current can enter the element and make obtaining stable readings difficult. The presence of stray currents must be eliminated at a site for the tests to be performed successfully. Potential sources of stray current may include buried electric transmission lines, railway tracks for electric-powered trains, waterfront structures in salt water, or nearby welding shops.
- Results are not meaningful if elements are not electrically isolated. If elements are not electrically isolated, components that are electrically connected should be identified. After identifying the components, the tests may be performed, but the location of corrosion, or compromised corrosion protection, will not be identified, and components other than the element (e.g., steel soldier piles) may be the site of corrosion.
- If results from half-cell potential and polarization current test are being compared at different times, (1) the position of the half cell and ground bed and (2) the dielectric properties of the electrolyte must be consistent. Although placement of instruments may be controlled, the moisture content of the soil/rock mass may vary and, thereby, affect the results.
- A good electrolyte must be present to successfully perform the electrochemical tests. If the electrolyte has a very high resistance, it will be difficult to achieve a high enough current level to observe the transition in the E versus log I response.
- None of the NDT described in this research can detect the occurrence of hydrogen embrittlement. This type of corrosion is important with respect to the service life of high-strength steels, typical of those used for pretensioned strand-type elements.
- The effect of different grout types on the measurement of half-cell potential is unknown. Criteria were established from observations made on steel elements within neutral soils and water. These may not apply to the different chemical environments for elements surrounded by cement, epoxy resin, or polymer-type grouts. Therefore, the relationship between corrosion and measured half-cell potential is uncertain.

For the wave propagation techniques, including impact and ultrasonic tests, the following limitations apply:

- Loss of less than 25 percent of cross section is not easily detected with impact or ultrasonic tests. This estimate of the sensitivity of measurement is conservative. Many

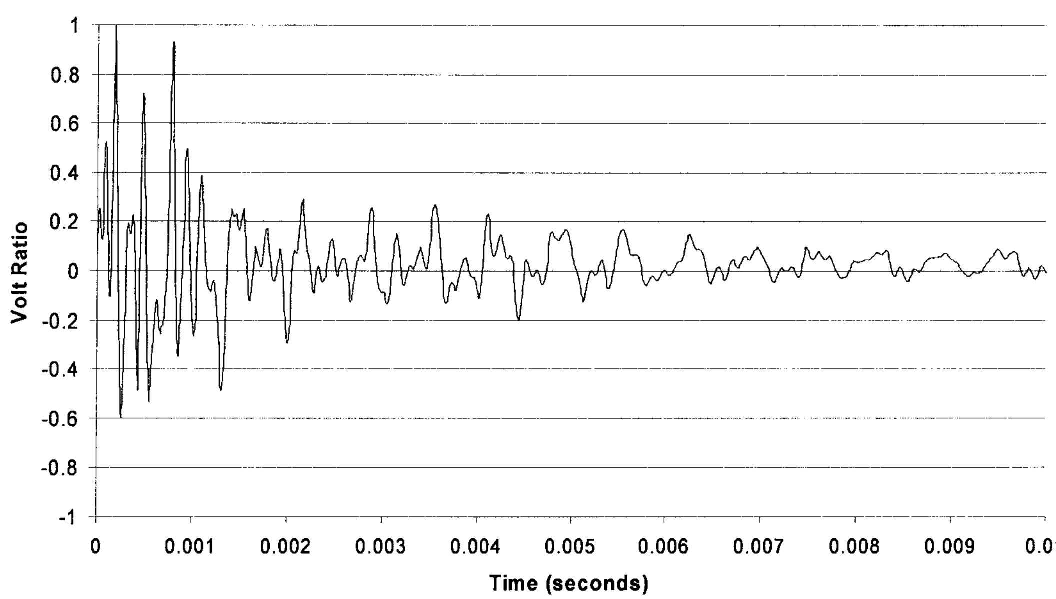


Figure 3-10. Typical time history for impact test.

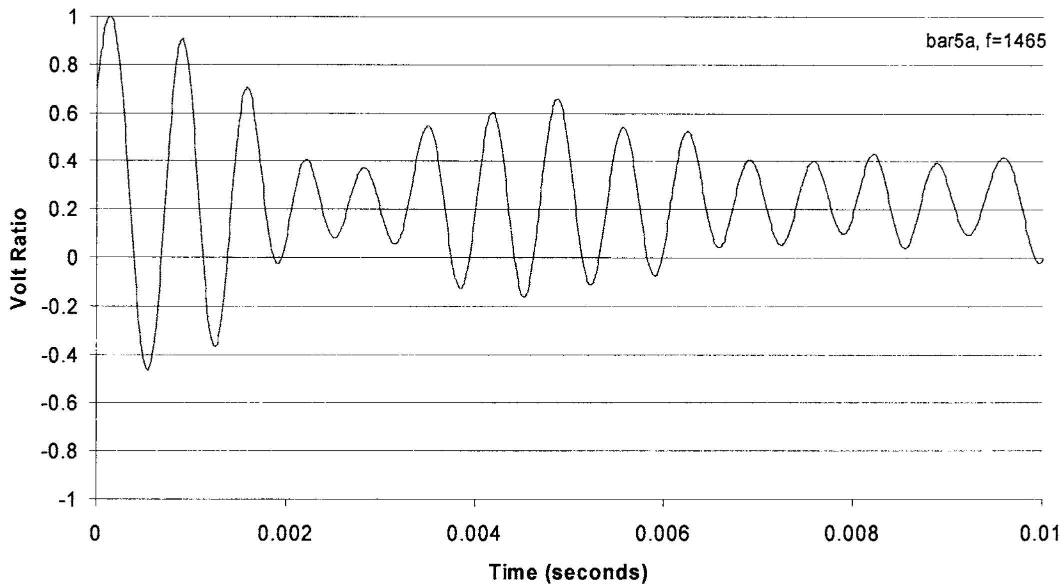


Figure 3-11. Impact test results in time domain for a band-pass filter centered about a fundamental frequency of 1,465 Hz.

specimens used in bench scale and in situ test evaluations of NDT included defects representing approximately 25 percent loss of cross section, and these defects were detected by NDT. Other researchers have reported that loss of less than 25 percent of cross section is difficult to observe with NDT. Improvements in NDT hardware, signal conditioning, and data interpretation may allow increased sensitivity of NDT measurements in the future.

- Element diameter must be at least approximately 25 mm for an impact test. The diameter at the base of the accelerometer is approximately 12.5 mm. The element needs to be approximately 25 mm in diameter to allow the accelerometer to be mounted near the perimeter, leaving room for the impact to be applied near the center of the element. This limitation may be overcome through improving the impact method.
- The nature of defects cannot be identified from the results of NDT. The data are interpreted to determine the arrival times of the reflections. Features may be located along the length of the element, but the reflection from a coupling is not discernable from a break in the element. Therefore, information about the element installation is required to interpret the data for locations of distress. Also, type of distress cannot be inferred from the data (e.g., the difference between a reflection caused by severe grout cracking is not recognized from a reflection caused by loss of element cross section).
- The length of an element that can be probed is limited. Impact test results show that it is difficult to probe element lengths beyond approximately 10 m. The limit depends on details of the installation, including the presence of grout surrounding the element.
- Data processing for an impact test requires special training. Some knowledge of wave mechanics and dynamics is needed.
- The current method for applying impact is not repeatable because impact is applied with a hand-held hammer, and two different operators may apply different levels of impact. Therefore, it is possible that, for the same element, a reflection from a distant source appears in one test record, but not in another.

CHAPTER **4**

RECOMMENDED PRACTICE

The following sections present details and background information in support of the recommended practice included in Appendix A. The recommended practice describes procedures, parameters, test methods, and criteria necessary for collecting and analyzing performance data.

The first step in the recommended practice is to establish an inventory of buried metal-tensioned systems. The inventory should include data for soil nail, tieback retaining wall, and rock bolt sites. The inventory provides information necessary for screening sites and establishing priorities for evaluation and condition assessment and serves as a repository to archive results from NDT performed on elements at a site.

Sites are screened to establish priorities for detailed evaluations. Information required for screening includes (1) geological conditions, including soil and rock properties at a site, and (2) details of the metal-tensioned systems. The screener first considers relevant site conditions and then evaluates the hazard with respect to corrosion and other factors that may affect the condition of metal-tensioned elements. A decision tree, which includes a corrosion assessment model, is presented. The decision tree allows the user to begin from a description of the character of a site and arrive at an opinion with respect to the degree of hazard presented by a site. The degree of hazard is subjectively described as high, moderate, or low.

Next, the screener reviews details of the metal-tensioned system. A second decision tree is presented that allows the user to begin from knowledge of the type of element, anchorage details, corrosion protection system employed, and date of installation, and arrive at an opinion regarding the vulnerability of the installation. Similar to site hazard, element vulnerability is subjectively described as high, moderate, or low.

The final part of the screening process is to combine the level of hazard and vulnerability at a site into an index describing risk of damage or distress to elements installed at a site. For instance, risk at a site with high hazard and low vulnerability may be relatively low compared with a site where the hazard is moderate but the elements are highly vulnerable. Depending on the risk assessment for a site, a recommendation is made to either (1) evaluate and perform a condition assessment of elements at the site or (2) do nothing except to rescreen the site at a later date. Risk assessment and screening processes are management tools that help ensure that scarce resources are allocated toward the greatest need.

A third decision tree is presented to describe the process when evaluation and condition assessment are recommended. The user begins by determining the number and location of elements to be tested and then performs several nondestructive tests to monitor the condition of the elements. Data from the NDT are analyzed and interpreted to determine whether corrosion is occurring and to locate any anomalies or signs of distress along the length of the element. The remaining service life is evaluated on the basis of the observed condition and of results from service-life prediction models. The user then makes recommendations that may include continued monitoring at selected intervals, more intensive monitoring at frequent intervals, invasive testing, or retrofit (such as replacement of anchors).

The following sections describe the recommended practice for selecting the levels of ground hazard, element vulnerability, risk assessment and screening, and condition assessment and evaluation of service life at a site.

4.1 ESTABLISHING INVENTORY

Installation details and descriptions of subsurface conditions should be included in the inventory. Installation details—including the type of element, anchorage details, date of installation, steel type, and level of corrosion protection afforded to the system—can be collected from agency construction records. If agency construction records are not available, information can be found on appropriately dated standard installation details distributed by suppliers.

For many sites, subsurface information related to the design and construction of the installation has been archived. Typically, subsurface explorations and soil data maintained by different agencies do not include parameters needed for assessment of corrosion. Therefore, it may be necessary to collect subsurface information in two phases. During the first phase, readily available information is collected, including (1) the elevation, fluctuation, and chemistry of the groundwater; (2) soil or rock type; and (3) the presence of artificial fills and nearby structures. Possible sources of this subsurface information include local and national geological surveys, the U.S. Department of Agriculture, and state departments of transportation. If information is not available on soil resistivity, pH, and sulfate and chloride content, such information should

TABLE 4-1 Soil testing

Chemical Tests	Physical Tests
• Resistivity (AASHTO T 288)	• Moisture Content (AASHTO T 265)
• pH (AASHTO T 289)	• Grain Size Analysis (AASHTO T 88)
• Sulfate Content (AASHTO T 290)	• Atterberg Limits (AASHTO T 89 & T 90)
• Chloride Content (AASHTO T 91)	

be obtained in the second phase, which includes sampling and testing for these parameters as described in the recommended practice (Appendix A). Sites included in the Phase II assessment are those that include clayey or layered sand/clay soils, artificial fills, or aggressive groundwater conditions or sites where only a portion of the element is above the groundwater level.

Soil, rock, and groundwater samples that represent materials surrounding a metal-tensioned element should be taken. Several different types of soil, rock, or both may need to be sampled if conditions vary along the length of the element. Soil samples should be tested for physical and chemical properties, as described in Table 4-1. Care should be taken during sampling to avoid contaminating the soil being sampled, mixing soil types, and losing moisture during storage and sample transport to the laboratory. A relatively large sample, approximately 1,500 g of soil sample finer than 2.00 mm (passing the U.S. #10 sieve) is needed because of the requirements of the soil resistivity test.

If possible, rock outcrops representative of rock bolt or ground anchor installations should be located and the rock type identified by visual inspection. If no outcrops are available, rock samples should be obtained by diamond core drilling techniques, as described in ASTM D2113 (ASTM, 2001). Rock joints should be observed, and those with infill materials that daylight at the outcrop should be sampled. Samples of joint infilling should be subjected to the soil tests described in Table 4-1.

Recommended test methods for analysis of water samples are described by AASHTO T263 (AASHTO, 2000). Table 4-2 lists five tests for qualitatively assessing the potential aggressiveness of groundwater.

4.2 RISK ASSESSMENT AND ASSIGNMENT OF PRIORITIES

Information from the inventory is used to screen sites according to the risk that remaining service life is affected by corrosion or loss of anchorage. The risk that metal-tensioned elements will fail to perform their function is evaluated in terms of (1) the hazard inherent to a site and (2) vulnerabilities related to the element installation details. A corrosion assessment model, based on past performance of buried metals (including ground anchors), is recommended for recognizing conditions corresponding to the occurrence of corrosion. The intent is to distinguish among different levels of ground hazard and element vulnerability. Ground hazard is the presence of conditions that make the occurrence of corrosion possible or increase the likelihood that corrosion may occur for unprotected, buried metal elements. Element vulnerability addresses the strengths or weaknesses of the installed metal-tensioned system in terms of its ability to resist attack from corrosion (e.g., the level of corrosion protection afforded to the system). Risk of loss of element service life from corrosion is the combined consideration of ground hazard and element vulnerability.

There is also a risk that elements will lose anchorage capacity from sources of distress other than corrosion, such as creep behavior of the soil in the anchorage zone, and from anchorage details that are not effective over the anticipated service life of the element.

4.2.1 Ground Hazard

Figure 4-1 is a decision tree to assess the ground hazard at a site. The decision tree describes a model to assess the

TABLE 4-2 Parameter limits for aggressive groundwater conditions

Test	Aggressiveness		
	Weak	Strong	Very Strong
pH	6.5 – 5.5	5.5 – 4.5	< 4.5
Lime-dissolving CO_2, mg/ℓ	15 – 30	30 – 60	> 60
Ammonium (NH_4^+), mg/ℓ	15 – 30	30 – 60	> 60
Magnesium (Mg^{2+}), mg/ℓ	100 – 300	300 – 1500	> 1500
Sulfate (SO_4^{2-}), mg/ℓ	200 – 600	600 – 3000	> 3000

Note: table modified after Xanthakos, P. P., 1991, *Ground Anchors and Anchored Structures*, John Wiley and Sons, Inc., New York, New York.

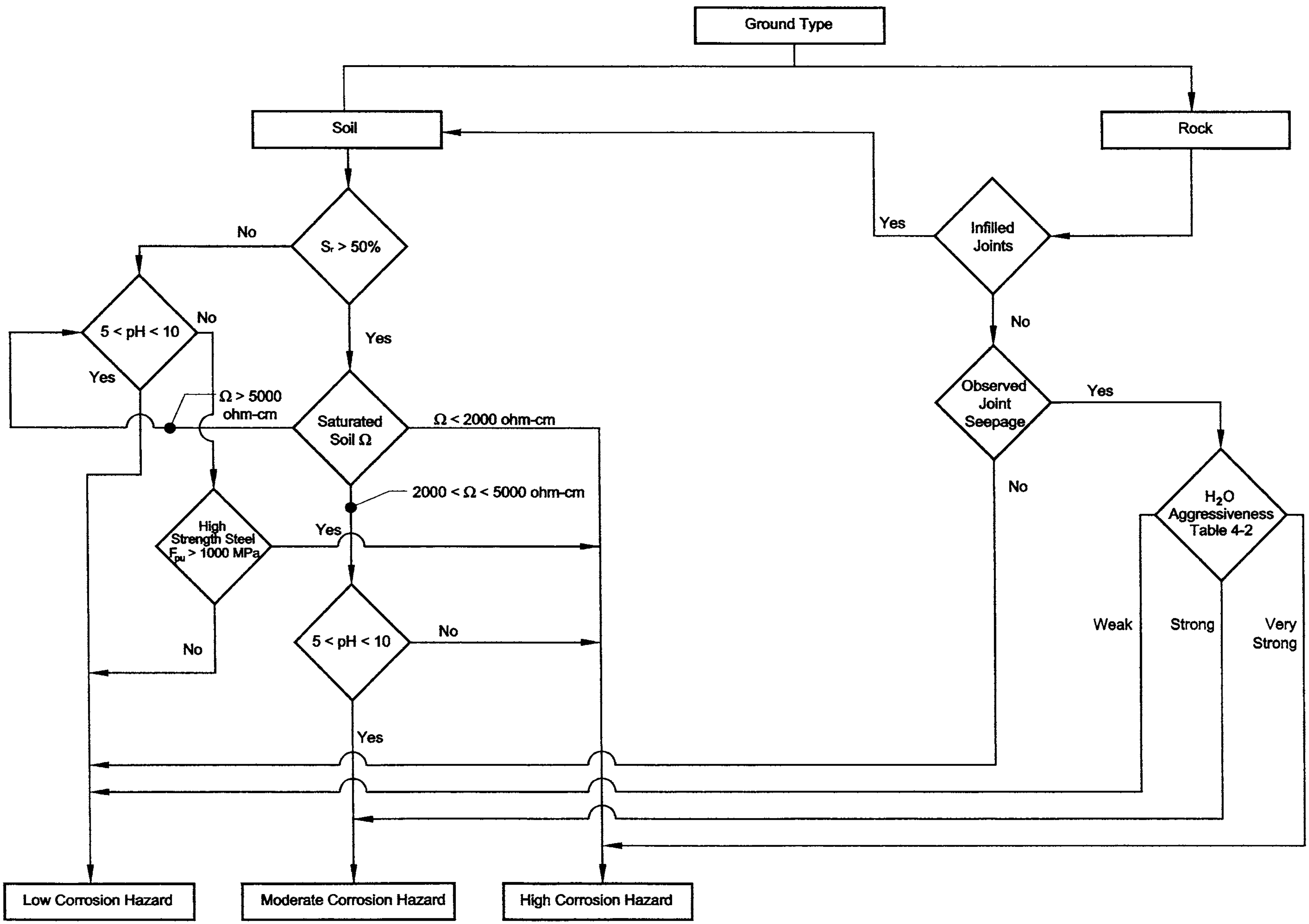


Figure 4-1. Decision tree for assessing ground hazard.

potential for corrosion. Parameters required by the corrosion assessment model include soil resistivity, pH, and sulfate and chloride ion content. Sites in the inventory that do not include corrosion assessment model parameters, and that do not require a Phase II subsurface exploration as described in Section 4.1, are categorized as low hazard relative to corrosion.

As shown in Figure 4-1, the ground hazard assessment begins with characterizing the site as either soil or rock. Some metal-tensioned elements may be entirely within soil or rock, while others may include a free length within soil and a bonded zone within rock. In the latter case, the most vulnerable part of the element is that within soil, and, therefore, the soil branch of the decision tree should be followed.

Although soil resistivity and pH are considered the most influential parameters affecting the aggressiveness of soil, the decision tree recognizes that AASHTO T288 (AASHTO, 2000) for measurement of soil resistivity is a laboratory test where moisture content is varied and the minimum resistivity of the soil samples is reported. In general, resistivity decreases as the water content and degree of saturation, S_r, increases. The corrosion of mild steel increases when the soil moisture content exceeds about 50 percent. Research data strongly suggest that maximum corrosion rates of mild steel occurs at $S_r \approx 60$–85 percent (Darbin et al., 1986). Therefore, the decision tree separates soils with $S_r < 50$ percent as being a low corrosion hazard for installation involving mild steel. For soils with $S_r > 50$ percent, the corrosion hazard is determined using measurements of soil resistivity and pH. Degree of saturation and the resistivity of the soil are environmental conditions that may change with the seasons. The evaluation should consider the degree of saturation at the worst time of year, and this may be correlated with known changes in groundwater conditions.

At low moisture contents, a pitting-type corrosion attack is more likely. For high-strength steels, pitting corrosion is more of a hazard, particularly in low-pH environments, and this fact is accounted for in the decision tree.

For rock materials, the hazard is determined on the basis of the characteristics of the infilling material that may be present in the joints, open bedding planes, or both and on the basis of the aggressiveness of the groundwater flowing along the open joint or bedding planes. If no infilling material or groundwater is present, the hazard relative to corrosion is low. If the infilled joint is a conduit for groundwater, the possibility exists for macrocell corrosion. For this case, the hazard condition for infilling is assessed, as for soils.

For potential problems posed by groundwater, Table 4-2 provides parameter limits from five tests for qualitatively assessing the potential aggressiveness of groundwater. The limits assume that the groundwater is stagnant or flowing very slowly and that the attack is immediate and unaffected by the presence of grout around the metal. To use Table 4-2, assign the highest level of aggression from the results of any single test or, if the results of any two tests are in the upper quarter of any level, assign the next higher aggression level.

The possibility of creep and poor drainage should also be considered as potential ground hazards. At existing sites, evidence of creep may be observed in soils from scarps along the ground surface, bulging at a wall face, or heaving at the base of a wall or stabilized slope. Although some softer rock deposits may exhibit evidence of creep movement, creep may be difficult to recognize from a visual inspection of harder rocks. For rock slope sites, therefore, the user will need to rely on historical records of incidents of creep failures to determine whether this hazard exists at a given site.

Drainage problems may be identified by observing seeps along a slope or wall face or at metal-tensioned element locations. Excess pore water pressures associated with poor drainage may contribute to loads not considered in the original design, and groundwater flow paths may contribute to the possibility of localized corrosion. Climate is a key factor, and the amount of precipitation and cycles of (1) wetting and drying and (2) freezing and thawing may affect element vulnerability.

The presence of either creep or poor drainage may mean that elements are subjected to load levels not considered in the original design. Creep may also cause a loss of resistance along the bond length, decreasing element capacity and contributing to an overloaded condition. If poor drainage or creep is recognized as a problem at the site, a condition assessment should be recommended.

4.2.2 Element Vulnerability

Element vulnerability is assessed relative to corrosion processes and details of the anchorage (e.g., grouted or mechanical). Figure 4-2 is a decision tree to be followed in assessing element vulnerability at a particular site. The assessment of element vulnerability requires information regarding the type of metal-tensioned element, anchorage details, and date of installation. Description of element type includes consideration of whether the element is a bar or strand and what type of steel (i.e., mild or high-strength steel) was used to manufacture the element. Anchorages are described as grouted (i.e., using cement- or resin-based grouts) or mechanical (which may include wedge sets or expansion shell anchorages).

The decision tree divides the elements into older and newer installations. Older installations were installed before 1985, when resin grouting became more popular and mechanical anchorages were used less. Also, the types of sleeves used in corrosion protection systems are different for newer systems. After 1985, the use of wrapped tendons declined, and extruded plastic sheathes became more popular.

Newer installations are not considered vulnerable, unless the element consists of high-strength steel and is not protected from corrosive environments. Steel with an ultimate tensile stress (F_{PU}) greater than 1,000 MPa is more vulnerable to attack from hydrogen embrittlement and corrosion stress cracking. Generally, high-strength steel is used to manufac-

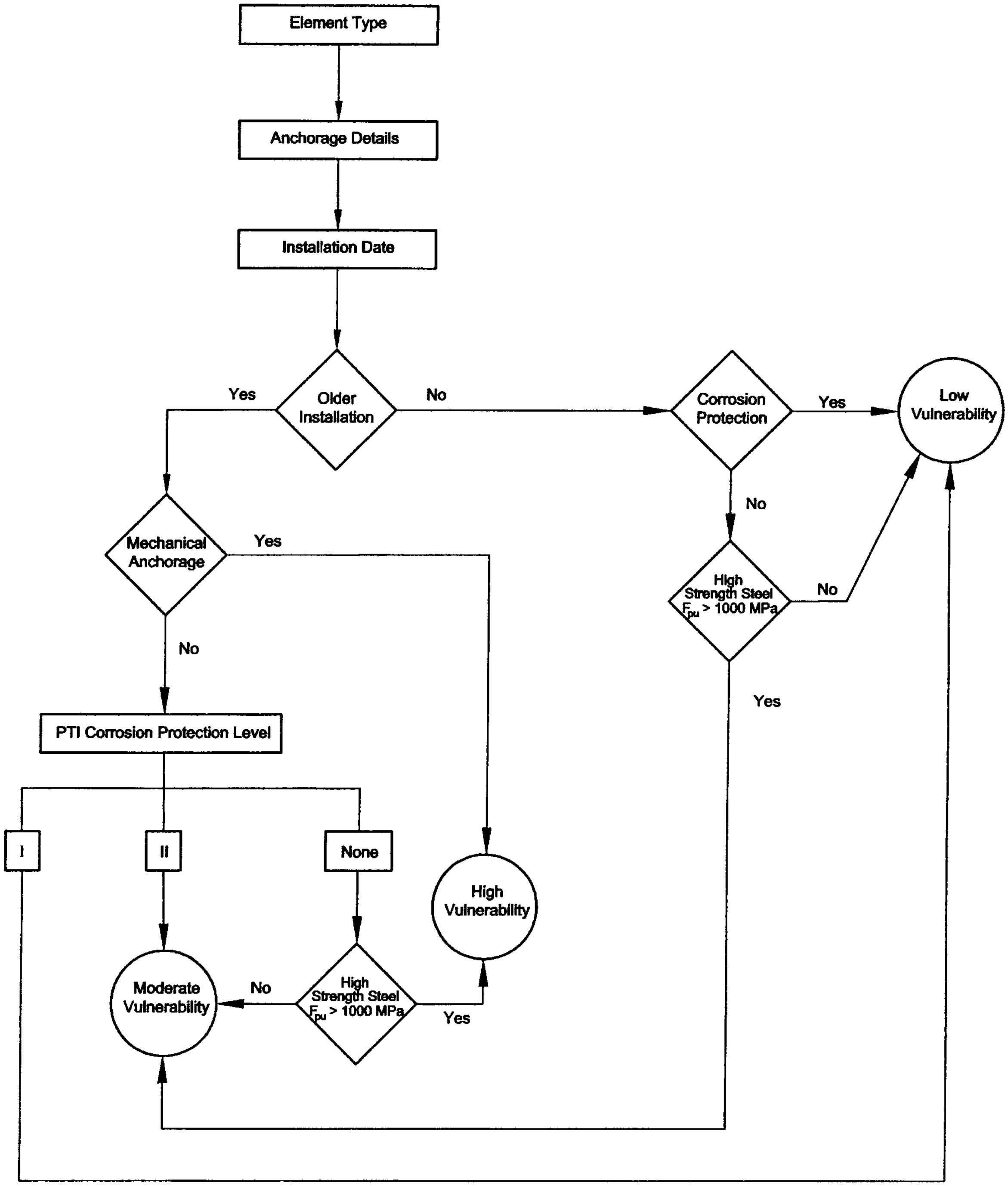


Figure 4-2. Decision tree for assessing vulnerability of elements to corrosion and loss of anchorage capacity.

ture wire strand–type elements ($F_{PU} \approx$ 1,700–1,900 MPa). Without adequate corrosion protection, strand-type elements are more vulnerable to corrosion than bar-type elements are because strand-type elements have more surface area than, but the same outside diameter and steel type as, bar-type elements.

The recommended practice assumes that reasonable care was used during construction, and so construction quality is not an issue. However, new installations should not be categorically considered in good condition if the inventory data suggest poor construction quality in the installation or suggest that the corrosion protection was compromised. Good construction quality includes quality control (to limit scratching and tearing of sheathing) and implementation of successful grouting practices (which preclude the existence of voids and ensure that the tendon is full of grout to its highest point without bleeding of the grout).

Installations that employ mechanical anchorages are considered vulnerable because older systems of this type were not fully grouted. Compared with grouted anchorages, mechanical anchorages are more dependent on unknown or uncertain rock mass characteristics.

For grouted anchorages, the vulnerability depends on the degree of corrosion protection (i.e., Class I, II, or none as defined by PTI [1996]), whether the element is mild or high-strength steel, and the level of quality during construction.

4.2.3 Assignment of Priority Rating

A rating system is recommended for establishing site priorities for condition assessment. The recommended practice is used to describe the site hazard and element vulnerability as high, medium, or low. Risk from corrosion attack is the combined consideration of hazard and vulnerability.

Table 4-3 is recommended for assigning priority ratings, relative to corrosion, for each site listed in the inventory. The agency may then proceed to perform condition assessments according to site priority as budget, time, and other resources permit.

A rating of 1, 2, or 3 is assigned to low, medium, or high element vulnerability; 0, 1, or 2 is assigned to site hazard conditions. Users compute the priority index by multiplying the vulnerability rating and the hazard rating. The resulting priority index ranges from 0 to 6.

Condition assessments should be recommended at sites with a priority index of 6, followed by those with lower priority indexes. Sites with an index of 0 are last on the list of sites recommended for condition assessments. With this approach, all sites with low site hazard relative to corrosion are given a low priority. This low priority reflects the fact that corrosion-induced failures of ground elements are mostly associated with sites having very aggressive ground conditions.

When an agency begins to perform condition assessments at sites with a priority index of 0, it may distinguish among sites with high, moderate, or low vulnerability and perform condition assessments at the most vulnerable sites first.

Sites where there are problems with creep or poor drainage should be assigned a priority index of 4 or 6.

TABLE 4-3 Screening assignment of priority index for condition assessment

(1) Vulnerability Level	(2) Hazard Level	(3) Vulnerability Rating	(4) Hazard Rating	(5) Priority Index (3) × (4)
L	L	1	0	0
M	L	2	0	0
H	L	3	0	0
L	M	1	1	1
L	H	1	2	2
M	M	2	1	2
H	M	3	1	3
M	H	2	2	4
H	H	3	2	6

4.3 PROTOCOL FOR SITE AND CONDITION ASSESSMENTS

The purpose of a condition assessment is to evaluate and monitor existing installations of metal-tensioned systems; apply NDT techniques in the field; and correlate results of the nondestructive tests with subsurface conditions, details of the installation, and expectations that are based on service-life prediction models. If expertise is not available in-house, agencies may need to seek outside professional advice. Measurement of half-cell potential and linear polarization resistance is becoming a routine practice for assessment of bridge decks. However, specialized equipment and techniques are employed to measure polarization current for buried metal-tensioned systems. The impact and UT techniques are less common, although an agency may have some experience with their application to condition assessment of concrete structures, unbonded prestressing strands and ducts, and cable stays.

The electrochemical tests described in Chapter 3, including measurement of the half-cell potential and E versus log I relationship, are useful in assessing the integrity of the corrosion protection system, the element surface area vulnerable to corrosion, and whether corrosion is occurring. Wave propagation techniques (i.e., impact and ultrasonic tests), as described in Chapter 3, are used to assess the existing condition of elements (i.e., severity of corrosion).

The NDT protocol is described in Table 4-4 as it applies to bar or strand elements. The protocol is similar for soil nails, rock bolts, or ground anchors. Soil nails are bar elements, and the faces of the elements are usually encased in grout, which must be removed for testing. Rock bolts are usually bar elements, but sometimes strand elements are used for transferring very high levels of prestress. Many times, the head of the element is exposed, but access may be challenging because of the terrain. Ground anchor tendons may be bar or strand elements, and anchor heads may be exposed, covered by a grease-filled cap, or encased in grout.

If access is easy and the element faces are exposed, many elements may be tested with electrochemical tests and fewer elements selected for impact and ultrasonic tests. Conversely, if access is difficult, electrochemical and mechanical tests should be performed on all the elements accessed and prepared for testing.

If relatively easy access to a grease-protected sheath in the free length of a strand-type element is available, an attempt should be made to obtain sterile grease samples. Grease samples will be tested for bacteria content, which may be correlated with biological activity. An attempt should also be made to obtain grease samples from the vicinity of areas where corrosion is identified. Biological activity in these types of systems may produce sulfides as a byproduct, contributing to the potential for hydrogen embrittlement of the steel strand. If a hydrogen sulfide gas odor is detected, it should be documented. Special kits are available for sampling and testing for biological activity; alternatively, a micro-

TABLE 4-4 NDT protocol

Bars	Strands
• Perform electrochemical tests	• Sample and test grease surrounding tendons
• Perform both impact and ultrasonic tests	• Perform electrochemical tests
• Correlate performance with service-life prediction model	• Perform ultrasonic test
• Lift-off or other destructive tests	• Correlate performance with service-life prediction model
	• Lift-off or other destructive tests

biologist may be consulted for advice on the best sampling strategies and test methods.

NDT results obtained from a number of different elements at a given site are compared with one another, with known installation details, and with signatures that represent the response of typical elements. Elements should first be screened with the electrochemical tests to determine locations where corrosion is likely to occur according to agency records or visual observations made at the site. The results from electrochemical tests can be supplemented with results from the impact response, ultrasonic tests, or both to identify and locate defects along the length of the element. The value of NDT is to screen and identify locations where more detailed invasive inspection may be recommended. Performance tests (i.e., load tests) are recommended at elements where signs of distress have been identified. Invasive observations, proof testing, or both are always preferred alternatives to NDT, and the use of NDT will sometimes be an inviable option.

4.3.1 Sampling Criteria

After a site has been screened and a decision has been made to monitor the existing condition of elements at the site, the user needs to determine the number and locations of elements to be tested. A sampling plan should be developed to determine how large to make the sample size and what to use as acceptance criteria according to the results of the testing program. In specifying an acceptance sampling plan, two risks must be balanced: (1) the risk of overestimating remaining service life and (2) the risk of specifying retrofit for elements that are actually in good condition. A statistical approach is recommended such that the risks associated with the sampling and acceptance criteria are considered with respect to the costs of retrofitting the elements and the potential loss from damage caused by failure of the elements.

The number of elements that should be tested (n) out of the total number (i.e., population) of available elements (N) must be established. The probabilistic approach described herein recognizes that risk is inherent to the decision-making process, although the acceptable risk can be quantified and controlled by the agency. An important risk factor to consider is the amount of randomly located, distressed elements with undesirable characteristics that the agency is willing to accept.

Depending on the results of the NDT, the element condition will be broadly designated as conforming or nonconforming. Conforming elements are elements for which NDT results correlate well with details of the installation and no significant distress can be identified. NDT results for nonconforming elements display an anomaly, which may be correlated with element distress. This designation recognizes that the results from NDT may indicate an anomaly reflected by an unexpected feature (i.e., nonconformity), not necessarily distress.

The decision on the number of elements to test (n) depends on the allowable percentage of nonconforming elements, the allowable sampling error, and the confidence that can be placed on the overall result. The agency should answer the following questions to define the input necessary to develop the sampling plan:

- *How many nonconforming elements, expressed as a percentage of the entire population, are expected?* The expectation is based on the age of the system, knowledge of past performance of similar systems, and the level of quality during construction. This percentage should be considered a threshold, beyond which some action should be taken by the agency. As an example, the agency might decide that if less than 2 percent of the elements are nonconforming, no changes will be made to the monitoring plan. If 5 percent are nonconforming, the agency will order more frequent monitoring. If more than 10 percent are nonconforming, aggressive action in the form of performance testing, or retrofit measures, may be implemented.
- *What is the allowable sampling error?* The sampling error is the difference between the percentage of nonconforming elements in the sample and the percentage of nonconforming elements in the entire population.
- *What confidence limit is acceptable?* Confidence limit defines the probability that the acceptable percentage of nonconforming elements will be exceeded.

A number of factors may affect the agency's decision with respect to adopting the acceptance criteria, allowable sampling error, and confidence limits. These factors include the following:

- *The importance of an individual element relative to the overall system performance.* For instance, a soil-nailed slope may have more redundancy than tiebacks for an anchored wall system. Therefore, an anchored wall system may require more detailed testing than a soil-nail system.
- *The system performance history given the physical and environmental conditions at the site.* If the history of performance for a given system is available, the user may anticipate potential problems. If problems are suspected, more detailed testing may be ordered.
- *The cost or restricted schedule*. (This factor is to be determined by the agency.)
- *The site access and test feasibility.* At some difficult sites, accessing an element for testing may be a major expense, making the cost differential between testing or retrofitting an element relatively small.

The following formulas are recommended to compute the sampling interval meeting the requirements for acceptance criteria, sampling error, and confidence limits. First, the sampling interval (n_o) required for a population of infinite size is computed, and then it is adjusted for a finite-size population (N).

$$n_0 = \frac{t^2 pq}{d^2} \tag{4-1a}$$

$$d = |p - P| \tag{4-1b}$$

$$a = \text{prob}(|p - P| \geq d) \tag{4-1c}$$

where

α = confidence limit;
prob = probability;
p = maximum acceptable percentage of nonconforming elements in a selected sample, on the basis of which a follow-up decision should be made;
q = minimum acceptable percentage of conforming elements and is equal to $1 - p$;
d = allowable sampling error;
P = percentage of nonconforming elements in the entire population; and
t = student t-distribution value, which is a function of n_o and α.

Obtain the number of tests, n, by adjusting n_o according to population size, N, as follows:

$$n = \frac{n_0}{1 + \frac{(n_0 - 1)}{N}} \tag{4-2}$$

Equation 4-1(a) demonstrates that a zero-tolerance acceptance criterion, which does not allow for any nonconforming elements, is unrealistic. Equation 4-1(a) can be rewritten as $t^2 = n_0 d^2/pq$, and if no nonconforming elements are allowed ($p = 0$ and $q = 1$), $t = \infty$, for which there is a zero probability of exceeding the acceptance criteria.

Equation 4-1 accounts for the expected percentage of nonconforming elements and the acceptance criteria, sampling error, and confidence limits adopted for the site. Sampling requirements increase as the acceptance criteria becomes broader (i.e., as p increases). That is, the greater the percentage of nonconforming elements anticipated, the more testing is necessary. Thus, at a site where conditions are expected to be poor, more testing should be required.

The number of tests, n, can be decreased as the required confidence interval is increased. Fewer tests are required if a lower precision can be tolerated. Increasing the confidence limit, α, from 0.05 to 0.10, reduces n_0 by approximately 40 percent.

If no sampling error is allowed, n_0 is infinity, and NDT must be performed on each element of the system (i.e., $n = N$). Doubling the sampling error, d, reduces n_0 by approximately 75 percent. Equation 4-2 describes the relationship between sample size and population size. For the range of practical interest, the sample size is always less than the population. Because n_0 is associated with an infinitely large population, as N increases, n approaches n_0. As the population becomes small, the value of n approaches N.

Application of this approach is illustrated by the following examples for a ground support system consisting of 50 elements:

- *Case 1:* Access is available to all elements, and the agency decides to optimize the number of samples to reduce the chances of overestimating or underestimating the percentage of success of the tested elements.
- *Case 2:* Access is available to all elements, and the agency decides to accept a higher risk in tested elements because of limited budget, restricted schedule, or both. The agency decides to reduce the number of tested elements and doubles the confidence interval (i.e., 2α), which means that the percentage of nonconforming elements in the sample can be overestimated or underestimated by α.
- *Case 3:* Access is limited to selected elements because of difficulty in reaching some elements (e.g., a rock slope where bolts are higher than the reach of available lift trucks) or because of the effort required to expose the ends of the elements for testing. For this case the agency could decide to conduct NDT, mechanical tests, or both on all or just some of the accessible elements or on a limited number of elements that are exposed by preparation. The agency thereby estimates the probability of success of the whole system by testing a sample of all the elements. For this case, the agency decides to accept higher risk and use a confidence interval equal to 2α.

To determine the required number of samples to test for Cases 1 and 2, assume the following:

- $p = 0.05$ (if 5 percent of the elements are nonconforming, monitoring will continue),
- $q = 0.95$,
- $d = 0.1$ (acceptable sampling error is 10 percent), and
- $\alpha = 0.05$.

Standard tables from texts on statistics and probability give values of t for different values of ν ($n_0 - 1$) and α (for example, see the table in Appendix B). Because t depends on n_0, Equation 4-1 must be solved by iteration. The standard normal distribution (Z-distribution) may be used to obtain n_0 for the first iteration. If n_0 is estimated to be more than 30, using a normal standard distribution will provide sufficient accuracy for most applications.

To use the Z-distribution to find n_0 for Case 1:

1. Adopt $t = Z\ (\alpha/2 = 0.025) = 1.96$
   Substituting:

$$n_0 = \frac{(1.96)^2 \times 0.05 \times 0.95}{(0.1)^2} = 18.25 \approx 18$$

2. $t\ (n_0 = 18, \alpha/2 = 0.025) = 2.110$
   Substituting:

$$n_0 = \frac{(2.11)^2 \times 0.05 \times 0.95}{(0.1)^2} = 21.15 \approx 21$$

3. $t\ (n_0 = 21, \alpha/2 = 0.025) = 2.086$
   Substituting:

$$n_0 = \frac{(2.086)^2 \times 0.05 \times 0.95}{(0.1)^2} = 20$$

4. $$n = \frac{21}{1 + \frac{(21-1)}{50}} = 15 \text{ elements}$$

Therefore, 15 elements should be tested to meet the agency-specified sampling error, confidence level, and acceptance criteria. For this example, note the difference in the computed value of n using the t-distribution and the normal distribution. For $n_0 = 18$ (i.e., initial estimate made using the normal distribution), $n = 14$, which is approximately 7 percent less than n using the t-distribution. Therefore, in some cases, even when n is less than 30, the normal distribution can provide a practical estimate of n. However, for better refinement of n, the t-distribution should be used.

To use the Z-distribution to find n_0 for Case 2:

1. Initially, adopt $t = Z\ (\alpha = 0.05) = 1.645$
   Substituting:

$$n_0 = \frac{(1.645)^2 \times 0.05 \times 0.95}{(0.1)^2} = 12.85 \approx 13$$

2. $t\ (n_0 = 13, \alpha = 0.05) = 1.782$
   Substituting:

$$n_0 = \frac{(1.782)^2 \times 0.05 \times 0.95}{(0.1)^2} = 15.08 \approx 15$$

3. $t\ (n_0 = 15, \alpha = 0.05) = 1.761$
   Substituting:

$$n_0 = \frac{(1.761)^2 \times 0.05 \times 0.95}{(0.1)^2} = 14.73 \approx 15$$

4. $$n = \frac{15}{1 + \frac{(15-1)}{50}} = 11.7 = 12 \text{ elements}$$

Therefore, 12 elements should be tested to meet the agency-specified sampling error, confidence level, and acceptance criteria. Note the difference in n for Cases 1 and 2. For Case 2, the agency decided to be more conservative by accepting the percentage of success to be underestimated (i.e., the number of nonconforming elements can be overestimated), and n for Case 2 was 20 percent less than n for Case 1, where the agency decided to reduce the chances of overestimating or underestimating the percentage of success in tested samples.

To determine the required probability of success for Case 3, assume the following:

- $n = 5$ (anchors that can be accessed),
- $p = 0.05$ (if 5 percent of the elements are nonconforming, monitoring will continue),
- $q = 0.95$, and
- $d = 0.1$ (acceptable sampling error is 10 percent).

Then take the following steps:

1. $$n = \frac{n_0}{1 + \frac{(n_0 - 1)}{50}} = 5 \text{ elements}$$

   $n_0 = 5.44 \approx 5$

2. $$n_0 = \frac{t^2 * 0.05 * 0.95}{(0.1)^2} = 5$$

   $t(\alpha = ?, n_o = 5) = 1.025$

3. $\alpha = 0.196$
4. Probability of success (i.e., percentage of conforming elements)

   $P_s = 100\% - 19.6\% = 80.4\%$

4.3.1.1 Potential Site Risk

The overall risk of failure of a structure supported by metal-tensioned elements is a combination of the probability of failure of an individual nonconforming element (or number of nonconforming elements) and the consequences of failure. Thus, while the probability of failure may be high according to NDT results, the potential site risk could still be low if the consequences of failure are small. This concept is described for soil nail, rock bolt, and anchored structures.

A distressed soil nail wall is expected to significantly deform before failure, which should give enough warning time to take action and fix a potential problem. A soil nail typically supports a relatively small area, and a loss of service from a relatively small percentage of randomly located soil nail elements is unlikely to cause catastrophic failure. Therefore, a site having a soil nail system may be characterized as having a low consequence from failure of individual elements. For an anchored wall system, an anchor is usually supporting a larger area than that of a soil nail and typically has relatively high internal stresses.

If an anchor system does not have redundancy, the loss in load-carrying capacity from distressed anchors may not be redistributed to surrounding in-service anchors. With loss of service from individual anchors (i.e., elements), excessive wall deformation or local failure may occur at the location of these elements, or a catastrophic failure of parts of the wall may occur. Because the consequences of having distressed anchors are relatively significant and usually costly, a site having an anchored wall system may be characterized as a site with high consequences from element failure.

A rock bolt usually supports part of a rock block and likely acts independently from other rock bolts. A distressed rock bolt may cause an isolated and localized rock instability or failure. The significance of potentially failed rock bolts can be determined by level of exposure of public safety. For instance, an interstate highway with a limited clearance from a rock bolt system may be characterized as a high potential risk site. However an interstate highway having an adequately designed catchment system below a rock bolt support system and a steel net along the face of the supported slope may be characterized as having low consequences from individual rock bolt failures.

4.3.1.2 Simplified Sampling Criterion

A simplified criterion is presented. It is based on the statistical analysis presented in the previous sections, but does not require the user to have a background in statistics. An engineer will estimate the percentage of nonconforming elements at a site (p) and decide the consequences of failure at the site. The following statistical parameters are recommended:

- $d = 0.1$;
- For low consequences of failure, $\alpha = 0.1$ and $t = 1.282$;
- For moderate consequences of failure, $\alpha = 0.05$ and $t = 1.645$; and
- For high consequences of failure, $\alpha = 0.01$ and $t = 2.326$.

Figures 4-3, 4-4, and 4-5 summarize the number of required samples (n_0) for low, moderate, and high consequences of failure, respectively, using $p = 0.01$, 0.05, and 0.1. Note that n_0 was adopted to equal the total number of elements (N) when N is less than or equal to 10. The recommended sampling criterion is simplified and summarized in Table 4-5. Note that ranges of n_0 are recommended for different levels of risk (defined in terms of probability of element failure and consequences of failure). For the given ranges, the lower value of n_0 corresponds to $N = 10$, and the upper value of n_0 corresponds

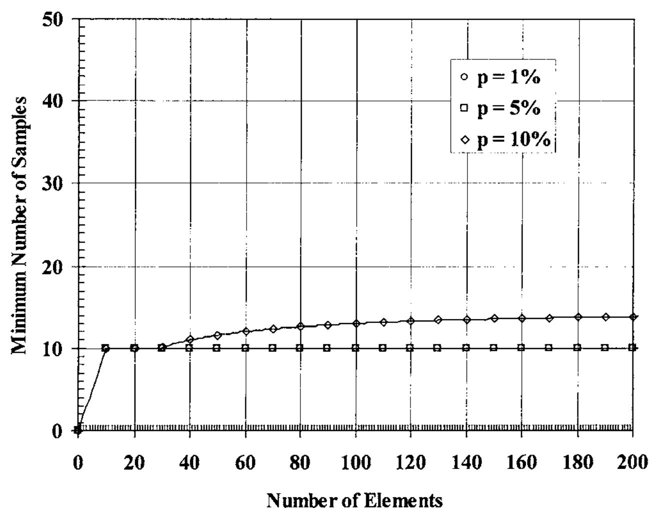


Figure 4-3. Sampling criteria for low consequences of failure.

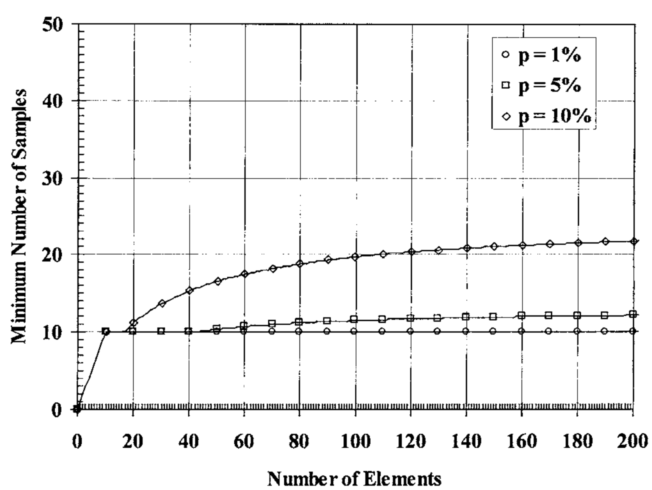


Figure 4-4. Sampling criteria for moderate consequences of failure.

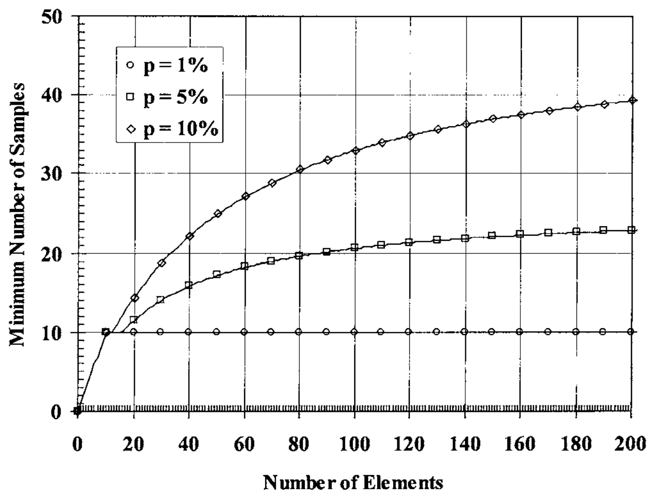


Figure 4-5. Sampling criteria for high consequences of failure.

to $N = 200$. For an intermediate N value between 10 and 200, n_0 can be approximately obtained using linear interpolation.

4.3.2 Condition Assessment

Figure 4-6 illustrates the process for condition assessment and service-life evaluation of buried metal-tensioned elements. The user begins by determining the number and location of elements to be tested and then performs several nondestructive tests to monitor the condition of the elements. Data from the NDT are analyzed and interpreted to determine whether corrosion is occurring and to locate any anomalies or signs of distress along the length of the element. The remaining service life is evaluated using the observed condition and the results from service-life prediction models. The user then makes recommendations that may include continued monitoring at selected intervals, more intensive monitoring at frequent intervals, invasive testing, or retrofit (such as replacement of anchors).

4.4 SERVICE-LIFE PREDICTION

After examining the test data for anomalies, results are compared with the estimated service life based on mathematical models. Currently available corrosion models do not directly account for any type of corrosion (i.e., localized or environmental cracking) other than uniform attack.

Romanoff (1957) proposed the following power law to predict rates of uniform corrosion of buried metal elements:

$$X = Kt^r \tag{4-3}$$

where

X = loss of element thickness or radius (μm),
K = constant (μm),
t = time (years), and
r = constant.

Equation 4-3 can be rearranged to compute the time for a given loss of element thickness as

$$\ln(t) = \frac{\ln(X) - \ln(K)}{r} \tag{4-4}$$

For a round bar, loss of radius corresponds to symmetric loss of element thickness. The critical radius, which defines the service life of the bar element undergoing corrosion loss, is determined by computing the critical radius at which the yield stress is reached under constant load. The yield stress for steel types used to manufacture ground anchors and rock bolts are often relatively close to the ultimate stress. For a 25.4-mm-diameter steel bar (A_0 = 507 mm^2) conforming to the specifications described in ASTM A722-95, the minimum yield strength is 880 MPa and the working stress, $\sigma = 0.6\sigma_y$, is 528 MPa. For the 25.4-mm-diameter bar:

$$r_{critical} = \sqrt{\frac{A_0(0.6)\sigma_y}{\sigma_y(\pi)}} = \sqrt{\frac{507(528)}{880(\pi)}} = 9.84 \text{ mm} \tag{4-5}$$

The critical radius computed above represents a symmetrical loss of thickness of the element equal to 24.5 mm/2 – 9.84 mm = 2.86 mm = 2860 μm.

No reliable correlation exists between the soil model parameters (K, r) and the electrochemical properties of the soil. However, there are limited data showing a dependence on corrosion rate with respect to measured soil parameters, including conductivity, pH, and salt concentration. The information contained in the recommended practice is intended to provide general guidance on adjustment of parameters relative to soil conditions that may be considered minimally aggressive

TABLE 4-5 Minimum number of samples (n_0) for $10 \leq N \leq 200$

p(%)	Low Consequences of Failure	Moderate Consequences of Failure	High Consequences of Failure
1	10	10	10
5	10	10 – 15	10 – 25
10	10 – 15	10 – 25	10 – 40

Note that for $N < 10$, $n_0 = N$.

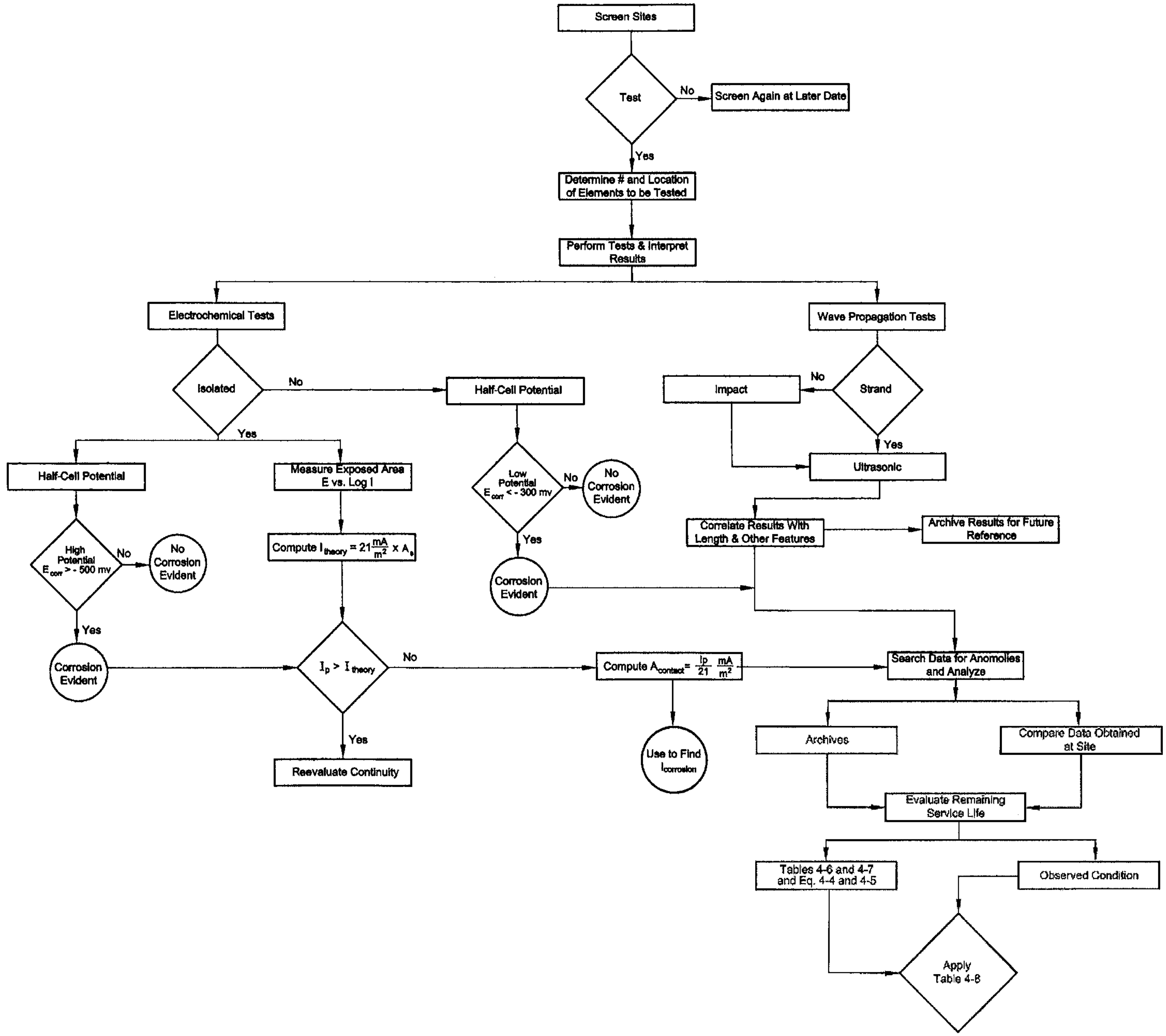


Figure 4-6. Process for condition assessment and service-life evaluation of buried metal-tensioned elements.

(i.e., normal), aggressive, and very aggressive. Table 4-6 provides general measures of corrosion potential using the results from resistivity and pH testing of soil and groundwater.

For low-carbon steels, Romanoff (1957) recommends values of *r* ranging between 0.5 and 0.6 and *K* ranging between 150 μm and 180 μm. The data used by Romanoff were developed from a wide range of burial conditions, which, for the purpose of the recommended practice, are considered normal ground conditions. According to these parameters, a 25-mm-diameter bar with a yield strength of 880 MPa has an estimated service life of approximately 100 years (≈30 μm/year). Using these same parameters, a 25-percent loss of cross section, which is approximately the sensitivity of the recommended NDT, would be estimated after approximately 42 years of burial.

More aggressive ground conditions were considered during French laboratory tests (Darbin et al., 1986). Because of this work, *r* values for carbon steel range from 0.65 to 1, and the constant *K* ranges from 3 μm to 50 μm. The highest values of the corrosion parameters correspond to an aggressive environment, characterized by soils with relatively low resistivities and high concentrations of chlorides and sulfates. If $r = 1$ and $K = 50$ μm are assumed for an aggressive environment, a 25-mm-diameter bar with a yield strength of 880 MPa has an estimated service life of approximately 57 years. For the same aggressive soil, a 25-percent loss of cross section is anticipated to occur after approximately 34 years of burial.

For extremely aggressive ground conditions, there are limited data in the literature describing measurement of very high corrosion rates. Beavers and Durr (1998) monitored corrosion rates of steel piles that were embedded in very aggressive ground conditions with chloride concentrations between 2,000 mg/kg and 5,000 mg/kg. Measurements indicate corrosion rates of approximately 340 μm/year. Assuming uniform corrosion, symmetric loss of cross section, and a corrosion rate of 340 μm/year, a 25-mm-diameter bar with a yield strength of 880 MPa has an estimated service life of 8.4 years.

The preceding discussion describes recommendations to estimate corrosion rate using a general description of the aggressiveness of the soil environment. Although this estimation relates to electrochemical properties of the soil, a more direct correlation incorporating measurements of soil conductivity and pH would be a more powerful tool. A number of studies relating to the corrosion of buried metal culverts have attempted to draw some conclusions regarding this relationship (TRB, 1978). However, there is no consensus on the validity of the results that have been presented to date. However, since some of the information is in reasonable agreement with observations, it is worth including observations from studies of buried metal culverts as additional guidance for estimating service life of buried metal elements.

TABLE 4-6 Corrosiveness of soils

Corrosiveness	Resistivity (ohm/cm)	pH
Normal	2000 – 5000	5 – 10
Aggressive	700 – 2000	5 – 10
Very Aggressive	< 700	< 5

Figure 4-7 is a nomogram that shows a relationship between corrosion rate and soil environment, described by pH and resistivity. According to Figure 4-7, for low to moderately aggressive soil conditions, average rates of corrosion vary from 10 μm to 35 μm per year (for steel: 1 g/m^2/year ≈ 0.127 μm/year). Rates of corrosion for aggressive soils range from 20 μm to 58 μm; and for very aggressive soils, rates range from 77 μm to 200 μm. These ranges are somewhat consistent with corrosion rates predicted with the Romanoff (1957) equation, discussed in the preceding paragraphs.

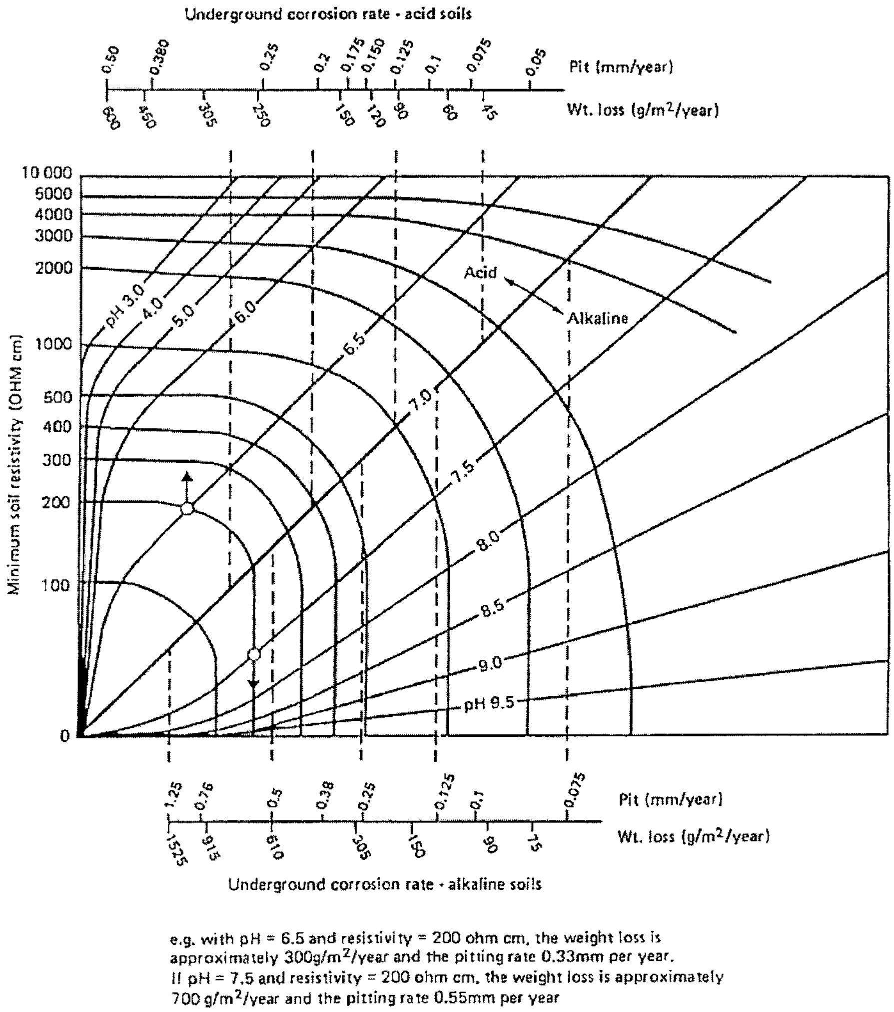
Cracks often initiate at sites showing evidence of pitting corrosion. The effect of pitting corrosion needs to be considered to adapt the models to metal-tensioned elements that may be vulnerable to this type of corrosion. This need is particularly true for elements made from high-strength steel, subjected to high levels of prestress. This problem is more for strand than for bar-type elements.

The results from several studies where the effects of pitting corrosion have been considered are described below to provide some guidance on these effects. Pitting corrosion has a greater effect than uniform corrosion on the service life of metal-tensioned elements.

The effect of nonuniform corrosion losses has been considered statistically (Elias, 1990) using test results that relate the relative loss of tensile strength to relative average thickness loss. Elias (1990) studied data obtained from samples of buried metal reinforcements that had undergone significant corrosion. Weight loss measurements were divided by the total surface area to compute average loss of thickness. The data strongly suggest that the loss of tensile strength, expressed as a percentage of the original strength, is twice as high as the average loss of element thickness.

Using the factor of two suggested by Elias (1990), the service life of a 25-mm-diameter bar element subject to pitting corrosion corresponds to an average loss of cross section of approximately 21 percent. Considering an aggressive soil environment with $r = 1$ and $K = 50$ μm and conditions that favor pitting corrosion, a 25-mm-diameter bar element has an estimated service life of approximately 28 years (≈ 100 μm/year). If very aggressive ground conditions are considered, the estimated service life of the same bar subjected to pitting corrosion is 4.2 years (≈ 680 μm/year). These service lives are considerably less than those anticipated assuming uniform corrosion.

The nomogram shown in Figure 4-7 estimates the rate of pitting corrosion that may occur in ground anchor applications. The steels used for ground anchors are different from those used for metal culverts, and rates of pitting corrosion may be significantly different. However, Figure 4-7 is considered a good place to begin to estimate rates of pitting corrosion for bar-type elements.



Note: Figure taken from King, 1977.

Figure 4-7. Nomogram for estimating the corrosion rate of steel.

4.4.1 Estimated Service Life

Equations 4-4 and 4-5 and Tables 4-6 and 4-7 are recommended for estimating the useful service life of bar elements assuming uniform corrosion and symmetric loss of cross section. The estimated service life serves as a benchmark that may be compared with the observed performance of the elements. These service-life predictions do not consider the presence of corrosion protection systems. If the service-life prediction model described in this section estimates significant loss of cross section, but NDT results do not indicate the presence of corrosion or element distress, the corrosion protection system may be intact and functioning as intended.

Loss of element thickness corresponding to the end of the useful service life and appropriate values for the parameters K and r are needed for input to Equation 4-4. Loss of thickness for use in Equation 4-4 may be computed as the original radius minus the critical radius ($r_o - r_{crit}$). Equation 4-5 is recommended for computing the critical radius of the bar element corresponding to the yield strength of the steel and the initial cross-sectional area of the bar. Finally, the constants K and r for use in Equation 4-4 may be estimated using knowledge of soil or rock mass electrochemical properties (e.g., resistivity and pH) and the data from Tables 4-6 and 4-7.

Pitting corrosion should be considered for low pH environments (i.e., when pH < 5). To consider pitting corrosion, the critical thickness loss (X) computed from Equation 4-3 should be divided by two. To consider service life, $X/2$ should be used in Equation 4-4 in place of X. For a bar with a diameter of approximately 25 mm, the critical loss of cross section is approximately 20 percent at the estimated end of

TABLE 4-7 Recommended parameters for service-life prediction model

Parameter	Normal	Aggressive	Very Aggressive
K (μm)	35	50	340
n	1.0	1.0	1.0

TABLE 4-8 Recommended action plan

Case	Conditions	Recommended Action Plan
1	• No distress is observed with NDT • Service-life prediction model estimates <25% loss of bar cross section • For wire strands, the corrosion assessment model indicates that hydrogen embrittlement and corrosion stress cracking are not likely	• Replacement of existing elements is not recommended • If test results indicate that grout does not reach the back of the anchor plates, the existing void should be filled with grout • Future monitoring is recommended at a selected monitoring interval based on anticipated service life
2	• No distress is observed with NDT • The service-life prediction model estimated more than 25 percent loss of bar cross section. • For wire strand elements, the corrosion assessment model indicates that hydrogen embrittlement and corrosion stress cracking are likely	• Verify results of NDT with invasive observations • If verified, continued monitoring at the site is recommended • A reduction in the frequency of testing may be considered
3	• Distress is observed with NDT • The service-life prediction model estimates less than 25 percent loss of bar cross section • For wire strands, the corrosion assessment model indicates that hydrogen embrittlement and corrosion stress cracking are not likely	• Apply the acceptance criteria described in Section 4.3.1 • If the existing condition is deteriorated below the acceptance criteria, verify results of NDT with invasive observations • If results from NDT are confirmed, retrofit and more frequent test intervals are recommended
4	• Observations and service-life prediction models are consistent with the conclusion of no remaining service life	• Confirm results from NDT with invasive observations • If confirmed, retrofit is recommended

the service life. This loss of cross section is near the sensitivity of NDT measurement. Therefore, it is possible for NDT to indicate the problem, but with little warning before failure, meaning remedial action must be taken immediately.

Because of the increased surface area of a strand element compared with a bar element, the estimated service life will be much lower for strand elements than that for bar elements. This analysis does not consider the substantial benefit on service life from properly installed grease and sheathing surrounding the elements. Because the integrity of the corrosion protection system is known to significantly affect the service life of strand elements, the condition assessment should focus on obtaining information on the system's integrity (e.g., electrochemical tests, sample and test grease for microbiological activity, and ultrasonic test for continuity of grout in trumpet assembly).

Corrosion processes, such as hydrogen embrittlement and stress-corrosion cracking, are known to lead to sudden failure in strand-type elements without any significant loss of element cross section. Therefore, NDT described for condition assessment will not be useful for indicating the severity of distress from hydrogen embrittlement or stress-corrosion cracking. If a user observes conditions that can contribute to hydrogen embrittlement or stress-corrosion cracking—such as a low pH soil environment, high concentrations of sulfides and chlorides, and a compromised corrosion protection system—immediate action should be recommended.

4.5 RECOMMENDED ACTION PLAN

The estimated remaining service life is compared with the observed condition of elements at the site. Four results from interpretation of the test data are possible, each leading to different recommended actions, as described in Table 4-8.

Where there is little or no consequences of failure, a "no action" alternative may be appropriate for Cases 2, 3, and 4 of Table 4-8. "No action" means monitoring conditions by visual observation and allocating budget, time, and other resources as required in response to events such as element failure, excessive slope or wall movement, and rock falls. At some sites, where there are little or no consequences of failure, this action may be appropriate.

CHAPTER 5

FIELD INVESTIGATION AT SELECTED SITES

Eight sites were included in a field study to demonstrate application of the recommended practice for condition assessment and estimation of remaining service life for existing metal-tensioned systems. Table 5-1 summarizes pertinent information for each site, including (1) the application as rock bolts, tiebacks, or wall anchors (tiebacks are anchored with a grouted zone or self-contained mechanical anchorage, and wall anchors are tied to deadmen); (2) the type of element (i.e., either bar or strand); (3) the date of installation; (4) the element vulnerability; (5) the site hazard; (6) the prestress level; and (7) special comments.

The age of the elements included in the field study range from 2 years old to 33 years old. Different anchorage types—including mechanical, cement, and resin-grouted anchorages within a variety of soil and rock types—are represented in the site domain depicted in Table 5-1. Not all the tendons were installed with corrosion protection systems that meet current standards, and this fact is reflected in the vulnerabilities of the different elements. A range of site conditions is also present, and the study includes sites corresponding to hazard conditions ranging between low and high. In addition to potential hazard due to corrosion, several of the sites have hazards related to distress from creep movement or poor drainage conditions.

The first site (Site 1) listed in Table 5-1 is the test bed constructed at the State University of New York at Buffalo that was used to evaluate the NDT methods during Phase I of the project. Results from tests performed during Phase I are presented in *NCHRP Web Document 27*. Subsequent testing was performed at the testing facility prior to exhuming of the elements. Anchors were also exhumed as part of planned reconstruction/demolition at the Buffalo Inner Harbor Site (Site 2).

Preliminary studies were conducted at the Inner Harbor Project in Buffalo, New York (04/19/00), and at the reconstruction of New York State (NYS) Route 5 in Sennet, New York (05/03/00). These studies are considered preliminary because during field studies for this project, excavation for reconstruction was underway, the elements were partially exhumed, and only a few elements were included in the condition assessment. In addition to NDT, visual inspection over part or all of the length of selected elements was performed at these sites. At the Buffalo Inner Harbor site, bar-type tendons were studied; in Sennet, New York, strand-type tendons were evaluated.

Four sites involve tieback or anchored wall systems, and the remaining three are rock bolt sites. The rock bolt site in Ellenville, New York, was selected because of the presence of different types of rock bolts, the density of the rock bolt pattern, the rock conditions, and the ease of access to the site. In 1992, a rock slide occurred at this location, and a report was prepared by NYSDOT (Johnston, 1996) describing the condition of rock bolts exhumed from the site subsequent to the slide. The site at Dresden, New York, was evaluated because load cells had been installed on several of the rock bolts at this site. The wall site at Texas A&M University National Geotechnical Experimentation Site (TAMU-NGES) was also selected because a number of the ground anchors were instrumented.

5.1 DESCRIPTION OF DATABASE

The performance database includes details of the subsurface conditions and installation details similar to that included in the inventory. Additional information is collected as part of the site evaluation, performance monitoring, and element condition assessment.

Table 5-2 presents the format for the database and includes information from the inventory of sites included in the field studies listed in Table 5-1. The following is a brief summary of the information archived in the performance database:

- **Site evaluation**
  - Soil parameters for condition assessment, including soil resistivity, pH, chloride and sulfate content, and groundwater chemistry
  - Level of site hazard, element vulnerability, and priority rating, as described in Figures 4-1 and 4-2 and Table 4-3
  - History of maintenance, performance monitoring, and retrofit
  - Estimated remaining service life based on application of Equations 4-4 and 4-5 and Tables 4-6 and 4-7
- **Performance monitoring**
  - Performance criteria—expected percentage of distressed elements
  - Consequences of failure at site—high, moderate, or low
  - Total number of elements at site

TABLE 5-1 Sites for Phase II field studies

Site No.	Site Location	Appl.[1]	Type	Install. Date	Elem. Vul.[2]	Site Hazard[3]	Pre-Stress (kN)	Comments
1	State University of New York at Buffalo; Buffalo, NY	Test Bed	Bar and Strand	1999	Low to Mod.	High (corr.)	None	3-m-long bars and strands w/ and w/o defects installed in soil
2	Buffalo Inner Harbor; Buffalo, NY	Wall anchor	Bar	1967	Mod.	Low	None	Sheet-pile quay wall; tiebacks were exhumed
3	NYS Route 5, Sennet, NY	Tie-backs	Strand	1994	Low	Mod. (corr.)	270	Sheet-pile bulkhead; wall demolished for reconstruction
4	O Street; Washington, DC	Tie-backs	Bar	1975	Mod.	High (corr. & creep)	445	Concrete diaphragm wall; known failures of anchors
5	Route 52; Ellenville, NY	Rock Bolts	Bar	1972 to 1999	Mod. to High	High (corr. & creep & drain)		Different ages and types of bolts; previous slide area; case study
6	Route 22; Dresden Station, NY	Rock Bolts	Bar	1990	Mod.	High (creep)	225 to 450	Four bolts installed with load cells to monitor changes in prestress
7	I-40 West, along Pigeon Gorge, NC	Rock Bolts	Bar	1985	Mod.	High (creep)	180	Bars installed near slide area. pyrite deposits in rock, suspected acidic groundwater conditions
8	NGES- Texas A&M Riverside Campus; College Station, TX	Tie-backs	Bar	1991	Low	High (drain)	85 to 380	Soldier-pile and lagging; easy access; wall built for experiment; instrumentation is installed; case study

[1] Appl. = application.
[2] Elem. Vul. = element vulnerability: low, moderate, or high.
[3] Site hazard is low, moderate, or high from corrosion, creep, or drainage problems.

  – Number of elements tested at the site
- **Element condition assessment**
  – Half-cell potentials—percentage of elements where corrosion is likely
  – Polarization current—percentage of elements with compromised corrosion protection systems
  – Impact testing and UT—percentage of elements with suspected voids behind the bearing plate
  – Impact testing and UT—percentage of elements with anomalous reflections or suspected defects
  – Number of elements tested with invasive tests
  – Number of distressed elements observed from invasive tests
- **Summary of recommended actions**
  – Compare condition assessment with estimated service life and performance criteria—assign case number according to Table 4-8
  – Is invasive testing recommended?
  – Is retrofit recommended?
- **General comments**

5.2 SUMMARY OF SITE EVALUATIONS AND CONDITION ASSESSMENTS

The following sections are summaries of the site evaluations and condition assessments performed at each of the sites listed in Table 5-1. The summaries include details of the installations, subsurface conditions, results from previous performance monitoring (if available), description of NDT and condition assessment, and conclusions from the site evaluation and element condition assessment.

5.2.1 SUNY at Buffalo, Buffalo, New York

In 1999, a special test bed was constructed for this project at the State University of New York at Buffalo (UB), North Campus. The test bed is located near the northwest corner of the civil engineering laboratory (Ketter Hall), within an open grassy area, which is approximately 1,500 m^2 in plan. The test bed features different types of metal elements, with known features and initial conditions, installed in the ground.

TABLE 5-2 Performance database for condition assessment and evaluation of service life of metal-tensioned system

1. Location	**O Street, Wash. D.C.**	**Ellenville, NY**	**Ellenville, NY**	**Dresden, NY**	**Pigeon Gorge, NC**	**TAMU-NGES**
Route No.	Between Carpenter St.	NYS 52	NYS 52	NYS 22	US I-40	sand site
Milepost	and Branch Ave.	1119	1119	1642	1	NA
Date of last evaluation	May-00	Aug-00	Aug-00	Sep-00	Oct-00	Dec-00
2. Element Data						
Application	tiebacks	rock bolts	rock bolts	rock bolts	rock bolts	tiebacks
Date of installation	1978	1972	1992	1992	1985	1991
Type	bar	exp. shell	resin grouted	resin grouted	resin grouted	bar
Diameter (mm)	32	19	25	32	25	32
Number of Elements	176	17	35	15	> 100	19
Steel Grade (MPa)	1030	550	1030	1030	1030	1030
Prestress (kN)	445	-	-	225 to 450	180	85 to 380
Corrosion Prot. (PTI Class)	II	none	grout	grout	grout	II
3. Site Data						
Rock Type	NA	sandstone	sandstone	gneiss	siltstone	NA
Soil Type (USCS)	CL and CH	infill-SC	infill-SC	infill-SW-SM	clay seams	SP
Resistivity (ohm-cm)	2250	10,400	10,400	5900	-	8200
pH	3.6 to 4.1	3.1	3.1	6.9	-	6.4
Cl (mg/kg)	11-12.5	5.5	5.5	4.3	-	17.6
SO4 (mg/kg)	< 10	< 10	< 10	< 10	-	< 10
Creep	yes	no	no	no	no	no
Poor Drainage	yes	yes	yes	no	yes	yes
Stray Current	no	no	no	no	no	no
4. Maintenance Data						
Observed Failure	SCC in 1979	rock fall 1992	rock fall 1992	load cells	slides in 1985	load cells
and retrofit	Creep in 1995	installed	installed	monitored	and 1997	monitored
	retro. anchor head 1980	new bolts	new bolts	quarterly	catchments installed	1991-1996
	retrofit wall in 2000					lift-off test 1996
5. Risk Assessment						
Site Hazard (Figure 4-1)	high	low	low	low	moderate	low
Element Vulnerability (Figure 4-2)	moderate	high	moderate	moderate	moderate	moderate
Priority Rating (Table 4-3)	6 (creep)	4 (drainage)	4 (drainage)	0	2	4 (drainage)
6. Estimated Remaining Service Life						
conditions favor pitting	yes	yes	yes	no	no	no
K (Tables 4-6 and 4-7) for Eq. 4.4	50	35	35	35	50	35
total service life from Eq. 4-4 (years)	36	30	40	100	57	100
remaining service life	14	0	32	92	42	92
7. Performance Monitoring						
p - acceptance criterion (%)	5	5	1	1	1	5
consequence of failure	moderate	high	high	moderate	moderate	high
recommended no. of samples (Table 4-5)	10 to 15	10 to 25	10	10	10	10
no. of samples tested	15	5	24	6	9	10
8. Condition Assessment						
% with corrosion	50	100	17	33	22	NA
% compromised corrosion protection	N.O.	100	100	100	100	N.O.
% with voids behind bearing plate	0	NA	100	100	100	NA
% with suspected distress	17	25	4	0	0	0
along free length	yes	yes	yes	NA	NA	NA
along bonded length	yes	no	no	NA	NA	NA
invasive test	NA	NA	NA	NA	NA	lift-off O.K.
9. Recommended Actions						
Case (Table 4-8)	2 (creep)	2	3	1	1	1
Action	invasive obs.	retrofit	invasive obs.	continue	continue	continue

SCC = stress corrosion cracking.
N.O. = not observed.
NA = not applicable.

The anchor elements were installed in vertical test holes. Auger borings with a diameter of 150 mm were advanced to a depth of 2.75 m at each element location. The specimens were centered within the hole, and the auger borings were backfilled with native soil. As the holes were advanced, soils were sampled continuously using a split-barrel sampler, and the standard penetration test (SPT) resistance was obtained at 0.6-m intervals in general conformance with ASTM D1586 (ASTM, 2001). The soil samples were tested in the laboratory for moisture content, Atterberg limits, and grain size.

5.2.1.1 Subsurface Information

Soil samples collected at the site were identified as fill. The fill is predominantly a fine-grained soil with varying amounts of gravel. The SPT N values of the fill range from 15 to 44 (blows per 0.3 m). The higher N values are apparently inflated because of the gravel content, and the lower N values, between 15 and 20, indicate a relatively stiff, fine-grained soil. Measured moisture contents ranged from 8 to 13.5 percent, most likely corresponding to the moisture content during compaction of the fill, and the degree of saturation ranged between 60 and 90 percent. Grain size analysis confirmed that the material is predominantly fine grained, with 60 to 70 percent finer grain than the 200 sieve. The liquid and plastic limits of the fill soil were measured as 22.9 percent and 10.8 percent, respectively. Using these laboratory test results, the fill is classified as CL by the Unified Soil Classification System (USCS) in conformance with ASTM D2487 (ASTM, 2001).

5.2.1.2 Element Construction and Installation

The UB test facility is pictured in Figure 5-1. Eight elements, each with a length of approximately 3 m, were installed. The elements were placed along two rows and separated by approximately 4.5 m. All of the elements have a 0.3-m-long grout bulb at their lower end to simulate anchorage of the bars and tendons in soil. The grout bulbs were precast at the bottom end of the specimens prior to installation.

Four types of elements were installed at the UB Test Facility. These included

- 32-mm-diameter, plain Dywidag bars;
- 32-mm-diameter, epoxy-coated Dywidag bars;
- 32-mm-diameter, plain Dywidag bars surrounded by grout encased in a 2.7-m-long, 10-mm-diameter plastic pipe; and
- 15-mm-diameter, seven-wire Polystrand coated with grease, surrounded by an extruded high-density polyethylene (HDPE) sheath.

Two specimens were installed for each type of element, the first intact without any defect and the second 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 extends through the grout and into the bar. For strand elements, the defect was created by stripping a 75-mm length of the HDPE sheath and exposing the strand to the subsurface environment.

5.2.1.3 NDT Performed at Site

Results from initial NDT conducted in the fall of 1999 and spring of 2000—including electrochemical testing, impact testing, and UT—were presented in *NCHRP Web Document 27* (D'Appolonia Consulting Engineers et al., 2001). A few of the important results are presented here to describe the observed trends.

Electrochemical test results include half-cell potentials and observation of the E versus log I relationship. The ground bed used for the E versus log I test consisted of three 1-m-long, copper-plated rods, located approximately 30 m south of Element 1. The half cell was located on the ground surface within approximately 0.3 m of each respective element during testing. Table 5-3 is a summary of results from electrochemical tests, where

- E_{corr} = free corrosion potential with reference to the half cell (mV),
- I_p = polarization current observed from the E versus log I relationship (mA),
- $A_s = I_p/21$ = estimated surface area of the element in contact with the ground (mm^2),
- $L_e = A_s/(d$ = length of the element corresponding to A_s (m^2), and
- d = element diameter (mm).

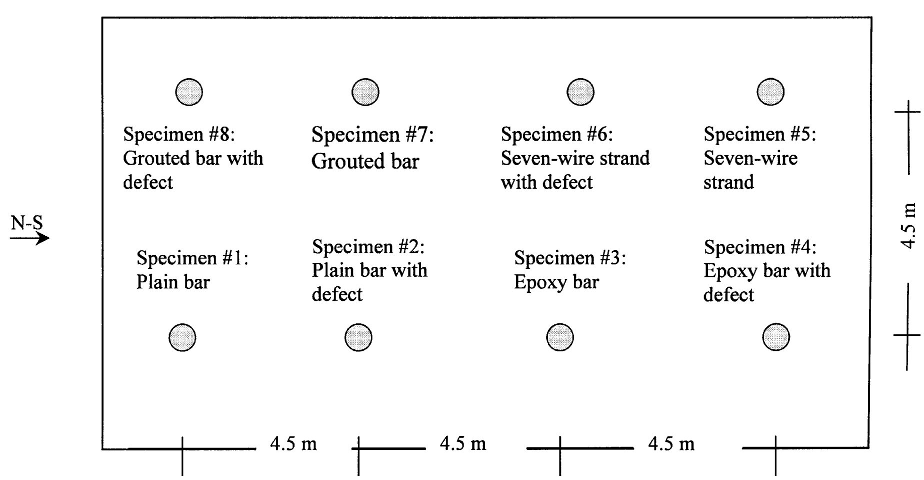


Figure 5-1. Plan view of in situ specimens at UB test facility.

TABLE 5-3 Summary of electrochemical test results from UB test bed specimens

Element No.	Description	E_{corr} (mV)	I_p (mA)	A_s (m^2)	L_e (mm)
1	Plain Bar (d = 32 mm)	–515	4.4	0.2100	2084
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

Compared with the potential of –200 to –500 mV CSE typically observed for rusted low-carbon steel in neutral soils and water, the corrosion potentials shown in Table 5-3 indicate that corrosion has occurred in Elements 5 and 8. The lengths of exposed element (L_e), estimated from the E versus log I test, are consistent with the known conditions of the elements. Element 1 is in direct contact with the ground for the majority of its length, and, by contrast, Element 5 is insulated with grease and plastic sheathing. This insulation is reflected in the relatively large and small values of L_e for Elements 1 and 5, respectively. Elements 4 and 6 have L_e roughly corresponding to the existence of the 75-mm-long defect. The larger L_e of Element 8 includes some exposed metal near the top of the element in contact with the ground.

Figures 5-2 and 5-3 show results from impact tests performed on epoxy-coated bars, with and without defects (Elements 3 and 4). Reflections at intervals of approximately 0.0011 s are observed in the time domain. These reflections correspond to the time for compression waves to travel the length of the bar and back (2 × 3 m ÷ 5,500 m/s = 0.0011 s). For the bar with the defect, an additional reflection appears that becomes more apparent as energy from reflections at the end of the bar attenuate. The presence of a defect along the length of the bar is also characterized in the frequency response presented in Figures 5-4 and 5-5. Compared with the intact specimen, the lower fundamental frequencies are relatively more predominant in the frequency response of the defected bar. Table 5-4 is a summary of the fundamental frequencies observed in the impact test results for each of the elements at the UB test bed.

Figures 5-6 and 5-7 show results from ultrasonic tests conducted on the same epoxy-coated elements (Elements 3 and 4) for which impact test data were presented. Similar to the results obtained with the impact test, a distinct reflection is observed approximately 0.0011 s after the ultrasonic excitation pulse. Compared with the results from the intact bar, the relatively small amplitude of the reflection from the end of the defected bar is readily apparent. The smaller amplitude is due to reflections and refractions from the defect and associated loss of energy. An additional reflection is perceptible for the bar with the defect. Thus, the presence of the defect is manifested in the test results. The data from the ultrasonic test confirm the results obtained from the impact test. The data are very useful because similar results are obtained when the

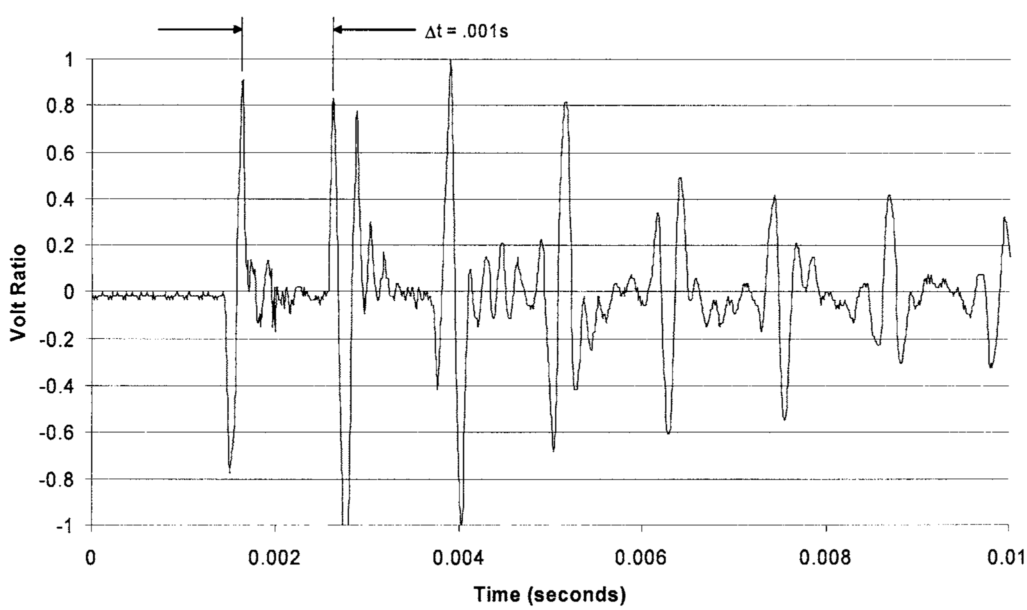


Figure 5-2. Impact test on epoxy-coated bar without defect at UB test facility.

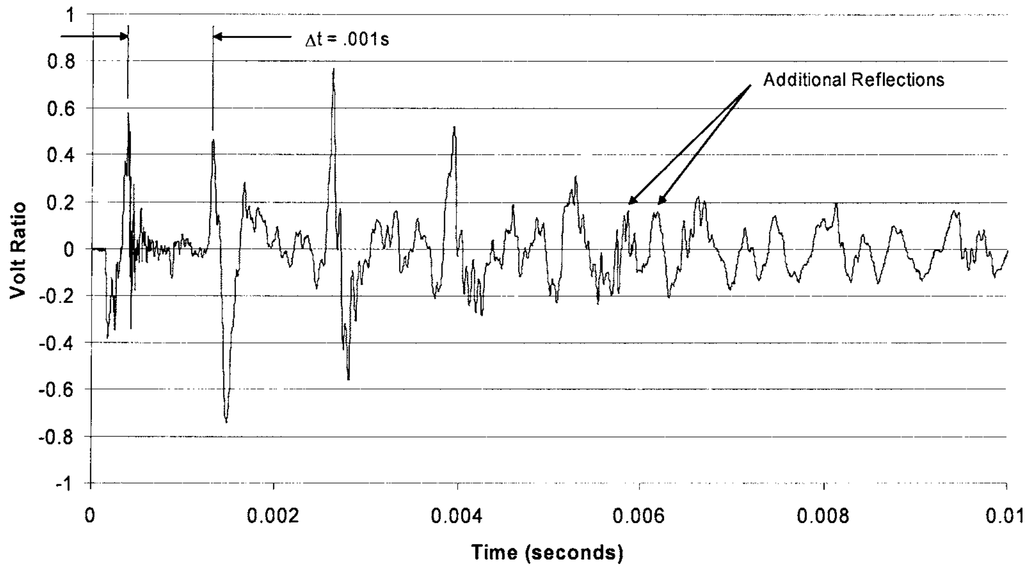


Figure 5-3. Impact test on epoxy-coated bar with defect at UB test facility.

same elements are tested using different NDT methods. This similarity demonstrates the validity of either test method.

5.2.1.4 Conclusions from NDT

The half-cell potentials are consistent with the known conditions of the elements, and the measured polarization current correlates with the known lengths of the elements exposed to the surrounding soil. Results from impact and UT are consistent with the known lengths of the elements and presence of defects.

5.2.2 Buffalo Inner Harbor, Buffalo, New York

In 1967, a 12-m-high, sheet-pile quay wall was constructed at the future site of the Buffalo Inner Harbor Development Project. The wall was supported with a single row of anchors located approximately 1.0 m below the top of the wall. The wall anchors consisted of smooth bar elements (i.e., upset rods) threaded at each end. The bars were threaded at each end, made from mild steel with a diameter of 64 mm at the upset end and 51 mm along the shaft. The bars were anchored to a sheet pile deadman located approximately 17 m behind the wall face. Double-channel walers were attached to the

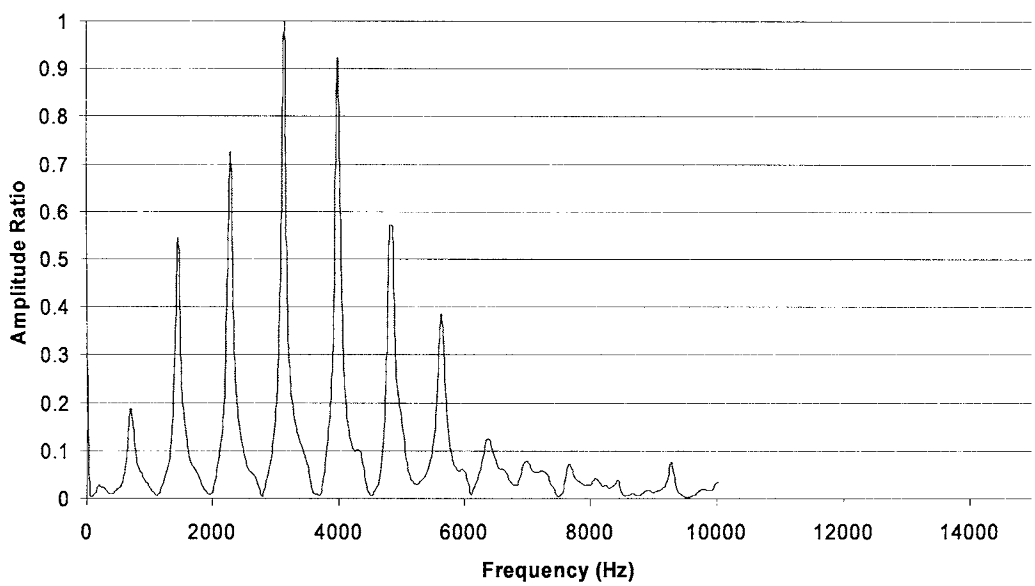


Figure 5-4. Amplitude spectrum from impact test on epoxy-coated bar without defect at UB test facility.

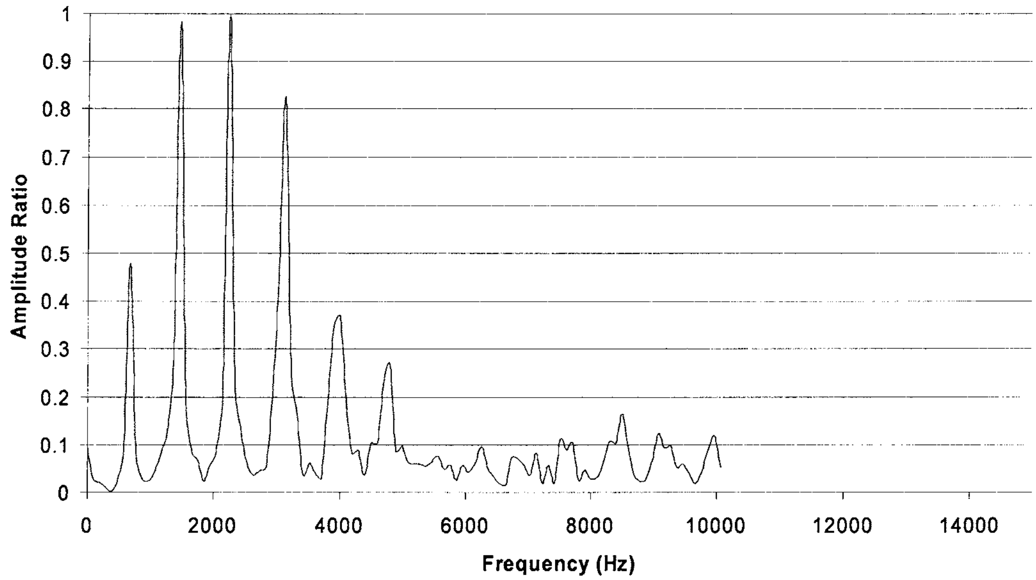


Figure 5-5. Amplitude spectrum from impact test on epoxy-coated bar with defect at UB test facility.

inside of the wall face and along the back side of the sheet-pile deadman. Bars were attached to the walers with nuts and bearing plates at each end.

Approximately 8.5-m-long bars were coupled to span between the wall face and the anchor sheets. A corrosion protection system was not included in the design of the anchors. Because of the age (more than 15 years old), level of corrosion protection (none), and steel type (mild), element vulnerability was considered moderate.

Figure 5-8 is a photograph showing the partially exhumed anchors. Couplings can be seen in the photograph at the right side of the excavation. The waler along the inside of the wall face is visible at the left edge of the excavation.

5.2.2.1 Subsurface Information

In support of the Buffalo Inner Harbor Development Project, test borings were advanced behind the existing quay wall and soil samples were retrieved for laboratory testing. Bedrock was located at the toe of the wall at a depth of approximately 12 m from the ground surface. Soil deposits on top of the bedrock include approximately 9 m of natural soil deposits followed by 3 m of granular fill. The mean lake elevation, and corresponding groundwater level, was approximately 3 m below the ground surface (and the top of the wall). Using this information and observations of the elements made during the NDT at the site, the wall anchor elements were located within the fill and completely above the groundwater table.

Laboratory testing conducted on the granular fill included moisture content analysis, grain size analysis, and chemical analysis (including pH), as well as measurement of trace compounds typical of those conducted for environmental assessment. The pH measured on two samples of the fill was 7.0 and 6.4, and sulfides were not present above the detection limit of 11 mg/kg. The results of the grain size analysis are presented in Table 5-5.

In summary, the fill is relatively free draining and is classified as poorly graded gravel, GP (ASTM, 2001). Although the soil resistivity was not measured directly, a free-draining gravel material with neutral pH has an estimated resistivity of more than 5,000 ohm-cm. Because of this information,

TABLE 5-4 Summary of f_n for bars tested at UB

Element No.	Description	f_1 (Hz)	f_2 (Hz)	f_3 (Hz)	f_4 (Hz)	f_5 (Hz)	f_6 (Hz)	f_7 (Hz)
1	Plain bar	683	1464	2246	3125	3906	4785	5566
2	Plain bar w/defect	716	1464	2246	3092	3971	–	–
3	Epoxy-coated bar	732	1465	2295	3125	3955	4834	5615
4	Epoxy bar w/defect	684	1465	2246	3125	4004	4785	–
7	Grouted bar	683	1464	2050	2636	3222	3906	4882
8	Grouted bar w/defect	683	1367	1953	2539	3125	3808	4980

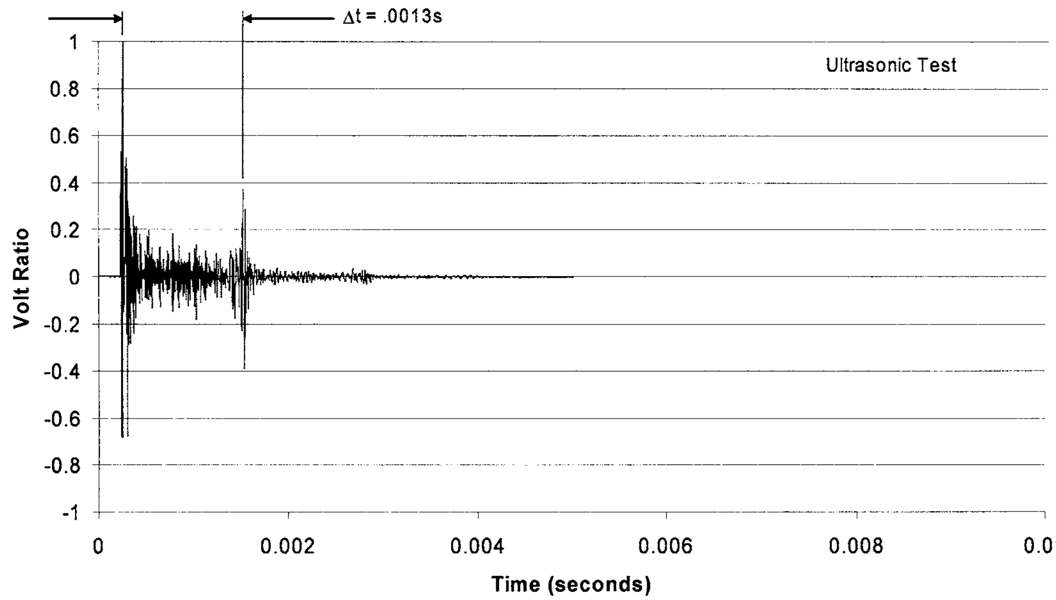


Figure 5-6. Ultrasonic test results for epoxy-coated bar without defect at UB test facility.

and because the wall anchor elements were located above the water level, site hazard is considered low.

5.2.2.2 NDT Performed at Site

During the spring and summer of 2000, the existing quay wall was demolished and a new quay wall was constructed approximately 15 m behind the location of the original wall. During construction of the new quay wall, the backfill behind the old wall was excavated and the anchor bars were exhumed. The coupling was unearthed, and its location (approximately 8.5 m from the end of the bar) was documented. Visual observation of the exposed bars indicated that the bars were in very good condition and had a relatively uniform coating of rust surrounding the bars. No significant loss of cross section or surface pitting was observed. Access to the end of the anchors was available from the excavation on the back side of the sheetpile deadman.

NDT was performed on three adjacent elements at the site that were located at a spacing of approximately 2 m. Bar 1 was located in the middle, and Bars 2 and 3 were located approxi-

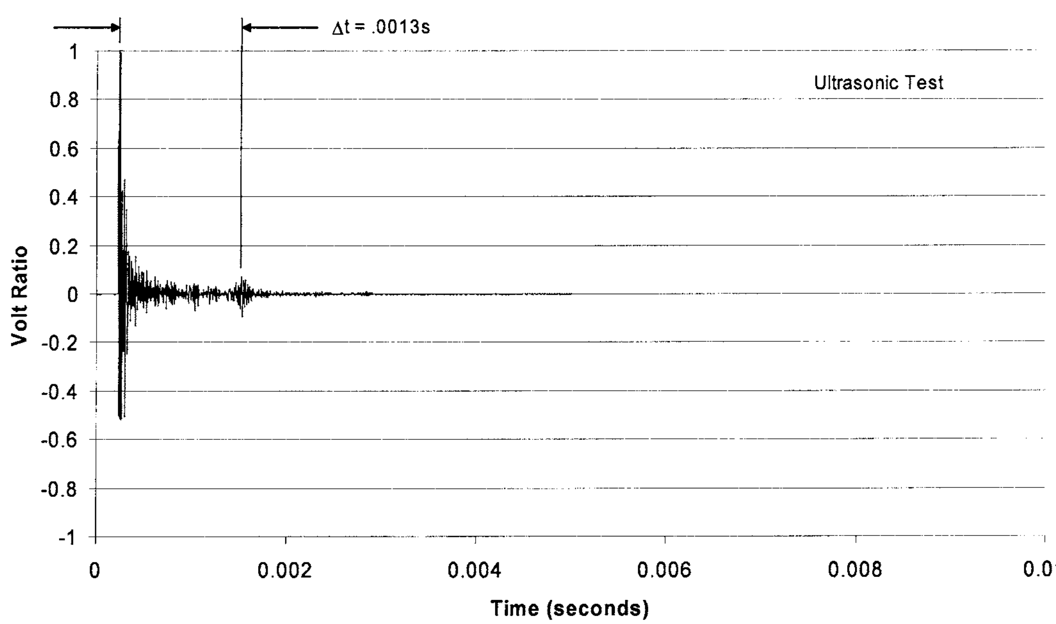


Figure 5-7. Ultrasonic test results for epoxy-coated bar with defect at UB test facility.

Figure 5-8. Thirty-three-year-old bars in free-draining gravel behind old quay wall unearthed at Buffalo Inner Harbor project.

mately 2 m east and west of Bar 1, respectively. NDT at the site of the Buffalo Inner Harbor included impact and ultrasonic tests. Electrochemical tests were not performed.

For the impact test, a PCB Model 086C05 steel-tipped, instrumented hammer (described in Chapter 3) was used to apply the impact force. The accelerometer was glued to the end of the bar using a special detachable base with a threaded hole in the middle. Figure 5-9 compares time histories from testing performed on Bars 1, 2, and 3. A reflection at 3.1 ms is clearly evident in the test results from Bars 1 and 3, and, although present for Bar 2, it is less discernible. The reflection at 3.1 ms is consistent with the observed location of a mechanical coupling about 8.5 m from the end of the bar ($8.5 \text{ m} \times 2 \div 5{,}500 \text{ m/s} = 0.0031 \text{ s}$).

Results from impact testing were transformed to the frequency domain, and the amplitude spectra for Bars 1, 2, and 3 are shown in Figure 5-10. The amplitude spectra for the three bars are similar, although there is a shift in the predominant frequencies, particularly at frequency levels above 5,000 Hz. It appears that Bar 1 has the highest frequency response, followed by Bar 3. Bar 2 does not show significant peaks in the higher frequency response range of the amplitude spectrum. According to the results of research conducted at the University of Aberdeen (Famiyesin et al., 1997), the frequency response of tensioned metal elements is related to tension level in the bar. An increase in the main frequency content is realized when tension applied to the element is increased. This fact may explain the differences noted in the frequency response presented in Figure 5-10. Because of the results presented in Figure 5-10, Bar 1 is believed to have a higher level of tension than Bars 3 and 2.

TABLE 5-5 Grain size of gravel fill at Buffalo Inner Harbor

Sieve Size (U.S. Standard)	% Passing
50 mm	100
6.3 mm	45.3
No. 40	15.3
No. 200	6.8

Ultrasonic tests were performed on bar ends that did not receive any special preparation or surface treatment. Valvoline grease was used as an acoustic couplant between the ultrasonic transducer and the end of the bar. The ultrasonic transducer was a Panametrics Model V1011. This device has an operating frequency of approximately 100 kHz and a contact surface with 38-mm diameter.

Results from testing Bars 1, 2, and 3 are shown in Figure 5-11. A clear reflection is indicated at approximate 3.2 ms in the test results for Bar 1, which is consistent with the reflections observed in the impact test results. The same reflection is not clearly visible in the test results for Bars 2 and 3. This lack of clearly visible reflection may be due to the relatively poor, uneven end conditions for Bars 2 and 3 compared with the end conditions for Bar 1. Poor end conditions can negatively affect acoustic coupling between the transducer and the end of the bar, which lessens the ability of the transducer to transmit and receive the sound wave signals. Because of this observation, preparation of bar ends is recommended prior to performing the ultrasonic test. Surface preparation may be necessary to achieve a flat surface parallel to the cross section of the bar. This preparation involves (1) squaring the bar ends with a power saw (i.e., a gas-powered chop saw) suitable for cutting through steel and (2) surface grinding to remove irregularities as necessary.

Tests on new bar elements. In addition to tests performed on the existing quay wall anchor elements, a few nondestructive tests were performed on new bars stockpiled on-site for construction of the new quay wall. The new bars were approximately 45-mm-diameter, Grade 150, Dywidag bars that were protected by a grout-filled plastic sheath. Ultrasonic tests were performed on two bar elements designated New Bar 1 (length $\approx$ 7.5 m) and New Bar 2 (length $\approx$ 5.0 m). Ultrasonic test results for New Bars 1 and 2 are compared in Figure 5-12. The time of arrival of the reflected waves was approximately 0.0022 s and 0.0033 s for New Bars 2 and 1, respectively. The ratio of the arrival times (New 1/New 2 = 0.0033/0.0022 = 1.5) corresponds to the ratio of the element lengths. Given the length of wave propagation, the travel wave velocity is computed as 4,400 m/s, which is less than the compression wave velocity along a cylindrical steel bar. The travel time of the bar surrounded by grout is reduced compared with the travel time of bare steel because the grout affects the dynamic response of the element.

Bars exhumed during archeological dig. During construction activities for the Buffalo Inner Harbor Development Project, foundation relics were being studied as part of an archeological dig east of the old quay wall. Three additional anchor elements were exhumed that, according to archeologists conducting the excavation, were installed around 1930. Details of the element type and installation are similar to details observed behind the old quay wall. Although NDT was not conducted at the archeological dig, the observed

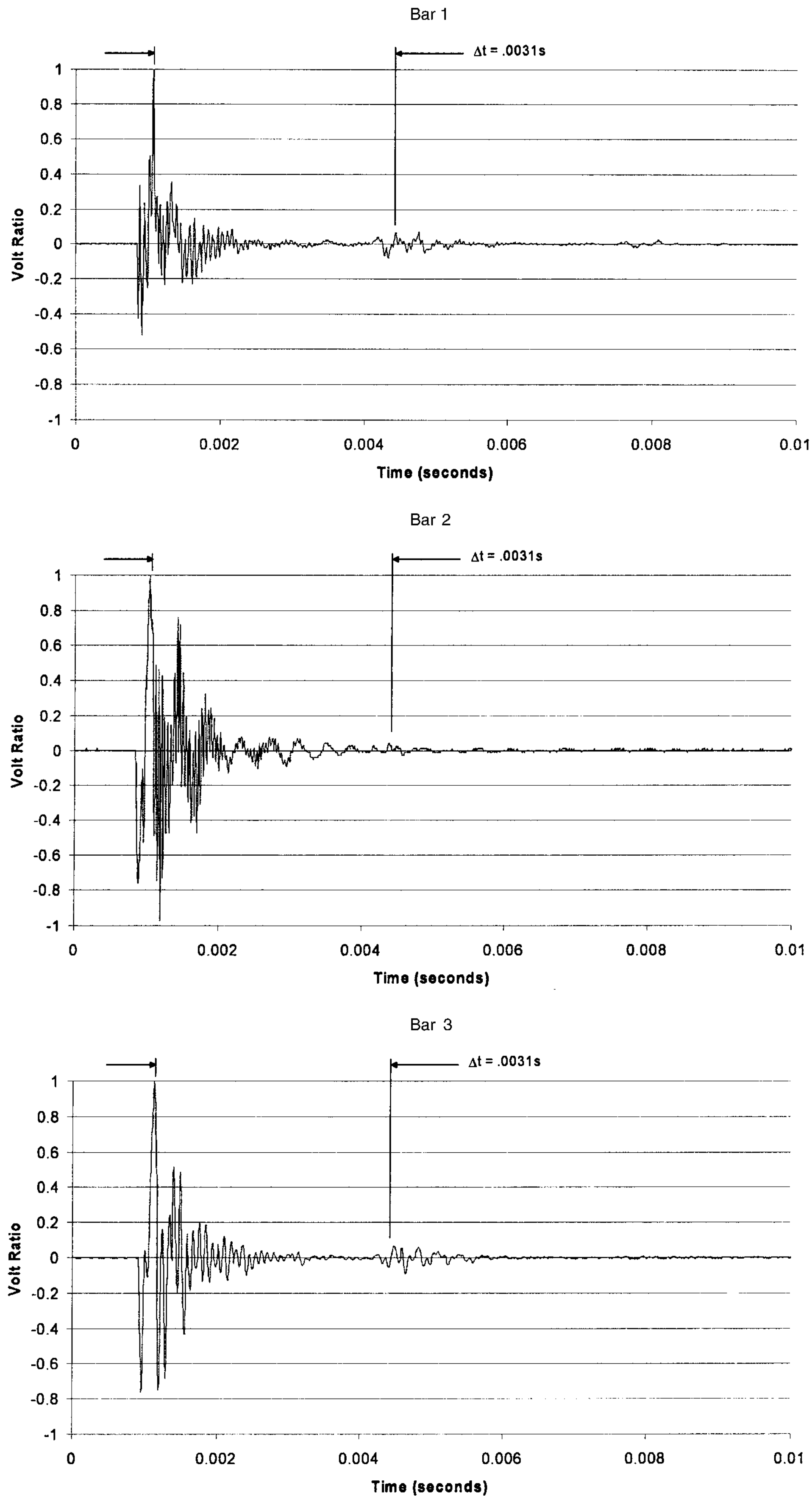


Figure 5-9. Comparison of time histories from impact tests performed on Bars 1–3 at Buffalo Inner Harbor.

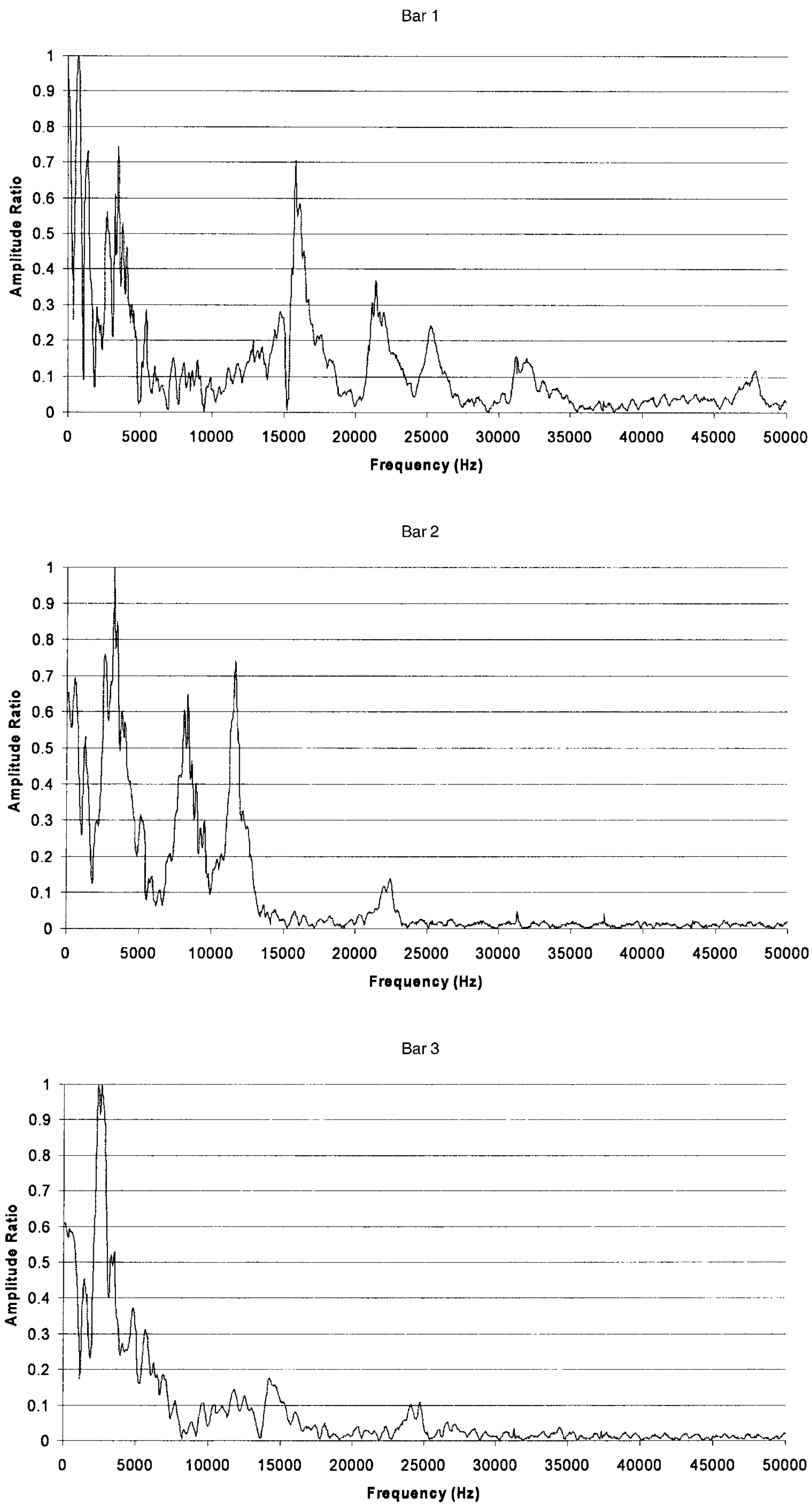


Figure 5-10. Amplitude spectra from impact tests on Bars 1–3 at Buffalo Inner Harbor.

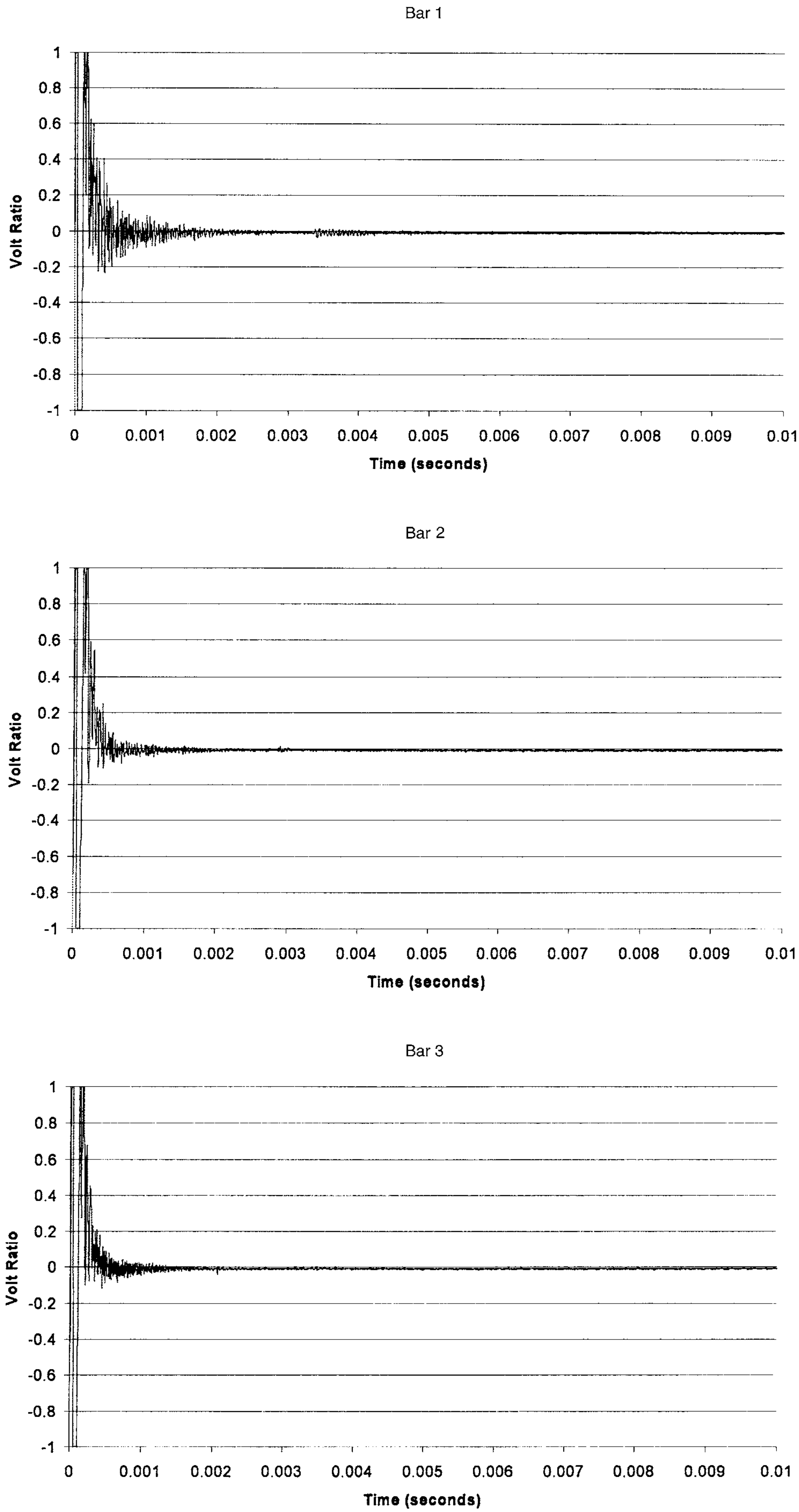


Figure 5-11. Results from ultrasonic tests on Bars 1–3 at Buffalo Inner Harbor.

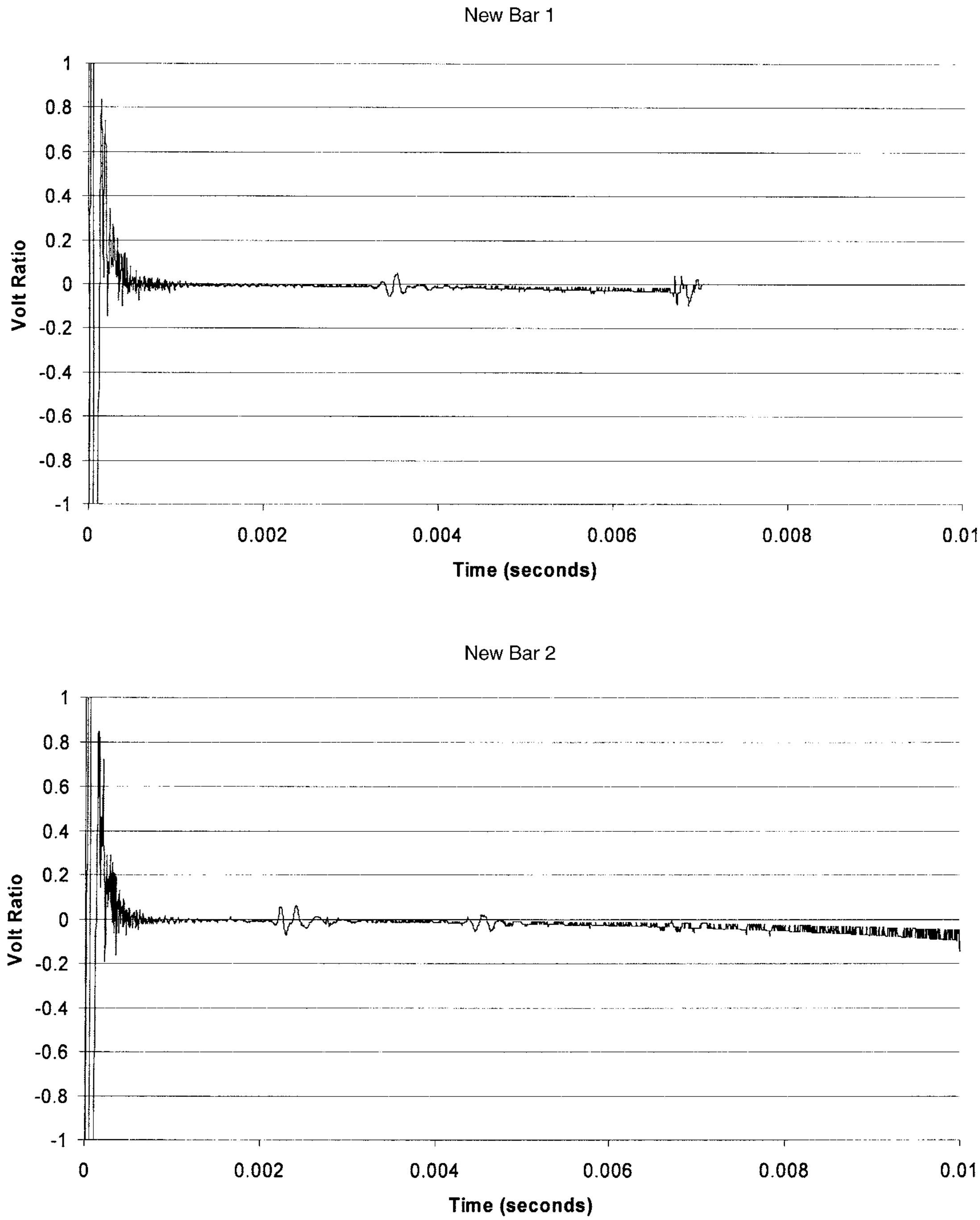


Figure 5-12. Ultrasonic tests on new bars at Buffalo Inner Harbor.

conditions of the exhumed elements and surrounding soil environment are reported herein. Conditions surrounding the elements discovered during the dig starkly contrast conditions surrounding the bars exhumed behind the old quay wall.

Severely corroded elements were discovered at the dig site. One element in particular, shown in Figure 5-13, was completely corroded at the location of the coupling. The other two bars, both located east of the element shown in Figure 5-13, exhibited loss of cross section, and measurements of diameter taken at a number of different locations along the length of the elements range from 41.9 mm to 50.8 mm.

Backfill within the area of the dig was much different from what was observed behind the quay wall. Within the dig area, slag and cinder ash were used as backfill. These materials, which are waste products from steel manufacturing, are characterized by neutral to slight alkalinity and high salt concentrations. Samples of the slag backfill were taken and tested for pH, resistivity, sulfates and chlorides. Test results are presented

Figure 5-13. Corroded 70-year-old element backfilled in slag, exhumed at east end of Buffalo Inner Harbor project.

in Table 5-6. The test results confirm that the backfill is very aggressive and show a high hazard relative to corrosion.

5.2.2.3 Conclusions from Observations and NDT at Site

According to the information gathered at the Buffalo Inner Harbor site, the soil environment and corresponding level of corrosion hazard has an important effect on the service life and condition of buried metal elements. Similar elements were buried in very different ground conditions, and the effect of aggressive ground conditions on the service life of the elements is evident. The data presented relative to this site illustrate the profound effect that ground condition has on the service life of buried metal elements.

5.2.3 NYS Route 5, Sennet, New York

The site is located in the town of Sennet, New York, along NYS Route 5 in Region 3 of the NYSDOT. A tieback sheet-pile wall constructed in 1994 supports a railway embankment that serves as the approach to the elevated crossing of Conrail over NYS Route 5. Figure 5-14 shows the face of the wall as it looked in the winter of 2000.

The height of the wall ranges from approximately 1 m to 4 m, and the wall supports a 4- to 7-m-high slope at a grade

TABLE 5-6 Electrochemical properties of backfill sampled at the Buffalo Inner Harbor archeological dig

Description	Value
pH	7.7
Resistivity (Ω-cm)	850
Sulfates (kg/mg)	10,000
Chlorides (kg/mg)	5

Figure 5-14. Tieback wall for railway embankment along NYS Route 5 in Sennet, New York.

of 2.5H : 1V. The wall is supported by one or two rows of grouted ground anchors. The tendon elements are seven-wire strands, inclined at 15° with respect to the horizontal. At the wall face, the strands are surrounded by a metal sheath that extends through a wide flange beam waler. The anchor head is covered by a grease-filled metal end cap. Each anchor assembly includes two 12.7-mm-diameter, seven-wire strands conforming to Grade 270 of ASTM Specification A416. Anchors were spaced approximately 3.5 m c-c along the length of the wall and were designed for an ultimate capacity of 270 kN. Each strand is coated with grease and surrounded by an extruded plastic sheath. Along the free length, the strand pairs are surrounded by a smooth polyvinyl chloride (PVC) pipe sleeve.

5.2.3.1 Subsurface Conditions

A subsurface investigation was conducted in support of reconstruction of NYS Route 5. Test boring logs were obtained from the NYSDOT Region 3 Soils Engineer. Soil samples were collected, and standard laboratory tests (including moisture content, grain size analysis, and Atterberg limits) were conducted. The soil behind the wall above the dredge line is a loose-to-medium-dense silty gravel with SPT *N* values ranging from 4 to 19 blows per 0.3 m and in-situ moisture contents ranging from 8 to 12 percent. Below the dredge line is a stiff soil consisting of alternating layers of clayey silt and silty clay, having SPT *N* values ranging from 19 to 40 blows per 0.3 m and moisture contents ranging from 6 to 21 percent. Measured groundwater elevations are variable and were observed within the alternating layers of clayey silt, silty clay soils. No chemical analyses of the soils were performed.

The free length of the ground anchors passes through the gravel zone, and the bonded zone is located within the stiff soil strata. According to information provided in the test boring logs, ground anchors appear to be located above the observed groundwater levels. Given the stratigraphy, however, perched groundwater within the gravel layer of the free-length zone is possible.

5.2.3.2 NDT Performed at Site

As part of a highway reconstruction project that involved establishing a grade crossing for the railroad, the embankment was excavated and the wall was demolished in spring 2000. When the embankment excavation reached the elevation of the tiebacks, the elements were severed along the exposed back face of the sheetpile. During the site visit, the severed ends of the tendons were exposed within the clayey silt soil, which was wet and soft at the surface from a recent rain. The exposed strands appeared to be in good condition. Uniform corrosion was observed surrounding the exposed cut ends, but this corrosion probably occurred after the demolition.

Three pairs of tendons were accessible at the east end of the project. The equipment van was parked along the shoulder of the highway near the exposed tendons, and necessary wiring was extended from the back of the parked van. A portable generator was used to power the equipment.

Electrochemical measurements were made on one strand. Three copper rods were driven into the ground at a distance of approximately 6 m from the strand to serve as a ground bed. The copper/copper-sulfate half cell was placed within a half meter of the strand. A half-cell potential of −180 mV was measured, indicating that corrosion had occurred along the strand. Results from the E versus log I test rendered a corrosion protection current of approximately zero, suggesting that the plastic sheath was intact and that the strand was insulated from the surrounding soil environment over the remainder of its length.

The ends of the tiebacks were ground flat and smooth with a surface grinder in preparation for UT, as shown in Figure 5-15. Four strands were tested, which were from two pairs of adjacent tiebacks. The test results, shown in Figure 5-16, were processed with a scaling function to enhance the observed reflection. The reflections from the four strands were observed at approximately 0.0012 s, which corresponds to a distance of 3.3 m from the cut end of the element. The observed reflections are likely due to the interface between the bonded and unbonded zones. At this point, the plastic sheath is discontinued and the steel strands are surrounded by grout. The different impedance between the bonded and unbonded zones along the length of the element results in a strong reflection of the propagating wave.

Figure 5-15. Strand ends tested at Sennet, New York.

5.2.3.3 Conclusions from NDT

NDT conducted at the site served to demonstrate application of NDT to strand-type elements. Features of the anchor head assembly were not present, but the ability to test isolated lengths of strand was established.

Electrochemical test results were consistent with the anticipated conditions. Some corrosion was evident because of unprotected elements exposed to the environment subsequent to demolition. These observations demonstrate the vulnerability of strand-type elements when corrosion protection is compromised. Attempts to measure polarization current indicated that the plastic sheath was intact for most of the remaining length of the unbonded zone.

Ultrasonic test results indicated that reflections could be obtained from distant sources (approximately 3.3 m from the face of the element) if the ends of the strand were cut and ground smooth to obtain good acoustic coupling.

5.2.4 O Street, Washington, D.C.

Slope instabilities and creep-related slope movements have been documented between Highland Drive and O Street in Washington, D.C., since the late 1950s. With development of the area through the early 1970s, the extent of the slope instabilities and failures increased, and by 1974, some structures on Highwood Drive had reportedly been in danger of serious structural damage. The O Street retaining wall was constructed in 1978 to stabilize the slopes between Highland Drive (North), O Street (South), Carpenter Street (East), and Branch Avenue (West) in the Southeast quadrant of Washington, D.C. The O Street retaining wall is a diaphragm-type concrete structure that was constructed using slurry trench methods by ICOS Corporation of America (ICOS). Apparently, slope movements continued after construction of the wall, and, in 1995, significant distress was observed that led to a geotechnical investigation and recommendations for repairing the wall.

The site was selected for evaluation as part of NCHRP Project 24-13 because of the potential presence of damaged or distressed ground anchors and because of the information collected relative to the installation and history of the wall system.

5.2.4.1 Details of the Wall Installation

Background information and details of the wall construction and soil profile were obtained from the report prepared by Thomas L. Brown Associates for the District of Columbia Department of Public Works (1998). Figure 5-17 shows the location of the wall, which was constructed with fifty

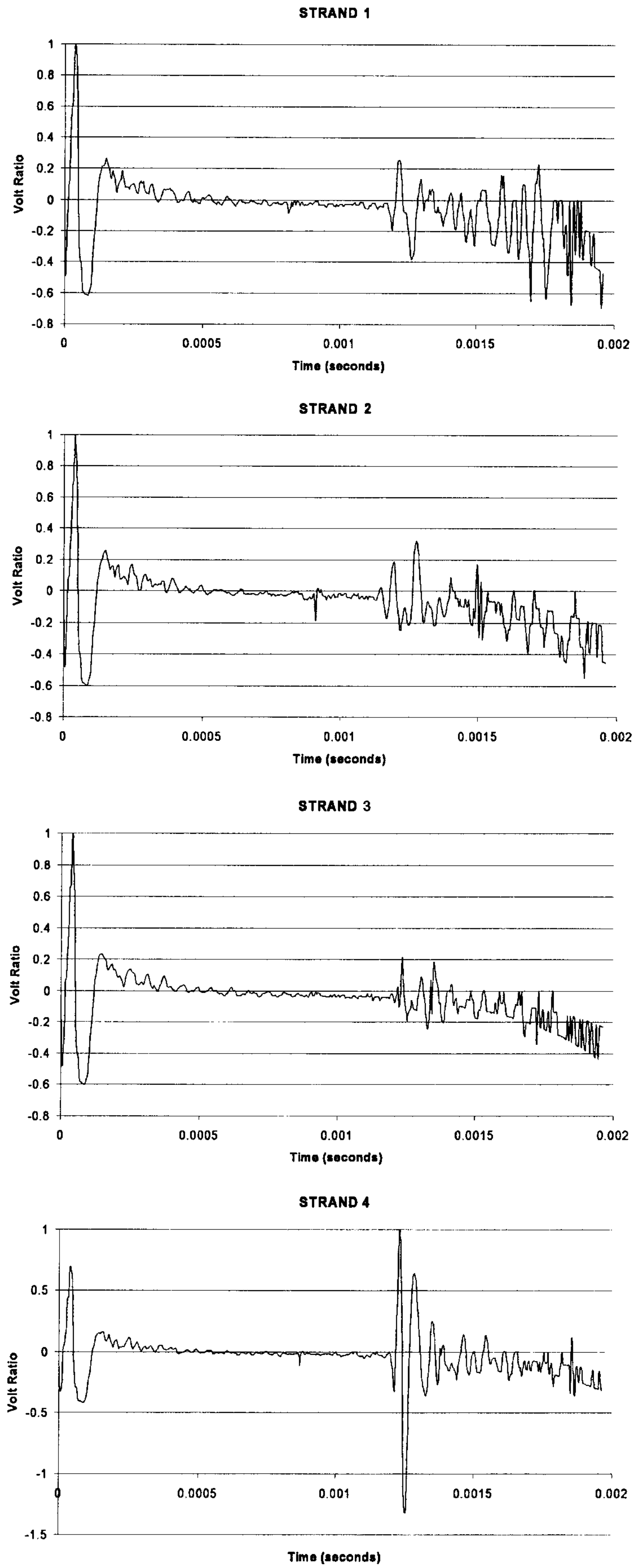


Figure 5-16. Ultrasonic test results from strand elements at Sennet, New York.

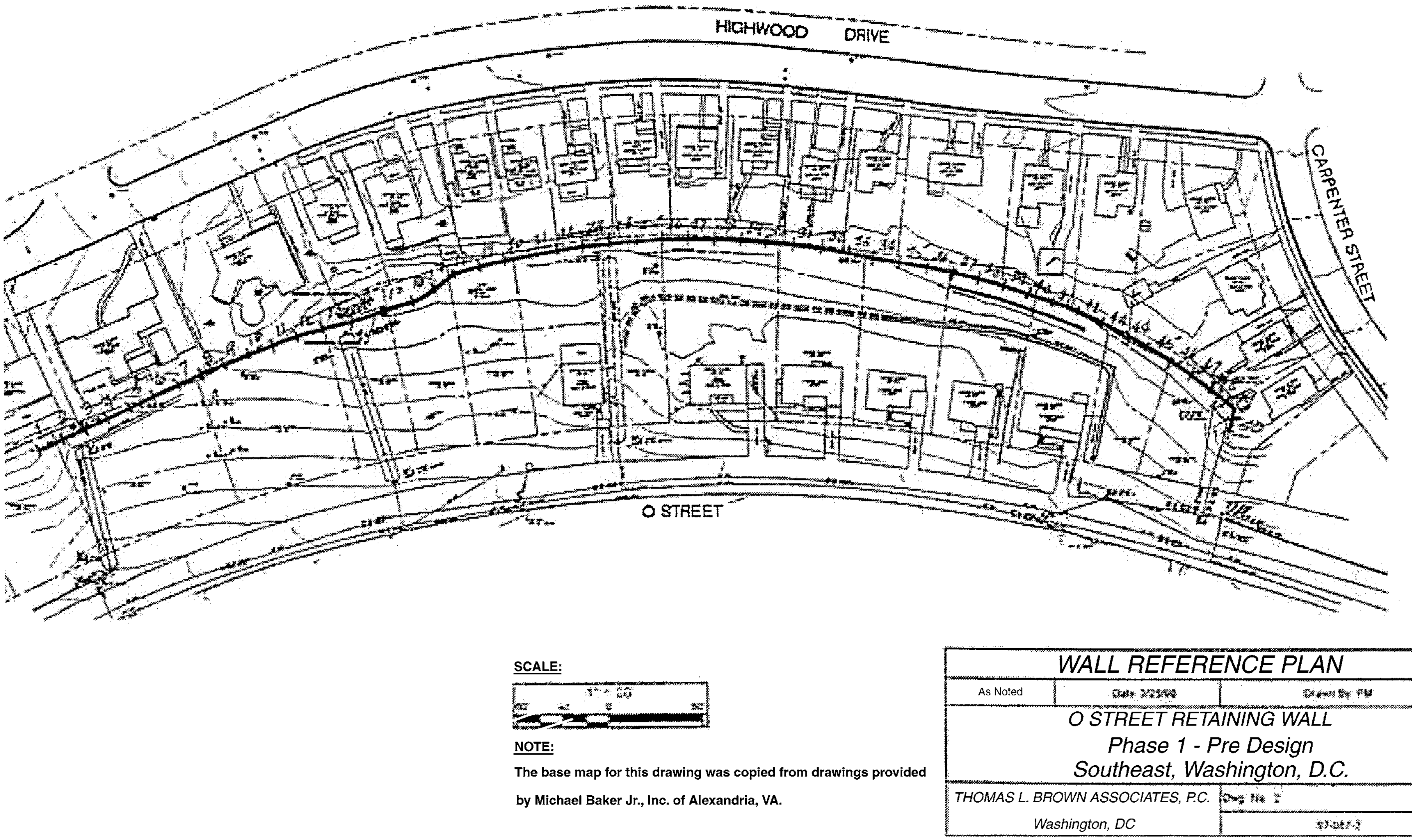


Figure 5-17. Plan view of O Street wall, Washington, D.C.

6.7-m-wide, concrete diaphragm panels, numbered 1 through 50 starting at the west end of the site. The wall is 335 m long and extends most of the length of O Street between Branch Avenue to the west and Carpenter Street to the east. In general, the ground surface at the top and base of the wall rises toward the east end of the wall. The wall supports a sloping ground surface (1V : 3H) and ranges in height from 2.3 m to 9.6 m. The toe of the wall has an embedment depth ranging from 5.2 m to 10.2 m. Table 5-7 shows the elevations at the top, base, and toe along the east and west end of the wall.

Each concrete diaphragm panel is 0.6 m thick and constructed with concrete having a minimum unconfined compressive strength equal to 21 MPa. The horizontal reinforcing steel placed in each wall panel consists of Number 5 bars spaced 457 mm center to center on each face. Vertical reinforcement in the diaphragm wall reportedly consists of Number 5 bars spaced 457 mm center to center along the front face of the wall, and Number 9 bars spaced 406 mm center to center along the back face of the wall. Weep holes, 100 mm in diameter, were drilled along the base of many of the wall panels, and a sloping asphalt swale was constructed immediately uphill of the wall and at the base of the wall to divert any runoff.

Panels are supported with one to three rows of tiebacks for a total of three to seven tiebacks per wall panel. Tiebacks are 32-mm-diameter Dywidag rods that conform to ASTM Specification A722. Tiebacks reportedly were installed at angles ranging from 0° to 16° from horizontal. A 305-mm-diameter, 6-mm-wall-thickness pipe sleeve with a 533-mm-square, 6-mm-thick plate was set into the wall at each anchor bar location. The sleeve was filled with grout and covered with a 356-mm-square, 13-mm-thick anchor plate. Beveled washers, having a diameter of 114 mm, and anchor nuts were installed at the exposed ends of the anchors. The ends of the anchor rod and nut were not protected with an end cap. Along the free length, bars are surrounded by grease, a plastic sheath, and an 8-in.-diameter grouted annulus. Because the corrosion protection is Class II as defined by PTI (1996), element vulnerability is considered moderate.

5.2.4.2 Subsurface Information

Test borings were advanced at the site as part of the initial investigation and for investigations into the cause of distress observed along the wall and slope. Test borings were advanced along the top of the wall and along its toe. Two fairly well defined soil strata underlie the site.

TABLE 5-7 Wall elevations along O Street wall, Washington, D.C.

Panel No.	Elevation (m)		
	Top	Base	Toe
1 (west)	44.68	42.40	36.14
10	51.39	41.79	36.60
14	50.02	44.38	38.89
28	52.41	46.06	40.87
41	53.07	48.95	38.74
50 (east)	52.77	48.19	42.70

Typically, the first stratum was observed from below the surface topsoils, pavement, and fill materials to elevations ranging from 43 m (west) to 48 m (east) along Highwood Drive. It slopes to elevations from 34 m (west) to 43 m (east) along O Street. Compared with the wall elevations cited in Table 5-7, at most locations the interface between Strata 1 and 2 is located near the dredge line of the wall, becoming close to 3 m below the dredge line at the east side of the site. Stratum 1 consists of interbedded layers of sand, silt, clay, and some gravel. Water, believed to have been perched, was somewhat erratically observed within this stratum.

The second stratum was typically observed to the depth of the explorations. The soils within this stratum are predominantly clays, which exhibit a variety of colors, consistency, and plasticity. Highly plastic and slickensided zones, as well as water-bearing sand lenses, were observed frequently within this stratum between elevations approximately 43 m and 34 m north and south of the wall. Below the clay stratum, sands were observed in the lower depths of several borings.

Soils were sampled and sent to the laboratory for testing, which included moisture content, Atterberg Limits, and gradation analysis. Table 5-8 summarizes the results from testing soils from Strata 1 and 2. Sand layers observed within Stratum 1 were medium to fine grained with 20 to 30 percent passing the Number 200 sieve. The in-situ moisture content was measured to be 6 to 8 percent.

Soil samples from Strata 1 and 2 were collected during January 2000 as part of a subsurface investigation conducted by Gannett Fleming, Inc., in support of plans for the remedial design. Gannett Fleming, Inc., provided several jars of split spoon soil samples representative of soils within Strata 1 and 2. Samples were sent to Geotechnics, Inc., for chemical analysis, including pH, sulfate and chloride content, and resistivity. Results are presented in Table 5-9. Because of the low pH's and resistivity measured for the soils at this site, the corrosion hazard is considered high.

5.2.4.3 Wall Performance History

Shortly after wall construction was completed, several tiebacks failed and stress crack corrosion was identified as the cause. During June 1979, the Washington, D.C., Department of Public Works (DPW) noted that Tiebacks 1A (Panel 1, Tieback A) and 14C were broken, and Tieback 4A was bent and its washer was missing. In response to DPW's concerns about the failed tiebacks, ICOS asked Mueser, Rutledge, Johnston, and Desimone (MRJD) to investigate the problems with broken tiebacks. MRJD (1979) concluded that the high-tensile-strength Dywidag anchor rods were quite rigid and brittle. When these rods were subjected to large strains, or when the loads were not aligned longitudinally with the axis

TABLE 5-8 Soil moisture content and Atterberg limits at O Street, Washington, D.C.

Stratum No.	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
1	19 – 20	23 – 31	17 – 20	3 – 14
2	13 – 22	24 – 65	13 – 25	8 – 43

of the bar, high stress concentrations could result. Those high stress concentrations, coupled with the mild corrosive effects of the natural elements, caused failure of several anchor rods by stress crack corrosion. Several problems with the original installation were also cited as potentially contributing to the observed distress. The anchor assembly did not include a bearing washer with a seat to accept the anchor nut; this could have contributed to slight rotation in the bar, which could also create unacceptable stress levels and stress concentrations. Additionally, an unacceptable level of deformation in the 356-mm-square, 13-mm-thick anchor plate was observed at several anchors, and that occurrence could also be related to anchor failures. Consequently, the MRJD report recommended that each anchor assembly be removed and replaced. With respect to the corrosion issue, the MRJD recommended that the entire anchorage assembly be protected against corrosion by epoxy-based coating of all exposed parts, by protective shield, or by other special devices. Reportedly, ICOS returned during 1980 and performed most of the work in accordance with the MRJD report.

Although the stress crack corrosion problem appeared to be remedied, in 1995 the wall began to show evidence of creep-type failure. Between 1995 and the presentation of the report by Thomas L. Brown, Inc., in 1998, a significant amount of creep-type movements of the wall occurred. The area between Panels 10 and 17 experienced the most distress. Within that distress area, wall movement included as much as 2.44 m of outward (i.e., downslope) wall movement, accompanied by as much as 5° of rotation of the wall panels with respect to the base. Because of this wall movement, Panels 15 and 16 displaced as much as 4 m from their original alignment. Apparently, all bar anchors in these wall panels failed, and the soils downslope of the wall were pushed toward O Street. The toe of the slope failure was evidenced by a bulge that developed along the ground surface approximately 30.5 m downslope of the wall. Immediately behind (i.e., upslope) of the diaphragm wall in the distressed area, the ground had settled as much as 3.05 m, and downslope of the wall several scarps were observed that further demonstrated the downslope movement of the wall.

Portions of the wall near Panels 32–34 displaced, but to a much lesser degree compared with Panels 15 and 16. Although no structural cracking was observed within or between these panels, sinkholes were identified in several areas on the Highwood Drive side of the wall, and soggy at-grade surface conditions were identified at several areas immediately adjacent to the O Street side of the wall.

Because of the observed distress, a remedial investigation was conducted under the direction of T.L. Brown Associates. Instrumentation installed along the wall in 1996 included four monitoring wells, three inclinometers, and a number of survey points.

According to the data collected from the monitoring wells, there was a perched water table behind the wall that fluctuated near elevation 47.3 m. Data also indicated a separate groundwater table below the wall near elevation 33.5 m. Throughout the site, various areas of seepage were apparent from both the face of the retaining wall and existing grades below the wall. Generally, the occurrence of the seeps varies with extent of precipitation, although some seeps are always active.

Inclinometer data were used to estimate the location of a slide plane. Figure 5-18 is a cross section of the wall taken near Panel 32, showing the wall cross section and location of the slide plane. According to the information presented in Figure 5-18, if the slide plane is present, it intersects the free length of the anchor rods at a distance of approximately 6.7 m from the anchor head along the lower tier of anchors and 9.15 m from the head along the top tier of anchors.

A number of factors may contribute to the observed wall distress and condition of ground anchors. According to the ICOS construction drawings, the only provision for precluding or preventing groundwater buildup behind the diaphragm wall was the placement of four weep holes per diaphragm panel. The weep holes were located approximately 150 mm above the excavation line in front of the wall. Many of the weep holes in the diaphragm wall appeared to be clogged and/or ineffective, and many of these weep holes appeared to not have been constructed or were beneath the existing grades on the downhill side of the wall (i.e., the wall had settled more than 150 mm).

TABLE 5-9 Chemical analysis of soils at O Street, Washington, D.C.

Stratum No.	pH	SO_4 (mg/kg)	Cl^- (mg/kg)	Ω (ohm-cm)
1	3.6	11	10.2	Not measured
2	4.1	12.5	1.7	2250

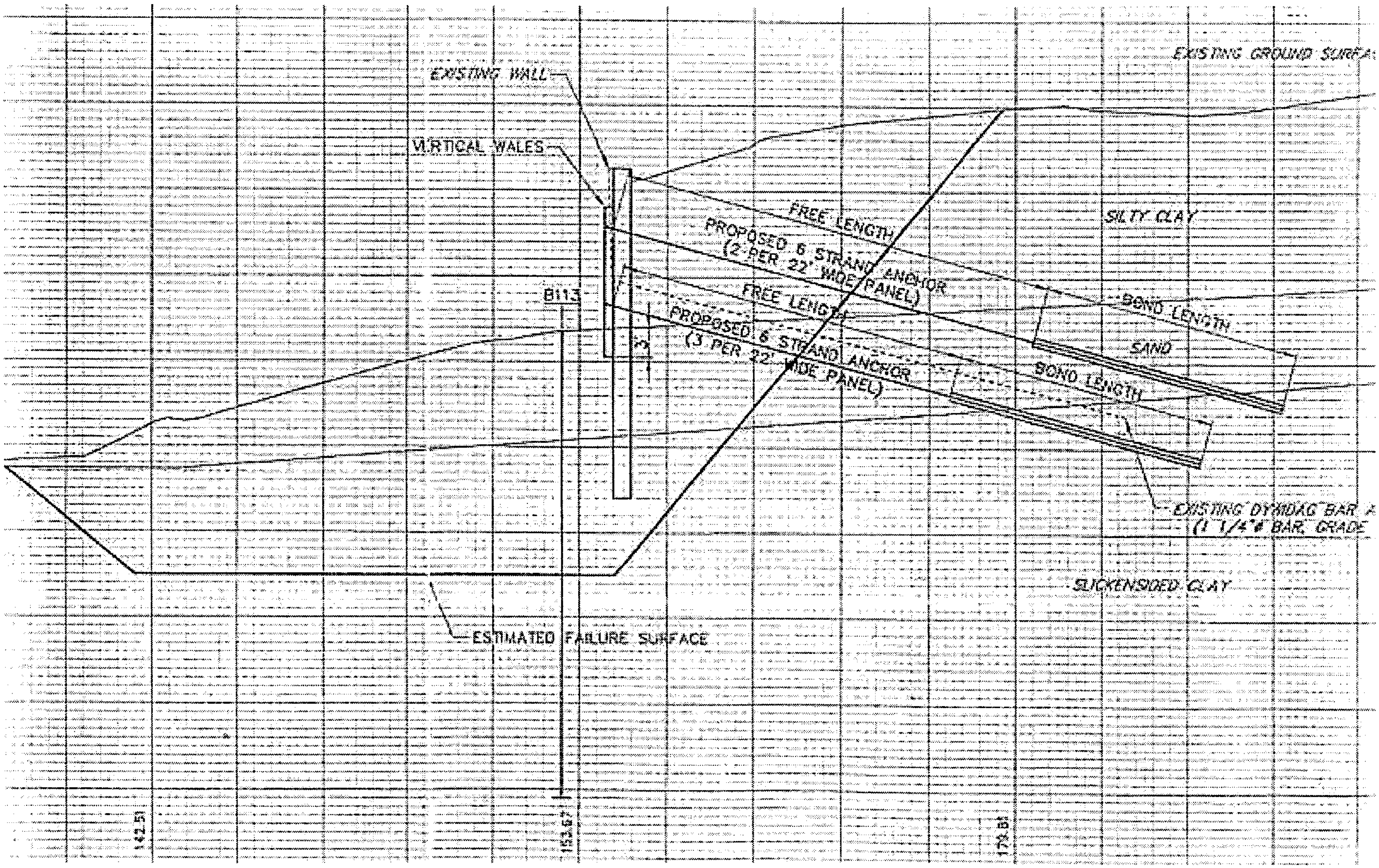


Figure 5-18. Cross section of O Street retaining wall near Panel 32, Washington, D.C.

Each tieback was designed to carry an anchor load of 445 kN. Anchor elements were reportedly 9.0–34.5 m long, and the bonded zone of many of the anchors penetrated into the second clay stratum. Considering that these anchors had been in place in a relatively wet environment for more than 20 years, it was reasonable to speculate that the ground anchors could have been subjected to distress from corrosion. Using available service-life prediction models as discussed in Chapter 4 (Romanoff, 1957; TRB, 1978; Darbin et al., 1986; Elias, 1990), and considering that the ground conditions were aggressive, the estimated loss of cross section for unprotected ground anchors over the 22-year life of the system was estimated to be approximately 27 percent.

In general, with the notable exception of distressed Panels 10–17, the remaining tiebacks appeared to be in place. Exceptions included Anchors 37A and 41B, which were listed as having possibly failed according to visual observations conducted by Thomas L. Brown Associates (1998).

5.2.4.4 NDT Performed at Site

A detailed field evaluation was conducted at the site of the retaining wall. Fifteen bar elements were tested out of 176 tiebacks installed along this approximately 335-m-long, tiedback, concrete diaphragm wall. The locations of the tested elements were evenly distributed along the length of the wall. Twelve of the elements were located near the base of the wall along Panels 2, 33, 41, and 44; three of the elements were located along the upper level from Panels 6 and 7. According to the sampling strategy described in Chapter 4, this sampling interval means that if one is searching for a failure rate of 1 out of 10 elements ($p = 0.1$), there is a 96.6-percent probability that this sampling domain will correctly represent the entire domain (i.e., there is a 4.4-percent chance that more than 1 out of every 10 anchors at the site are distressed), although less than this proportion is represented in the sampling interval. This analysis assumes a sampling error of approximately 10 percent. At the time of the evaluation, the wall was being retrofit so that access to the front of the wall was available via a ramp installed by the contractor. Therefore, the equipment van could be located within several feet of the wall face. The ends of the anchor rods were exposed at the anchor head, and connections could be made without the need to remove any of the existing wall face. The ends of the anchors were cut square with a chop saw and ground smooth prior to testing. Testing consisted of electrochemical measurements, impact tests, and ultrasonic tests.

Electrochemical tests were performed by clamping one end of an eight-gauge wire to the exposed end of the anchor rod, as shown in Figure 5-19. The half cell was pushed into the ground in front of the wall panel. Different placements of the half cell were also evaluated, including using a wet sponge to make contact with the wall face. Measurements of half-cell potential were not sensitive to placement of the half cell.

Figure 5-19. Electrical measurements being made at O Street wall, Washington, D.C.

To polarize the elements and measure the E versus log I relationship, a ground bed was established, and a 6-V source was applied to impress current on the system. For most of the tests, copper-plated rods were used as the ground bed. The rods were pushed into the ground at distances of approximately 15 m from the front of the wall face. A range of current from approximately 0.2 to 200 mA was impressed on the system. In several instances, neighboring elements were used as ground beds, and results were compared with those obtained using the copper-plated rods. Test results did not appear sensitive to the type of ground bed employed.

Using the diameter of the elements and the known length obtained from the as-built records, the current required to polarize the elements assuming they are electrically isolated was computed. All of the observed test results indicated that the current required for polarization was much higher than the computed value, and, given the voltage source used at the site, it was not always possible to reach a polarized state. This limitation indicates that the surface area involved in the circuit was large and electrical continuity existed between elements. This information is useful for interpreting the results from half-cell potential measurements of the individual elements. Table 5-10 presents measurements of half-cell potential at the site.

If electrical continuity exists, the potential for galvanic corrosion relates to the difference in half-cell potential between neighboring elements. Given the data presented in Table 5-10,

TABLE 5-10 Half-cell potential observed at O Street, Washington, D.C.

Element No.	E_{corr} (mV)
2a	–348
2b	–348
2c	–299
6f	–373
7e	–391
33a	400
33b	398
33c	–272
41a	389
41b	389
41c	616
44a	–358
44b	–349
44c	–574

the occurrence of corrosion is not likely for elements along Panel 2 because the half-cell potentials for Elements 2a, 2b, and 2c are relatively close. It is likely that corrosion is present along Panels 33 and 44. Apparently, Elements 33c and 44c are corroding because they have the most negative potential relative to neighboring elements, such that they act as anodes. During the corrosion process, anode potentials shift, becoming less negative. Positive potentials observed for Elements 33a, 33b, 41a, 41b, and 41c may be due to previous corrosion and corresponding shift in potential.

Impact test accelerometers were attached to the ends of the anchor rods using glue-mounted, threaded baseplates. The baseplates were mounted off center such that impacts could be applied at the center of the elements. The test sequence on each bar included the use of different impact hammers and methods of applying impact to the ends of the bars. Hammer types included an instrumented, modally tuned hammer, which allowed observation of the hammer response, and a ball peen hammer that was not instrumented. A centering punch was employed for some of the tests to focus the energy near the center of the element face. Impact was applied in both the longitudinal and transverse directions. The repeatability of the tests was observed by performing at least three impacts for each case (e.g., three impacts with a ball peen hammer and punch and three impacts for the ball peen hammer with no center punch). Test results obtained from using the ball peen hammer and punch will be discussed here because this technique appeared to direct the highest energy down the axis of the element, and reflections could be observed for longer time intervals.

Figure 5-20 displays a typical impact test result for a bar element tested at the O Street site. Reflections are observed from two locations at relatively long distances from the face of the element. Figure 5-21 is a schematic of the bar installation and indicates points along the length of the bar where wave reflection and refraction may occur because of a change in geometry. The first reflection is presumed to be from the end of the unbonded (i.e., free) length of the bar, where the plastic sheath is terminated, and the return signal from the refracted wave is from the end of the bar. The impact test results in Figure 5-20 indicate that Bar 33c has an unbonded length of approximately 12 m and a total length of approximately 21.5 m. Construction records from the installation of the elements document that the length of the bar is approximately 20 m, which compares well with the length indicated from the impact test. Given the total and the unbonded length of the element, the bonded length is computed as approximately 9.5 m.

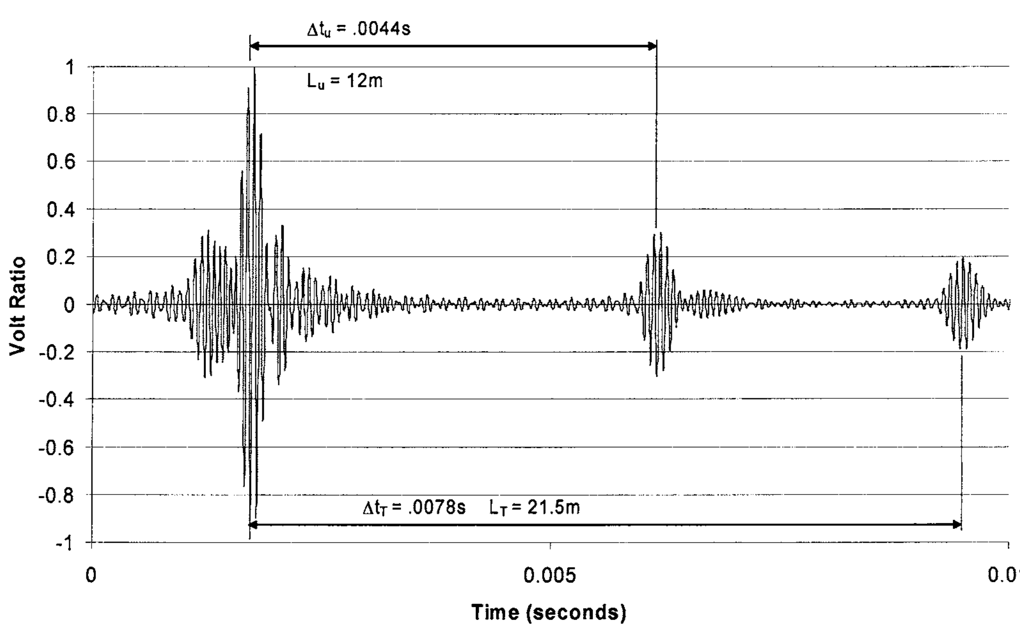


Figure 5-20. Typical impact test results from bar at O Street site, Washington, D.C.

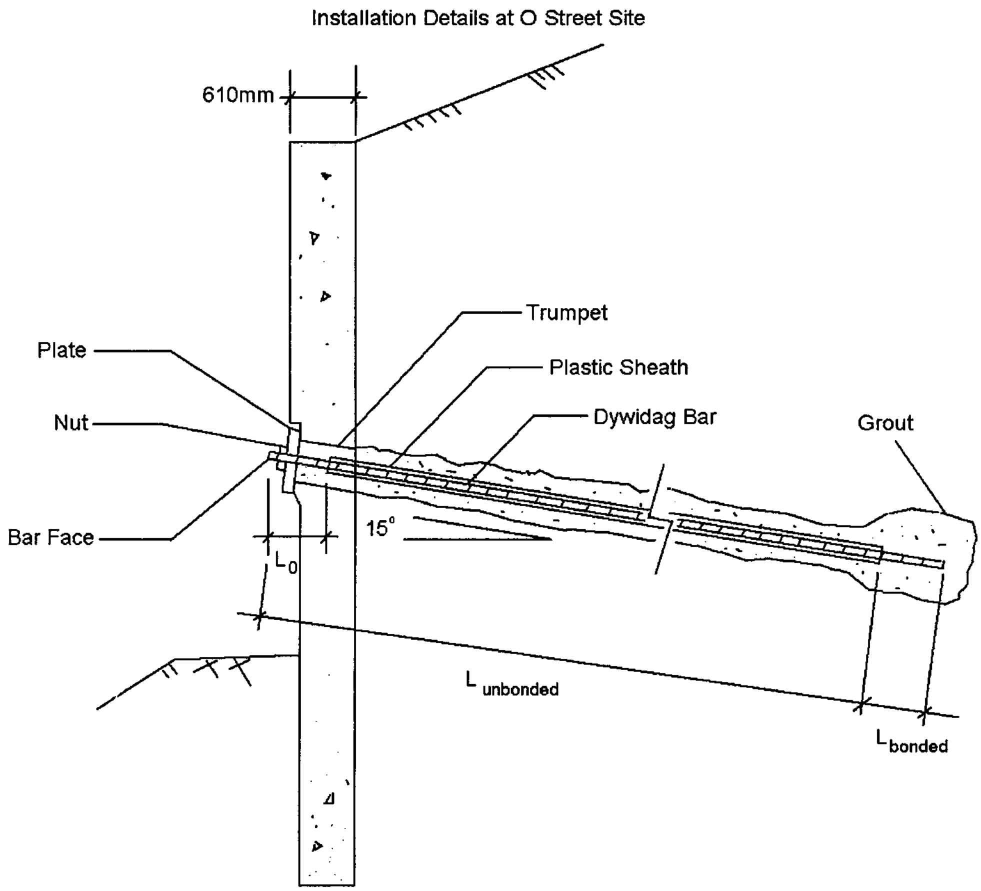


Figure 5-21. Schematic of bar element installed at O Street, Washington, D.C.

Table 5-11 is a summary of results from impact tests performed on 12 bar elements at the O Street site. For each element tested, the unbonded and total lengths observed from the test results are presented, along with the computed bonded length and the documented length of the bar obtained from the construction records. Observed bar lengths are within 5 percent of the documented lengths of the bar elements. However, for Bar 33a, the research team did not observe reflections corresponding to the end of the bar. This lack of visible reflections may be due to a severe loss of cross section, fracture of the bar within the bonded zone, or severe cracking of grout within the bonded zone.

As described in the condition report for the wall prepared prior to the NDT evaluation, Panel 33 is damaged and is displaced relative to its original position. Therefore, it is very likely that some element distress exists at this location.

Table 5-11 also indicates that Bar 41a may be distressed because the length of 4.5 m does not correspond very well to the anticipated length of the unbonded zone. If 4.5 m were the correct length of the unbonded zone, the length of the bonded zone would be computed as 11.5 m, which is several meters longer than the bonded lengths for other elements determined from the impact test.

The interpretation of test data from the O Street site demonstrates how distress is indicated in the test results. Although the severity of the distress is not quantified, the fact that a reflection is not evident from features at distances corresponding to observations from other, nearby elements is considered an indication of an anomalous condition along the length of the bar element.

Figure 5-22 presents typical results from UT performed on bar elements at the O Street site. The high-frequency signals transmitted to the bar are attenuated within a relatively short distance compared with results obtained from impact testing. However, the test can be useful for observing features within the first few meters from the face of the element. Figure 5-22 shows that a first reflection is observed at approximately 384 mm from the face of Bar 6D. This observation is assumed to correspond to the point within the trumpet assembly where the plastic sheath begins, designated as L_0 in Figure 5-21.

Table 5-12 summarizes the results from UT performed on 15 bar elements tested at the O Street site. Measurements of L_0 range from 229 mm to 439 mm, with an average L_0 of 343 mm. This range seems reasonable given that the length of the trumpet assembly is approximately 600 mm.

TABLE 5-11 Results from impact tests at O Street, Washington, D.C.

Bar No.	Measured from Impact Test			From Construction Records
	$L_{unbonded}$ (m)	L_{total} (m)	$L_{bonded} = L_{total} - L_{unbonded}$ (m)	L_{total} (m)
2a	5.5	10.5	5.0	11.0
2b	6.5	10.5	4.0	11.0
2c	3.5	11.5	8.0	11.0
6d	7.0	13.0	6.0	14.0
6f	4.5	11.0	6.5	11.0
33a	5.5	15.0	--	20.0
33b	5.5	11.0	5.5	11.0
33c	12.0	21.5	9.5	20.0
41a	4.5	16.0	--	15.5
41b	16.5	24.5	8.0	23.0
44a	18.5	27.5	9.0	29.0
44b	8.0	12.5	4.5	12.0

No impact test data for Bars 41c, 44c, and 7e.

Ultrasonic test measurements are compared with results from the impact test, where L_0 is determined from the frequency response of the bar and application of Equation 3-2(a). The range of L_0 measured from the impact test is 256 mm to 488 mm, with an average of 389 mm. Generally, the measurements from UT and from impact testing are within ±75 mm.

The test results presented in Table 5-12 demonstrate that the ultrasonic test method is useful for detecting features that are relatively close to the face of the element and that results from UT compare reasonably well with those obtained from the impact test.

5.2.4.5 Conclusions from NDT

Electrochemical tests performed at the O Street site indicate that corrosion may be occurring because of galvanic action between some of the elements. Panels 33, 41, and 44 were identified as areas where corrosion may be occurring or has occurred in the past. Further evidence of corrosion is suggested by rust stains observed on the face of these panels, and the rust stains could be seen directly beneath the bars at some locations.

According to results from impact testing, 2 out of 12 bars tested appear to be distressed. The distressed bars are iden-

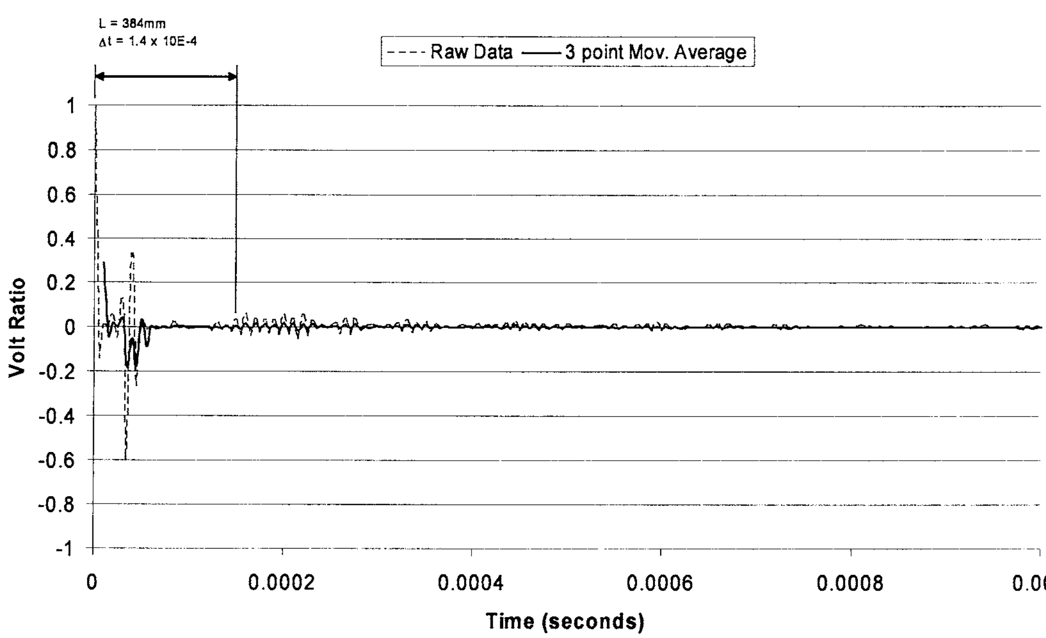


Figure 5-22. Typical ultrasonic test results from bar element at O Street, Washington, D.C.

TABLE 5-12 Results from ultrasonic tests at O Street, Washington, D.C.

Bar No.	Observed L_o (mm)		Difference (mm)
	Ultrasonic Test	Impact Test	
2a	384	439	55
2b	305	378	73
2c	412	466	54
6d	384	461	77
6f	305	343	38
7e	439	384	–55
33a	305	259	–46
33b	305	354	49
33c	229	305	76
41a	439	454	15
41b	229	256	27
41c	439	N.M.[1]	N.M.
44a	305	488	183
44b	229	N.M.	N.M.
44c	439	476	37

[1] Not measured.

tified as Bars 33a and 41a. Bar 33a appears to be distressed along the bonded length; Bar 41a appears to be distressed along the free length.

For most of the elements, impact test data displayed a reflection corresponding to the known length of the bar element. This display demonstrates that the bars are not severed or subject to such severe distress that wave energy cannot be transmitted to the end of the bar. Along Panel 33, where some deformation was observed at the wall panel, the bar elements most likely failed in shear within the bonded zone, along the interface between the grout and soil. It is not clear whether the shear resistance was overcome because of a reduced shear resistance along the bonded length or because load transferred to the wall face was not anticipated in the original design. The source of the reflection, observed approximately 5 m from the known end of Element 33a, may have been from fractured grout or from a kink in the bar along the bonded length.

Impact test data also exhibit a reflection assumed to roughly correspond to the beginning of the bonded length of the element. However, at many locations, this is where inclinometer data approximately indicate a slip surface behind the wall. Therefore, this reflection may also correspond to the location of a bend, or kink, in the bar due to relative movement of soil on either side of the slip surface.

Half-cell measurements conducted on Bar 41a indicate that corrosion may have occurred, and so an early reflection may indicate a loss in cross section from corrosion along the free length of Bar 41a. However, given the history of soil movement at the site, the early reflection may also be from a kink or bend along the free length.

Data obtained from the ultrasonic test indicate that conditions within the trumpet assembly are as expected given the installation details described in the engineering reports. Results obtained from the ultrasonic test are consistent with results from the impact test.

5.2.5 Ellenville, New York

Rock bolts support a large rock cut along the alignment of NYS Route 52 for a distance of approximately 2.2 km between Milepost Markers 1,119 and 1,132 (1 mi = 1.67 km). The rock slope is supported with up to three rows of rock bolts, as shown in the photograph in Figure 5-23 and as shown schematically in Figure 5-24. Several benches were cut along the face of the rock slope, and rock bolts were installed through the benches. The rock bolt anchorages extend below the bedding joints passing beneath the base of each bench.

There are hundreds of rock bolts at this site, including older expansion-shell, mechanical-anchorage-type and newer resin-grouted bolts. Bethlehem Steel manufactured the 19-mm-diameter, expansion-shell-type rock bolts, conforming to ASTM Specification A306 (now A675) Grade 80 standards. These bolts have a minimum ultimate tensile strength of 240 kN and were installed in 1972. On February 24, 1992, a rockslide occurred involving 475–650 m^3 of rock that had been stabilized with these bolts.

Subsequent to the rockslide, expansion shell anchorages were replaced with resin-grouted bolts. Resin-grouted bolts are 25-mm-diameter, Williams R71, Grade 150, all-thread

Figure 5-23. Rock slope with benches and different levels of rock bolts at Ellenville, New York.

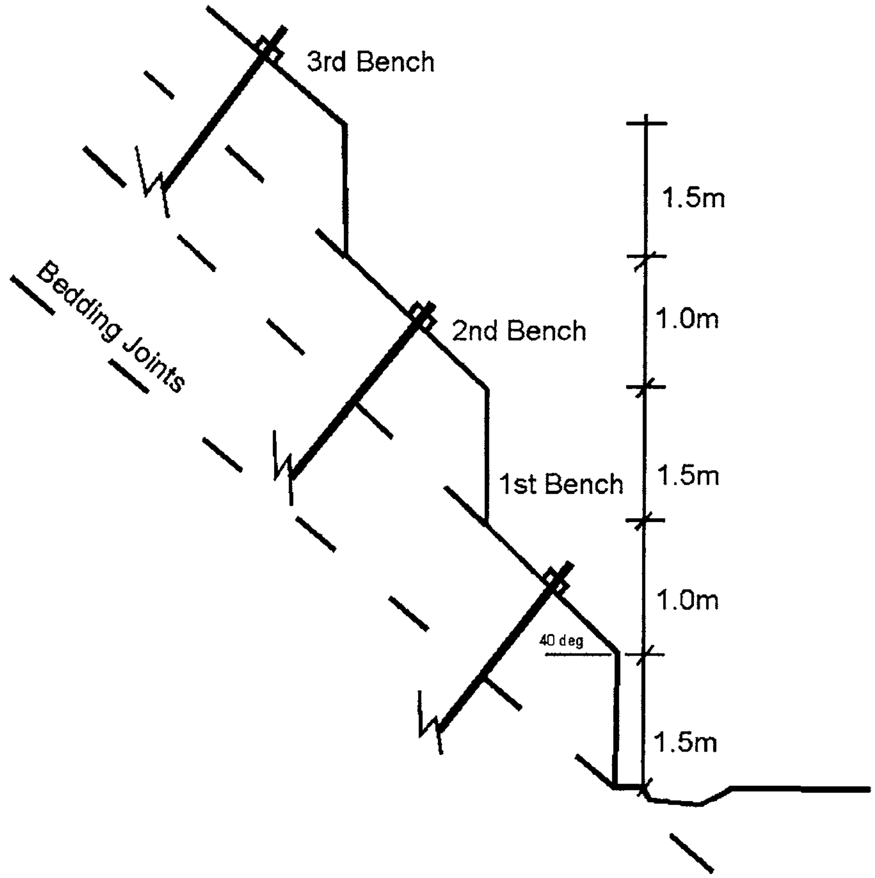


Figure 5-24. Schematic cross section of rock slope at Ellenville, New York.

bars with a minimum ultimate tensile strength of 570 kN. The newer, resin-grouted bolts were installed between 1992 and 1999.

Although no longer in service, many of the expansion shell rock bolts were left in place, allowing evaluation of both expansion shell and resin-grouted rock bolts at this site. The subsurface conditions, including the characteristics of the rock mass, are described in the next section, followed by information collected by NYSDOT after the rockslide and a description of NDT performed at this site.

5.2.5.1 Subsurface Information

Rock bolts at the slope in Ellenville, New York, are installed within a sandstone conglomerate, which is part of the Shawangunk formation. The rock mass has steeply dipping bedding planes with a dip direction approximately perpendicular to the alignment of NYS Route 52. Open bedding joints, having a dip angle of approximately 40°, are spaced vertically at approximately 1- to 1.5-m intervals.

The bedding joints are infilled with a light brown, clayey sand. Samples of infilling were collected by hand during the site visit and were sent to the geotechnical testing laboratory for determination of moisture content, grain size distribution, Atterberg limits, and chemical analysis (including pH, resistivity, and concentrations of sulfates and chlorides). Results from soil testing are summarized in Table 5-13. Using the information in Table 5-13, the infill material is classified by the USCS as clayey sand with gravel (SC). Although the measured pH is considered low, the infilling does not present a high corrosion hazard because of its relatively high electrical resistance.

A complete chemical analysis was performed on groundwater from one well that tapped the Shawangunk formation, located approximately 22 mi to the southwest of the Ellenville site (Frimpter, 1970). The pH of the water at the time the original fieldwork was done (June 1957) was 6.0. In March 1995, a field determination of pH by Johnston (1996) from 10 readings of water dripping from the slope were taken at various locations using litmus paper. Readings obtained were a consistent 6.0.

5.2.5.2 Condition of Rock Bolts Retrieved from Rockslide

As reported by Johnston (1996), twenty-two 19-mm-diameter, expansion-shell-type rock bolts were recovered from the slide area. Laboratory measurements performed on

TABLE 5-13 Rock joint infilling test results in Ellenville, New York

Sieve Size (U.S. Standard)	% Passing
50 mm	100
#4	73
#40	42
#200	31
Atterberg Limits	
Liquid Limit	28 %
Plastic Limit	20 %
Plasticity Index	8 %
Chemical Analysis	
Measured Moisture Content (w%)	15.4
Resistivity (ohm-cm)	10400 (min. at w = 27.3%)
pH	3.1
Sulfate Content (mg/kg)	5.5 mg/kg
Chloride Content (mg/kg)	< 10 mg/kg

each included photographs, length, and diameter of rock bolts and diameter and depth of any visible pit on the surface. Eight of the bolts were subjected to tensile strength tests.

All bolts had evidence of corrosion. The average loss of diameter of the bolts was 2 mm, which corresponds to an average loss of approximately 0.1 mm/year.

Estimated loss of load-carrying capacity ranged from 16 percent to 27 percent. None of the bolts exhibited a yield point, indicating that hydrogen embrittlement, or some alteration of the steel crystalline lattice structures, had embrittled the steel.

No direct correlation was found between loss of material (i.e., cross section) and loss of load capacity.

5.2.5.3 NDT Performed at Site

Two sections of the alignment, designated as Section I and Section II, were selected for the NDT evaluation. A wide shoulder and catchment ditch were available along the roadside at these locations. This availability allowed the evaluation team to secure a work area along the shoulder of the highway.

The sections were approximately 0.5 km apart, and each had a length of approximately 30 m. Section I was close to Milepost 1119, encompassing approximately 30 bolts (including 20 resin-grouted bolts and approximately 10 expansion-shell-type bolts). Section II was between Mileposts 1,122 and 1,123, encompassing approximately 22 bolts (including 15 resin-grouted and 7 expansion-shell-type bolts).

The bolt locations were referenced to stationing that was previously established by NYSDOT along the alignment of the highway. Stations were marked in 20-m intervals along the west shoulder of the highway. Station 14+540 is close to Milepost 1,119. Section I is between approximate Stations 14+540 and 14+570; Section II is between approximate Stations 15+100 and 15+120.

The top of the ditch at the base of the rock slope was used as a datum for referencing rock bolt elevations. As shown schematically in Figure 5-24, the lower bench is approximately 1–2 m above the ditch; the second bench is approximately 3–5 m above the ditch; and the third bench is approximately 6–7 m above the elevation of the ditch. Many of the bolts along the first bench were accessible while standing at ground level. Those along the level of the second bench required the use of a ladder. Access to many of the bolts along the third bench required mountain climbing techniques using ropes and special harnesses.

Sixteen bolts were tested from Section I, and 13 bolts were tested from Section II. Tables 5-14 and 5-15 summarize the locations and the tests performed on the older expansion shell and newer resin-grouted anchorages, respectively. Bolt locations were numbered and bearing plates labeled with a white permanent marking stick during the site visit.

Expansion shell bolts were tested near companion resin-grouted bolts. Expansion shell bolts were identified with a number followed by the letter "a." The number is correlated with the nearby resin-grouted bolt (e.g., expansion shell Bolt 2a is near resin-grouted Bolt 2).

The research team observed that Bolt 22a, one of the expansion-shell-type rock bolts, was noticeably distressed. The bolt head was not in contact with the bearing plate, and the bolt appeared to be loose. The team extracted the upper portion of the bolt and measured its length as approximately 1 m.

As a check on the repeatability and consistency of the testing, tests were repeated on one of the resin-grouted bolts on different dates. These tests were performed on Bolt 8 from Section I, designated as Bolts 8 and 8R.

TABLE 5-14 Summary of expansion shell rock bolts tested in Ellenville, New York

Section	Bolt No.	Electro-chemical Test	Impact Test	UT	Approx. Location Station (m)	Approx. Location El.[1] (m)
I	2a	Yes	No	No	14+546	3.8
II	16a	Yes	No	No	15+098.5	2.13
II	18a	Yes	No	No	15+104.7	2.13
II	21a	Yes	Yes	No	15+112.6	2.44
II	22a	Yes	Yes	No	15+111.5	5.48

[1] Elevation is with respect to the top of pavement drainage ditch.

TABLE 5-15 Summary of tests on resin-grouted rock bolts in Ellenville, New York

Section	Bolt No.	Electro-chemical Test	Impact Test	UT	Approx. Location	
					Station (m)	El.[1] (m)
I	1	Yes	Yes	Yes	14+540	2.9
I	2	Yes	Yes	Yes	14+545.7	2.9
I	3	Yes	Yes	Yes	14+547.6	3.0
I	4	Yes	Yes	Yes	14+550.8	3.0
I	5	Yes	Yes	Yes	14+545.7	5.6
I	6	Yes	Yes	Yes	14+549.2	3.8
I	7	Yes	Yes	Yes	14+560.4	2.1
I	8	Yes	Yes	Yes	14+565.9	1.5
I	9	Yes	Yes	Yes	14+571.4	2.0
I	10	Yes	Yes	Yes	14+540	7.0
I	11	Yes	Yes	Yes	14+550.8	3.1
I	12	Yes	Yes	Yes	14+560.44	3.2
I	13	Yes	Yes	Yes	14+565.6	2.7
I	14	Yes	Yes	Yes	14+543.1	7.0
I	15	Yes	Yes	Yes	14+555.6	4.3
I	8R[2]	Yes	Yes	Yes	14+565.9	1.5
II	16	Yes	Yes	Yes	15+097.7	2.1
II	17	Yes	Yes	Yes	15+100	1.5
II	18	Yes	Yes	Yes	15+105.3	2.4
II	19	Yes	Yes	Yes	15+108.1	1.5
II	20	Yes	Yes	Yes	15+111.6	2.4
II	21	Yes	Yes	Yes	15+113.2	2.3
II	22	Yes	Yes	Yes	15+112.6	5.5
II	23	Yes	No	Yes	15+114	3.7
II	24	Yes	No	Yes	15+115	3.7

[1] Elevation is with respect to the top of pavement drainage ditch.
[2] Bolt 8R is a repeat of Bolt 8.

Electrochemical tests. The moist and porous rock formation, which included the presence of dripping, wet, infilled bedding joints, acted as an electrolyte for performing electrochemical tests. The half cell was placed within the infill of a seam intersecting the rock bolt, at the base of the rock slope, within a nearby water-filled core hole (if available), or on the surface of the rock near the bolts. Responses from the same bolt using different positions of the half cell were observed; in most instances, results were not sensitive to the position of the half cell.

Polarization measurements (E versus log I) were performed using either a distant rock bolt or copper rods as the ground bed. Copper rods could be inserted only where soft soil was present at the base of the rock slope. For Section I, Bolt 2 was used as ground bed for testing Bolt 1; subsequently, Bolt 1 was used as ground for testing Bolts 2, 2a, 3, 4, 5, 6, and 8R. Bolt 4 was used as a ground for Bolts 7, 8, 9, 10, 11, 12, 13, and 15. Bolt 7 was used as a ground for Bolt 14. For Section II, copper rods were used as the ground bed.

Results for expansion shell anchorages. These results are presented in Table 5-16, including half-cell potential and the observed polarization current, I_p. The half-cell potential for Bolt 22a is more positive compared with the other measurements. Given that the rock bolts are electrically isolated from each other, more positive half-cell potential measurements indicate a higher likelihood that corrosion has occurred. Half-cell potentials for Bolts 2a, 16a, 18a, and 21a are relatively close, ranging from –399 mV to –475 mV. These half-cell potentials are close to the borderline of –500 mV separating the range of potentials associated with rusted and nonrusted low-carbon steel in neutral soils and water. Therefore, according to the half-cell measurements, although corrosion may have occurred for other elements tested, more severe corrosion occurred along Bolt 22a.

TABLE 5-16 Summary of electrochemical test results for expansion shell anchorages in Ellenville, New York

Element No.	E_{corr} (mV)	I_p (mA)
2a	–399	0.316
16a	–475	0.220
18a	–402	0.220
21a	–473	0.316
22a	–136	0.100

Observed polarization currents are relatively low compared with those corresponding to the length of the elements. According to the polarization current requirement of 21 mA per m^2 of surface area, for a 19-mm-diameter element, the polarization current is approximately 1.25 mA/m of element in contact with the surrounding electrolyte. The observed range, between 0.1 mA and 0.316 mA, corresponds to 80–250 mm of contact length. Because the joint infilling material is more conductive than the surrounding rock, the contact between the element and the joint infilling material is assumed to dominate the response along length of the bolts. The results for Bolts 2a, 16a, 18a, and 21a indicate that the bolts are in contact with joint infilling material for a length of 175–250 mm, which is approximately equal to the thickness of infilled seams. Bolt 22a appears to have a shorter contact length. This phenomenon may be explained by the fact that Bolt 22a is severed at approximately 1 m from its face. If the severed end of the bolt does not fully penetrate the infilled seam, a lower contact length will be measured. Therefore, the lower polarization current measured for Bolt 22a may indicate that the bolt is broken.

Results for resin-grouted anchorages. These results are presented in Table 5-17. Bolt 2 has a very high half-cell potential compared with the other measurements. This high half-cell potential could indicate that corrosion is taking place, or it may be a manifestation of the use of Bolt 2 as a ground bed prior to performing the half-cell measurement. During the E versus log I test, the impressed current affects the potential of the ground bed from the migration of positive ions, which tends to increase its potential. Test results for other bolts used as ground beds—including Bolts 1, 4, and 7—are not similarly affected because testing was completed on these elements prior to their use as ground beds.

The observed half-cell potentials for most of the tested bolts (Numbers 1, 4–7, 9, 10, 11, 13, 14, 15, and 17–24) were within the range (–800 mV < E_{corr} < –500 mV) associated with nonrusted, low-carbon steels. The observed half-cell potentials for Bolts 3, 8, 12, and 16 were between –200 mV

TABLE 5-17 Summary of electrochemical test results for resin-grouted anchorages in Ellenville, New York

Element No.	E_{corr} (mV)	I_p (mA)
1	–556	0.33
2	–238	0.47
3	–416	0.26
4	–501	0.25
5	–617	0.23
6	–661	0.30
7	–539	0.37
8	–305	0.44
8R	–297	I[1]
9	–600	0.63
10	–503	0.63
11	–692	0.32
12	–491	0.43
13	–612	0.68
14	–565	0.41
15	–664	0.17
16	–494	0.17
17	–570	0.21
18	–688	0.20
19	–533	0.31
20	–545	0.21
21	–631	0.35
22	–610	0.47
23	–523	0.34
24	–638	0.54

[1] Inconclusive.

and –500 mV, which indicates that corrosion may have occurred at these locations. Bolt 8 had the highest measured half-cell potential (except for Bolt 2); on the basis of the readings taken on two different dates, this result was very repeatable. Because of this observation, it appears that the occurrence of corrosion is more likely for Bolt 8 relative to the other resin-grouted bolts tested.

Compared with half-cell measurements for expansion shell anchorages, generally lower half-cell potentials were observed for resin-grouted bolts. The free lengths of the expansion shell bolts were not grouted, but the resin-grouted bolts are surrounded by grout for most of their lengths. For the latter, the grout may afford some corrosion protection, and the expansion shell bolts are 20–27 years older than the resin-grouted bolts. Therefore, it is reasonable to conclude that, at this site, the occurrence of corrosion is more likely for the expansion shell bolts.

Polarization currents observed for the grouted anchors range from 0.17 mA to 0.68 mA. For a 25-mm-diameter element, the polarization current is approximately 1.68 mA/m, so the corresponding contact lengths range from 120 mm to 405 mm. Similar results were obtained for the expansion shell bolts. Table 5-18 compares contact lengths (L_c) for companion expansion shell and resin-grouted rock bolts.

The comparison of polarization current measurements for Bolts 8 and 8R is inconclusive because a different ground bed was used for the repeat test. Bolt 2 was used as a ground bed for Test 8R, and the maximum current impressed on the

TABLE 5-18 Comparison of contact lengths (L_c) measured for companion expansion shell and resin-grouted rock bolts in Ellenville, New York

Bolt No.	L_c (mm)
2	280
2a	252
16	101
16a	176
18	119
18a	176
21	208
21a	253

system, with the 6-V power supply, was 0.43 mA. For Test 8, Bolt 4 was used as the ground bed, which is relatively closer to Bolt 8, and 0.89 mA was achieved. If the results plotted in the E versus log I space for Test 8R are extrapolated, I_p is approximately 0.43 mA, which compares well with the I_p observed for Test 8.

Impact tests were also performed on both expansion shell and resin-grouted rock bolts. For the impact test, two different methods of attaching the accelerometer to the end of the rock bolts in the field were employed. In the first method, the end of the rock bolt was drilled and tapped to receive the threaded end of the accelerometer base directly. This method is recommended by the manufacturer of the accelerometer. In the second method, a special mounting base was glued to the end of the rock bolt with threads that fit the base of the accelerometer. For Section I, bolts were drilled and tapped, and the mounting base was used in Section II. Bolt 20 was tested with and without the mounting base. No significant difference in the results was observed from testing the same bolt with the different methods of attaching the accelerometer.

The bolt head was impacted with a tack hammer, as well as with a hand-held punch and tack hammer. Some impact testing was also performed with an instrumented hammer. Only a few impact tests were performed on the expansion-shell-type bolts because the ends of bolts have a dish-shaped cap that makes attachment of the accelerometer difficult.

Expansion shell bolts. Figures 5-25 and 5-26 present the time history from impact tests performed on Bolts 21a and 22a, respectively. The response of Bolt 21a appears to include vibration at the bolt head that decays with time, reaching relatively small amplitude at approximately 2 ms. A reflection is apparent in the signal at approximately 2.5 ms. This length is assumed to correspond to the length of the element (6.9 m). In contrast to Bolt 21a, the test results from Bolt 22a display much less signal attenuation, and reflections at time intervals of approximately 0.3 ms are dominant throughout the record. A return signal at 0.3 ms corresponds to a length of approximately 0.8 m, which compares very well with direct measurement of the severed portion of the bolt extracted from the drill hole. These results clearly demonstrate that the impact test correctly identified the location of the severed end of Bolt 22a.

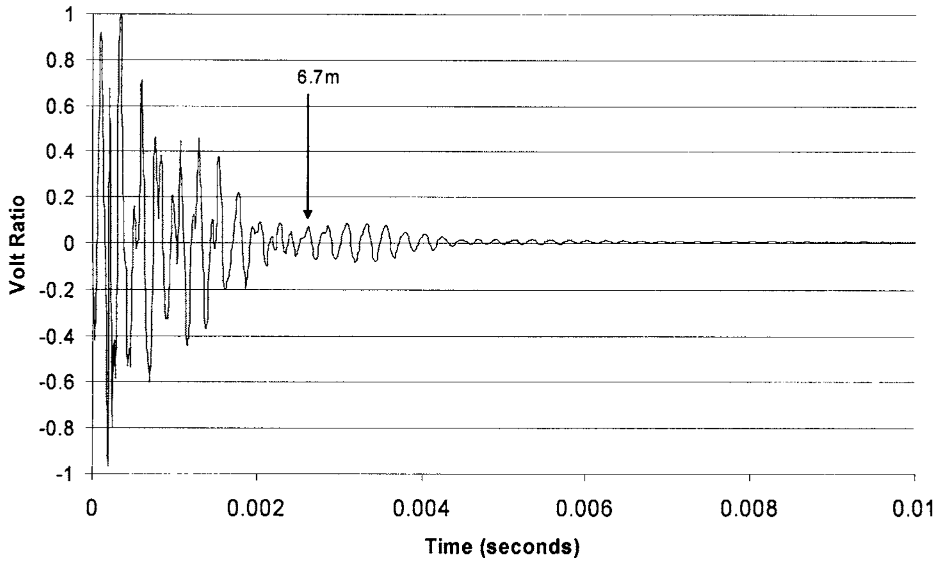


Figure 5-25. Impact test on Expansion Shell Bolt 21a (intact) at Ellenville, New York.

Results for the resin-grouted bolts. Waveforms generated from impact testing contained a number of peak frequencies, as shown by the typical amplitude spectrum presented in Figure 5-27. In general, amplitude spectra observed from different bolts exhibited similar frequency peaks. Table 5-19 is a summary of the range of peak frequencies observed and the presumed sources of these characteristic vibrations. Assuming that the theory of one-dimensional wave propagation can be applied, the length to each reflection source is related to frequency by Equation 3-2(a). Four levels of peak frequency content are described, including low, middle, upper middle, and high. The lower and higher frequency contents correspond to features farther away and closer to the bolt face, respectively.

The length of the bolt corresponds to a relatively low frequency, which exhibits a very small peak within the amplitude spectrum, as shown in Figure 5-27, and is not determined with a high degree of accuracy. The low-frequency range of approximately 450 Hz, shown in Table 5-19, is an average from results of all the tested bolts. The corresponding average bolt length of 6 m is considered reasonable considering the spacing of seams within the rock mass (≈ 1.5 m), the bond length required to achieve the capacity of rock bolts (≈ 4 m), the stock lengths of all-thread bar elements (12 m),

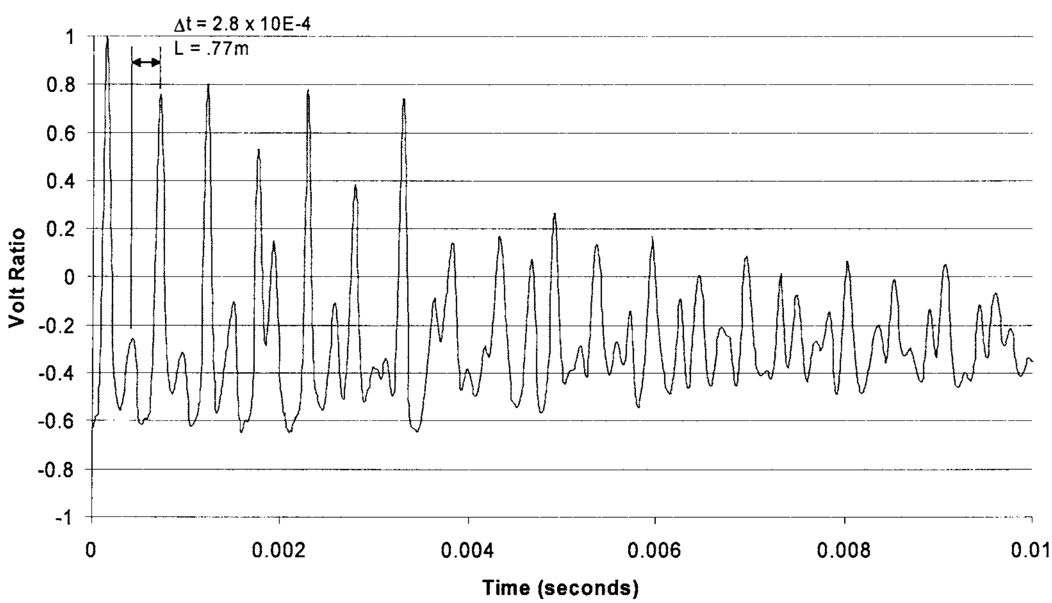


Figure 5-26. Impact test on Expansion Shell Bolt 22a (broken) at Ellenville, New York.

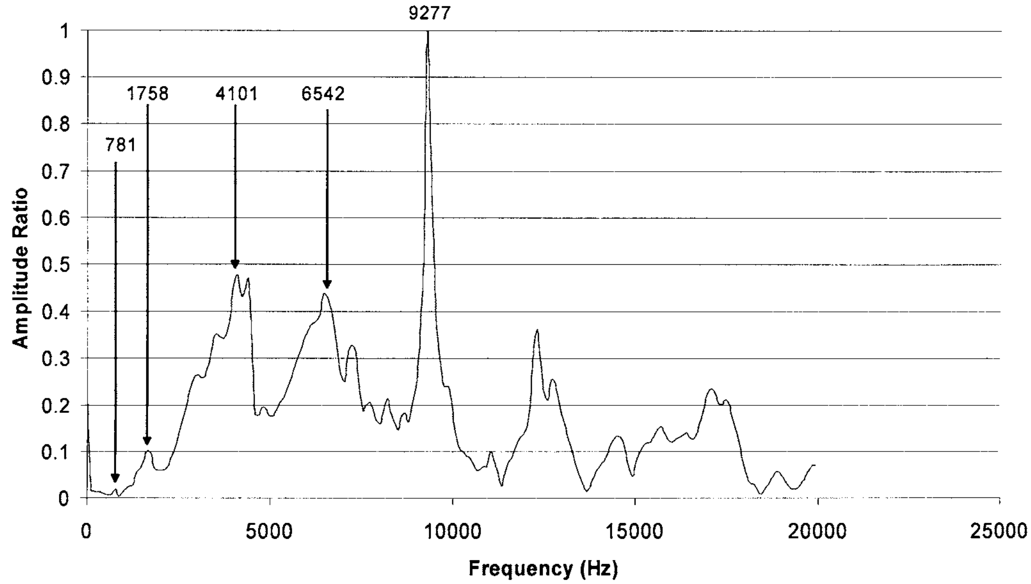


Figure 5-27. Typical amplitude from impact test of rock bolt at Ellenville, New York.

and the fact that couplings are not employed for resin-grouted anchors.

To evaluate the bolt lengths in more detail, the research team studied data in the time domain after filtering the frequency content and presenting a window of the time history surrounding peak frequencies from approximately 7,500 Hz to 12,500 Hz. A typical example of filtered data is shown in Figure 5-28. Two main reflections of the waveform can be seen in Figure 5-28 corresponding to lengths of approximately 1.8 m and 6.3 m. These lengths are presumed to correspond to a reflection from the location of the seam pierced by the rock bolt and from the length of the bolt, respectively. Table 5-20 is a summary of distances to the seam location (L_S) and rock bolt lengths (L_T) observed for all the resin-grouted bolts tested by the impact method.

For Bolt 8, no reflection from the distant end was observed. The repeatability of this result was checked, and Figure 5-29 shows the time histories from the impact tests performed on August 8, 2001 (Bolt 8), and on August 21, 2001 (Bolt 8R). The test results are consistent, showing reflections corresponding to distances approximately 1.2 m from the bolt face. If Bolt 8 has a length of approximately 6 m, similar to other elements at this site, distant reflections from the far end of the bolt may be masked by relatively strong reflections from the seam locations. The test results may indicate (1) distress at the seam location from corrosion or (2) a more abrupt change in geometry of the grout column compared with the other bolts tested at this site.

In addition to the joint seam locations and the ends of the bolts, wave reflections may also occur where the element begins to be surrounded by grout. For resin-grouted bolts, the grout column does not necessarily extend to the top of the drill hole. During construction, resin grout packages containing a predetermined volume of material are inserted into the drill hole. As the resin grout is mixed, some of the non-viscous resin grout may seep into rock joints or seams that intersect the drill hole. Thus, upon completion of the installation, the drill hole may not necessarily be filled with grout to the top, and there may be a gap behind the back of the bearing plate and the top of the drill hole.

For most of the impact test results, the strongest wave reflection was from the beginning of the grout column, and the upper-middle frequency was the dominant frequency within the amplitude spectrum (e.g., f_p = 9,277 Hz in Figure 5-27).

TABLE 5-19 Range of frequency peaks for bolts tested in Ellenville, New York

Frequency Level	Source of Wave Reflection Corresponding to Observed Frequency Range	Peak Frequency Range (Hz)
Low	≈ 6-m length of rock bolt	≈ 450
Middle	Seam location 1–2 m from bolt face	1500–3000
Upper Middle	Beginning of grout column 300–500 mm from bolt face	5000–9000
High	Nut and plate 140–250 mm from bolt face	10000–20000

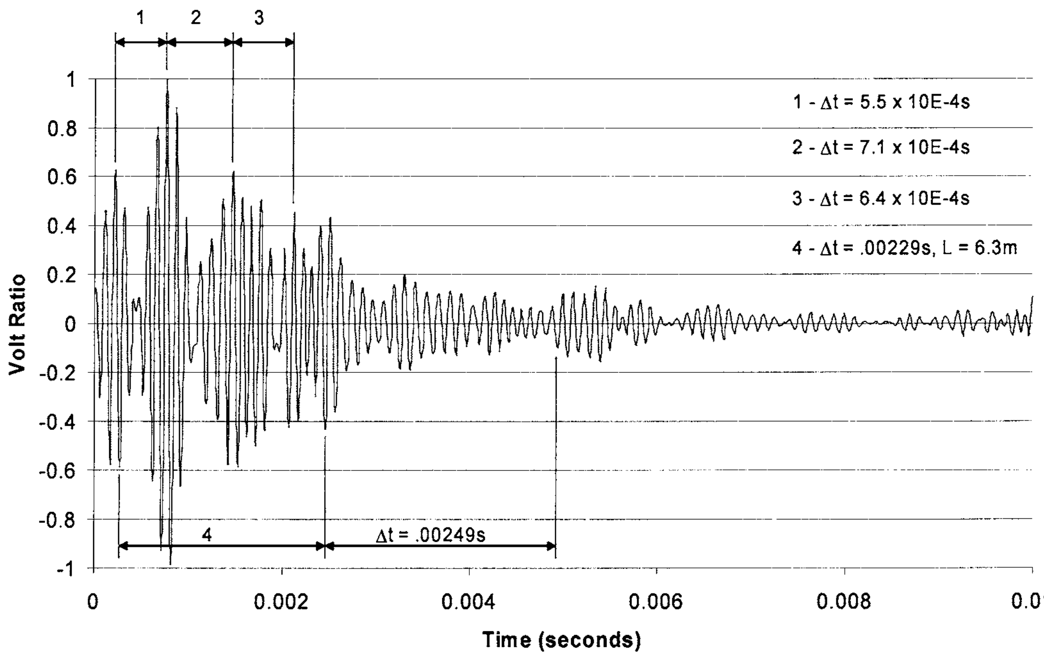


Figure 5-28. Typical filtered time history from impact test on rock bolt at Ellenville, New York.

Table 5-21 summarizes the dominant frequencies observed from amplitude spectra of each bolt tested. The table also shows the length to the top of the grout column, L_g, computed with the dominant frequency. The dominant frequencies observed for Bolts 5, 8, 11, 16, 18, and 19 do not correspond to the range of values inherent in the remaining test results (4,500 Hz < f_p <10,000 Hz). Although not the most dominant frequency, most test results for Bolts 5, 8, 18, and 19 exhibit a peak frequency consistent within the range corresponding to the top of the grout column. For these bolts, the less dominant frequency was used to compute L_g. The presence of a significant feature, beyond the beginning of the grout column, that returns a reflection with a relatively high-energy content may be the source of the lower dominant frequency for Bolts 5, 8, 18, and 19. Such a feature may be a defect along the length of the element or cracking or change in cross section of the grout column at a distant location.

TABLE 5-20 Observed distance to seam location (L_s) and total length of bolt (L_T) from impact tests performed on resin-grouted bolts in Ellenville, New York

Bolt No.	L_S (m)	L_T (m)
1	0.9	7.0
2	2.8	5.5
3	1.2	5.5
4	1.5	5.5
5	1.5	5.2
6	1.4	6.1
7	0.9	4.6
8 & 8R	1.2	–
9	1.1	5.5
10	1.1	4.9
11	1.8	6.3
12	1.0	5.5
13	1.4	4.8
14	1.0	7.3
15	1.4	4.9
16	1.5	6.7
17	2.0	5.6
18	1.8	5.9
19	1.4	5.5
20	0.9	6.9
21	1.5	6.7
22	1.8	6.1

Apparently, a reflection from the grout column was not observed in test results for Bolts 11 and 16. For these bolts, the grout column may extend to the back of the plates, and no gap exists behind the back of the plate and the beginning of the grout column.

Ultrasonic tests. Ultrasonic tests were performed only on resin-grouted bolts. Waveforms obtained from the ultrasonic tests showed considerably more damping than did waveforms from the impact test. Most of the energy from waves propagating within the grout column is dissipated before being reflected back to the transducer. The fact that this dissipation occurs indicates the integrity of the grout along the length of the bolts. Typical results from the ultrasonic test are presented in Figure 5-30. Reflections were observed corresponding to the beginning of the grout column and from the seam location. Table 5-22 compares between results obtained from UT with results from impact testing. The comparison in Table 5-22 is presented in terms of the ratio of the ultrasonic test results to the impact test results for observations of L_g and L_s.

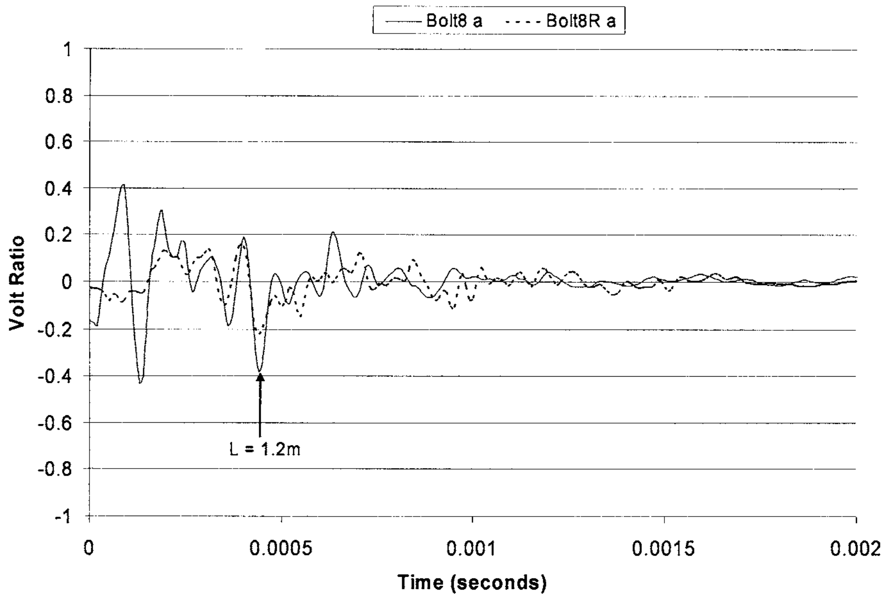


Figure 5-29. Impact tests on different dates on Bolt 8 at Ellenville, New York.

The average and standard deviation of the ratios presented in Table 5-22 is 0.96 and 0.19, respectively. These results demonstrate that the ultrasonic test yields similar results as the impact test results for reflections observed from locations within approximately 2 m from the face of the element.

Although impact tests were not performed on Bolts 23 and 24, the ultrasonic test data indicate that L_g for these elements are 275 mm and 550 mm, respectively; and L_s is 1.8 m and 1.1 m, respectively.

Impact test results performed on Bolt 8 on August 7, 2001, and August 21, 2001, are similar, and the results exhibit the same values for L_g and L_s.

TABLE 5-21 Summary of most dominant frequencies for bolts tested in Ellenville, New York

Bolt No.	Dominant Frequency	L_g (mm)
1	9277	300
2	6380	431
3	6641	414
4	5468	503
5	2930	429[1]
6	5469	503
7	8984	306
8	2344	403[1]
8R	2344	403[1]
9	9766	281
10	7162	384
11	1725	N.O.[2]
12	7812	352
13	6250	440
14	9668	284
15	4720	312[1]
16	1823	N.O.
17	9472	290
18	1563	275[1]
19	3385	274[1]
20	6400	430
21	10246	268
22	6348	433

[1] Computed from next highest observed predominant frequency.
[2] Not observed.

5.2.5.4 Conclusions from NDT

Electrochemical test results appear to be good indicators of the occurrence of corrosion. The *E* versus log *I* relationship and half-cell potential are affected by changes in conductivity of the surrounding soil or rock mass (i.e., changes in moisture content), and this fact should be considered when comparing results from electrochemical tests performed at different times. For polarization measurements, the same ground bed should be used for each test interval.

The impact and ultrasonic test results provide useful signatures that can serve as baselines with which future measurements may be compared. For the Ellenville, New York, site, element signatures are described in terms of (1) dominant frequencies inherent to the amplitude spectra computed from the results of impact tests and (2) reflections inherent to the time history of the motions observed from ultrasonic and impact test results.

In general, the electrochemical test results indicate that the occurrence of corrosion is more prevalent for older, less protected expansion shell rock bolts compared with resin-grouted rock bolts at this site. Two bolts are identified at sites where, relatively speaking, the occurrence of corrosion is more likely. One of the bolts (Bolt 22a) is an expansion shell bolt, and the other (Bolt 8) is a resin-grouted bolt.

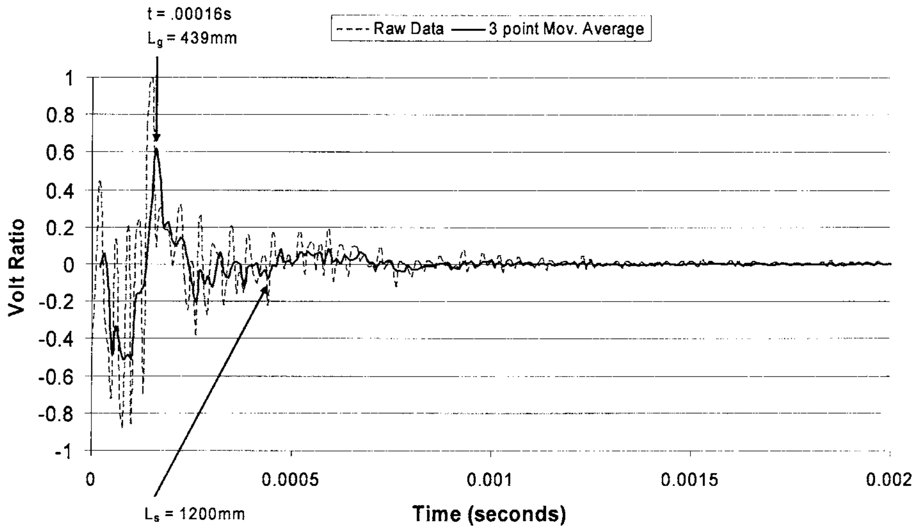


Figure 5-30. Typical results from ultrasonic test on rock bolt at Ellenville, New York.

The signatures obtained from the impact and ultrasonic tests support the conclusions from electrochemical tests. Similar signatures are obtained for most of the rock bolts tested. Elements with anomalous signatures are identified, and the potential for distress along the lengths of these elements is considered. Expansion shell Bolt 22a was observed to be severed, and its signature is very different when compared with nearby Bolt 21a.

The signature for Bolt 8 indicates that there may be loss of cross section, a fracture, or other distress present within 2 m of the face. Wave reflections corresponding to the presumed length of the bolt are not observed, and, compared with other bolts tested, a unique dominant frequency is observed for Bolt 8. This observation may be because the presumed length for Bolt 8 is incorrect or because the bolt is distressed. Bolt 8 is a candidate for further evaluation, which may involve more NDT, invasive observations or performance testing (such as a load or lift-off test), or both.

Future measurements are recommended to evaluate the possibility of corrosion-induced distress over time and to obtain information relative to the remaining useful service life of the rock bolts at Ellenville, New York. For future measurements to be meaningful, test results should be reproducible. Measurements were performed on Bolt 8 at 2-week intervals. NDT test results—including half-cell measurements, polarization measurement, impact testing, and UT—were all compared for Bolt 8 and found to be repeatable.

TABLE 5-22 Ratio of observed ultrasonic test results to impact test results in Ellenville, New York

Bolt No.	L_g Ratio (UT/Impact)	L_S Ratio (UT/Impact)
1	0.915	1.280
2	0.640	N.O.[1]
3	1.330	0.960
4	0.928	1.080
5	0.767	0.897
6	0.873	1.235
7	1.075	1.068
8 & 8R	1.090	1.000
9	0.783	0.873
10	0.859	0.823
11	–	1.00
12	0.781	0.851
13	0.750	0.843
14	0.968	1.235
15	0.934	1.000
16	–	1.373
17	0.948	0.825
18	0.800	0.833
19	0.803	0.786
20	0.958	1.444
21	0.821	0.867
22	0.635	1.111

[1] Not observed.

5.2.6 NYS Route 22, Dresden Station, New York

The Dresden Station site is located along NYS Route 22 near Mile Post 1,642, in Dresden Station, which is north of Whitehall, New York. A set of fifteen 32-mm-diameter, 6-m-long, resin-grouted rock bolts were installed at this location in 1992. The bolts are installed at the base of a near vertical cut, as shown in Figures 5-31 and 5-32. A lower rock joint is exposed near the base of the cut, which served as a bench for accessing the bolt heads with climbing ropes and a ladder. The site is of particular interest because four of the bolts are instrumented with load cells. Wiring from the load cells to the top of the slope can be seen in Figure 5-31. NYSDOT has monitored the load cells on a quarterly basis since the installation of the load cells. Three of the bolts are prestressed to 220 kN, and one bolt is prestressed to 440 kN. Generally, the measured loads remained stable, exhibiting only minor fluctuations over the monitoring period.

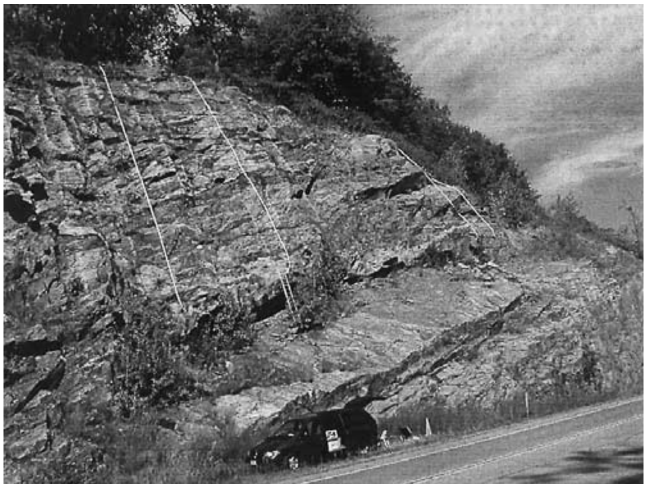


Figure 5-31. Rock cut along NYS Route 22 north of Whitehall, New York.

5.2.6.1 Subsurface Information

The highway cut is made within a gneiss rock formation that has a major joint, dipping obliquely to the alignment of the highway. The joints are spaced at approximately 3- to 6-m intervals and have an aperture of approximately 50 mm, infilled with fine sand.

Samples of infilling and rock were collected by hand during the site visit. Infill samples were sent to the geotechnical engineering laboratory for testing. Table 5-23 summarizes the results of laboratory testing performed on the samples. On the basis of these test results, the infilling was classified by the USCS as well-graded sand with silt (SW-SM). Results from chemical testing of the infill indicates that the pH is close to neutral and the maximum measured resistance is high enough that high rates of corrosion are not expected. Furthermore, resin grout surrounding the rock bolts tends to isolate elements from the surrounding rock mass. However, because resin grout may not completely surround elements, some of the bolt lengths may be exposed. NDT is used to assess the condition of the resin grout and to determine whether there has been any significant loss of cross section.

5.2.6.2 NDT Performed at Site

Six rock bolts were evaluated by NDT at the site, and each of the instrumented bolts (Bolts 1, 2, 3, and 4) was tested. Bolts 1 and 2 are at the south end of the site, and Bolts 3 and 4 are 40 m away at the north end. Bolts 5 and 6 (not instrumented) are near Bolts 1 and 2. Bolt 6 is approximately 1.5 m north of Bolt 1, and Bolt 5 is approximately 1 m below Bolt 6.

Half-cell potentials were observed for all six bolts, and polarization measurements were taken on Bolts 5 and 6. Impact tests were carried out on Bolts 1, 2, 3, and 4. The ultrasonic test was performed on all six bolts.

Electrochemical tests. In continuity checks, the bolts appeared to be electrically isolated from one another. Half-cell measurements from Bolts 1 through 6 are summarized in Table 5-24. The observed half-cell potentials for Bolts 1, 2, 5, and 6 range from –555 mV to –633 mV. With these low potentials, the presence of corrosion is not likely. Half-cell measurements from Bolts 3 and 4 were more positive, –144 mV and –200 mV, respectively. The more positive readings for Bolts 3 and

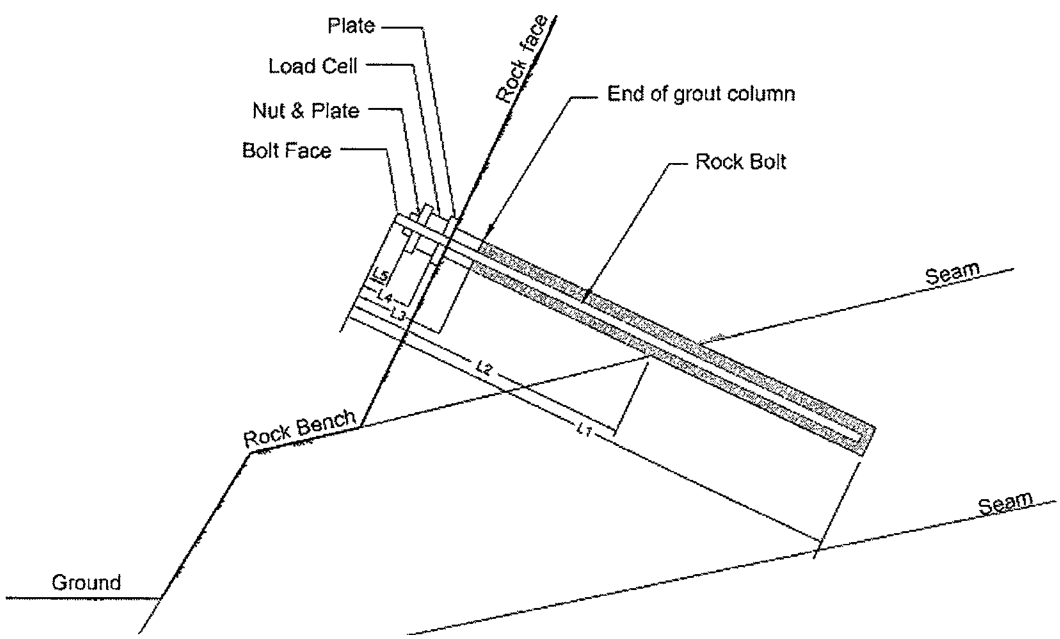


Figure 5-32. Schematic of rock bolt at Dresden Station, New York.

TABLE 5-23 Summary of test results for rock joint infilling from Dresden Station, New York

Sieve Size (U.S. Standard)	% Passing
19 mm	100
#4	94
#40	51
#200	8
D_{60} = 0.6 mm	
D_{30} = 0.3 mm	C_c = 1.2
D_{10} = 0.1 mm	C_u = 6.2
Atterberg Limits	
Liquid Limit	N.A.
Plastic Limit	N.A.
Plasticity Index	N.A.
Chemical Analysis	
Measured Moisture Content (w%)	16.1 %
Resistivity (ohm-cm)	5900 (min. at w = 23.6%)
pH	6.9
Sulfate Content (mg/kg)	4.3 mg/kg
Chloride Content (mg/kg)	< 10 mg/kg

4 indicate that corrosion may have occurred at some point along their length, most likely close to the rock face.

Table 5-24 also presents polarization current (I_p) measured for Bolts 5 and 6. For a 32-mm-diameter bolt, approximately 2.1 mA/m is required to polarize the surface of the bolt. The measured I_p is approximately 1.0 mA, which indicates that Bolts 5 and 6 are not electrically insulated by resin grout for a length of approximately 475 mm ($L \approx 1.0$ mA $\div$ 2.1 mA/m). Results from impact and UT indicated that this phenomenon was likely because the drill hole was not completely filled with resin grout.

Impact tests. The evaluation of the impact test data indicates that the amplitude spectrum for each bolt tested is a reasonable signature of the element condition. The signature appears to be repeatable and correlates well to known features of the rock bolt installations. Figure 5-33 depicts the frequency response in terms of the amplitude spectrum for Bolt No.1, which is typical for all four of the bars evaluated by impact testing. The amplitude spectrum exhibits five or six peak frequencies.

Table 5-25 is a summary of peak frequencies observed for all of the bolts tested by the impact method. Scatter in the data corresponding to f_1, f_4, and f_5 is relatively small, having a coefficient of variance of approximately 10 percent. For f_2 and f_3, the coefficient of variance is higher: 15 to 20 percent. In what follows, the relative scatter in the data is rationalized considering the precision associated with the locations of physical features, which are correlated with the observed peak frequency ranges.

TABLE 5-24 Summary of electrochemical test results for rock bolts tested along NYS Route 22 near Dresden Station

Element No.	E_{corr} (mV)	I_p (mA)
1	−555	-
2	−611	-
3	−144	-
4	−210	-
5	−633	1.0
6	−576	0.9

Five main features of the installation include the length of the bolt, seam location, top of the grout column, and two plates (one at each end of the load cell). These locations are designated as L_1 to L_5 in Figure 5-30. Table 5-26 correlates peak frequency ranges observed in the amplitude spectrum and features of the rock bolt installations.

The range of observed rock bolt lengths (L_1) correlates well with the known installed bolt lengths of approximately 6.1 m. The measurements corresponding to L_5 and L_4 for different bolts do not vary by more than 50 mm and 100 mm, respectively. Features such as bolt length (L_1) and locations of bearing plates (L_4 and L_5) are controlled quantities related to the supplied bolt length, specified plate thickness, load cell profile, and length of drill hole. Therefore, data from these measurements appear to have lower scatter relative to data for more uncertain features, such as those corresponding to L_3 and L_2.

According to the observed reflections corresponding to L_3 and L_4, the presumed gap between the back of the bearing plate and the beginning of the grout column ($L_3 - L_4$) ranges between 150 mm to 330 mm. The measured distance from the rock face to the joint seam ($L_2 - L_4$) ranges from 1.3 m to 2.3 m. Because the locations of the top of the grout column (L_3) and the joint seam (L_2) are not controlled during construction, relatively high scatter in the data corresponding to these measurements (i.e., f_2 and f_3) is expected.

The impact test results do not indicate significant levels of distress along the lengths of the bolts. This finding is particularly evident in the signature amplitude spectra, which clearly exhibit peak frequencies corresponding to the ends of the elements. If a loss of cross section was present, reflections from

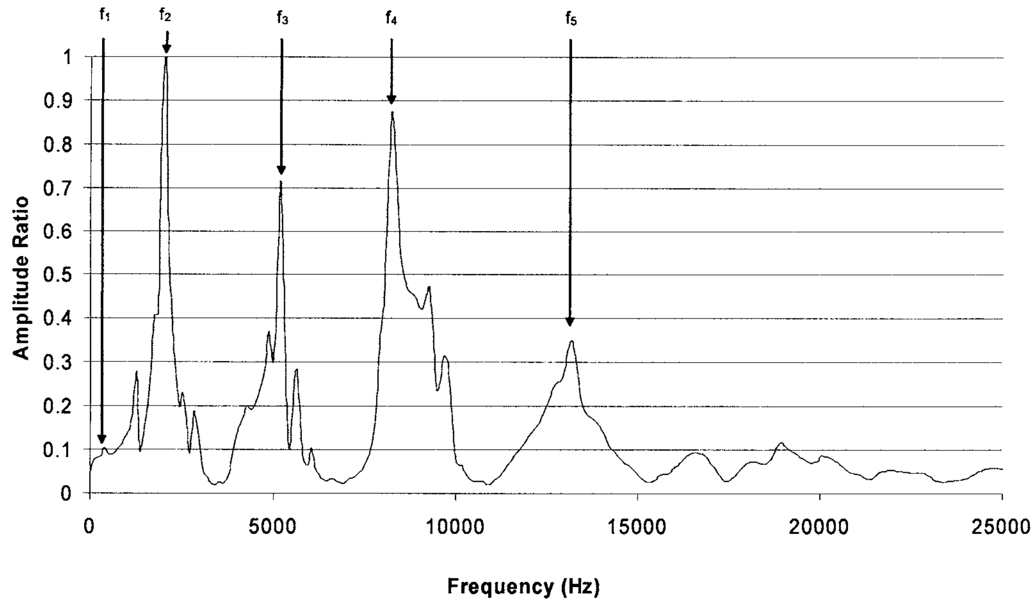


Figure 5-33. Typical amplitude spectrum for rock bolt at Dresden Station, New York.

the end of the bar would be weakened, and the corresponding low energy levels would not prevail in the amplitude spectra.

Ultrasonic tests. Reflections were evident in the ultrasonic test results corresponding to features within approximately 2 m from the bolt face. Figure 5-34 shows the time history of the transducer response at the face of the element for Bolt 2, which is typical of all the bolts tested.

The signal shown in Figure 5-34 decays rapidly and damps out within approximately 2 ms. Strong reflections are evident corresponding to locations L_5 and L_4, which are relatively close to the bolt face. Reflections at intervals of approximately 220 μs are also evident, corresponding to the top of the grout column (L_3). Much weaker reflections at intervals of approximately 790 μs, corresponding to the seam location (L_2), are also evident. Reflections from the end of the element, corresponding to L_1, were not detected in the ultrasonic test results. The amplitudes of these distant source reflections are very small and may be masked by the larger amplitude reflections from features closer to the face of the rock bolts.

Table 5-27 is a summary of the data from UT of Bolts 1 through 6. For Bolts 1, 2, 3, and 4, measurements from ultrasonic and impact testing are compared. The comparison of

TABLE 5-25 Summary of peak frequencies for rock bolts tested in Dresden Station, New York

Bolt No.	f_1 (Hz)	f_2 (Hz)	f_3 (Hz)	f_4 (Hz)	f_5 (Hz)
1	488	1269	5175	8203	13183
2	488	1000	5151	7080	10400
3	390	1220	4687	7763	10500
4	488	1562	3710	6689	11621

TABLE 5-26 Correlation of peak frequencies to physical features of the rock bolt installations in Dresden Station, New York

Range of Peak Frequency (Hz)	Description of Physical Feature Correlated to Range of Peak Frequency	Computed Length from Face[1] of Rock Bolt to Physical Feature Shown in Figure 5-30
400 – 500	End of rock bolt	L_1 (5.5–6.9 m)
1000 – 1500	Seam location	L_2 (1.83–2.75 m)
3500 – 5000	Beginning of grout column	L_3 (550–785 mm)
6500 – 8000	Nut and plate at bottom of load cell	L_4 (344–423 mm)
10500 – 13000	Nut and plate at top of load cell	L_5 (211–262 mm)

[1] Accelerometer is mounted to the exposed face of the rock bolt.

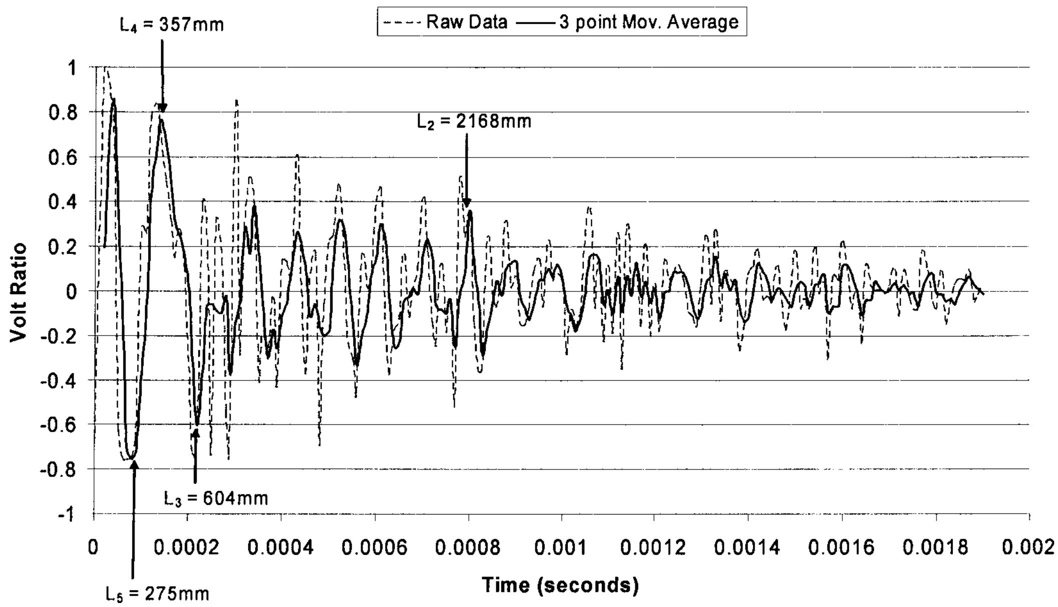


Figure 5-34. Typical ultrasonic test result for rock bolt at Dresden Station, New York.

observations from the different test methods is considered good. The average ratio of the ultrasonic test result to the impact test results is 0.99, and the corresponding coefficient of variation is 11 percent.

The ultrasonic test data provide a useful verification of the impact test data obtained at the Dresden Station site. This verification affords a high degree of confidence in the element signatures obtained from the impact test results.

5.2.6.3 Conclusions from NDT

The measured loads in Bolts 1 to 4 appear stable, indicating that the elements are not currently distressed. At the time of the NDT evaluation, the bars had been in service for only 10 years; given the environment (i.e., pH, resistivity, and sulfate and chloride content), this time is not enough for the elements to have undergone significant corrosion. Results from electrochemical tests indicate that corrosion may be occurring, but results from wave propagation tests do not reveal any significant loss of cross section. However, the impact and ultrasonic test results correlate well with physical features of the installation, including the observation of a gap between the backside of the anchor plate and the grout column.

Because of the observed half-cell potentials indicating the presence of corrosion along some of the bolts, and because of the element vulnerability to corrosion near the anchor head, future testing is recommended. The NDT results presented in this report can be useful as baseline measurements with which future NDT results can be compared. Future NDT will be useful to monitor the condition of the elements over time and to assess the potential impact of corrosion on the service life.

5.2.7 Pigeon Gorge, I-40, North Carolina

Interstate 40 follows the Pigeon River through its deep gorge for about 35 km through western North Carolina and into eastern Tennessee. The route crosses rugged terrain in the Smokey Mountains, where relief between the river and

TABLE 5-27 Comparison of ultrasonic and impact test data from bolts tested in Dresden Station, New York

Bolt No.	L_5 (mm)		L_4 (mm)		L_3 (mm)		L_2 (mm)	
	UT	Impact	UT	Impact	UT	Impact	UT	Impact
1	220	207	275	305	494	534	2043	2135
2	275	264	381	381	702	534	2580	2750
3	275	261	357	366	604	610	2168	2288
4	192	236	439	409	732	762	1537	1753
5	-[1]	-[1]	137	N.M.[2]	329	N.M.	1037	N.M.
6	-[1]	-[1]	110	N.M.	220	N.M.	2074	N.M.

[1] No load cell.
[2] Not measured.

the surrounding mountaintops often exceeds 600 m. Along the roadway, rock cuts up to 100 m high are common. Many bolts are located at elevations difficult to achieve without using a special lift or undertaking difficult rock climbing.

Both planar and wedge-type rock slope failure mechanisms have been problematic along an approximate 7-km stretch of I-40 from the state line eastward into North Carolina (Glass, 1998). Slope repair projects were conducted in the 1980s and after a 1997 rockslide. Work completed around 1985 included relocation of the highway to provide for a catchment area, excavation of unstable material, scaling, and installation of approximately 12,000 linear meters of rock bolts, wire mesh, and horizontal drains.

In 1997, a large rock wedge failure occurred approximately 1 km east of the state line. Rock bolts were installed around the head of the slide and down the line of the wedge intersection. Hanging nets were also suspended across the slope to slow falling rock and direct it into the existing catchment area. Also, the catchment area was improved by construction of a mechanically stabilized earth (MSE) wall at the edge of the roadway.

An approximately 300-m-long section near the location of the 1997 rockslide was selected for the evaluation. Figure 5-35 shows a picture of the area, including the rock slope and the MSE barrier wall at the west end. Rock bolts at the section selected for evaluation stabilize (1) potential failure mechanisms, including wedge failures, along the line of intersection of the joint and bedding surfaces, which dips approximately 40°, oblique to the highway alignment and (2) planar sliding along the bedding surfaces. Rock bolts are 25-mm-diameter, resin-grouted bars manufactured by Dywidag. The bolts were installed approximately 15 years ago, and lengths range from 3 m to 20 m. The bars were reportedly prestressed to 178 kN (Glass, 2000).

5.2.7.1 Subsurface Information

The rock unit at the Pigeon George site is the Pigeon Siltstone—a gray, thin-to-thick bedded metasiltstone containing some layers of fine-grained metasandstone. Bedding layers strike 20° northwest and dip southwest 30° to 35°. In places, clay seams, up to 50 mm thick, have been observed along the bedding surface. Joints, generally striking northeast and dipping 60° to 65° to the southeast, intersect the bed to form an unstable wedge of rock material dipping obliquely into the roadway. Samples of rock were collected by hand during the site visit. Clay seams were inaccessible, and infill material was not sampled at this site.

Figure 5-35. Rock slope at Pigeon Gorge, I-40 in North Carolina.

5.2.7.2 NDT Performed at Site

The approximately 10-m-wide catchment at the side of the road allowed the project team to pull off the highway and set up equipment between the roadside Jersey barrier and the rock face. Some of the bolts in this area were accessible with a ladder supplemented with mountain climbing techniques using ropes and special harnesses. Nine bolts were tested at the site. The eastern edge of the MSE barrier wall, constructed along the edge of the roadway, is used as a reference for location. Bolts 1 and 2 were located approximately 100 m east of the MSE wall, along a bedding plane, at an elevation approximately 6 m from the base of the slope. Bolts 3 and 4 were located near the east end of the MSE wall, along the southeast dipping joint planes, at elevations of approximately 6 m and 9 m, respectively, from the base of the slope. Bolts 5 through 9 were located in an area approximately 250 m east of the MSE wall. These bolts were located to either side of the line of intersection, formed by a bedding plane and rock joint, daylighting approximately 5 m above the toe of the slope. Bolts 5, 6, and 7 were spaced approximately 6 m apart along the southwest dipping bedding plane. Bolts 8 and 9 were spaced approximately 3 m apart along the southeast dipping joint plane.

Bolts 1–9 were all evaluated with half-cell and electrical resistance measurements, as well as impact and ultrasonic tests.

Electrochemical tests were performed using a nearby bolt as the ground bed. The half cell was placed in colluvial deposits near the base of the slope. Table 5-28 summarizes

TABLE 5-28 Summary of electrochemical test results for rock bolts tested along I-40 near Pigeon Gorge, North Carolina

Element No.	E_{corr} (mV)	I_p (mA)
1	–570	0.52
2	–494	1.00
3	–519	0.55
4	–385[1]	0.49
5	–378	4.10
6	–271[1]	6.30
7	–341	5.00
8	–542	4.10
9	–446	5.50

[1] Used as ground bed prior to half-cell measurement.

the electrochemical test measurements, including the corrosion potential and polarization current for all of the bolts tested at the Pigeon Gorge site. High half-cell potentials observed for Bolts 4 and 6 may be due to the use of these elements as ground beds prior to making half-cell measurements. With the exception of Bolts 5 and 7, the measured half-cell potentials for the remaining bolts (Numbers 1, 2, 3, 8, and 9) are close to −500 mV. The higher half-cell potentials observed for Bolts 5 and 7, −378 mV and −341 mV, respectively, indicate that corrosion may be present for these bolts.

Measurements of polarization current ranged from approximately 0.5 mA to 1.0 mA at the western and central portions of the site (Bolts 1, 2, 3, and 4); and from 4 mA to 6.3 mA at the eastern end of the site (Bolts 5, 6, 7, 8, and 9). For 25-mm-diameter bar elements, the computed polarization current requirement is approximately 1.65 mA/m of element in contact with the surrounding electrolyte. Therefore, along the western and central portions of the site, the measured polarization currents correspond to approximately 300 mm to 600 mm of contact length. This length corresponds to the distance from the back of the bearing plate to the top of the grout column, which is consistent with measurements from the impact test described in the next section.

Significantly more contact lengths (i.e., between 2.40 m and 3.82 m) are apparent in the data from the eastern end of the site. This observation indicates that bolts installed along the eastern end of the site may be more vulnerable to corrosion. Joints and seams at the east end of the site appear to be more abundant and/or more open compared with joints and seams observed along the central and western portions of the site. Thus, it is speculated that at the east end of the site, during installation more grout seeped into the surrounding rock mass through the joints and seams, resulting in relatively less grout cover along the lengths of the bolts.

According to the results from electrochemical testing, Bolts 5 and 7 are candidates for more detailed evaluation because polarization current measurements have identified them as vulnerable to corrosion, and half-cell potential measurements indicate that corrosion is present.

Impact tests. Figure 5-36 is the amplitude spectrum obtained from the impact test performed on Bolt 2. A predominant frequency of 4,101 Hz is evident in Figure 5-36, and Table 5-29 is a summary of the predominant frequencies, f_p, observed from the amplitude spectra of all bolts tested, which range from 3,613 Hz to 6,993 Hz. The predominant frequencies are presumed to correspond to wave reflections associated with the beginning of the grout column. As shown in the third column of Table 5-29, the lengths between the beginning of the grout columns and the faces the elements (L_g), computed from the predominant frequencies, range from 393 mm to 761 mm. These lengths need to be adjusted by approximately 300 mm, corresponding to the protrusion of the element faces beyond the back sides of the bearing plates, to render the range of gap distances between the backs of the bearing plates and the tops of the grout columns of approximately 100 mm to 450 mm.

Peak frequencies ranging from 250 Hz to 600 Hz are apparent in many of the amplitude spectra, which are presumed to correspond to the lengths of the elements (L_T). These low

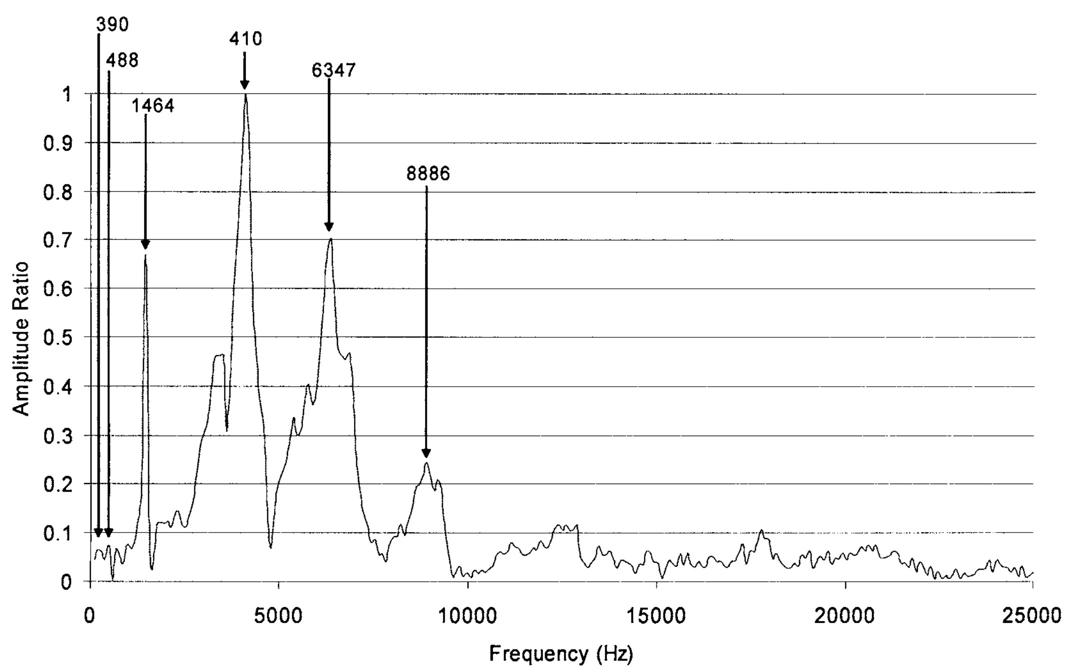


Figure 5-36. Amplitude spectrum for Bolt 2a at central portion of site along I-40, North Carolina.

TABLE 5-29 Summary of impact test results for site along I-40 near Pigeon Gorge, North Carolina

Bolt No.	f_p (Hz)	L_g (mm)	L_s (mm)	L_T (mm)
1	4882	562	1830	6405
2	4101	671	1525	7015
3	5175	531	2961	5704
4	5712	481	2288	18300
5	4663	590	2135	9760
6	4931	558	1830	5185
7	4594	599	1373	7625
8	6993	393	4118	13725
9	3613	761	3355	9760

peak frequencies are present to varying degrees depending on the lengths of the elements tested, and, apparently, the degree to which the bolts are encapsulated by grout. A low peak frequency of approximately 400 Hz is apparent for Bolt 2; but this bolt has a very small amplitude compared with the predominant frequency shown in Figure 5-36. In contrast, Figure 5-37 is the amplitude spectrum obtained for Bolt 6, which exhibits a peak frequency at 512 Hz with nearly the same amplitude as the predominant frequency. The predominance of lower frequency content in the amplitude spectrum for Bolt 6 may relate to the condition of the grout surrounding the element. If less of the element is surrounded by grout, relatively more energy is reflected from the end of the element. A similar trend is apparent from the amplitude spectra of Bolts 5, 7, 8, and 9, which have higher amplitudes associated with low frequency peaks compared with amplitude spectra corresponding to Bolts 1, 2, 3, and 4. This observation is consistent with polarization measurements, which indicate that elements at the east end of the site are not encapsulated by grout to the same degree as those at the central and west ends of the site.

Because of the lack of precision and the difficulty of defining low frequency peaks in the amplitude spectra, the lengths of the elements (L_T) and distant reflections from seam locations (L_s) are determined from time history data. After application of a high-pass frequency filter, reflections corresponding to lower frequencies are apparent in the waveforms. Figure 5-38 is the time history of accelerations for Bolt 2 corresponding to a frequency window centered about the 13,000-Hz frequency range. Reflections are apparent in the waveforms corresponding to distances of 1,525 mm and 7,015 mm. These distances are presumed to correspond to reflections from a distant seam (L_s) and from the far end of the element (L_T), respectively. The last two columns of Table 5-29 summarize the values of L_s and L_T observed for all the elements tested. The observed range of L_T is about 5–18 m, which correlates well with the knowledge that installed lengths at this location range from 3 m to 20 m.

The data presented in Table 5-29 do not indicate any significant distress along the lengths of the elements. This conclusion is based on the observation of reflections, corresponding to the ends of the element, that would otherwise be masked by strong reflections from loss of element cross section along the element length.

Reflections corresponding to L_T for Bolts 5 and 7 are distinct, and, although data suggest that these bolts are more vulnerable, the bolts do not appear to be more distressed than other bolts tested.

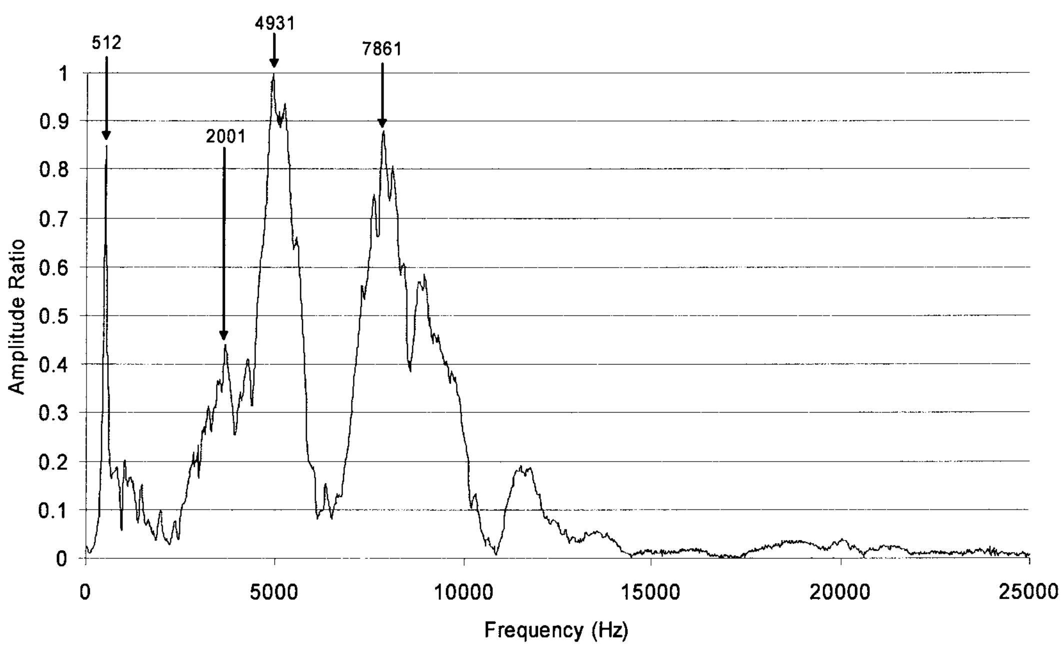


Figure 5-37. Amplitude spectrum for Bolt 6a at eastern portion of site along I-40, North Carolina.

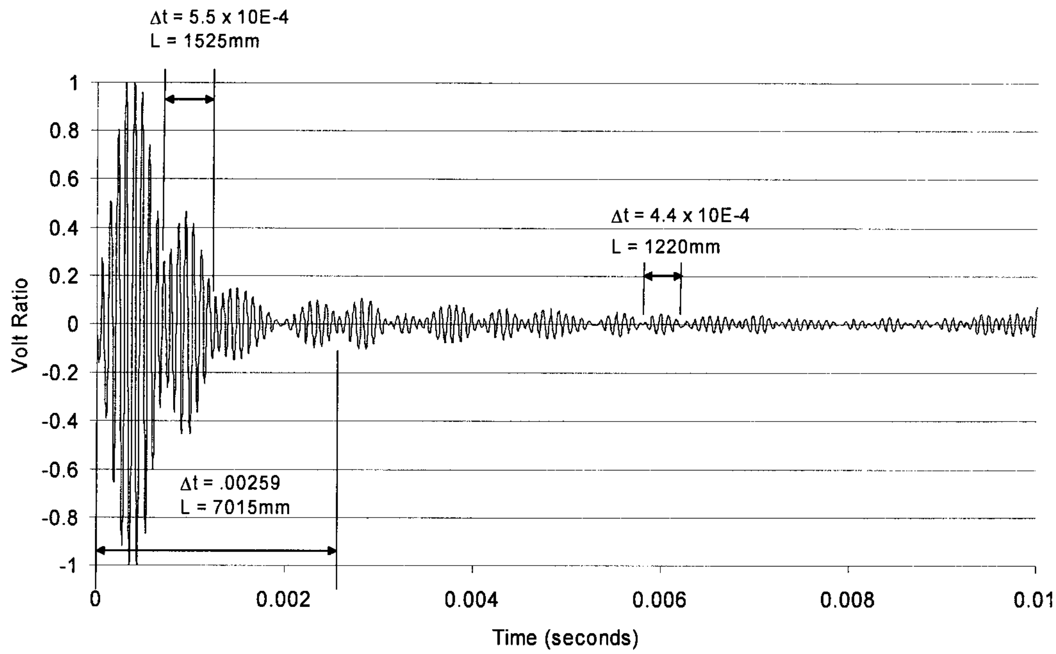


Figure 5-38. Typical filtered time history of acceleration from impact test of rock bolt at Pigeon Gorge site along I-40, North Carolina.

Ultrasonic tests. Figure 5-39 is the time history of the ultrasonic transducer signal obtained from testing Bolt 1. Reflections corresponding to L_g and L_s of lengths 576 mm and 2,135 mm, respectively, are identified in Figure 5-39. These lengths compare reasonably well with measurements from impact testing. Table 5-30 is a summary of observations from ultrasonic tests performed on all the elements tested at the site compared with those from impact testing. The average ratio of ultrasonic test result to impact test result is 1.046, and the data have a coefficient of variation of approximately 12 percent.

These results demonstrate that consistent results can be obtained from impact and UT. Either test may be useful to verify the results from the other. When both tests provide similar data, more confidence may be placed upon the overall results.

Although reflections corresponding to distances more than 2 m from the faces of the elements were not evident in the

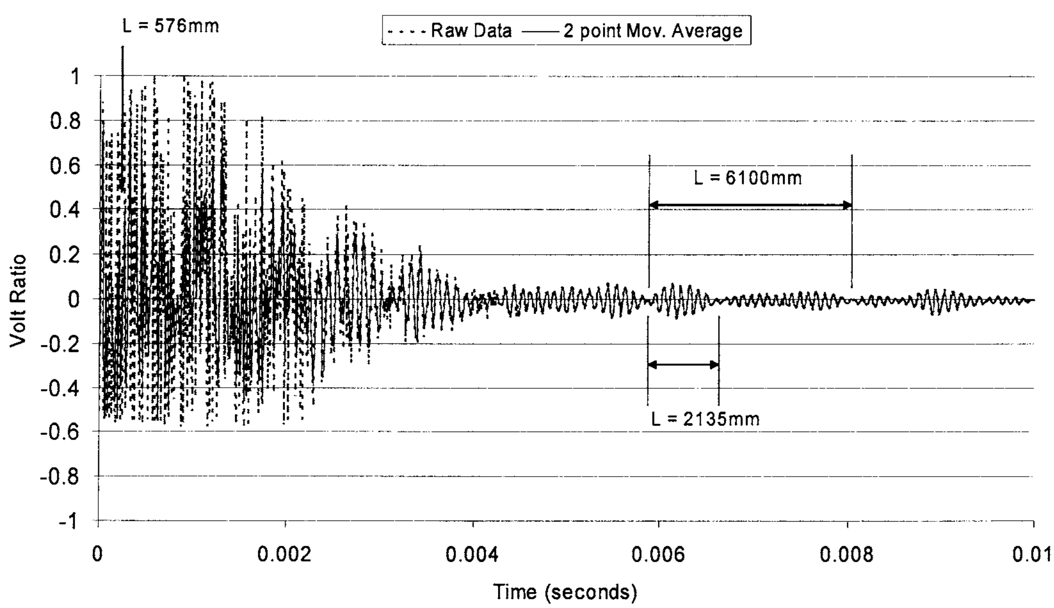


Figure 5-39. Ultrasonic test result from Pigeon Gorge site along I-40, North Carolina.

TABLE 5-30 Summary of ultrasonic test data for site along I-40 near Pigeon Gorge, North Carolina

Bolt No.	Ratio of Ultrasonic to Impact Test Measurement (L_g^{UT}/L_g^{impact})	(L_s^{UT}/L_s^{impact})	(L_T^{UT}/L_T^{impact})
1	1.01	1.17	0.95
2	0.94	0.90	0.91
3	1.09	N.O.[1]	N.O.
4	1.14	1.20	N.O.
5	0.93	1.24	N.O.
6	1.13	1.17	1.12
7	0.92	1.11	1.28
8	0.87	N.O.	N.O.
9	0.87	N.O.	N.O.

[1] Not observed.

majority of the ultrasonic test data, some of the data did exhibit reflections corresponding to the length of the element. This occurrence is particularly evident in the results for Bolt 1, shown in Figure 5-39. Reflections from the ends of the elements were also apparent in the test results for Bolts 2, 6, and 7.

5.2.7.3 Conclusions from NDT

The electrochemical test results indicate that some of the elements at this site are vulnerable and corrosion may be occurring. Element vulnerability appears to correlate with location, and elements installed along the eastern portion of the site are apparently more vulnerable.

Impact test results confirmed the vulnerability of elements along the eastern portion of the site. Results from impact and UT were consistent, and significant levels of distress were not identified for any of the elements tested.

Given that elements at this site are identified as vulnerable, future monitoring is recommended. Data presented in this report may serve as baseline measurements with which future data may be compared.

5.2.8 TAMU-NGES

In 1991, a tied-back soldier pile wall with wood lagging was constructed at the Texas A&M University National Geotechnical Engineering Experimentation Site (TAMU-NGES). The wall was instrumented with strain gauges, load cells, and inclinometers. Details of the wall construction and performance monitoring during and after construction are reported by Briaud et al. (1998). Salient details are presented here to support the description of NDT performed at the site and condition assessment of the tieback elements.

The wall, pictured in Figure 5-40, is approximately 7.6 m high and 50 m long. One-half of the wall is supported by a single row of ground anchors, while the other half is supported by two rows of ground anchors. Anchor forces are transferred to soldier piles by walers, which are not continuous along the length of the wall; each waler supports a separate pair of piles, as shown in Figures 5-40 and 5-41.

Figure 5-40. Tieback wall at TAMU-NGES.

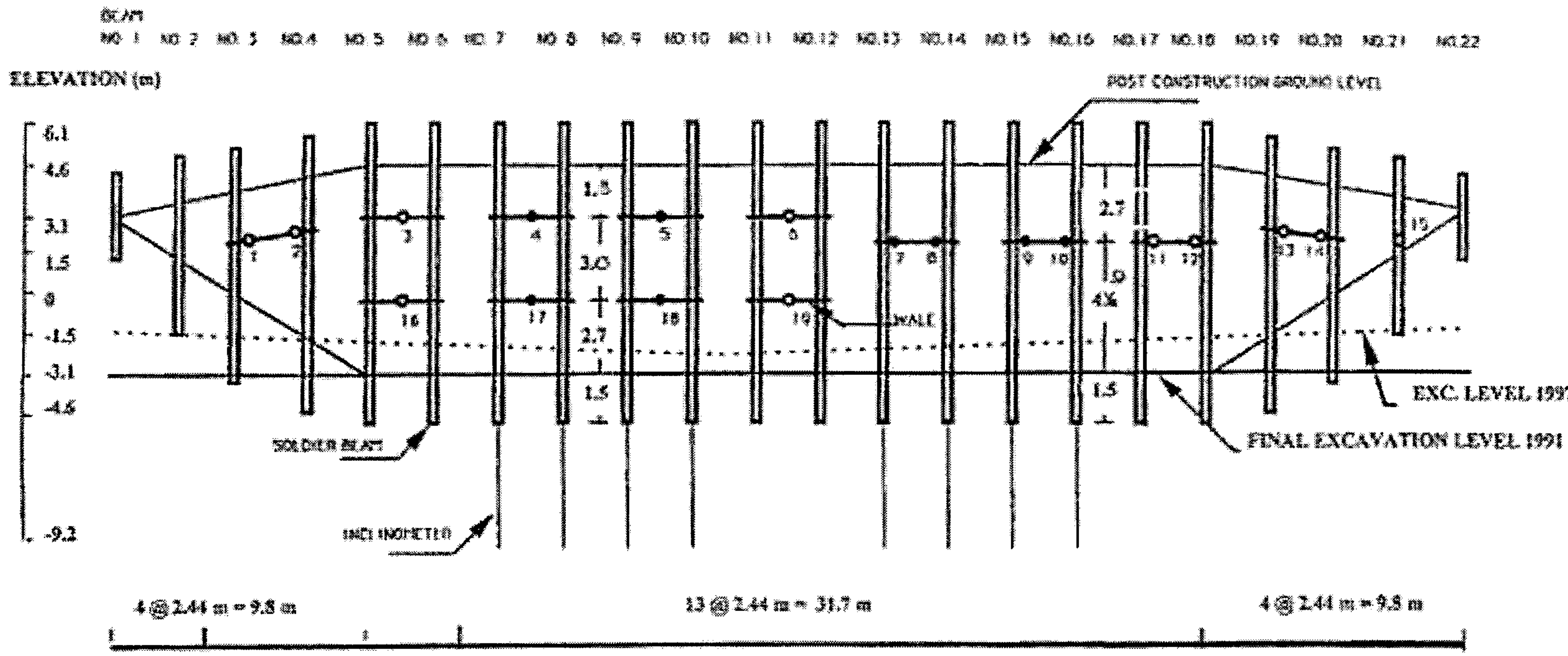
Tieback elements are 32-mm-diameter, Grade 150, Dywidag bars that were installed in 89-mm-diameter holes. Bonded lengths are 7.3 m, and unbonded lengths are either 4.6 m or 4.9 m, depending on location (Briaud et al., 1998). Tieback elements extend beyond the wall face for lengths of approximately 1,300 mm, which includes the depth of the walers as depicted in Figure 5-40. 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. In the unbonded zone, elements were surrounded by grease and a plastic sheath.

Generally, anchor loads, monitored with the load cells, remained relatively constant for the 5-year period over which the wall behavior was studied. A special experiment was conducted during which anchor loads were released at two locations, and the corresponding redistribution of load was observed. Results from this experiment, and the documented load recovery, suggest that the ends of the tiebacks were properly bonded to the soil (Briaud et al., 1998).

Figure 5-40 shows that, at the time of the research team's site visit, the wall lagging was in poor condition. Sand behind the retaining wall had raveled and sloughed; some lagging boards were missing, and many were deformed from bending. In places, voids behind the wall coalesced into chasms, forming a chimney along the back side of the lagging. This back side daylighted near the base of the wall. Large chasms could be seen in several places along the wall. Beneath these chasms, piles of silty sand remained. If these chasms were viewed from behind the wall face, ground anchor elements were visible and one could inspect the bar and plastic sheath surrounding the bar.

5.2.8.1 Subsurface Information

Soils at the site consist primarily of alluvial sand deposits. Soils are described as medium-dense clayey sand or silty sand



Note: Figure taken from Briaud et al., 1998.

Figure 5-41. Full-scale tieback wall at TAMU-NGES: front view.

from the ground surface to a depth of 3.5 m; medium-dense, clean, poorly graded sand from 3.5 m to 7.5 m; medium-dense clayey sand from 7.5 m to 13 m; and hard clay below a depth of approximately 13 m. In 1991, groundwater was observed at a depth of 9.5 m below the top of the wall.

Several in situ tests were performed at the wall site during 1990 and 1991, including standard penetration tests (SPTs), cone penetrometer tests (CMTs), preboring pressuremeter tests (PPTs), dilatometer tests, and borehole shear tests. The sand has the following average properties: total unit weight of 18.5 kN/m^3, SPT blow count increasing from 10 blows/0.3 m at the surface to 27 blows/0.3 m at a depth of 9.15 m, borehole shear test friction angle of 32° with no cohesion, CPT point resistance of 7 MPa, PPT modulus of 8 MPa, and PPT limit pressure of 0.5 MPa. Also, a sand pH of 6.2 was measured following the procedure in ASTM Standard D-4972 (ASTM, 2001). During the site visit, samples of sand from behind the retaining wall were retrieved for further testing. The samples were sent to the geotechnical engineering test laboratory for chemical analysis, including pH, resistivity, and sulfate and chloride ion content. Results from testing are summarized in Table 5-31.

According to the results from the chemical analysis, including a pH close to neutral and a relatively high resistivity, the soil at this site is not considered aggressive relative to corrosion.

5.2.8.2 NDT Performed at Site

During the site visit, access to the wall was accomplished by driving the equipment van to the top of the excavation. Anchor heads were accessed by ladder from the base of the excavation and necessary wiring extended to the van at the top. The site is equipped with a trailer and power supply, so use of a generator was unnecessary.

Table 5-32 summarizes the NDT performed at this site, and Figure 5-41 sketches the corresponding anchor locations (Briaud et al., 1998). Ten anchors out of 19 available at this site were tested. Impact and ultrasonic tests were performed on all 10 anchors.

As shown in Figure 5-41, six of the tested anchors were from the single-tier section of the wall (Bolts 9, 10, 11, 12, 13, and 14); three were from the bottom row (Bolts 17, 18, and 19) and one from the top row (Bolt 6) along the two-tiered section.

Four of the tested bolts were monitored with load cells, and lock-off loads are documented for all the tiebacks. Loads in the tested elements, indicated in the fourth column of Table 5-32, correspond to load cell readings from April 1996 or to lock-off loads as reported by Briaud et al. (1998).

TABLE 5-31 Summary of soil tests from samples retrieved from TAMU-NGES

Chemical Analysis	
Resistivity (ohm-cm)	8200 (min. at w = 23.6%)
pH	6.4
Sulfate Content (mg/kg)	17.6 mg/kg
Chloride Content (mg/kg)	< 10 mg/kg

TABLE 5-32 Summary of NDT elements at TAMU-NGES

Test No.	Element No. From Figure 5-41	Load Cell	Load (kN)
1	10	Yes	315[1]
2	9	Yes	186[1]
3	19	No	320[2]
4	18	Yes	330[1]
5	17	Yes	204[1]
6	13	No	164[2]
7	14	No	85[2]
8	11	No	300[2]
9	12	No	300[2]
10	6	No	378[2]

[1] Measured April 1996 (Briaud et al., 1998).
[2] Lock-off load (Briaud et al., 1998).

Electrochemical tests. Because of the connections between anchor elements, walers, and steel soldier piles, single anchor elements were not electrically isolated. However, it was possible to isolate a section consisting of two soldier piles, two walers, and two ground anchors. The measured half-cell potential of the isolated section was –409 mV, which indicates that corrosion may be present. The corrosion is most likely along the length of the driven soldier piles at the test location.

Results from the E versus log I measurements confirmed that a large metal surface area was exposed to the environment.

Impact test. Several interesting features of the bar installation were apparent in the amplitude spectra. For example, in Figure 5-42, which is the amplitude spectra for Test 6, significant peak frequencies are not observed beyond a peak of 6,640 Hz, but Figure 5-43 exhibits a peak frequency of 8,300 Hz in the amplitude spectra for Test 8. This observation is consistent with the concept that higher loads are correlated with higher natural frequencies of vibration. Peak frequencies corresponding to higher frequency contents of approximately 10,000 Hz were apparent in Tests 1, 3, 4, 5, 8, 9, and 10, but were not apparent in Tests 2, 6, and 7. According to the loads presented in the fourth column of Table 5-32, measured loads in the elements corresponding to Tests 2, 6, and 7 range from 85 kN to 186 kN, which is lower than the range of 204 kN to 378 kN observed for the remainder of the elements.

The approximately 1,300-mm-long extension of the bar elements beyond the face of the wall had a strong influence on the frequency response of the elements. For a 1,300-mm-long cylindrical element with boundary conditions fixed at one end and free at the other, and with a compression wave velocity of 5,500 m/s, the first fundamental frequency of vibration is 1,057 Hz. Figures 5-42 and 5-43 both exhibit a predominant frequency of approximately 1,057 Hz.

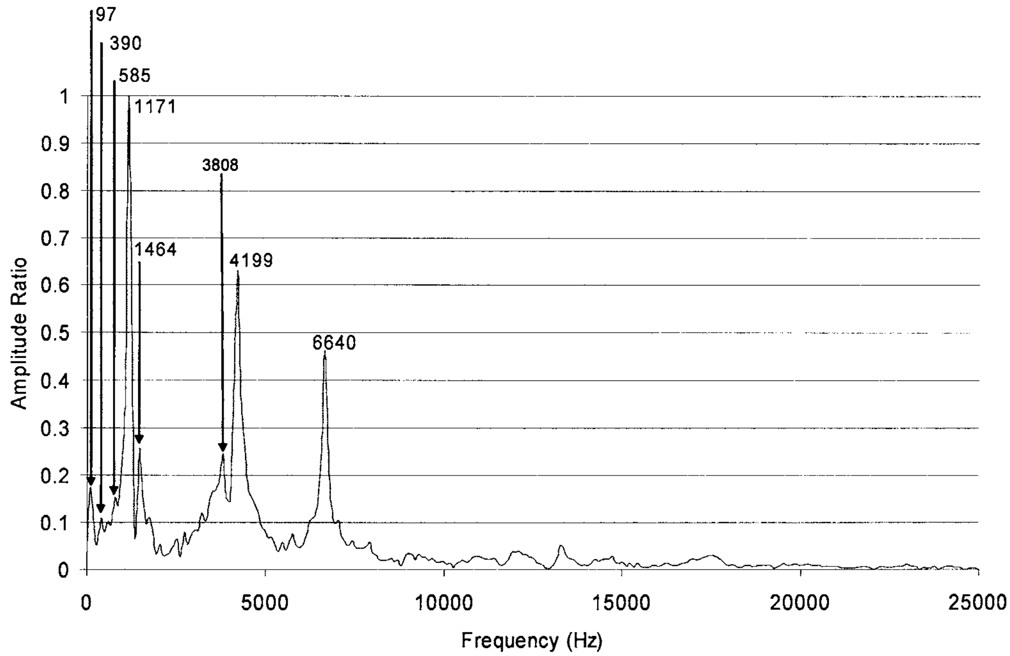


Figure 5-42. Amplitude spectrum for impact test on Bar 6a (prestress = 164 kN) at TAMU-NGES.

To help identify signals obscured by the very strong influence of vibrations associated with the element extension, a band-pass filter was applied to the data. The signal corresponding to a frequency bandwidth centered with respect to a peak frequency between 3,500 Hz and 4,500 Hz was converted to the time domain for analysis. Figure 5-44 is an example of filtered data from the results of Test 6; it is typical of the elements tested at TAMU-NGES. Reflections in the signal are apparent at times of 0.0019 s and 0.0043 s (the beginning of the bonded zone and the end of the element, respectively).

Table 5-33 compares observations from impact tests and as-built details of the wall system for all of the elements

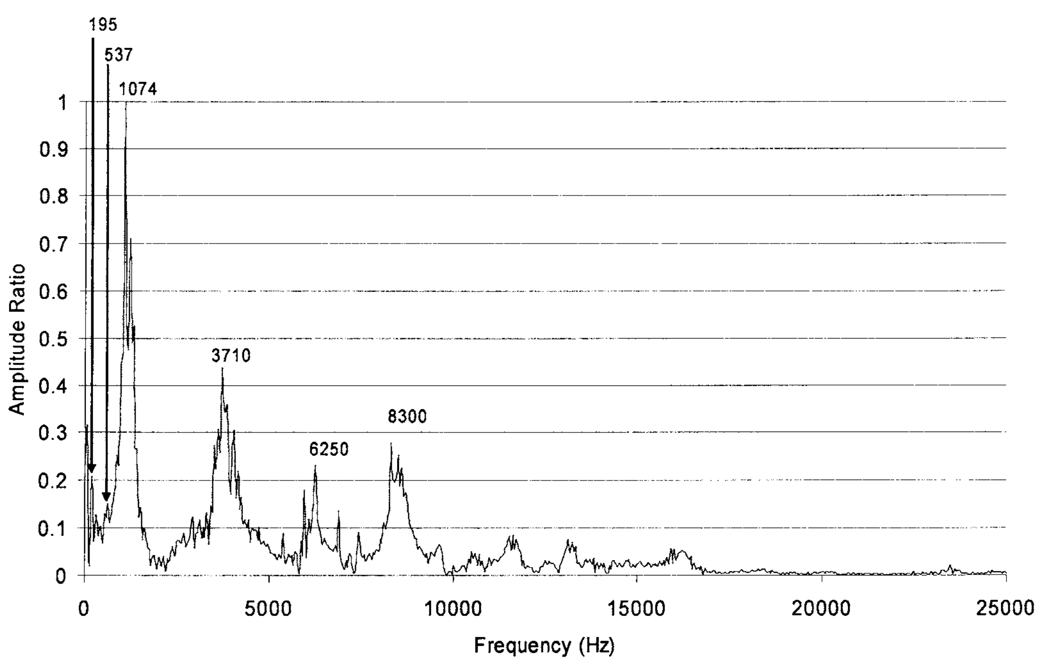


Figure 5-43. Amplitude spectrum for impact test on Bar 8 (prestress = 300 kN) at TAMU-NGES.

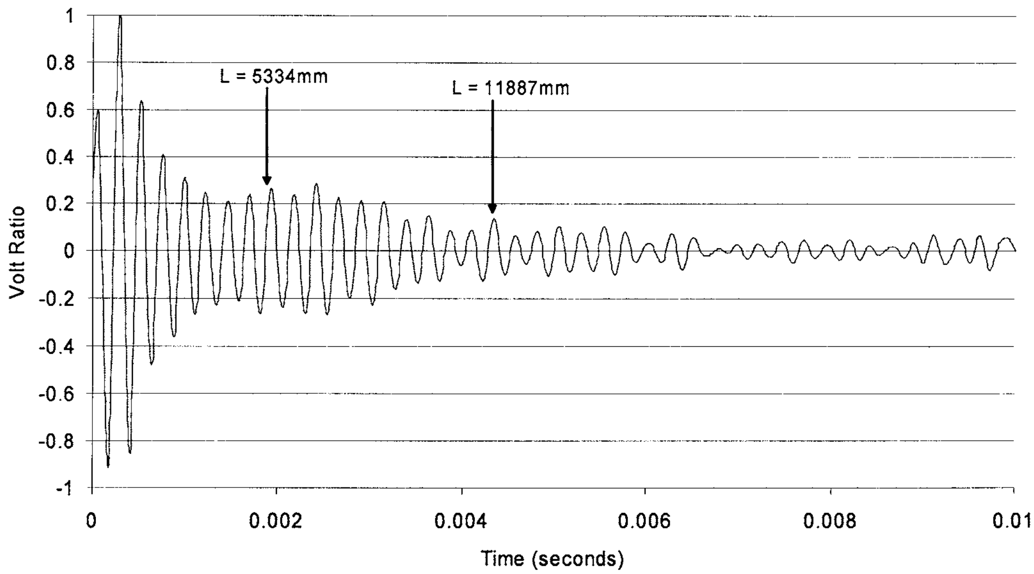


Figure 5-44. Typical filtered time history of impact test on Bar 6a at TAMU-NGES.

TABLE 5-33 Reflections observed from impact test results and as-built details 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.[1]	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

[1] Not observed.

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 the length that the tie-backs extend beyond the wall face. In Table 5-33, the bonded lengths are computed as differences between the total and unbonded lengths.

According to the data presented in Table 5-33, 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.

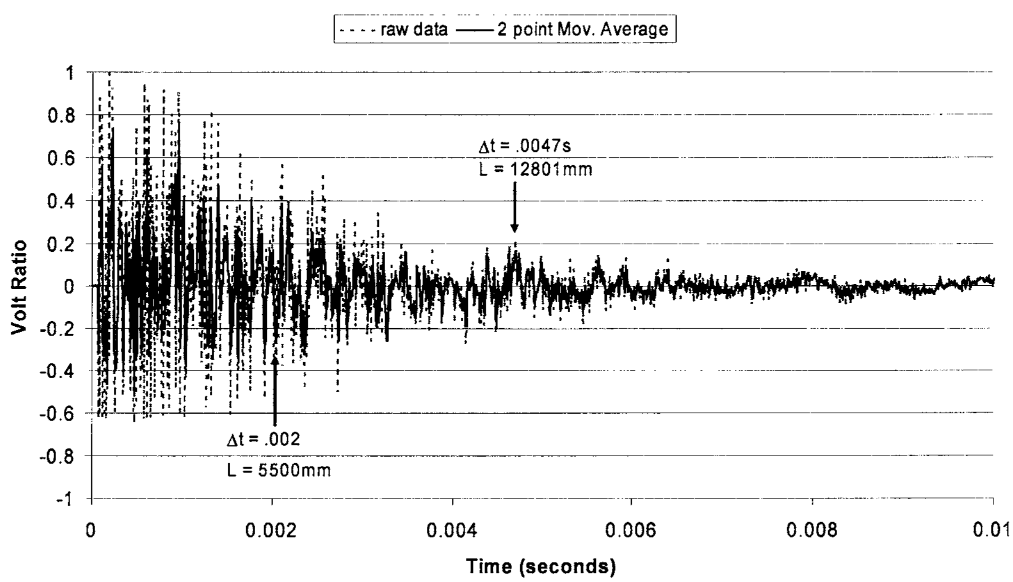


Figure 5-45. Typical ultrasonic test results from element at TAMU-NGES.

Ultrasonic tests. Figure 5-45 is the time history of the ultrasonic transducer signal obtained from Test 6. Reflections are observed at 0.002 s and 0.0047 s, which compares reasonably well with reflections observed from impact test data for the beginning of the bonded zone and the end of the element, respectively. Table 5-34 compares UT results for all the elements tested at the site with the results from impact testing. Reflections from the distant ends of the elements were not observed in every test result.

The comparison of results from ultrasonic and impact testing is considered good. The average ratio of UT result to impact test result is 1.026, and the data have a coefficient of variation of approximately 8.4 percent.

TABLE 5-34 Comparison of results from ultrasonic and impact testing at TAMU-NGES

Test No.	Ratio UT/Impact Test Result	
	($L_{unbonded}^{UT}/L_{ubonded}^{impact}$)	($L_{total}^{UT}/L_{total}^{impact}$)
1	1.04	N.O.[1]
2	1.07	N.O.
3	1.17	N.O.
4	0.87	1.04
5	1.00	N.O.
6	1.04	1.08
7	1.01	N.O.
8	1.06	1.01
9	1.13	1.00
10	1.04	0.83

[1] Not observed.

5.2.8.3 Conclusions from NDT

Results from NDT are consistent with the anticipated performance of the elements. Element distress was not observed from the results of NDT performed on elements at the TAMU-NGES. Given the soil conditions, the age of the elements, and the degree of corrosion protection afforded to the elements, distress was not anticipated.

Features observed in the impact test data are consistent with known details of the installation including levels of prestress, and the unbonded, bonded, and total lengths of the elements. Data from UT were in good agreement with results from impact testing. Thus, similar results were obtained from two independent testing techniques.

Results from the field study performed at TAMU-NGES indicate that the NDT technologies applied in this research can be useful for validating the existing condition of the elements and for identifying elements where geometry or load carried by the element is anomalous relative to other elements tested. Anomalous features so identified may be correlated with element distress. Therefore, the NDT technologies are useful devices for element condition assessment.

CHAPTER **6**

INTERPRETATION, APPRAISAL, AND APPLICATIONS

Transportation agencies are faced with the task of maintaining existing installations of buried metal-tensioned systems and managing the operation of transportation facilities. Reliable estimates of remaining service life are necessary for rational decisions relative to rehabilitation or retrofit. Available service-life prediction models must be verified and calibrated with well-documented performance data, and reliable methods of condition assessment must be developed for existing systems.

Major products of this research include the evaluation and application of several NDT techniques for condition assessments and the preparation of a working plan and recommended practice for obtaining performance data and estimating the remaining service life of existing metal-tensioned systems.

Field studies were conducted at eight different installations of metal-tensioned systems in the northeast, southeast, and southwestern United States. These field studies serve to validate the work plan and recommended practice, demonstrate application of NDT, and provide performance data for future reference.

6.1 SUGGESTED WORK PLAN AND RECOMMENDED PRACTICE

The suggested work plan in Chapter 3 and recommended practice in Chapter 4 describe the use of NDT for condition assessment of metal-tensioned systems and estimation of remaining useful service life. The NDT techniques are useful in determining whether corrosion can, or has, occurred and whether an element is currently distressed. The work plan describes administration of a systemwide agency program for evaluating existing metal-tensioned systems. The recommended practice describes details of implementing the condition assessment.

Four different NDT techniques are recommended, including measurement of half-cell potential, measurement of polarization current, impact echo testing, and UT. Half-cell potential and polarization current measurements are electrochemical tests used to determine whether corrosion is present and to evaluate the level of corrosion protection afforded to the system. The impact echo and ultrasonic tests are mechanical wave propagation techniques used to evaluate the current condition of an element.

6.1.1 Suggested Work Plan

The suggested work plan describes a rational approach to estimate future maintenance, rehabilitation, and retrofit needs for existing installations of metal-tensioned systems. The plan has four basic components: (1) develop an inventory of sites with installations of buried metal-tensioned systems within the agency's jurisdiction, (2) establish priorities regarding the need for detailed evaluation of site and element conditions, (3) formulate and implement a test protocol for condition assessment, and (4) formulate a recommended action plan.

Establishing an inventory is a necessary first step, but may represent a major effort on behalf of the agency. This inventory can help define the magnitude of the problem, identify the types of facilities inherent to construction practices in the region, and provide a means to screen facilities according to the need for more immediate attention. Screening is an assessment of risks associated with site corrosion hazards and installed metal-tensioned system vulnerabilities. The testing protocol describes the subsurface information required and the sequence of tests for NDT of the elements.

Results from the service-life prediction and the condition assessment are compared to formulate a recommended action plan. Recommended actions may include no action; further NDT; invasive testing; or design of rehabilitation for, or retrofit of, the existing metal-tensioned system.

6.1.2 Recommended Practice

The recommended practice describes procedures and input necessary for performing element condition assessments. The practice describes a corrosion assessment model, a sampling strategy for element condition assessment, and parameters and input required for service-life prediction modeling.

Corrosion is identified as a major source of distress for metal-tensioned systems. Chapter 4 presents simple decision trees that describe application of the corrosion assessment model. The model requires details of the installation, including the level of corrosion protection afforded to the system and the subsurface conditions. A few soil parameters are required for assessing the aggressiveness of the subsurface conditions relative to corrosion, and some sampling and laboratory testing of soils is also required.

A sampling strategy is needed because, at many sites, it is unfeasible to test every element. The strategy is based on a statistical analysis, but a background in statistics is not required for application of the charts, which are presented to select sample size. The charts allow the user to determine sample requirements on the basis of the total number of elements at the site, the importance of the facility relative to the consequences of failure, and the anticipated level of performance.

Available service-life prediction models are used to estimate the corrosion rate and the anticipated loss of element cross section. The prediction models require results from testing soil, groundwater, and rock samples. The practice cites relevant test standards for sampling and testing soil, groundwater, and rock. Equations and monographs from relevant literature are presented for estimating corrosion rate. The models are the result of previous studies conducted on uniform corrosion of buried metal specimens, corrosion of buried metal culverts, and corrosion of steel strips used for soil reinforcement.

6.2 PERFORMANCE DATA

Results from the field study are included within the framework of a database summarizing performance data of existing metal-tensioned systems. The database provides needed information for (1) validation, calibration, and improvement of risk assessment and (2) service-life prediction models.

Following are general interpretation comments based on data collected and evaluated at eight different field sites:

- Results from the field study support the need for system inventories and use of corrosion assessment models to correctly identify sites where the occurrence of corrosion is likely.
- In general, no significant loss of cross section is observed for installations less than 20 years old. This observation is consistent with estimates of remaining service life that indicate that distress is unlikely, even without corrosion protection, unless the ground conditions are very aggressive. For resin-grouted rock bolts, there may be loss of grout cover because of flow of grout into rock fractures during installation.
- Element distress can be identified by interpreting results from NDT with respect to known features of the element. If complete details of installations are unavailable, compare NDT results from a number of elements at a site to identify anomalies.
- Most of the observed element distress is located along the free length within 2 m of the element face. At a site with a history of creep movement, NDT results indicated distress along the bonded length of an element.
- Results from impact tests may be useful to detect loss of element prestress.

6.3 COSTS AND BENEFITS FROM PROPOSED WORK PLAN

The utility of maintaining the database may be appreciated by cost-benefit analysis. The costs of implementing the proposed work plan and maintaining a performance database are compared with the benefits (i.e., the associated cost savings related to maintaining, rehabilitating, or retrofitting an existing metal-tensioned system) and with the risks and costs associated with element failure.

Costs of condition assessments are those related to (1) planning the work, including a preliminary field visit; (2) acquiring equipment required for NDT; (3) getting access to element head assemblies; and (4) meeting workforce requirements for performing NDT, data interpretation, and reporting.

Benefits from implementing the proposed work plan and maintaining performance data on buried metal-tensioned systems include improved resource allocation and carefully planned capital expenditures. An effective plan for retrofit can be implemented when factors contributing to element service life are identified. Improved service-life estimates allow planning and evaluation of alternatives for the most cost-effective retrofit design.

Resource allocation based on observed performance and estimated remaining service life is an improvement over current practice, where resources are applied in reaction to an observed failure or to a severe loss in service. The latter practice may result in additional costs related to injury or consequential damages and to implementation of unnecessary retrofit measures during crisis. Element failures and unnecessary retrofit may be avoided if element condition is monitored over time.

6.4 LIMITATIONS OF THE WORK PLAN

Details of limitations for each test method are provided in Chapter 3. The limitations described here are more general, as they apply to the proposed work plan and recommended practice.

The ability to test strand-type elements is limited, and strand-type elements are sometimes employed for rock and ground anchor installations. The impact test is not applicable because no method has been developed to monitor vibration (i.e., it is unfeasible to attach an accelerometer to the surface of the strand). Application of the ultrasonic test appears possible, but there is limited experience monitoring strand elements. According to the observations documented so far, application of the ultrasonic test to strand elements is promising.

High-strength, quenched and tempered steels, which are sometimes used in the manufacture of strand-type elements, are particularly vulnerable to stress crack corrosion and hydrogen embrittlement. Brittle failure from stress crack corrosion or hydrogen embrittlement occurs suddenly, which is dangerous and aggravated by the fact that ground anchors may be subjected to high prestress. It is impossible to monitor these corrosion processes using the electrochemical tests described in the recommended practice.

Stray currents in the ground are a significant hazard relative to corrosion. The electrochemical tests cannot be applied where stray electrical currents are present unless the stray currents can be eliminated during the monitoring period.

Access to the ends of the element is required for NDT. At some sites, this access may be difficult to bet because of location (e.g., if the site is very high with respect to the highway) or because the head of the elements are encapsulated. Evaluation at some sites may be more expensive because of the need for special equipment—such as lifts, cranes, and scaffolding—and because of the time required to remove and replace the encapsulation. Specially qualified personnel may also be required to set up rigging and provide specific details of the elements.

NDT can be used to detect and locate a defect. However, loss of cross section less than 25 percent is difficult to detect with NDT. Therefore, the tests are useful to indicate when distress has reached significant levels, but the tests cannot indicate the initiation of a problem. The size, shape, and nature of the defect or anomaly are not determined using the data-processing techniques developed so far. It is difficult to distinguish an anomaly related to installation details from loss of cross section due to corrosion or other types of in-service distress.

The length of an element that can be detected with NDT is limited. Good results have been obtained for element lengths of approximately 10 m, and some element lengths as long as 20 m have been detected. Success depends on how much of the element is surrounded by grout. However, information along the free length of a long element may still be obtained, and many times problems occur within the first few meters from the anchor head assembly.

Results from NDT must be verified from direct physical observation of element condition, invasive testing, or both. NDT is useful because it identifies elements requiring application of more expensive, time-consuming invasive tests.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

7.1 CONCLUSIONS

This report describes the application of several NDT techniques for condition assessment and estimation of remaining service life of buried metal-tensioned systems, including ground anchors, rock bolts, and soil nails. Equipment for performing the NDT is commercially available, and the NDT may be performed by people with limited specialized training. However, knowledge of corrosion processes, wave mechanics, and signal processing are helpful for data processing and interpretation of the NDT, and these tasks should be performed by a qualified engineer.

Electrochemical tests, including half-cell potential and polarization measurements, are useful for indicating whether corrosion is present and for assessing the integrity of existing corrosion protection systems. However, results from these tests do not indicate the severity of corrosion or the location of the corrosion along the length of the element. Results from electrochemical tests can be supplemented with results from the impact response test, the ultrasonic test, or both to identify and locate defects along the length of the element.

NDT results are qualitative in the sense that data from each element at a site can be compared with known installation details, with each other, and with signatures that represent the response of typical elements. Test data may be evaluated for attributes such as good versus bad or conforming versus nonconforming. Results from NDT must be supplemented with more certain, detailed information from records documenting element construction and from invasive tests (e.g., performance load tests). Because invasive tests are more costly and time consuming to perform, the value of NDT is to screen and identify element locations where more detailed invasive testing should be recommended.

This report proposes a working plan and recommended practice that describe a program for evaluating existing metal-tensioned systems and implementing NDT, condition assessment, and estimation of remaining service life. The recommended practice includes a simple decision tree to identify sites with a high risk of corrosion. Remaining service life is estimated using equations and nomographs, which relate rate of corrosion to factors associated with corrosivity of the surrounding soil or rock mass.

Application of the working plan and recommended practice are demonstrated using an inventory of eight sites in the northeast, southeast, and southwest United States. The inventory represents a variety of different installations of metal-tensioned systems. Results from the field studies contribute to a database documenting the performance of in-service, metal-tensioned systems. Performance data obtained so far are consistent with (1) risk assessment models that identify sites where corrosion is likely and (2) mathematical models of service life that estimate rate of corrosion. Although corrosion was observed at many of the sites, significant distress was not identified at sites with installations less than 20 years old and with ground conditions that were not highly aggressive relative to corrosion.

7.2 RECOMMENDATIONS FOR FUTURE RESEARCH

The following sections address the need for (1) further verification of results from NDT; (2) personnel training to implement the work plan; (3) improvements in testing techniques, data processing, and interpretation; and (4) improved service-life prediction models.

7.2.1 Verification Studies

Results from NDT have been verified in the laboratory at bench scale and at an in-situ testing facility using relatively simple element installations and geometries. Condition assessments conducted in the field produced results that correlated very well with known features of the installation. Results from these studies serve as necessary, but insufficient, conditions for verifying NDT. In each of these trials, the person conducting the condition assessment was aware of features of the installation and aware of the location and nature of any anomalies present (i.e., the assessments were not demonstrations of “blind predictions”).

Element distress was indicated in the NDT data at two of the sites included in the field study. According to the site conditions and past performance of the facilities, the presence of distress at the locations indicated by NDT is very likely. However, a direct visual examination of the elements after NDT and invasive testing was impossible, so the actual condition of the elements is uncertain.

According to the laboratory test results, in situ testing, and performance data obtained so far, there is strong evidence that the NDT described in this research has merit, but uncertainty about the precision, accuracy, sensitivity, reliability, and limitations of the NDT remain. There is a need to examine the application of NDT under field conditions where corrosion conditions and details of distressed elements are known. To make the field observations serve as a "blind prediction," known details of the elements must not be shared with members of the evaluation team. Therefore, an agency separate from the evaluation team should be responsible for installation of the test elements.

Elements should be installed in the field using commonly employed construction techniques and hardware. Approximately 10 elements should be installed, some intact and others with corrosion and/or installation defects. The evaluation team should be provided with information normally obtained prior to a condition assessment (e.g., subsurface conditions; total, free, and fixed lengths of the elements; drill hole diameter; level of prestress; features of the corrosion protection system; grout type; and details of the anchor head assembly), but should not be given any information about the type or location of defects or corrosion conditions. The evaluation team will perform a condition assessment at the test site, process and interpret the data, and report its findings. A review panel will compare the findings with known details of element distress and tabulate the results.

The results of the study will be useful for evaluating the precision, accuracy, and reliability of the test methods. This information is valuable for assessing future results from NDT and quantifying error. Improved confidence in the results from NDT may reduce the number of invasive tests recommended before a decision is made to design a retrofit or implement a rehabilitation strategy. This reduction of invasive tests will lead to lower costs associated with performing site evaluations and implementation of the proposed work plan.

7.2.2 Training Program

Specialized training is necessary to provide awareness and promote implementation of the proposed work plan and recommended practice, including condition assessment and application of NDT. Development of training materials is recommended to familiarize people in charge of operations with the recommended working plan and to prepare the appropriate staff to carry out the working plan, perform NDT, and interpret data for condition assessment.

A 2-day workshop is envisioned, covering topics related to implementation of the work plan and recommended practice. A workshop format provides the opportunity for participants to gain "hands-on" experience with the tools included in the work plan and recommended practice, including performance of NDT.

The first day of the workshop could be devoted to a general overview of the products from this research. A case study might be developed wherein participants are afforded an opportunity to apply the proposed risk assessment and service-life prediction models. By following the proposed work plan and recommended practice, workshop participants using the case study would get an overview of condition assessment, including application of sample criteria, use of NDT, interpretation of results, and formulation of an action plan.

The second day of the workshop might be a primer on the recommended NDT technologies. Well-known and established concepts, theories, and technologies contribute to researchers' understanding of the corrosion process and the basis for NDT. Most engineers are familiar with the underlying principles of corrosion and NDT, which are covered within the basic coursework requirements at accredited four-year college and university engineering degree programs. However, many civil engineers involved in the operations of transportation facilities are not familiar with the specific applications of NDT, performance monitoring, and condition assessment. Furthermore, the equipment and instrumentation used in pursuit of the recommended NDT methods are not traditional tools applied within the civil engineering practice. The contents of the workshop should include the principles of the test technique; equipment details; and performance of NDT, data acquisition, and data processing. Participants could be given the opportunity to perform NDT on bench-scale test specimens using a specially constructed portable demonstration unit.

The anticipated audience for the workshop would be engineers and technical staff at state transportation agencies. Operation managers may be more interested in the first day of the workshop, and technical staff may be more interested in the second day. The workshop could be custom tailored for presentation at specific DOT sites in an effort to best address (1) the local inventory of metal-tensioned systems and (2) the efforts made toward management and operation of transportation facilities. Workshop materials could be made available to a wider audience through the development of a CD-ROM and internet-based media intended for individual study. The digitized versions would include the contents and supporting materials of the workshop prepared in an interactive form for self-training purposes.

7.2.3 Improvements in NDT Techniques

To increase data quality and consistency, improvements to hardware should be implemented. The impact tests and ultrasonic tests employ hardware (i.e., an impact device and ultrasonic transducer) for delivering the excitation at the face of the element. Important characteristics and parameters related to the excitation should be identified and incorporated in the design of the delivery system.

7.2.3.1 Impact Tests

Currently, the impact is administrated with a hammer and a hand-held punch. This method is useful for directing energy along the axis of the element, but the duration, amplitude, and frequency content of impacts are not controlled and are not repeatable.

The loss of wave energy at each reflection relates to the size of the defect and is manifested in the signal attenuation or change in amplitude associated with each arrival of the reflected wave. Wave scattering relates to the shape of the defect and may be observed by the length of the reflected signal. It is necessary to control the impact directed at the end of the element to study the trends associated with each of these features of the signal.

Future development of the impact test should include use of a hammer capable of applying a controlled, repeatable impact. If this development is accomplished, more subtle features of the signal—such as amplitude attenuation, damping, and scattering—could be compared between different tests. This ability would allow more detailed signature analysis and system identification to be performed, as described in Section 7.2.4.1.

Results from research performed by others on applications of the impact test to concrete plates and shells and to rock bolts describe the design and application of special devices for applying the impact. These devises are commercially available, although they are not necessarily designed to perform optimally for condition assessment of metal-tensioned systems. The recommended research should explore the use of existing designs and recommend modifications, if necessary, to achieve improved performance for condition assessment of metal-tensioned systems. If necessary, prototypes should be designed and manufactured, and the performance of the prototypes should be demonstrated on tendons installed using commonly employed construction techniques and hardware.

7.2.3.2 Ultrasonic Tests

Ultrasonic waves propagating in a slender element are dispersive in nature. Therefore, for elements that are surrounded by grout, wave energy is lost by dispersion into the surrounding medium. The effect of dispersion is to limit the length of the element that may be probed with the UT technique. Through improvements in transducer design, it may become possible to probe longer distances along the tendons.

Parameters associated with the design of the ultrasonic transducer, including the frequency content of the excitation signal and the size and mass of the transducer components, affect the transducer performance for a given application. Dispersion is less for high-frequency sound waves, which have a small wavelength compared with the cross-sectional dimensions of the specimen. However, a low-frequency sound wave suffers less signal attenuation traveling through steel. These two conflicting requirements must be balanced to achieve an optimum transducer design.

In this study, two commercially available transducers were employed having predominant frequencies of 100 kHz and 55 kHz. Because commercially available, “off-the-shelf” transducers were obtained, they are not necessarily optimized for this application. The optimum transducer design may vary depending on type of strand and details of the installation.

Research is recommended to identify the parameters of the transducer design that affect the performance of the ultrasonic test for condition assessment of buried metal-tensioned systems. After the significant parameters are identified, prototype ultrasonic transducers that are tuned to the specific application should be designed and manufactured. The performance of the prototypes should be demonstrated on tendons of different lengths installed using commonly employed construction techniques and hardware.

7.2.4 Improvements in Data Processing and Interpretation

Relatively simple techniques are described in this report for interpretation of NDT data. These techniques rely on an existing body of knowledge related to the system response and interactions with the surrounding environment. Information about element condition beyond what current mathematical models and techniques are able to identify and process is contained in the data obtained from the NDT. Limited interpretation does not allow full realization of the benefits of NDT. This section recommends research to extend the existing body of knowledge related to data interpretation, to identify relatively subtle features of the system response, and to provide more details on the nature and characteristics of observed element distress.

7.2.4.1 Electrochemical Tests

Measurement of half-cell potential appears to be a useful indicator of the presence of corrosion. However, ranges of half-cell potential cited in the literature for discerning corroding from noncorroding elements are based on data collected from carbon-based steel in neutral soil and water. These conditions are different from those surrounding metal-tensioned elements. Many metal-tensioned elements are surrounded by cement, polymer, or epoxy resin grouts. The effect of the grout type on measurement of half-cell potential should be investigated. This study is necessary to identify conditions where half-cell potential measurement may not be meaningful and to study how the potentials of the steel elements may be affected by different chemical environments.

7.2.4.2 Impact Tests

The method for interpreting impact test results described in this report is useful to identify the location of defects or anomalies along the length of an element. However, information about the size and shape of defects is not obtained. Unless details of the installation are available, it is difficult to distinguish element distress from features of the installation (i.e., reflections may be observed from couplings that are not distinguished from broken tendons). Data interpretation also does not, at present, distinguish among different types of distress (e.g., reflections caused by severe grout cracking from reflections caused by loss of element cross section).

The current data-processing technique applied to impact test results does not consider details of the dynamic response of the element, including signal attenuation, scattering, and damping. Although the size and shape of anomalies along the length of the element, as well as the level of prestress, influence the characteristics of the dynamic response, no simple relationship exists. The relationship needs to be studied using laboratory and numerical analyses. Numerical analyses allow for the use of computer simulation to estimate the mechanical behavior under sets of known conditions that vary over a very large domain. Laboratory experiments help to verify the numerical results and assumptions used in the analysis.

Applications of the impact echo test to concrete plate and shell-type structures described in the literature demonstrate the use of numerical analyses to develop signatures for system identification. Some work has also been done by others demonstrating how element prestress can be correlated with the dynamic response of the element. Numerical analysis packages are available and can be applied to this problem, but the number of different scenarios to analyze needs to be identified.

This numerical analysis is considered a long-term goal that involves a considerable research effort. The research should first be attempted as a pilot project, focused on a single type of element (e.g., resin-grouted rock bolts). A typical defect shape based on the observed condition of distressed elements exhumed from the field should be selected for the pilot study. Numerical analysis should be used to develop data showing how the size of the defect is represented in element signatures.

If the results are encouraging, the research could be continued to another, more detailed phase. The product of this recommended research effort will be a database of signatures describing different element conditions. These signatures could then be used for system identification and “finger printing” of element conditions observed from NDT.

7.2.5 Improvements in Service-Life Prediction Models

High-strength steels used to manufacture prestressing strand elements, and sometimes used as tendons for ground anchors and rock bolts, are vulnerable to hydrogen embrittlement and corrosion stress cracking. Service-life prediction models described in this report were mainly developed from observation of uniform corrosion of buried specimens of mild steel. There are limited data on rates of surface pitting, but no techniques exist to predict when an element may fail from hydrogen embrittlement or corrosion stress cracking. A major part of the problem is that the mechanisms contributing to these types of corrosion are not completely understood; therefore, factors controlling the rate of deterioration are not identified.

Environmental factors such as pH, soil resistance, the presence of sulfates and chlorides, and level of prestress are known to contribute to the occurrence of stress crack corrosion and hydrogen embrittlement. Ongoing research is attempting to achieve accelerated corrosion in the laboratory that may allow for scaling with respect to time. If successful, these techniques may be used to collect corrosion data under controlled laboratory conditions, leading to the development of service-life prediction models.

The recommended scope of future research is to identify, document, and evaluate results from recent and ongoing studies of hydrogen embrittlement and stress crack corrosion. The utility of these results and the need for further research to achieve a better understanding of the impact of these corrosion mechanisms on the service life of buried metal-tensioned systems should be evaluated.

A series of experiments should be performed under different environmental conditions and levels of prestress to understand (1) the relationship between relevant factors and time to failure of buried metal-tensioned, strand-type elements and (2) the effectiveness of currently applied corrosion protection systems. For these tests to be feasible, it may be necessary to apply techniques to accelerate corrosion in the laboratory. Therefore, the ability to perform this task may depend on the results of ongoing research.

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APPENDIX A

RECOMMENDED PRACTICE FOR EVALUATING METAL-TENSIONED SYSTEMS USED IN GEOTECHNICAL APPLICATIONS

AASHTO Designation

1. Scope

1.1 This Standard Guide is focused on condition assessment and estimation of remaining useful service life of rock bolts, ground anchors and soil nails used as buried metal-tensioned elements for geotechnical applications.

1.2 The Standard Guide describes a corrosion assessment model, a sampling strategy for element condition assessment, and parameters and input required for service-life prediction modeling.

1.2.1 The Standard Guide incorporates nondestructive test (NDT) methods including half-cell potential, polarization current, impact response, and ultrasonic wave reflection measurements for element condition assessment. Details of these test techniques are described in Appendices C, D, E and F.

1.3 The Standard Guide describes a rational approach for estimating future maintenance, rehabilitation and retrofit needs for existing installations of metal-tensioned systems (MTS).

2. Preamble

2.1 Transportation agencies are faced with the task of maintaining existing installations of buried metal-tensioned systems and managing the operation of transportation facilities. For many, the current practice is to react to situations as they arise. This approach is not cost-effective, makes no effort to manage risk, and does not allow the agency to plan for allocation of resources.

2.2 Reliable estimates of remaining service life are necessary for making rational decisions about rehabilitation, or retrofit.

2.2.1 Estimates of service life provide a basis for comparison with condition assessment.

2.2.1.1 Service-life prediction models are used to estimate the service life of unprotected elements. For systems installed within oxygen deprived environments, or protected by passivation with grout, the rate of corrosion may be considerably less than that anticipated by the service life prediction model.

2.3 NDT methods are used for condition assessment including the integrity of corrosion protection systems, and for detecting significant loss of cross section, or loss of anchorage.

2.3.1. Corrosion protection systems are necessary for the extended service life of buried MTS. Condition assessment of corrosion protection systems provides an indicator to element vulnerability.

2.3.2 Results from NDTs may be used to assess the distress to an element such as loss of cross section from corrosion, or other factors. NDT is not a direct measure of distress and results are subject to interpretation.

2.4 Performance monitoring and service-life prediction of buried metal-tensioned systems are complex problems. This Standard Guide describes a general approach and methodology to be followed.

2.4.1 NDTs and estimates of service life have limited application, and results are uncertain. Important limitations to the service life prediction model and NDTs used for condition assessment are described in Section 6.

3. Referenced Documents

3.1 AASHTO Standards:

T88	Particle Size Analysis of Soils
T89	Determining the Liquid Limit of Soils
T90	Determining the Plastic Limit and Plasticity Index of Soils
T263	Chemical, Biological and Physical Analysis of Water
T265	Laboratory Determination of the Moisture Content of Soils
T288	Determining Minimum Laboratory Soil Resistivity
T289	Determining pH of Soil for Use in Corrosion Testing
T290	Determining Water Soluble Sulfate Ion Content in Soil
T291	Determining Water Soluble Chloride Ion Content in Soil
R026	Standard Practice for Assessment of Corrosion of Steel Piling for Non-Marine Applications

3.2 ASTM Standards

D2113	Diamond Core Drilling for Site Investigation
D2488	Description and Identification of Soils (Visual-Manual Procedure)
D2487	Standard Classification of Soils for Engineering Purposes (Unified Soils Classification System)
D4220	Preserving and Transporting Soil Samples

3.3 Post Tensioning Institute (PTI)

"Recommendations for Prestressed Rock and Soil Anchors"

3.4 American Concrete Institute (ACI)

ACI 222.2R-01, "Corrosion of Prestress Steel"
ACI 423.4R-98, "Corrosion and Repair of Unbonded Single Strand Tendons"

4. Terminology

4.1 Active systems are prestressed during installation.

4.2 Passive systems are loaded as the soil or rock material deforms during installation.

4.3 Bar is a solid metal element having a cross section manufactured from a single piece of material. The bar may be smooth or corrugated with threaded ends, or continuously threaded.

4.4 Strand is an element type with a cross section comprised of multiple wires that are twisted around a central or king wire.

4.5 F_{PU} is the specified minimum tensile strength of the element.

4.6 Passivity refers to the loss of chemical reactivity experienced by certain metals and alloys under particular environmental conditions.

4.7 Pitting is localized corrosion where the corroded area has a width to depth ratio less than four.

4.8 Stray Direct Currents are present in the ground as a result of electrical leaks, or failure to provide positive and permanent electrical grounding.

4.9 Hydrogen embrittlement is the migration of atomic hydrogen into the metal lattice where hydrogen molecules are formed producing internal pressure in the metal.

4.10 Stress Corrosion Cracking is a type of locally concentrated corrosion defined as cracking that may result from the combined action of corrosion and static tensile stress.

4.11 Creep is the time-dependent deformation of material under the action of constant load.

4.12 Hazard is the presence of conditions that make the occurrence of corrosion possible, or increase the likelihood that corrosion may occur for unprotected or inadequately protected elements.

4.13 Vulnerability is the assessed ability of the installed metal-tensioned system to resist attack from corrosion.

4.14 Risk is the combined consideration of hazard and vulnerability.

4.15 Service-life is the length of time that a system or component performs its intended design function.

4.16 NDT are non-intrusive, nondestructive tests whereby the test element is not compromised by testing and does not need to be taken out of service during or after testing.

5. Significance and Use

5.1 Transportation agencies are faced with the task of maintaining existing installations of buried metal-tensioned systems and managing the operation of transportation facilities. Reliable estimates of remaining service-life of buried metal-tensioned systems are necessary for rational decisions relative to the need and schedule for rehabilitation or retrofit.

5.2 This Standard Guide is used to: a) establish priorities regarding the need for detailed evaluation of site and element condition, b) formulate and implement a test protocol for condition assessment and detailed evaluation, and c) formulate a recommended action plan.

5.3 Available service-life prediction models should be verified and calibrated with well-documented performance data and reliable methods of condition assessment for existing systems where available. Much of this data is difficult to obtain and not readily available.

6. Limitations

6.1 Condition assessment described in this Standard Guide is subject to the following limitations:

6.1.1 Access to the ends of the element is required for condition assessment and detailed evaluation. At some sites this may be difficult due to location (e.g., height with respect to the base of a slope or wall), or because the head of the elements are covered or encapsulated in grout, or concrete and may be blocked by structural and architectural elements.

6.1.1.1 Evaluation at some sites may be more expensive because of the need for special equipment, such as lifts, cranes, and scaffolding, and the time required to remove and replace the encapsulation.

6.1.2 Available test methods used for condition assessment are not generally applicable to all element types, particularly with respect to strand type elements.

6.1.2.1 The ultrasonic test may work well for strand elements, but has not been evaluated with wedge plates in place.

6.1.2.2 The impact test is not recommended for use with strand type elements.

6.1.3 Elements must be electrically isolated for electrochemical testing.

6.1.3.1 Wall tiebacks are not often electrically isolated due to waler connections, although isolation may be achieved for systems employing plastic trumpet assemblies. In some instances, elements may be decommissioned and isolated from wedge sets or bearing plates before testing.

6.1.3.2 Rock bolts are often electrically isolated.

6.1.3.3 Soil nails need to be isolated from steel mesh used to reinforce wall facing.

6.1.4 Available test methods have limited sensitivities.

6.1.4.1 Less than about a 25% loss of element cross-section is difficult to detect. This is a significant loss of cross section, which for stand elements is near failure, and for bar elements is approximately a 30 percent increase in stress level.

6.1.4.2 Test results may indicate when distress has reached significant levels, but cannot indicate the initiation of corrosion.

6.1.4.3 The size, shape and nature of the defect, or anomaly, cannot be determined using existing data processing techniques. It is difficult to distinguish an anomaly related to installation details from loss of cross section due to corrosion, or other types of in-service distress.

6.1.4.4 The length of an element that can be detected with NDT is limited. Good results have been obtained for element lengths of approximately 10 m and some element lengths as long as 20 m have been detected. Success depends on how much of the element is surrounded by grout. However, information along the free length of a long element may still be obtained, and most often problems occur within the first meter or two from the element head assembly.

6.1.4.5 Results from NDT must be verified from direct physical observation of element condition and/or invasive observations. The utility of the NDT is to identify elements requiring more expensive, and/or time-consuming, invasive observations.

6.1.5 It is not possible to monitor stress crack corrosion and hydrogen embrittlement using the condition assessment and detailed evaluation described in this Standard Guide. This is very important as it applies to strand type elements.

6.1.5.1 High-strength (F_{pu} > 1000 MPa) quenched and tempered steels are particularly vulnerable to stress crack corrosion and hydrogen embrittlement. These steels are not commonly used for ground anchor, rock bolt or soil nail installations in North America.

6.1.5.2 Brittle failure from stress crack corrosion or hydrogen embrittlement occurs suddenly, which is dangerous. Stress crack corrosion and hydrogen embrittlement are aggravated by the fact that ground anchors may be subjected to high prestress ($\sigma_{prestress} > 0.5F_{pu}$).

6.1.6 The electrochemical tests, which are part of the detailed evaluation, cannot be applied at sites where stray electrical currents are present, unless the stray currents can be eliminated during the monitoring period.

6.1.6.1 Stray electrical currents in the ground are a significant hazard relative to corrosion.

6.2 Service-life estimates described in this Standard Guide are subject to the following limitations:

6.2.1 Service life prediction models are empirical and not an exact prediction of service life. In general, they describe trends relating the effects of time and simple descriptions of soil aggressivity to element condition in terms of loss of cross section for bar elements.

6.2.2 Service-life predictions are based on uniform corrosion processes and cannot predict rate of corrosion from hydrogen embrittlement and corrosion stress cracking. They are a useful starting point, but for extreme conditions, actual rates of corrosion may be higher than estimated.

6.2.2.3 Results from corrosion assessment and NDT will provide useful performance data, which can be applied to calibrate service life predictions.

7. Types of Metal-Tensioned Systems

7.1 Geotechnical applications of metal-tensioned systems include ground anchors, rock bolts and soil nails. Table A-1 summarizes key features of the different types of metal-tensioned systems.

TABLE A-1 Summary of types of metal-tensioned systems

Type of Metal-Tensioned Systems	Tendon Type	Anchorage Type	Corrosion Protection
Ground Anchors	Strands or Bars	Cement Grout in Bonded Zone	More recent permanent installations use Class I or Class II Protection (PTI, 1996); older systems may have no protection other than grout cover.
Rock Bolts	Usually bars, but could be strand	Mechanical, Resin Grout, or Cement Grout	Epoxy coating, galvanization, grout cover, older installations may have none
Soil Nails	Bars	Cement grout entire length	Grout cover, bars may be epoxy coated

7.2 Tensioned elements of the system include bar and strand components. The steel grade and level of prestress employed in these systems are relevant to the type of corrosion problems that may occur and to the prediction of service life. Bar elements are available in a variety of steel grades ranging from Grade 400 to 1100. Strand elements are manufactured from Grade 1700 and 1950 high-strength steel. Wire tension systems, using the button head anchorage, have been used in some early applications.

7.3 Current guidance documents (FIP, 1996; FHWA, 1996, 1998; PTI, 1996) recommend incorporating corrosion protection measures into the design of metal-tensioned systems. Corrosion protection measures include the use of coatings, protective sheaths, passivation with grout, encapsulation and electrical isolation. Corrosion protection has been recommended for most permanent installations in the United States since approximately 1985.

7.3.1 Ground anchors include an anchored or "bonded" zone and a free length or "unbonded" zone. The bonded zone is anchored to the soil or rock with cement grout. Recent installations use Class I or Class II protection as recommended by PTI (1996). For Class I protection the anchor is encapsulated (often referred to as double corrosion protection) and for Class II the anchor is protected by grout (often referred to as single corrosion protection). Double corrosion protection is recommended for ground anchors in aggressive ground conditions and permanent installations. More protection is needed for active systems because these usually involve stress levels greater than 50% of ultimate ($\sigma_{prestress} > 0.5F_{pu}$). Many of the installations do not incorporate details that meet current standards, or were installed without any corrosion protection beyond the passivation of the grouted portion of the tensioned elements.

7.3.2 Rock bolts have mechanical or grouted anchorages using cement grout or resin. Older style rock bolts with mechanical anchorages may have no corrosion protection. Grouted or resin grouted rock bolts are surrounded by grout, but the bolt heads are often not encapsulated. The possibility exists for voids along the grouted length and under the anchor plates, particularly for resin grouted systems using pre-measured cartridges.

7.3.3 Soil nails typically have a full-length grouted anchorage using cement grout. Soil nails are passive systems that are not prestressed and generally subject to low working stress levels ($\sigma_{prestress} < 0.5F_{pu}$). As such, compared to other systems they are not as vulnerable to stress corrosion cracking or hydrogen embrittlement.

8. Performance

8.1 The main factors affecting the service life of metal-tensioned systems are corrosion, loss of prestress due to creep or loss of bond within the bonded zone, loading not considered in the design such as stress from bending, cyclic loading, ice loads or hydrostatic pressures, and anchorage failure.

8.1.1 Details of the corrosion process and types of corrosion are described by FIP (1996).

8.1.1.1 Particularly for the higher strength steel ($F_{pu} > 1000$ MPa), corrosion is often localized and evident in the form of pitting. Stress crack corrosion is aggravated by high tension from prestressing ($\sigma_{prestress} > 0.5\ F_{pu}$), which is often required for ground anchors and rock bolts.

8.2 The following observations are based on a review of the literature describing performance of metal-tensioned systems (FIP, 1986; NCHRP 2000):

8.2.1 Most reported corrosion problems are correlated with the presence of aggressive ground conditions as described in Section 14, or stray currents. For systems with a properly installed and intact corrosion protection system, corrosion problems have not been reported, even for aggressive ground conditions.

8.2.2 Nearly all documented corrosion problems were located within the free length of the element and most were within one meter of the element head. The performance and service life of metal-tensioned systems depend upon the details of the design, manufacture and workmanship during installation of corrosion protection systems, particularly with respect to encapsulation at the tensioned element head. If stray currents are present in the ground or in aggressive ground conditions, then the elements should be electrically isolated. For strand tendons, the sheathing should be extruded onto the strand stressing length. Care must be taken during transportation and installation of tendon elements not to damage sheathing or disturb the grease or corrosion inhibitor compound surrounding the metal element. If grease is heated by the sun, it may lose viscosity and flow, leaving the upper portions of the tendon element exposed. The type of grease or corrosion inhibitor should be selected such that it does not have an affinity for water, does not promote micro-bacterial induced corrosion and contains an effective corrosion inhibitor.

8.2.3 Very few documented cases describe corrosion problems located within the bonded zone. Cracking of the grout is anticipated and has been observed in the transition zone between the bonded zone and the free length. The cracks may compromise the ability of the grout to passivate the metal element and provide electric conductivity between the element and soil electrolyte, which facilitates corrosion. However, oxygen must be present for corrosion to occur, and the environment surrounding the bonded zone is often oxygen deprived. This may explain why loss of service from corrosion of ground anchors within the bonded zone is not an apparent problem.

8.2.4 If water is present within greased and sheathed strand type elements, the protective coating of grease may, eventually, undergo bacteriological degradation with associated byproducts including sulfur and organic acids. The environment created by these by-products is conducive to hydrogen embrittlement and stress crack corrosion. Based on this, it appears that metal-tensioned systems, with an unbonded grease protection system along the free length, may have a particular long-term possibility of corrosion, if the grease is not properly formulated and properly applied. Properly formulated grease includes a bacticide.

8.3 Compared to the small number of failures from corrosion, less information is available in the literature describing the effect of creep on service life of metal-tensioned systems. However, some information is available relative to evaluating conditions for which creep may be a problem (FHWA, 1996, 1998; PTI, 1996) and the performance testing of anchors used to evaluate the potential for creep deformations during the service life of the structure. Highly plastic clay-type soils surrounding the bonded zone are potentially susceptible to creep.

9. Summary of Approach

9.1 The systems approach described in this Standard Guide has four basic components:

9.1.1 Inventory of sites

9.1.2 Priorities for detailed evaluation

9.1.3 Detailed evaluation

9.1.4 Recommended action plan

9.2 It is strongly recommended that agencies catalog their inventory of rock bolt, ground anchor, and soil nail installations. The inventory should include site locations, installation records, and subsurface conditions.

9.2.1 Section 10 describes installation details needed for the inventory.

9.2.2 Parameters for assessing corrosivity of the subsurface environment are described in Sections 11 and 12.

9.2.2.1 Section 11 describes a preliminary investigation (Phase I) to obtain pertinent available information on the subsurface conditions at the site.

9.2.2.2 In the Phase II subsurface investigation described in Section 12, soil, rock and groundwater sampling and testing is performed as necessary.

9.3 Site priorities are established based on risk, as described in Section 14.

9.3.1 Site hazard is evaluated with respect to corrosion and creep. Section 14.3 describes the corrosion assessment model. A simple decision tree is used to identify the corrosion hazard at a site.

9.3.2 Section 14.4 describes the assessment of element vulnerability. A simple decision tree is used to identify installations that are vulnerable to corrosion or loss of anchorage.

9.3.3 Section 14.5 describes assignment of a risk-based priority index to each site in the inventory. Agencies may then schedule detailed evaluation at sites according to priority.

9.4 Detailed evaluation includes condition assessment and estimation of remaining service life as described in Sections 15 and 16.

9.4.1 It usually not feasible to test every element at a site, and a sampling strategy as described in Section 15.3 is needed. A sample size is recommended to establish a statistical basis for the test results. The recommended sample size is based on the total number of elements at the site, the importance of the facility relative to the consequences of failure, and the anticipated condition of the metal-tensioned system. More testing may be necessary for highly stressed elements ($\sigma_{prestress} > 0.5\ F_{pu}$).

9.4.2 A flow chart is presented in Section 15.4 describing the process for condition assessment and evaluation of buried metal-tensioned systems. Several NDTs are recommended. Data are analyzed and interpreted to determine if corrosion is occurring and locate any anomalies or signs of distress along the elements.

9.4.3 Remaining service life is estimated, as described in Section 16, using a mathematical model relating rate of corrosion to factors associated with the corrosivity of the surrounding soil or rock mass.

9.5 Results from service-life prediction and condition assessment are compared to formulate a recommended action plan, as described in Section 17.

9.5.1 Recommended action may include doing nothing, further NDT, invasive observation and testing such as lift-off tests, if possible, or design of rehabilitation or retrofit of the existing metal-tensioned system.

10. Installation Details

10.1 Possible sources of information for installation details include construction records available from State DOTs and/or element installers, and typical details provided by suppliers of metal-tensioned systems.

10.2 Pertinent information includes the element type, anchorage details, installation date, steel type, prestress and corrosion protection afforded to the system, and history of construction problems.

11.0 Phase I Preliminary Subsurface Investigation

11.1 The purpose of the Phase I Subsurface Investigation is to obtain pertinent available information on the subsurface conditions to aid in the assessment of soil corrosivity (corrosion hazard). Information may be available on (1) the presence of environmental contaminants, (2) characteristics of soil and rock at the site, and (3) location, fluctuation and chemistry of the groundwater.

11.2 Possible sources of information for the Phase I Subsurface Investigation include (1) local and U.S. Geologic Survey, (2) U.S.D.A. Soil Conservation Service, (3) U.S. and State DOTs, and (4) past or present construction activities near the project site.

11.2.1 The performance history of buried metal elements in the general area of the structure should be documented.

11.3 A Phase II Site Investigation is required unless:

11.3.1 The Phase I Site Assessment provides the necessary information outlined in the Phase II Site Investigation to establish the corrosion hazard at the site.

11.3.2 Priority for Phase II subsurface investigation should be given to those sites known to contain corrosive materials such as slag, cinders, ash, or other manmade products; clayey or layered sand/clay type soils; or aggressive groundwater conditions.

12 Phase II Site Investigation

12.1 The purpose of the Phase II Site Investigation is to obtain information on soil properties to characterize the aggressivity of soil and assess the potential for corrosion of buried metal-tensioned elements at the site.

12.1.1 The groundwater level should be measured and fluctuations in the level should be recorded.

12.2 Sample Collection

12.2.1 Soil, rock or groundwater samples should be retrieved that are representative of materials surrounding a metal-tensioned element. Several different soil and/or rock types may need to be sampled if conditions vary along the length of the element.

12.2.1.1 A relatively large sample is needed, due to the requirements of the soil resistivity test; approximately 1500 grams of soil sample finer than 2.00 mm (passing the #10 sieve).

12.2.2 Care should be taken during sampling to avoid contaminating the soil being sampled, mixing soil types, and loss of moisture during storage and sample transport to the laboratory. The intent, precautions and procedures of ASTM D4220 (Group B) are applicable to this Standard Guide.

12.2.2.1 Representative soil sample should be collected to a depth that is the lesser of 1 m below the water table or to the end of the element.

12.2.3 If possible, rock outcrops representative of rock bolt or ground anchor installations should be located, and the rock type identified by visual inspection. Rock joints should be observed and rock joints with infill materials that daylight at the outcrop should be sampled.

12.2.3.1 If no infilling is available for sampling, groundwater should be sampled. Groundwater should be sampled from monitoring wells if available, or from seepage zones at the face of the rock slope. Care should be exercised when sampling from seeps because the pH may be affected when water is oxidized upon contact with the atmosphere.

12.2.3.2 If no rock outcrops are available, rock samples should be obtained to a depth which is the lesser of 1 m below the water table or the end of the element by diamond core drilling techniques as described in ASTM D2113.

12.3 Laboratory Soil Testing

12.3.1 Description and identification

12.3.1.1 Soil samples should be visually examined in the as-received condition to determine uniformity/homogeneity and particle size.

12.3.1.2 A log of the various soil layers should be recorded and include description, identification, and thickness measurements of the soil layers. The location of rock infill should be recorded.

12.3.1.3 Applicable methods include ASTM D2488 and D2487. The Unified Soil Classification System (USCS) should be used to classify soil.

12.3.2 Soil Samples should be tested for physical and chemical properties as described in Table A-2.

TABLE A-2 Soil testing

Chemical Tests	Physical Tests
• Resistivity (AASHTO T288)	• Moisture Content (AASHTO T265)
• pH (AASHTO T289)	• Grain Size Analysis (AASHTO T88)
• Sulfate Content (AASHTO T290)	• Atterberg Limits (AASHTO T89 & T90)
• Chloride Content (AASHTO T291)	

12.3.2.1 Samples of rock joint infilling should be subjected to the same tests as soil samples.

12.3.3 Recommended test methods for analysis of water samples are described by AASHTO Test Standard T263. Table A-3 provides descriptions of five tests for qualitatively assessing the potential aggressiveness of groundwater.

13 Reporting

13.1 Report date, time, depth and method of sampling.

13.2 Report sample identification location and depth.

13.3 Report the position (depth) of the groundwater table.

13.4 Report test methodology if several methods are applicable.

14 Risk Assessment and Assignment of Priorities

14.1 The risk that metal-tensioned elements may fail to perform their function is evaluated in terms of: (a) the hazard inherent to a site, and (b) vulnerabilities related to the element installation details.

14.2 Separate ratings are assigned to hazard and vulnerability, which are combined into an index for screening and assigning site priorities.

14.3 Figure 1 is a decision tree that describes the corrosion assessment model. Corrosion hazard is described as low, medium, or high.

14.3.1 Table 3 provides parameter limits from five tests for qualitatively assessing the potential aggressiveness of groundwater. These parameters assess the aggressiveness of groundwater towards cement grout as well as metal surfaces.

14.3.1.1 The limits assume the groundwater is stagnant or flowing very slowly, and that the attack is immediate and unaffected by the presence of grout around the metal.

14.3.1.2 Table 3 is used by assigning the highest level of aggression from the results of any single test, or if the results of any two tests are in the upper quarter of any level, the next higher aggression level is assigned.

14.3.2 If poor drainage, creep, or excessive settlement at the wall face is recognized as a problem at the site, a condition assessment should be recommended. The presence of creep, settlement or poor drainage may mean that elements are subjected to loads

TABLE A-3 Parameter limits for aggressive groundwater conditions (Modified after Xanthakos, 1991)

Test	Aggressiveness		
	Weak	Strong	Very Strong
pH	6.5 - 5.5	5.5 - 4.5	< 4.5
Lime-dissolving CO_2, mg/ℓ	15 - 30	30 - 60	> 60
Ammonium (NH_4^+), mg/ℓ	15 - 30	30 - 60	> 60
Magnesium (Mg^{2+}), mg/ℓ	100 - 300	300 - 1500	> 1500
Sulfate (SO_4^{2-}), mg/ℓ	200 - 600	600 - 3000	> 3000

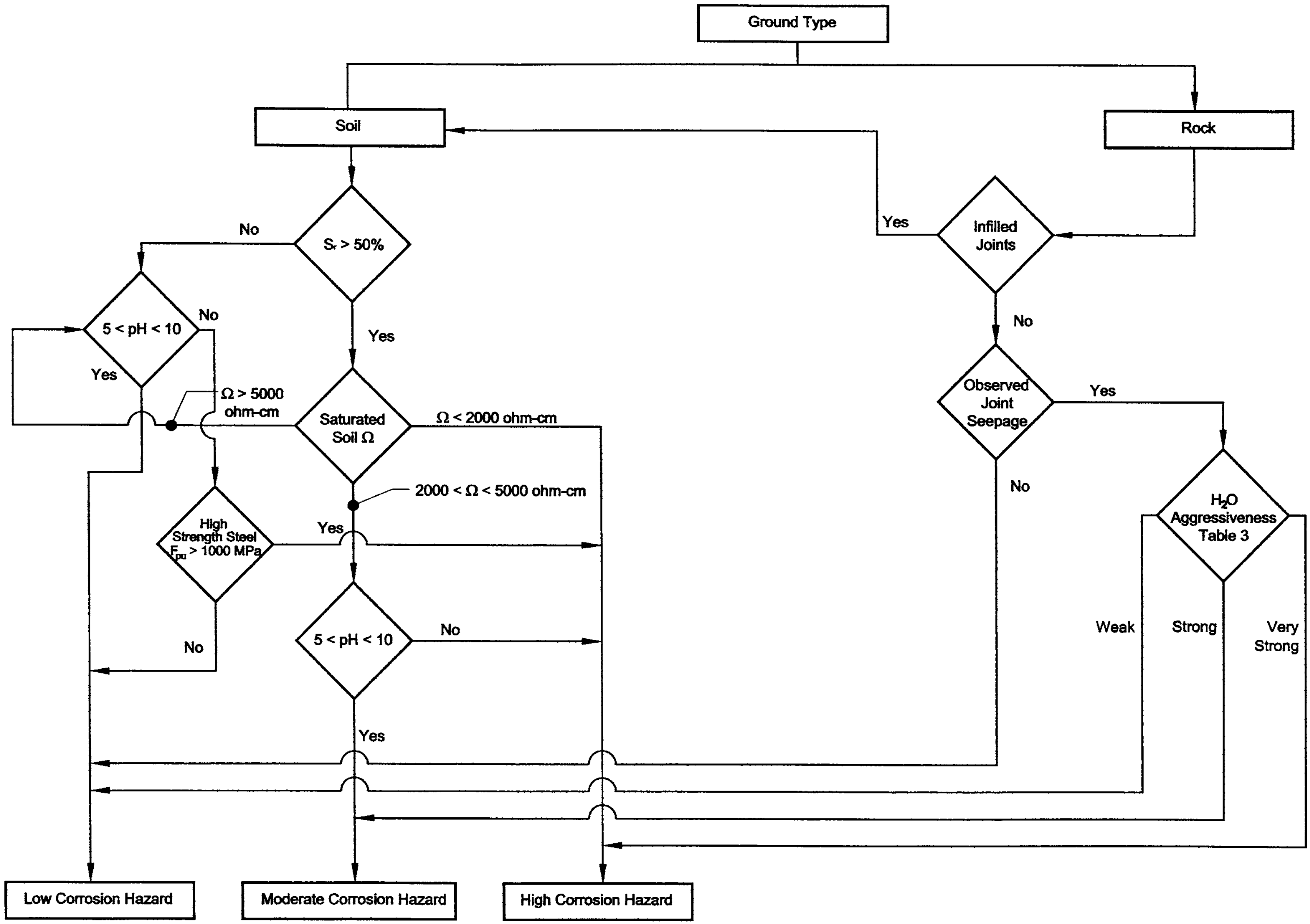

Figure A-1. Decision tree for ground hazard.

not considered in the original design. Creep may cause a loss of resistance along the bond length, thereby decreasing element capacity and contributing to an overloaded condition. Excess pore water pressures associated with poor drainage may contribute to loads not considered in the original design, and groundwater flow paths may contribute to the possibility of localized corrosion.

14.3.2.1 Evidence of creep may be observed in soils from scarps along the ground surface, bulging at a wall face, or heaving at the base of a wall or stabilized slope.

14.3.2.2 Soft rock deposits may exhibit evidence of creep movement, but creep may be difficult to recognize from a visual inspection of harder rocks. The user will need to rely on historical records of incidents of creep failures to determine if this hazard exists at a given site.

14.3.2.3 Drainage problems may be identified by observing seeps along a slope, wall face, or at anchor head locations. Climate is a factor and the amount of precipitation, cycles of wetting and drying, and freezing and thawing may have an impact on element vulnerability.

14.4 Figure A-2 is a decision tree to describe element vulnerability as low, medium, or high.

14.4.1 This Standard Guide assumes that reasonable care was used during construction so that workmanship and quality of detailing are not issues. However, new installations should not be categorically considered in good condition if data suggest that poor workmanship was present in the installation, or that the corrosion protection was inadequate or compromised.

14.4.1.1 Good detailing and workmanship during construction includes quality control to limit scratching and tearing of sheathing, and implementation of successful grouting practices to preclude the existence of voids and ensure that the tendon is full of grout to its highest point without bleeding of the grout.

14.5 Table A-4 is recommended for assigning priority ratings, relative to the potential for corrosion problems at a site. Site condition assessments may proceed according to site priority as budget, time and other resources permit.

14.5.1 When an agency begins to perform condition assessments at sites with a priority index of zero, it may distinguish sites with high, moderate or low vulnerability and perform condition assessments at the most vulnerable sites first.

14.5.2 Sites where problems with creep, excessive settlement at the wall face, or poor drainage have been identified should be assigned a priority index of four or six.

15 Evaluation of Metal-Tensioned Systems

15.1 The purpose of a condition assessment is to evaluate and monitor existing installations of metal-tensioned systems; apply NDTs in the field; and correlate results of the NDTs with subsurface conditions, details of the installation and expectations based on service-life prediction models.

15.1.1 Evaluation should include visual observation of the wall condition and conditions at the anchor head assembly. Observations may include the presence of corrosion products at the anchor head, cracking or bulging at the wall face, and the existence of efflorescence on the surface of cementitious materials. Voids behind the anchor plate may often be observed by soundings, or probing the perimeter of the plate with a thin wire.

15.2 If NDT expertise is not available in-house, agencies may need to seek outside professional advice. Measurement of half-cell potential and linear polarization resistance is becoming a routine practice for assessment of bridge decks. However, specialized equipment and techniques are employed to measure polarization current for buried metal-tensioned systems. The impact and ultrasonic test techniques are less common.

15.2.1 Specially qualified personnel may be required to set up rigging and be familiar with installation details of the elements.

15.3 After a site has been screened, as described in Section 14.5, and a decision has been made to monitor the existing condition of elements at the site, the user needs to determine the number and locations of elements to be tested. A sampling plan

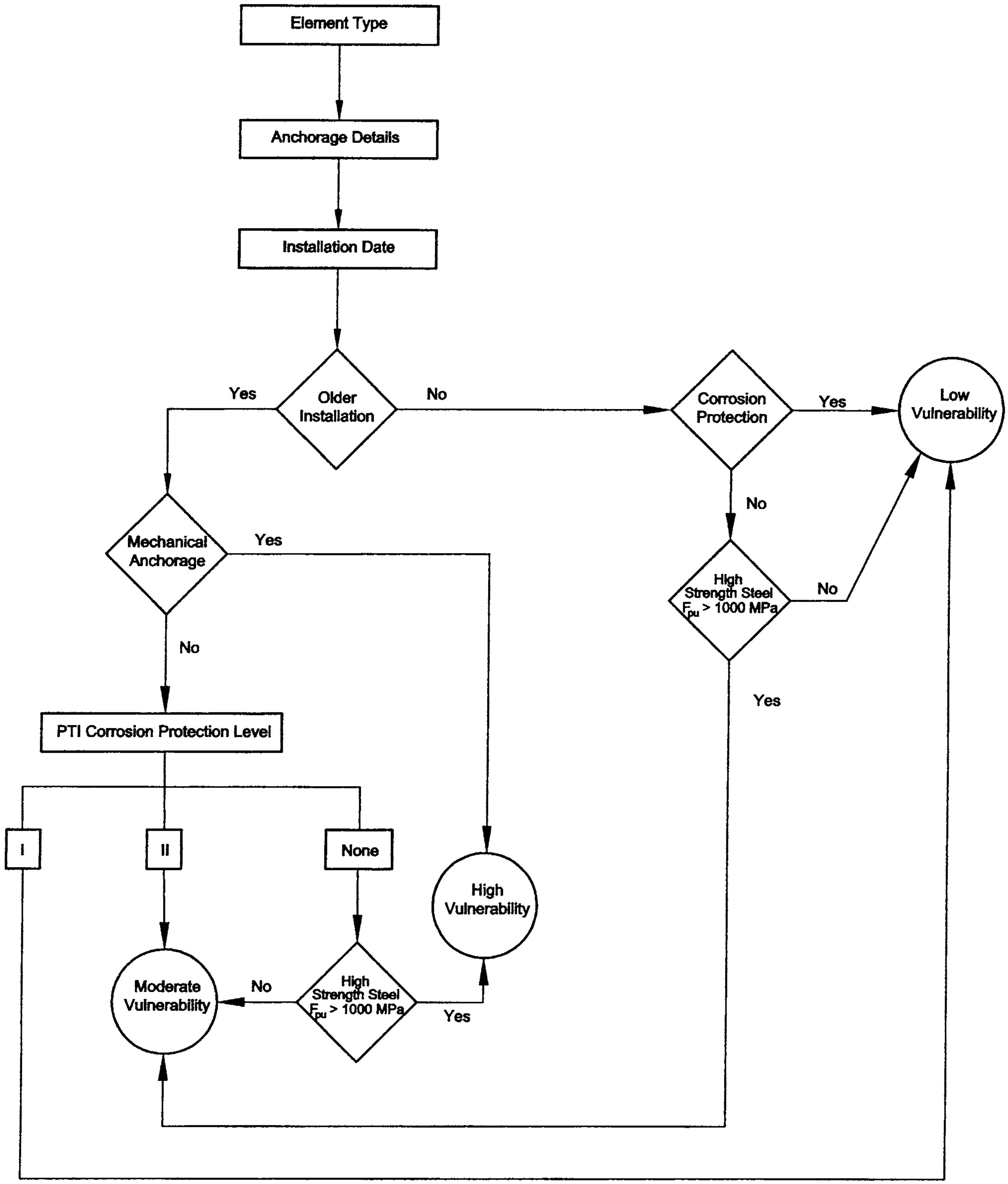

Figure A-2. Decision tree for vulnerability of elements to corrosion and loss of anchorage capacity.

TABLE A-4 Screening assignment of priority index for condition assessment

(1) Vulnerability Level	(2) Hazard Level	(3) Vulnerability Rating	(4) Hazard Rating	(5) Priority Index (3) × (4)
L	L	1	0	0
M	L	2	0	0
H	L	3	0	0
L	M	1	1	1
L	H	1	2	2
M	M	2	1	2
H	M	3	1	3
M	H	2	2	4
H	H	3	2	6

should be developed to determine how large to make the sample size and what to use as acceptance criteria based on the results of the testing program.

15.3.1 The recommended sampling criteria is summarized in Table A-5 where p% is the estimated percent of distressed elements, N is the total number of elements at the site, and n_o is the minimum number of samples to test to achieve a statistical basis for the test results.

15.3.1.1 The value of p% is an expectation based on the age of the system, knowledge of past performance of similar systems, detailing, and the level of workmanship during construction. The expected percentage of distressed elements may be considered a threshold beyond which some action should be taken by the agency.

15.3.1.2 Risk of failure of a structure supported by metal-tensioned elements is a combination of the probability of element failure, redundancy and the consequences of failure. Thus, while the probability of failure may be high, the potential site risk could still be low if the consequences of failure are small. For instance, an interstate highway with a limited lateral clearance from a rock bolt system may be characterized as a site with a high consequence of failure. However an interstate highway having an adequately designed catchment system below a rock bolt support system and a steel net along the face of the supported slope may be characterized as having low consequences from individual rock bolt failures.

15.3.1.3 Elements selected for testing should be distributed throughout the site. If there is reason to believe that problems exist within a limited area, then additional elements should be tested at that locale.

15.4 Figures A-3a and A-3b illustrate the process for condition assessment and service-life evaluation of buried metal-tensioned elements.

15.4.1 Recommended test procedures, data interpretation, and reporting requirements for half-cell potential measurement, polarization measurement, and impact and ultrasonic tests are described in Appendices C, D, E, and F.

15.4.1.1 Electrochemical tests such as measurement of half-cell potential and polarization current may indicate the presence of corrosion or the vulnerability of an element to corrosion (subject to the limitation of Section 6), but cannot indicate the severity of corrosion.

TABLE A-5 Minimum number of samples (n_0) for $10 \le N \le 200$

p (%)	Low Consequence of Failure	Moderate Consequence of Failure	High Consequence of Failure
1	10	10	10
5	10	10-15	10-25
10	10-15	10-25	10-40

Note that for $N < 10$, $n_0 = N$.

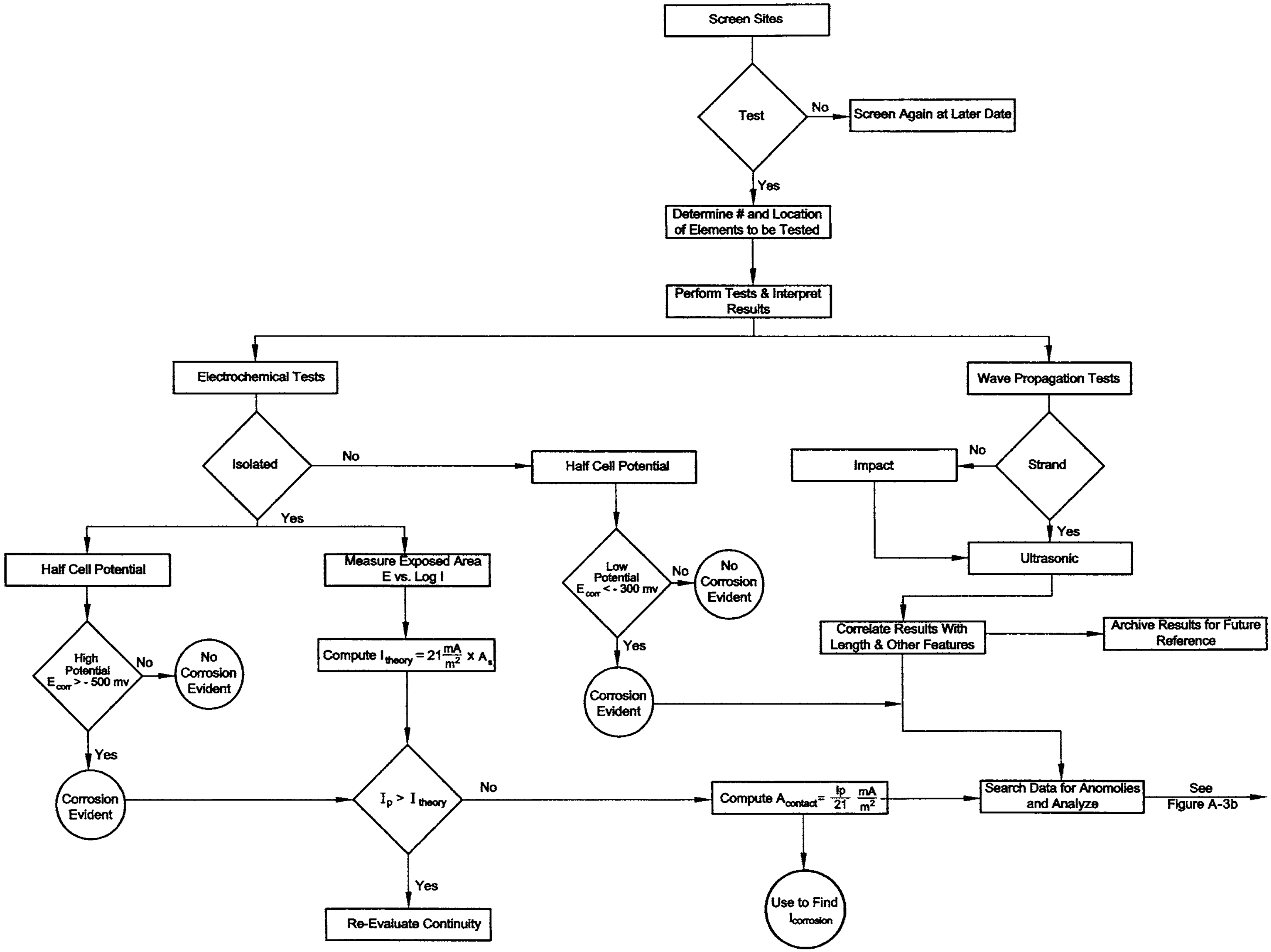

Figure A-3a. Process for condition assessment and service life evaluation of buried metal-tensioned elements.

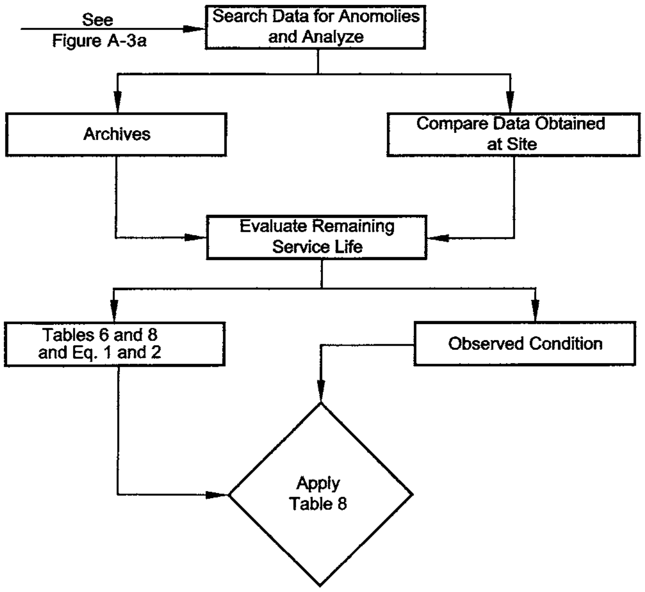

Figure A-3b. Process for condition assessment and service life evaluation of buried metal-tensioned elements.

15.4.1.2 Wave propagation techniques such as the impact echo and ultrasonic tests may be used to evaluate the severity of corrosion, e.g. loss of element cross section, subject to the limitations described in Section 6.

15.4.1.3 Alternative NDT technologies for probing the length of the element should also be considered for condition assessment.

15.4.1.4 Invasive observations and testing, such as lift-off tests, where practical, are always a preferred alternative to NDT. The value of NDT described in this Standard Guide is to screen and identify locations where more detailed, invasive observations, may be recommended. At some sites, implementation of NDT may not be practical and invasive observations may be prescribed without screening by NDT.

15.4.1.5 Lift-off testing will not indicate the degree of corrosion, and caution must be exercised during testing because of the danger associated with sudden failure of distressed elements having significant loss of cross section.

15.4.1.6 Lift-off testing is more difficult to implement on strand type elements.

15.4.2 Grease samples should be tested for bacteria and mold content, which may be correlated with biological activity.

15.4.2.1 Grease samples are very difficult to obtain and reliable testing for bacteria and mold content may not be possible. However, if a hydrogen sulfide gas odor is detected, this should be documented.

15.5 Data from nondestructive testing are analyzed and interpreted to determine if corrosion is occurring and to locate any anomalies or signs of distress along the length of the element as described in this report.

15.5.1 NDT provides limited information and results may only locate points where there is a change in element geometry, or condition.

15.5.2 Based on results from NDT alone, there is no way to identify what a change in signal (e.g. a wave reflection) has detected. Results from NDT must be compared to expectations to gain insight into the possibility of distress such as the existence of voids behind a bearing plate, loss of element cross section, etc.

15.6 The remaining service life is evaluated based on the observed condition, and results from service-life prediction models described in Section 16.

15.7 The user then makes recommendations that may include continued monitoring at selected intervals, more intensive monitoring at frequent intervals, invasive observations, or retrofit such as replacement of anchors as described in Section 17.

16 Estimation of Service Life

16.1 The estimated service life serves as a benchmark, which may be compared with the observed performance of the elements. These service-life predictions do not consider the presence of corrosion protection systems.

16.1.1 If the service life prediction model described in this section estimates significant loss of cross section for bar elements, but NDT results do not indicate the presence of corrosion or element distress, this may mean that the corrosion protection system is intact and functioning as intended.

16.1.1.1 The service life prediction model described in this Standard Guide is conservative because the benefits of corrosion protection are not considered.

16.1.1.2 It may be that corrosion protection is functioning well and elements are not vulnerable to aggressive ground conditions. NDT results may be a useful indicator of the integrity of corrosion protection.

16.2 Estimated service life for a bar element may be computed with Equation (1):

$$\ln(t) = \frac{\ln(X) - \ln(K)}{N} \tag{1}$$

where, X = loss of element thickness or radius (μm), K = constant (μm), n is a constant ranging from 0.6 to 1.0, and t = time (years).

16.2.1 Loss of thickness for use in Equation (1) may be computed as the original radius, r_o, minus the critical radius; $X = (r_o - r_{crit})$.

16.2.2 Equation (2) is recommended for computing the critical radius of the bar element corresponding to the initial cross sectional area of the bar, A_o.

$$r_{\text{critical}} = \sqrt{\frac{0.6A_o}{\pi}} \tag{2}$$

16.2.3 The constants K and n for use in Equation (1) may be estimated from knowledge of soil or rock mass electrochemical properties (resistivity and pH) and the data from Tables A-6 and A-7.

16.2.4 Due to the increased surface area of a strand element compared to a bar element, the estimated service-life will be less than that for bar elements.

TABLE A-6 Corrosiveness of soils

Corrosiveness	Resistivity (ohm/cm)	pH
Normal	2000-5000	5-10
Aggressive	700-2000	5-10
Very Aggressive	<700	< 5

TABLE A-7 Recommended parameters for service life prediction model

Parameter	Normal	Aggressive	Very Aggressive
K (μm)	35	50	340
n	1.0	1.0	1.0

16.3 Pitting corrosion should be considered for low pH environments (pH < 5). To consider pitting corrosion the critical thickness loss, (X) computed in Section 16.2.1, should be divided by two, and X/2 used in Equation (1) in place of X to compute service life.

16.4 A reliable mathematical model is not available for estimating service-life under conditions favorable for hydrogen embrittlement or stress-corrosion cracking.

16.4.1 Corrosion processes such as hydrogen embrittlement and stress-corrosion cracking may lead to sudden failure in strand-type elements without any significant loss of element cross section.

16.4.2 Because the integrity of the corrosion protection system is known to have a significant effect on the service life of strand elements, the condition assessment should focus on obtaining as much information on the integrity of this system as possible (e.g. electrochemical tests, sample and test grease for microbiological activity, ultrasonic test for detecting voids beneath bearing plates).

16.4.3 If conditions are observed which can contribute to hydrogen embrittlement or stress-corrosion cracking, such as a low pH soil environment, high concentrations of sulfides or chlorides, and a compromised corrosion protection system, immediate action should be recommended as described in Section 17.

17 Recommended Action Plan

17.1 The estimated remaining service life is compared to the observed condition of elements at the site.

17.2 Four results from interpretation of the test data are possible, leading to different recommended actions as described in Table A-8.

17.2.1 For sites where there is little or no consequences of failure, “no action” may be appropriate for cases 2, 3 and 4.

TABLE A-8 Recommended action plan

Case	Conditions	Recommended Action Plan
1	• No distress is observed with NDT • Service life prediction model estimates <25% loss of bar cross section • For wire strands, the corrosion assessment model indicates that hydrogen embrittlement and corrosion stress cracking are not likely	• Replacement of existing elements is not recommended • If test results indicate that grout does not reach the back of the anchor plates, the existing void should be filled with grout • Future monitoring is recommended at a selected monitoring interval based on anticipated service life
2	• No distress is observed with NDT • The service life prediction model estimated more than 25 percent loss of bar cross section. • For wire strand elements, the corrosion assessment model indicates that hydrogen embrittlement and corrosion stress cracking are likely	• Verify results of NDT with invasive observations • If verified, continued monitoring at the site is recommended. • A reduction in the frequency of testing may be considered
3	• Distress is observed with NDT • The service life prediction model estimates less than 25 percent loss of bar cross section • For wire strands, the corrosion assessment model indicates that hydrogen embrittlement and corrosion stress cracking are not likely	• Apply the acceptance criteria described in this Standard Guide • If the existing condition is deteriorated below the acceptance criteria verify results of NDT with invasive observations • If results from NDT are confirmed, retrofit and more frequent test intervals are recommended
4	• Observations and service life prediction models are consistent with the conclusion of no remaining service life	• Confirm results from NDT with invasive observations • If confirmed, retrofit is recommended

17.2.1.1 "No action" means monitoring conditions by visual observation and responding to events as needed; allocating budget, time and other resources as required in response to events such as element failure, excessive slope or wall movement, or rock falls.

18 Concluding Summary

18.1 This Standard Guide is a systems approach for identifying sites and element types where there is a high likelihood for performance problems, and a need for detailed evaluation.

18.2 Performance monitoring and service-life prediction of buried metal-tensioned systems are complex problems. This Standard Guide describes a general approach and methodology to be followed. More innovative test techniques, data interpretations, and enhancements to service-life prediction models are encouraged.

18.3 Detailed evaluation involves condition assessment and testing of individual elements. Although element testing is an important part of the process, considerable engineering judgment, risk assessment, and planning need to be exercised before recommending and implementing courses of action.

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APPENDIX B

PERCENTAGE POINTS OF THE *t*-DISTRIBUTION

TABLE B-1 Percentage points of the *t*-distribution

$\nu^{(1)}$ \ α	0.40	0.25	0.10	0.05	0.025	0.01	0.005	0.0025	0.001	0.0005
1	0.325	1.000	3.078	6.314	12.706	31.821	63.657	127.32	318.31	636.62
2	0.289	0.816	1.886	2.920	4.303	6.965	9.925	14.089	23.326	31.598
3	0.277	0.765	1.638	2.353	3.182	4.541	5.841	7.453	10.213	12.924
4	0.271	0.741	1.533	2.132	2.776	3.747	4.604	5.598	7.173	8.610
5	0.267	0.727	1.476	2.015	2.571	3.365	4.032	4.773	5.893	6.869
6	0.265	0.718	1.440	1.943	2.447	3.143	3.707	4.317	5.208	5.959
7	0.263	0.711	1.415	1.895	2.365	2.998	3.499	4.029	4.785	5.408
8	0.262	0.706	1.397	1.860	2.306	2.896	3.355	3.833	4.501	5.014
9	0.261	0.703	1.383	1.833	2.262	2.821	3.250	3.690	4.297	4.781
10	0.260	0.700	1.372	1.812	2.228	2.764	3.169	3.518	4.144	4.587
11	0.260	0.697	1.363	1.796	2.201	2.718	3.106	3.497	4.025	4.437
12	0.259	0.695	1.356	1.782	2.179	2.681	3.055	3.428	3.930	4.318
13	0.259	0.694	1.350	1.771	2.160	2.650	3.012	3.372	3.852	4.221
14	0.258	0.692	1.345	1.761	2.145	2.624	2.977	3.326	3.787	4.140
15	0.258	0.691	1.341	1.753	2.131	2.602	2.947	3.286	3.733	4.073
16	0.258	0.690	1.337	1.746	2.120	2.583	2.921	3.252	3.686	4.015
17	0.257	0.689	1.333	1.740	2.110	2.567	2.898	3.222	3.646	3.965
18	0.257	0.688	1.330	1.734	2.101	2.552	2.878	3.197	3.610	3.922
19	0.257	0.688	1.328	1.729	2.093	2.539	2.861	3.174	3.579	3.883
20	0.257	0.687	1.325	1.725	2.086	2.528	2.845	3.153	3.552	3.850
21	0.257	0.686	1.323	1.721	2.080	2.518	2.831	3.135	3.527	3.819
22	0.256	0.686	1.321	1.717	2.074	2.508	2.819	3.119	3.505	3.792
23	0.256	0.685	1.319	1.714	2.069	2.500	2.807	3.104	3.485	3.767
24	0.256	0.685	1.318	1.711	2.064	2.492	2.797	3.091	3.467	3.745
25	0.256	0.684	1.316	1.708	2.060	2.485	2.787	3.078	3.450	3.725
26	0.256	0.684	1.315	1.706	2.056	2.479	2.779	3.067	3.435	3.707
27	0.256	0.684	1.314	1.703	2.052	2.473	2.771	3.057	3.421	3.690
28	0.256	0.683	1.313	1.701	2.048	2.467	2.763	3.047	3.408	3.674
29	0.256	0.683	1.311	1.699	2.045	2.462	2.756	3.038	3.396	3.659
30	0.256	0.683	1.310	1.697	2.042	2.457	2.750	3.030	3.385	3.646
40	0.255	0.681	1.303	1.684	2.021	2.423	2.704	2.971	3.307	3.551
60	0.254	0.679	1.296	1.671	2.000	2.390	2.660	2.915	3.232	3.460
120	0.254	0.677	1.289	1.658	1.980	2.358	2.617	2.860	3.160	3.373
∞	0.253	0.674	1.282	1.645	1.960	2.326	2.576	2.807	3.090	3.291

(1) $\nu = n - 1$

APPENDIX C

RECOMMENDED TEST METHOD FOR HALF-CELL POTENTIAL MEASUREMENT OF ROCK BOLTS, GROUND ANCHORS AND SOIL NAILS (2002)

1.0 SCOPE

1.1 This document describes procedures for making half-cell potential measurements of existing installations of rock bolts, ground anchors and soil nails; and guidance for interpretation of data. Figure C-1 is a schematic of a half-cell measurement.

2.0 SIGNIFICANCE

2.1 The free corrosion potential is the potential of a rock bolt, ground anchor or soil nail with respect to a reference electrode when no current flows from or to it. For a given material in a given environment, the potential is an indicator of corrosion activity. Interpretation of the data needs to consider whether the element being tested is electrically isolated. In general, the half-cell potential is more positive at sites where corrosion is occurring. For cases where electrical continuity exists between metal elements, potential differences may indicate areas where galvanic corrosion could occur.

3.0 PURPOSE

3.1 The purpose of this document is to establish a field procedure for measuring free corrosion potentials of rock bolts, ground anchors and soil nails with respect to a reference electrode. Potential measurements can be used to indicate the likelihood that corrosion has occurred, or can occur.

4.0 LIMITATIONS

4.1 This method cannot be used in rock formation unless it is extensively jointed with water intrusions that are conductive. It works well in soils that have electrical resistances less than 20,000 ohm-cm. The location of the reference electrodes is important.

4.2 Half-cell measurements should not be performed when temperatures are below 0° C.

5.0 APPLICABLE DOCUMENTS

5.1 American Society for Testing and Materials (ASTM), 2000, “Test Method C876-91(1999) Standard Test Method for Half Cell Potentials of Uncoated Reinforcing Steel in Concrete,” *Annual Books of ASTM Standards, Volume 04.02, Concrete and Aggregates*, ASTM, West Conshohocken, PA, 6p.

5.2 National Association of Corrosion Engineers (NACE), 1997, “Measurement Techniques Related to Criteria for Cathodic Protection on Underground or Submerged Metallic Piping Systems,” *NACE Standard TM0497-97, Item N0. 21231*, NACE International, Houston, TX.

5.3 National Cooperative Highway Research Program (NCHRP), 2002, “Recommended Practice for Evaluation of Metal-Tensioned Systems in Geotechnical Applications,” *NCHRP Report 477*, NCHRP, Washington, D.C.

5.4 AASHTO, “Recommended Test Method for Measurement of Polarization Current for Rock Bolts, Ground Anchors and Soil Nails,” *Appendix D of NCHRP Report 477.*

6 EQUIPMENT

6.1 *Half Cell*. A rigid tube or container composed of a dielectric material that is nonreactive with copper or copper sulfate, a porous ceramic plug that remains wet by capillary action, and a copper rod that is immersed within the tube in a saturated solution of copper sulfate. The solution shall be prepared with reagent grade copper sulfate crystals dissolved in distilled or

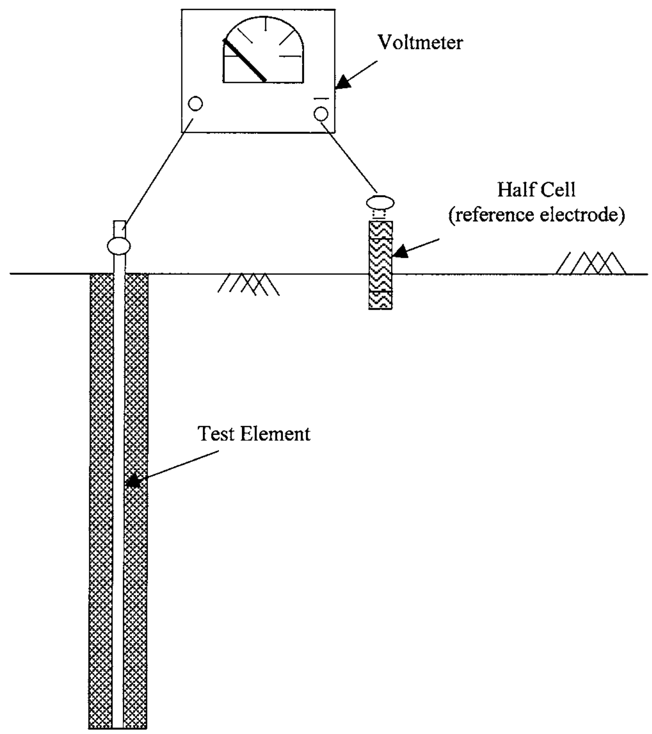

Figure C-1. *Half-cell potential measurement.*

deionized water. The solution may be considered saturated when an excess of crystals (undissolved) lies at the bottom of the solution.

6.2 *Voltmeter*. The voltmeter shall have the capacity of being battery operated and have ±3% end-of-scale accuracy at the voltage ranges in use. The input impedance shall be no less than 10MΩ when operated at a full scale of 100 mV. The divisions on the scale used shall be such that a potential difference of 0.02 V or less can be read without interpolation.

6.3 *Electrical Lead Wires*. The electrical lead wire shall be of such dimension that its electrical resistance for the length used will not disturb the circuit by more than 0.0001 V. This can be accomplished by using no more than 150 linear meters of at least AWG No. 24 gauge wire. The wire shall be suitably coated with direct burial type of insulation.

7 EQUIPMENT CHECK

7.1 Inspect half cell and check that the tube is filled with solution.

7.2 Check for excess crystals at the bottom of the solution.

7.3 Follow procedures for care of the half cell as described in ASTM C876.

7.4 The porous tip on the half cell should be saturated.

7.5 Check battery on voltmeter.

7.6 Check that the voltmeter is set to the proper range. Voltage measurements should be made using the lowest practical range on the instrument. A voltage measurement is more accurate when it is measured in the upper two-thirds of a range selected for a particular instrument.

7.7 Check the performance of the voltmeter by comparing measurements made with different meters, or making measurements from a known voltage source.

7.8 Check electrical lead wires for continuity.

8 PREPARATION OF TEST ELEMENT

8.1 Access is required to the anchor head assembly for attachment of necessary wiring. Protective caps, if present, should be removed from the ends of the elements, and for encapsulated anchorages, grout may need to be chipped away from the end of the element. For concrete encapsulated elements, concrete may need to be chipped away.

8.2 Scale, coatings or rust, if present, shall be cleaned from the end of the element to achieve good electrical contact.

8.3 Attach lead wire to the end of the element using clamps or other device. Do not weld to the prestressing steel or anchorage.

9 CONTINUITY CHECK

9.1 Check for electrical continuity between elements and/or between elements and the wall face. This may be accomplished by measuring the resistance between elements or by comparing half-cell potentials between elements after a current has been impressed upon the element to be tested.

9.1.1 If the measured resistance between elements is greater than 5 Ω, elements may be considered to be isolated. Alternatively, using the same leads for the resistance check, the potential between two elements can be measured and should be greater than 5 mV to verify that continuity does not exist.

9.1.2 If a current is impressed on one of the elements (see description of measurement of polarization current, Appendix D), the half-cell potential measured for other elements should not be affected if the element is electrically isolated.

10 TEST PROCEDURE

10.1 Connect the test lead from the element to be tested to the positive terminal of the voltmeter as shown in Figure C-1.

10.2 Locate the half cell in close proximity to the element. To the extent possible, find a path between half cell and element along a low resistance electrolyte. Examples include pressing the tip of the half cell into a clay-filled seam intersecting a rock bolt, or within soil material along the toe or backfill of a tieback wall.

10.3 Connect the test lead from the half cell to the negative terminal of the voltmeter as shown in Figure C-1.

10.4 Turn on the voltmeter and read the value.

11 REPORTING

11.1 Record the potential value, sign, type of electrode used, the location of the element and details about the anchor head, and the location of the reference electrode relative to the element.

11.2 If elements are electrically isolated, identify potentials that are more positive than the range of −500 mV to −800 mV. Potentials more positive than −500 mV indicate that corrosion is present.

11.3 For elements that are not electrically isolated, evaluate data for changes in potential, i.e. identify areas of relatively high or low potential measurements. Elements with relatively lower half-cell potentials are sites where corrosion is likely to occur.

APPENDIX D

RECOMMENDED TEST METHOD FOR MEASUREMENT OF POLARIZATION CURRENT FOR ROCK BOLTS, GROUND ANCHORS AND SOIL NAILS (2002)

1.0 BACKGROUND

1.1 For corrosion to occur, there must be an electrical path from the element through an electrolyte. The soil/water or rock mass environment surrounding a rock bolt, ground anchor or soil nail (element) may serve as an electrolyte and contribute to the corrosion process. Metal-tensioned elements are often installed with corrosion protection to isolate the element from the surrounding ground. The integrity of the corrosion protection system may be evaluated by observing the response of the element to impressed current.

1.2 The electrolyte provides a current path between an element and an established ground bed as shown in Figure D-1. Negatively charged ions within the electrolyte migrate towards the positively charged element. A level of impressed current, I_p, is reached for which the surface of the element is fully polarized. The surface area of the metal element in contact with the surrounding ground is estimated from the measured I_p.

2.0 SCOPE

2.1 This document describes procedures for making polarization current measurements of buried metal-tensioned elements. This recommended test method describes details for impressing current upon the system, measuring I_p, and interpretation of data.

2.2 The test procedure is only applicable to elements that are known to be electrically isolated. An element is considered electrically isolated if its potential is unaffected when current is impressed upon any other element.

3.0 SIGNIFICANCE

3.1 Approximately 21 mA/m^2 is required to polarize buried bare metal surfaces. Using this constant, the surface area of steel in contact with the ground can be computed using the measured I_p. As described in Section 12.4, this information can be used to assess the integrity of existing corrosion protection systems, which may involve plastic sheathing, or other dielectric material surrounding, or coating the element.

3.2 The polarization current for a fully protected system is close to zero.

4.0 PURPOSE

4.1 The purpose of this document is to establish a field procedure for measuring the approximate surface area of rock bolts, ground anchors, or soil nails which is not protected, and, therefore vulnerable to corrosion.

5.0 LIMITATIONS

5.1 This method cannot be used in rock formation unless it is extensively jointed with water intrusions that are conductive. The method works well in soils that have electrical resistances less than 20,000 ohm-cm. The location of the reference electrodes is important.

5.1.1 Tests should be performed when the ground is saturated. It is difficult to perform the test during dry periods, and the test is not suitable for arid climates.

5.2 Polarization measurements should not be performed when temperatures are below 0° C.

5.3 Elements must be electrically isolated prior to testing.

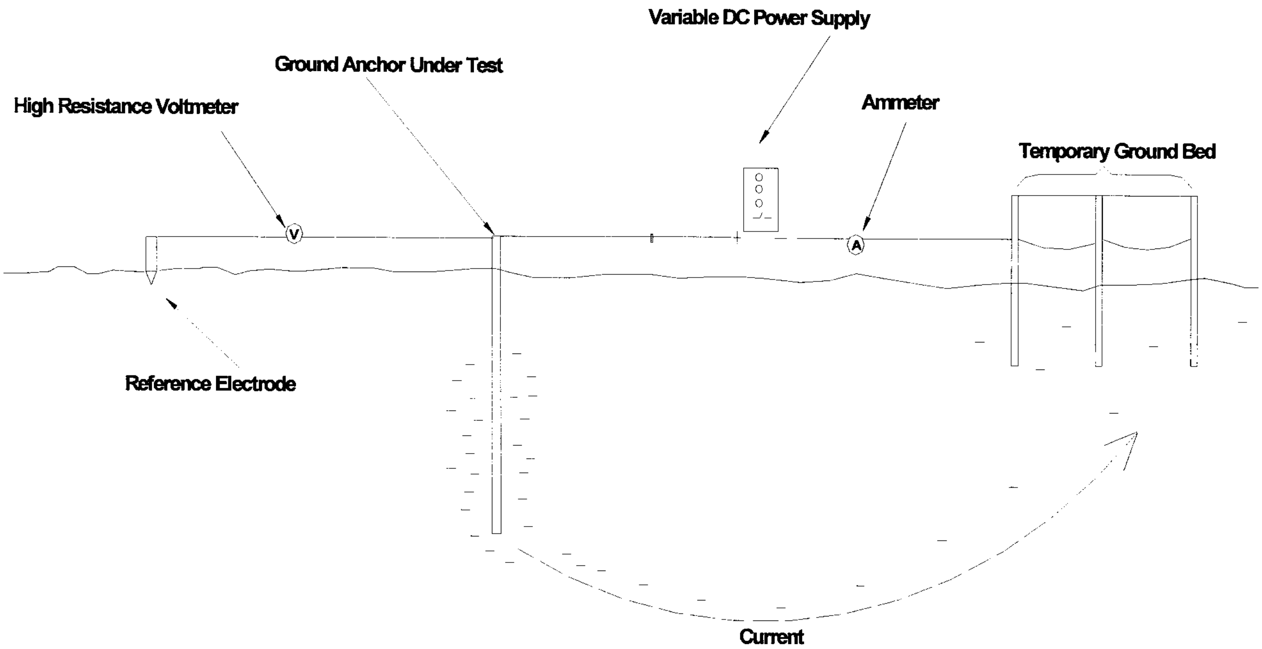

Figure D-1. Test arrangement for measuring polarization current.

6.0 APPLICABLE DOCUMENTS

6.1 American Society for Testing and Materials (ASTM), 2001, "Test Method C876-91(1999) Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete," *Annual Books of ASTM Standards, Volume 04.02, Concrete and Aggregates,* ASTM, West Conshohocken, PA, 6 p.

6.2 National Association of Corrosion Engineers (NACE), 1997, "Measurement Techniques Related to Criteria for Cathodic Protection on Underground or Submerged Metallic Piping Systems," *NACE Standard TM0497-97, Item N0. 21231,* NACE International, Houston, TX.

6.3 National Cooperative Highway Research Program (NCHRP), 2002, "Recommended Practice for Evaluation of Metal-Tensioned Systems in Geotechnical Applications," *NCHRP Report 477,* NCHRP, Washington, D.C.

6.4 American Association of State Highway and Transportation Officials (AASHTO), "Recommended Test Method for Half-Cell Potential Measurement of Rock Bolts, Ground Anchors and Soil Nails," *Appendix C of NCHRP Report 477.*

7.0 EQUIPMENT

7.1 The equipment needed for the test includes a power supply with a rheostat, an ammeter, a high impedance voltmeter and a reference electrode (half cell).

7.2 A rheostat can be assembled using a battery pack and a set of variable resisters to control the output of current. The rheostat, voltmeter, ammeter, and three bus bars may be housed within a portable casing. As shown in Figure D-2, the portable casing can be arranged such that only three external connections are required corresponding to the test bar, half cell, and ground bed.

7.3 *Half cell.* A rigid tube or container composed of a dielectric material that is nonreactive with copper or copper sulfate, a porous ceramic plug that remains wet by capillary action, and a copper rod that is immersed within the tube in a saturated solution of copper sulfate. The solution shall be prepared with reagent grade copper sulfate crystals dissolved in distilled or deionized water. The solution may be considered saturated when an excess of crystals (undissolved) lies at the bottom of the solution.

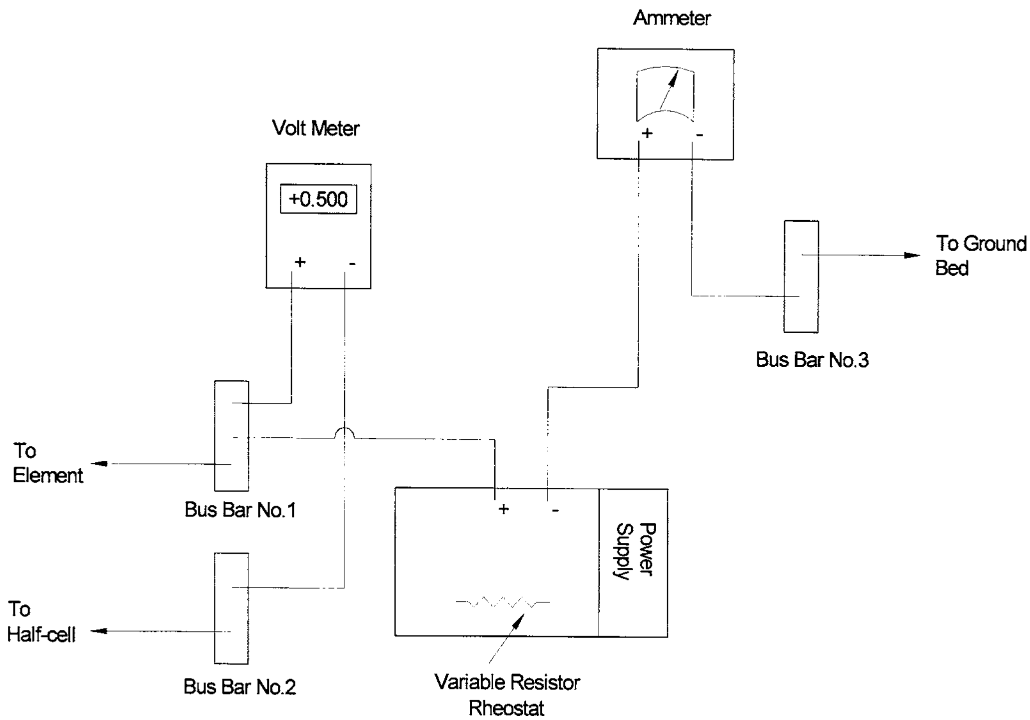

Figure D-2. Schematic of components and connections for measurement of polarization current.

7.4 *Ground bed*. At least three copper-plated rods, approximately 16-mm diameter, and 1-m long should be used. The three rods should be embedded at least 600 mm into the ground, spaced approximately 300 mm apart, and connected in series. For rock bolts, it may not be possible to embed the rods in an area with good electrical communication to the elements being tested. In this case, another rock bolt may be used as a ground if it is not electrically insulated. However, caution should be exercised because the use as a ground will affect the measurement of half-cell potential, and some amount of protective corrosion product surrounding the element will be discharged from the surface, increasing the vulnerability of the rock bolt to future corrosion.

7.5 *Rheostat*. A voltage source and a means to apply variable resistance may be used to vary the current applied to the circuit shown in Figure D-2. The rheostat should include an on-off switch to control the application of current to the circuit. A 12 V source is recommended.

7.6 *Voltmeter*. The voltmeter shall have the capacity of being battery powered and have ± 3% end-of-scale accuracy at the ranges in use. The input impedance shall be no less than 10 MΩ when operated at a full scale of 100 mV. The divisions on the scale used shall be such that a potential difference of 0.02 V or less can be read without interpolation.

7.7 *Ammeter*. The ammeter is used to measure applied current. A zero resistance ammeter should be used such that including the ammeter in the circuit has no effect on the current being measured.

7.8 *Electrical wire leads*. The electrical lead wires shall be of such dimension that its electrical resistance for the length used will not disturb the circuit by no more than 0.0001 V. This has been accomplished by using no more than 150 linear meters of at least AWG No. 24 gauge wire. The wire shall be suitably coated with direct burial type of insulation.

8.0 PREPARATION OF TEST BAR

8.1 Access is required to the anchor head assembly for attachment of necessary wiring. Protective caps must be removed if present. Grout may need to be removed to provide an area to attach wires.

8.2 Scale, or rust, must be removed if present to provide a good electrical contact.

8.3 Attach lead wire to the element using clamps that insure a tight connection.

9.0 EQUIPMENT CHECK

9.1 Inspect the half cell and check that the tube is filled with solution. Check for excess crystals at the bottom of the solution. The porous tip on the half cell should be saturated. Follow procedures for care of the half cell as described in ASTM C876.

9.2 Check battery on voltmeter and ammeter. Check the performance of the voltmeter and ammeter by comparing measurements made with different meters, or by making measurements from a known voltage or current source.

9.3 Check that voltmeter and ammeter are set to the proper range. Measurements should be made using the lowest practical range on the instrument. A voltage measurement is more accurate when it is measured in the upper two-thirds of a range selected for a particular instrument.

9.4 Inspect the rheostat, and check that the batteries are charged.

9.5 Check electrical lead wires for continuity and for proper connections as shown in Figure D-2.

10.0 Continuity Check

10.1 Check for electrical continuity between elements and/or between elements and the wall face. This may be accomplished by measuring the resistance between elements or by comparing half-cell potentials between elements after a current has been impressed upon the element to be tested.

10.1.1 If the measured resistance between elements is greater than 5 Ω, elements may be considered to be electrically isolated. Alternatively, using the same leads for the resistance check, the potential between two elements can be measured and should be greater than 5 mV to verify that continuity does not exist.

10.1.2 If a current is impressed on one of the elements, the half-cell potential measured for other elements should not be affected if the element is electrically isolated.

10.2 If the element is not electrically isolated, the test procedure described in Section 11 and data interpretation described in Section 12 are not applicable.

11.0 TEST PROCEDURE

11.1 Connect the lead from the test element to Bus Bar #1 (see Figure D-2) in common with the positive lead from the rheostat and the positive lead from the voltmeter.

11.2 Locate the half cell in close proximity to the element. To the extent possible, find a path between half cell and element along a low resistance electrolyte. Examples include pressing the tip of the half cell into a clay filled seam intersecting a rock bolt, or within soil material along the toe or backfill of a tieback wall.

11.3 Connect the lead from the half cell to Bus Bar #2 (see Figure D-2) in common with the negative lead from the voltmeter.

11.4 Establish the ground bed far enough away so as not to affect the measurement of half-cell potential. A distance of approximately 30 m between the half cell and ground bed is usually adequate. Connect the lead from the ground bed to Bus Bar #3 (see Figure D-2) in common with the negative lead from the ammeter.

11.5 Connect the negative lead from the rheostat to the positive terminal of the ammeter.

11.6 With the current off, check the half-cell potential.

11.7 Turn the current on, and with the variable resistance set to zero, record the maximum current. The maximum current that can be obtained depends on the degree of corrosion protection offered to the system, and the resistance of the surrounding soil or rock mass.

11.8 Turn the current off and wait for at least one minute.

11.9 Apply increments of current in two-minute intervals. The first increment should be approximately 1/100 of the maximum current. At the end of each two-minute interval, record the half-cell potential and then increase the current for the next interval. Apply approximately 15 intervals. A curve similar to Figure D-3 is developed by applying increasing levels of current until a definite break in the curve is defined. It is important to refine the current intervals to get a good definition of the break in the curve. The first few intervals may be increased by doubling the amount of current each time, but near the break in the curve, equal intervals should be applied. For the curve shown in Figure D-3, appropriate current intervals would be 0.12, 0.25, 0.50, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0, 11.0 and 12.0 mA.

12 REPORTING

12.1 The following information can be used to assess the integrity of existing corrosion protection systems, which may involve plastic sheathing, or other dielectric material surrounding, or coating the element.

12.2 Record the initial potential value, sign, type of electrode used, the location of the element and details about the element head, and the location of the reference electrode and ground bed relative to the element.

12.3 Plot the measured potential versus the log of the applied current as shown in Figure D-3. The result should be a curve having an initial straight-line section curving into a second straight-line section. If this shape is not obtained it is possible that the test did not cover a wide enough range of current.

12.3.1 The slope of the second straight-line section should not be greater than 0.1 volt/log cycle.

12.3.2 The first point on the second straight-line portion of the curve is the polarization current.

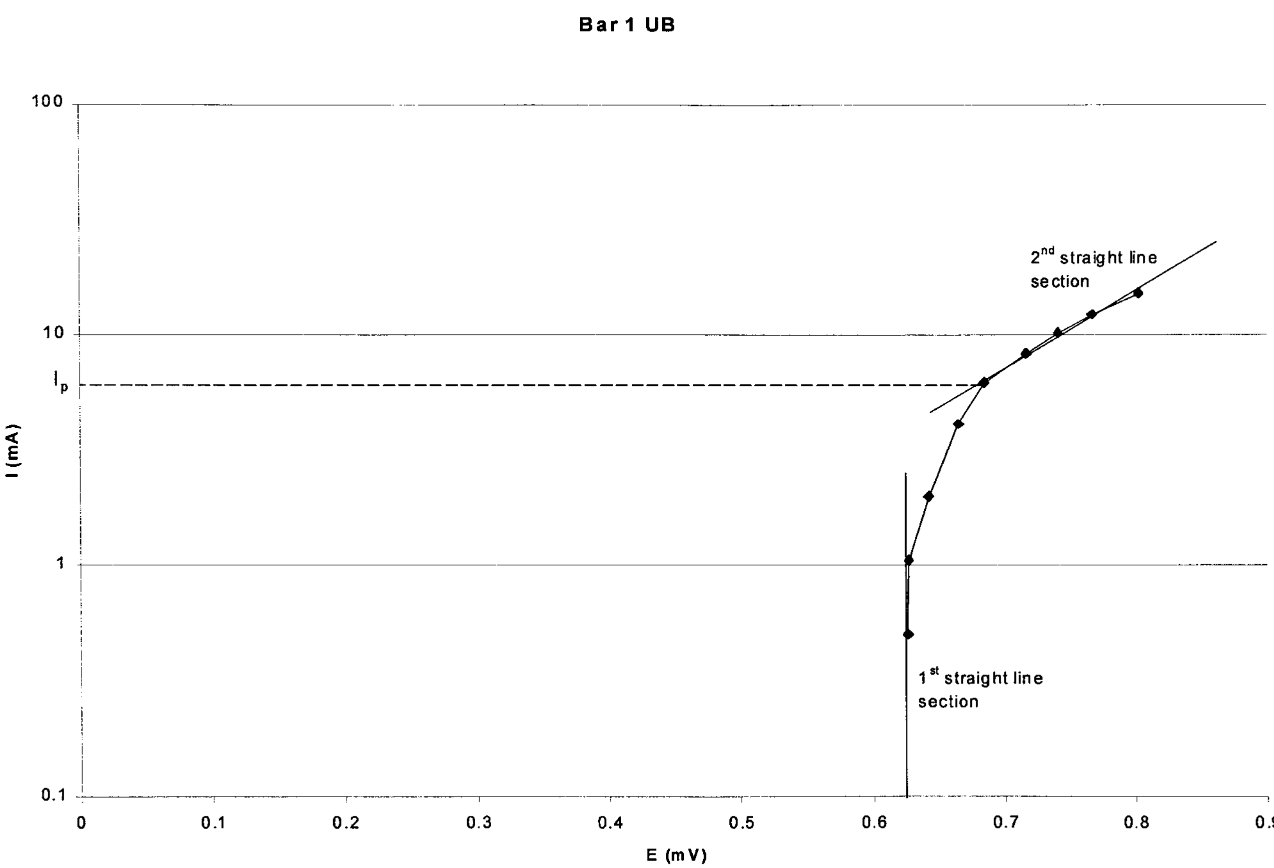

Figure D-3. Typical polarization measurement showing characteristic curve.

12.3.2.1 Report the measured polarization current, I_p, as shown in Figure D-3.

12.4 Using Equation D-1, compute the theoretical polarization current (I_{theory}) which assumes that the element is in contact with the ground over the entire surface area of the element, ignoring the existence of plastic sheathing and/or coatings such as grout and epoxy.

$$I_{theory} = A_s \times 21 \qquad \text{(D-1)}$$

where,

A_s is the surface area of the element (m^2)
I_{theory} is in milliamperes

12.4.1 The estimated current requirement, I_{theory}, can be compared to measured current requirement, $I_{p.}$ The report should include one of the following conclusions:

12.4.1.1 If $I_p << I_{theory}$, the element is probably electrically well insulated and well protected.

12.4.1.2 If $I_p < I_{theory}$, the element is probably coated or protected over just some part of its surface. Using the measured protection current (I_p), the unprotected length of the element (meters) can be estimated with Equation D-2 and the unprotected length should be reported.

$$L_{unprotected} = I_p/(21 \times \pi \times d) \qquad \text{(D-2)}$$

where,

d is the element diameter (m).

12.4.1.3 If $I_p > I_{theory}$, this suggests that more surface area is involved than initially assumed and could be an indicator that electrical contacts with other elements having surface areas in contact with the ground have not been considered.

APPENDIX E

RECOMMENDED TEST METHOD FOR IMPACT ECHO TEST OF BAR-TYPE ROCK BOLTS, GROUND ANCHORS AND SOIL NAILS (2002)

1.0 SCOPE

1.1 This test method describes equipment and procedures for in-situ measurement of the impact response of rock bolts, ground anchors and soil nails.

1.2 Distances to features along the length of an element are calculated from measured arrival times of reflected compression waves and the known compression wave velocity of the element material.

1.3 This test may be applied to solid bar type elements where access is available to the element head. Knowledge of installation details is needed for data interpretation.

2.0 SIGNIFICANCE

2.1 The face of the element is impacted using a hammer that generates elastic compression waves. The traveling waves are reflected whenever a change in material or geometry is encountered along the length of the element. The arrival of these reflected waves at the face of the element produces accelerations, which are measured by a receiving transducer, as shown in Figure E-1. The acceleration waveform can be used to determine travel time from the initiation of the impact to the arrival of the wave reflection, T_R. If the compression wave velocity, V_p, in the test element is known, the distance, L_R, to the reflector can be calculated. An alternative approach is frequency analysis of the acceleration waveforms. If the frequency content of a waveform is determined, distances to reflectors can be calculated.

2.1.1 The locations of reflectors, $L_{R,}$ are compared to the locations of known installation features involving changes in geometry or materials. Reflector locations that are not correlated with installation details are suspected distress locations, such as voids in the grout surrounding the element or reduced element cross section.

2.1.2 If a number of elements with similar installation features are tested at the same site, reflections observed for different elements may be compared to identify elements with anomalous reflections. Further evaluation of elements with anomalous reflections is recommended to determine if the anomalies are related to element distress.

2.2 Results from this test should be verified by invasive observations or performance testing such as lift–off tests of the elements.

3.0 PURPOSE

3.1 The impact test may be used to evaluate fracture of elements, loss of element cross-section, and cracking of grout surrounding an element.

3.2 Electrochemical tests such as measurement of half-cell potential and polarization current may indicate the presence of corrosion or the vulnerability of an element to corrosion. The impact-echo test may be used to evaluate the severity of corrosion, e.g. loss of element cross-section.

4.0 LIMITATIONS

4.1 The element diameter must be at least 25 mm to allow impact and instrumentation to be placed at the face of the element.

4.1.1 The impact test is not suitable for testing strand type elements.

4.2 The sensitivity of the impact test is limited.

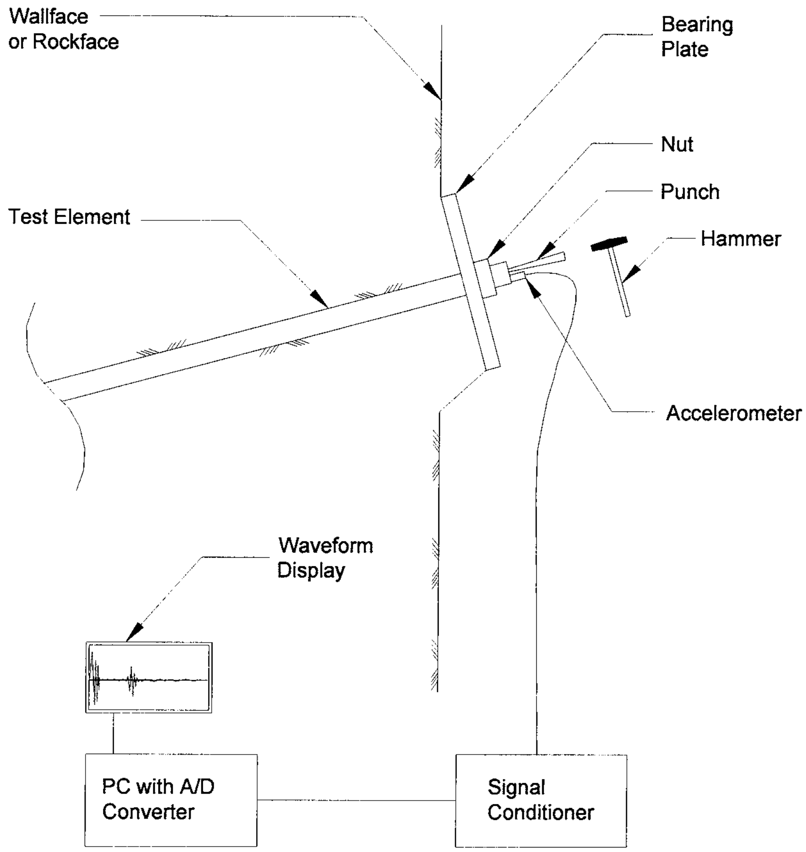

Figure E-1. Schematic of impact-echo test.

4.2.1 Less than about 25% loss of bar cross section is difficult to detect. This means that the element may be close to failure when loss of cross section is detected.

4.2.2 The length of an element that can be probed with the impact test is limited. Good results have been obtained for element lengths of approximately 10 m and some element lengths as long as 20 m have been detected. Success depends on how much of the element is surrounded by grout.

4.3 The size, shape and nature of the defect or anomaly cannot be determined using existing data processing techniques. It is difficult to distinguish an anomaly related to installation details from loss of cross section due to corrosion or other types of in-service distress.

4.3.1 Knowledge of installation details is necessary for data interpretation.

4.4 Identifying signals from multiple reflectors can be difficult. Near source reflectors tend to mask the reflections from more distant sources.

5.0 APPLICABLE DOCUMENTS

5.1 National Cooperative Highway Research Program (NCHRP), 2002, "Recommended Practice for Evaluation of Metal-Tensioned Systems in Geotechnical Applications," *NCHRP Report 477*, NCHRP, Washington, D.C.

5.2 American Association of State Highway and Transportation Officials (AASHTO), "Recommended Test Method for Half-Cell Potential Measurement of Rock Bolts, Ground Anchors and Soil Nails," *Appendix C of NCHRP Report 477.*

5.3 American Association of State Highway and Transportation Officials (AASHTO), "Recommended Test Method for Measurement of Polarization Current for Rock Bolts, Ground Anchors and Soil Nails," *Appendix D of NCHRP Report 477.*

6. EQUIPMENT

6.1 Equipment for the impact echo test includes an impact device, a sensor for measuring the element response, a signal conditioner, and a computer or waveform analyzer for recording data in real-time and storing the data.

6.2 *Impact Device.* A light (2N to 5N) hand-held hammer, with a small face (approx. 350 mm^2) that does not exceed the available impact area, may be used. A tack hammer or small ball peen hammer works well. A hand-held punch may also be used with the hammer to direct impact energy to a smaller area at the face of the test element.

6.3 *Sensor.* A general-purpose shock accelerometer with a frequency range from 0.4 Hz to 7500 Hz and a sensitivity of at least 10 mV/g is recommended. An integrated-electronics piezoelectric (IEPE), shear-structured accelerometer with very low sensitivity to transverse motion is desirable.

6.3.1 The amplitude range of the accelerometer should exceed the acceleration from impact; at least ±100 g is recommended when a metal tipped hand held hammer is used for impact.

6.3.2 The resonant frequency of the sensor should be outside the range of measured response. This may be accomplished by obtaining an accelerometer with a relatively high resonant frequency of at least 25 kHz.

6.3.3 An IEPE sensor includes built-in microelectronics with a preamplifier to drive output signals with low impedance. This makes it possible to use long cables with minimal loss in signal quality.

6.3.4 The transducer should be supplied with a National Institute of Standards and Testing (NIST) traceable calibration certificate.

6.4 *Signal Conditioner.* A battery powered, portable, low noise signal conditioner is recommended to couple the transducer to standard readout instrumentation. The unit should have a meter to monitor sensor operation, detect cable faults and indicate a low battery condition. The signal conditioner serves three basic functions:

6.4.1 It provides constant current excitation to power sensor microelectronics, which may be either built-in or in-line with the sensor. IEPE sensors require constant current excitation for proper operation.

6.4.2 It incorporates a coupling capacitor to remove the DC bias voltage from the analog signal and provide a drift-free AC coupled output compatible with standard readout instrumentation.

6.4.3 It includes a selectable gain to amplify the sensor output signal for use with low sensitivity readout equipment. An in-line preamplifier may be required if the transducer is not supplied with internal micro-circuitry.

6.5 *Readout Instrumentation.* A waveform analyzer, or computer with high-speed digital data acquisition (DAQ) software, should be used to capture the transient output of the sensor, store the digitized waveforms, and perform signal analysis. A suitable waveform analyzer, or data acquisition card, should have a sampling frequency of at least 100 kHz.

6.5.1 The readout instrumentation must be triggered upon impact.

6.6 *Cable.* Standard coaxial cable equipped with coaxial plugs and/or BNC connectors is recommended.

6.6.1 Cable capacitance and connectors between the sensor and signal conditioner must meet the specifications provided by the manufacturer of the sensor and signal conditioner units. Ruggedized cable with tin-plated copper braid and heat shrink tubing is recommended for use in the field. Use of couplings should be avoided, but may be necessary for long lengths of cable. Cables in excess of 30-m long should be avoided.

6.6.2 A standard RG-58/U coaxial cable may be used to connect the output from the signal conditioner to the readout instrumentation. Standard BNC plug/jacks are recommended for making necessary connections. Minimum length of cable (approximately 1 meter) should be used between the signal conditioner and the readout instrumentation.

6.7 *Mounting bases*. These may be used to mount the accelerometer using an adhesive to the test element surface. The base should have a cross section as large, or slightly larger, than the base of the accelerometer, be manufactured with a smooth surface for application of adhesive, and include male or female threads in the center to engage the base of the accelerometer.

6.8 *Adhesive*. “Super Glue” may be used for mounting the base to the test element surface. “Super Glue” provides strong bond but can easily be removed with a quick, “sharp” force in the shear direction.

7. EQUIPMENT CHECK

7.1 Perform a battery check on the signal conditioner.

7.2 Cables should be inspected for damage. Cable should be surrounded with insulation and bare cable should not be exposed at any point along the length. Damaged cable should be replaced.

7.3 Cables are equipped with BNC and/or coaxial connectors. Be sure connections are tight including couplings. The output of the instrument or signal conditioner should not be affected by touching or moving the cable. If the signal is affected, recheck the connection and the condition of the cable.

7.4 Apply a known voltage to the data acquisition system and confirm that the reading displayed is consistent with the applied voltage.

7.5 Connect the cable from the accelerometer to the signal conditioner and the output from the signal conditioner to the data acquisition system. Using the calibration supplied by the manufacturer of the accelerometer, and considering the gain from the signal conditioner, compute the voltage output for an acceleration change of two times the acceleration due to gravity. Observe the voltage recorded by the DAQ with the accelerometer in the upright position. Invert the accelerometer and observe the corresponding change in voltage recorded by the DAQ. The change in voltage should correspond to twice the acceleration due to gravity. If not, look for problems with the signal conditioner or the accelerometer. Courses of action may include checking the equipment with alternate signal conditioners and/or accelerometers. *Note: Accelerometers that rely on piezoelectric crystals respond to changes in acceleration. Therefore, when the crystals are inverted a change in response will be recorded, but if left inverted the output will return to null.*

7.6 Performance of the measurement system should be evaluated on a bench-scale specimen representative of the elements installed at the site being evaluated. The bench-scale specimen should be 1- to 2-m long and supported on low impedance material such as expanded polystyrene or foam rubber. The ends of the specimen should be prepared, instruments attached and electrical connections made as described in Sections 8.2 to 8.4. Bench scale testing and data interpretation should follow the procedures described in Sections 9 and 10. The L_R described in Section 10.3 should correspond to the known length of the bench-scale specimen. If not, this indicates a problem with the equipment, electrical connections, or data acquisition software.

8 PREPARATION OF TEST ELEMENT

8.1 Access is required to one end of the test element for attachment of the accelerometer. Protective caps or grout must be removed if the element face is covered.

8.2 The face of the element must allow the accelerometer to be mounted with full contact. The face may need to be ground with a surface grinder to remove high edges and rust scale may need to be removed to achieve a good bond with the adhesive used to mount the threaded base for the accelerometer.

8.3 Mount the accelerometer to the end of the specimen by drilling and tapping a hole to receive the thread at the base of the accelerometer, or mounting the threaded base to the element face with adhesive.

8.4 Make necessary connections between the accelerometer and the signal conditioner; and the output of the signal conditioner to the data acquisition system.

9 TEST PROCEDURE

9.1 Determine the compression wave velocity of the test element. This may be determined from the literature, or from measuring the travel time of reflected compression waves along a known length of a similar element. For reference, the compression wave velocity of low carbon steel is approximately 5,950 m/s. For elements surrounded by grout, the compression wave velocity will be less than that of an element in air, or surrounded by a low stiffness material such as grease or plastic.

9.2 Turn on the power supply and data acquisition system. Set the gain of the signal conditioner and the sensitivity of the data acquisition equipment to an optimum level. The optimum level is just below that at which electromagnetic noise reaches an intolerable level or triggers the data acquisition system at its lowest triggering sensitivity. The noise level shall not be greater than one tenth of the amplitude of the first peak signal received from the reflected wave.

9.3 Set recording time.

9.4 Set trigger level.

9.5 Strike the element with the impact device to generate compression waves along the specimen. The impact should be administered at or near the center of the element cross-section.

9.6 Observe the reflected waveform from the transducer output.

9.7 Store the data.

9.8 Repeat the impact until three repeatable signals are observed and recorded.

10. CALCULATION

10.1 Plot the time history of the accelerations.

10.2 Identify reflections apparent in the time-history.

10.2.1 Reflections may be obscured and data processing as described in this report may be necessary to enhance the signals.

10.2.2 Determine the arrival time of each observed reflection.

10.3 Calculate the location of the reflectors as follows:

$$L_R = V_P \times T_R/2$$

where:

L_R = the location of the reflector from the face of the element (m)
V_P = velocity of compression wave propagation (m/s)
T_R = the arrival time of the observed reflection (s)

10.3.1 Compute L_R for each reflection observed in the time history. Multiple reflections may be observed from the same reflector, particularly if the reflector is relatively close to the face of the element. Only consider the first arrival from each source.

10.4 Alternatively, data may be transformed to the frequency domain and the lengths corresponding to particular frequencies computed as described in this report.

11 REPORTING

11.1 The report shall include the following:

11.1.1 Identification of the test element, including element type and location (site location and relative location of element at site).

11.1.2 Details of the anchorage head, trumpet assembly, and corrosion protection such as sheathing and/or encapsulation; unbonded length; bonded length; and level of prestress.

11.1.3 Calculated locations of reflectors.

11.1.4 Comparison of calculated location of reflectors and known details of the installation; and, identification of reflectors that are not correlated with known details of the installation.

APPENDIX F

RECOMMENDED TEST METHOD FOR ULTRASONIC PROBE OF ROCK BOLTS, GROUND ANCHORS AND SOIL NAILS (2002)

1.0 SCOPE

1.1 This test method describes equipment and procedures for probing to find discontinuities along the length of rock bolts, ground anchors and soil nails, in situ, using ultrasonic waves.

1.2 Distances to features along the length of an element are calculated from measured arrival times of reflected sound waves and the known sound wave velocity of the element material.

1.3 This test may be applied to bar or strand type elements where access is available to the anchor head. Knowledge of installation details is needed for data interpretation. Good wave transmission may be obtained along the free lengths of greased and sheathed tendons.

2.0 SIGNIFICANCE

2.1 Ultrasonic waves are radiated when an ultrasonic transducer applies periodic strains on the face of the test element, which propagate as stress waves. The traveling waves are reflected whenever a change in material or geometry is encountered along the length of the element.

2.2 With the pulse-echo method (single-probe operation) shown in Figure F-1 the times for sound pulses, generated at regular intervals, to pass through the specimen and return, are measured. The transducer, which is acoustically coupled to the exposed end of the element, receives a shock excitation and generates a short ultrasonic pulse. The transducer receives echoes of the pulses after reflection. The return of the leading edge of the first echo can be easily detected by visual means from the time-history of transducer output.

2.3 From the reflection arrival times, T_R, the distance, L_R, to the reflector can be calculated if the velocity of sound wave propagation, V_P, is known.

2.3.1 The locations of reflectors, L_R, are compared to the locations of known installation features involving changes in geometry, or materials. Reflector locations that are not correlated with installation details are suspected distress locations; such as voids in the grout surrounding the element, or reduced element cross section.

2.3.2 If a number of elements with similar installation features are tested at the same site, reflections observed for different elements may be compared to identify elements with anomalous reflections. Further evaluation of elements with anomalous reflections is recommended to determine if the anomalies are related to element distress.

2.4 Results from this test should be verified by invasive observations and/or performance testing such as lift–off tests of the elements.

3.0 PURPOSE

3.1 The ultrasonic test may be used to evaluate fracture of elements, loss of element cross-section, and cracking of grout surrounding an element.

3.2 Electrochemical tests such as measurement of half-cell potential and polarization current may indicate the presence of corrosion or the vulnerability of an element to corrosion. The ultrasonic test may be used to evaluate the severity of corrosion, e.g. loss of element cross-section.

3.3 Results from this test may be used to verify or supplement results from impact echo testing as described in Appendix E.

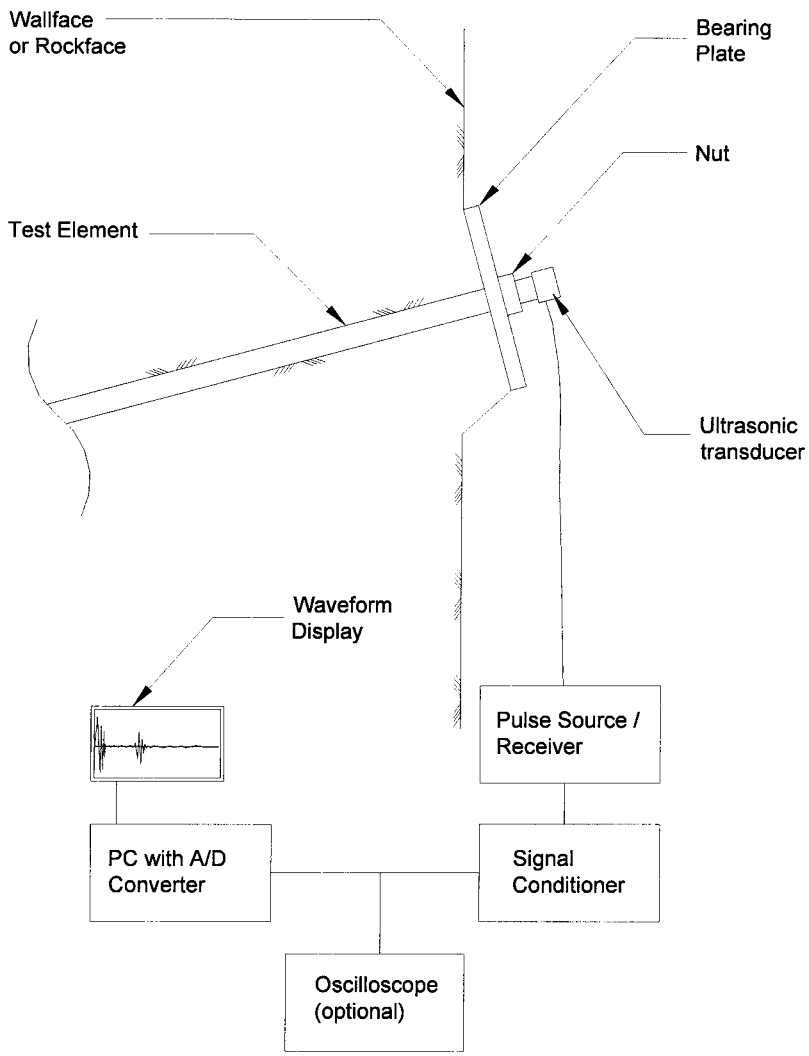

Figure F-1. Schematic of ultrasonic test.

4.0 LIMITATIONS

4.1 The test described herein may not detect loss of bar cross section less than 25%. This means that the element may be close to failure when loss of cross section is detected.

4.2 The length of an element that can be probed is limited. Research results indicate difficulty probing element lengths beyond approximately 2 m. The limit depends on details of the installation, including the presence of grout surrounding the element. However, the majority of corrosion problems that have been documented for ground anchors occurred within the first one or two meters from the anchor head assembly.

4.3 The size, shape and nature of the defect, or anomaly cannot be determined using existing data processing techniques. It is difficult to distinguish an anomaly related to installation details from loss of cross section due to corrosion, or other types of in-service distress.

4.3.1 Knowledge of installation details is necessary for data interpretation.

4.4 Identifying signals from multiple reflectors can be difficult. Near source reflectors tend to mask the reflections from more distant sources.

5.0 APPLICABLE DOCUMENTS

5.1 National Cooperative Highway Research Program (NCHRP), 2002, "Recommended Practice for Evaluation of Metal-Tensioned Systems in Geotechnical Applications," *NCHRP Report 477*, NCHRP, Washington, D.C.

5.2 American Association of State Highway and Transportation Officials (AASHTO), "Recommended Test Method for Half-Cell Potential Measurement of Rock Bolts, Ground Anchors and Soil Nails," *Appendix C of NCHRP Report 477.*

5.3 American Association of State Highway and Transportation Officials (AASHTO), "Recommended Test Method for Measurement of Polarization Current for Rock Bolts, Ground Anchors and Soil Nails," *Appendix D of NCHRP Report 477.*

5.4 American Association of State Highway and Transportation Officials (AASHTO), "Recommended Test Method for Impact Echo Test on Rock Bolts, Ground Anchors and Soil Nails," *Appendix E of NCHRP Report 477.*

6. EQUIPMENT

6.1 Equipment for the ultrasonic test includes an ultrasonic transducer, a pulse source/receiver, and a computer or waveform analyzer for recording data in real-time and storing the data.

6.2 *Transducer*. The transducer shall serve the dual function of a transmitter, which converts electrical pulses into mechanical pulses; and a receiver, which converts mechanical pulses into electrical pulses. Piezoelectric elements are usually recommended, but magnetostrictive elements may be suitable. Thickness-expander piezoelectric elements that generate and sense predominantly compression waves are recommended.

6.2.1 Low frequency, broadband ultrasonic transducers that operate at a frequency of approximately 50 kHz or 100 kHz are recommended.

6.2.2 The transducer diameter should be as large, or slightly larger than, the diameter of the element face.

6.3 *Pulse Source/Receiver*. The pulse source/receiver supplies a pulse of excitation to the transducer; and receives and conditions the return signal from the transducer.

6.3.1 The capability of generating pulses with selected pulse repetition frequency rate of 20 Hz, and the ability to damp and adjust the amplitude of the pulse is desirable.

6.3.2 Signal conditioning including high and low pass filters and signal attenuation is desirable.

6.3.3 The pulse/source receiver should have the ability to operate in pulse/echo mode.

6.4 *Readout Instrumentation*. A waveform analyzer, or computer with high-speed digital data acquisition software, should be used to capture the transient output of the pulse source/receiver, store the digitized waveforms, and perform signal analysis. A suitable waveform analyzer, or data acquisition card, should have a sampling frequency of at least two times the operating frequency of the transducer.

6.5 *Cable.* Standard coaxial cable equipped with coaxial plugs and/or BNC connectors is recommended.

6.5.1 Cable capacitance and connectors between the sensor and signal conditioner must meet the specifications provided by the manufacturer of the sensor and signal conditioner units. Use of couplings should be avoided, but may be necessary for long lengths of cable. Cables in excess of 30 m long should be avoided.

6.5.2 A standard RG-58/U coaxial cable may be used to connect the output from the signal conditioner to the readout instrumentation. Standard BNC plug/jacks are recommended for making necessary connections. Minimum length of cable (approximately 1 m) should be used between the signal conditioner and the readout instrumentation.

6.6 *Couplant*. A high viscosity lubricant is a good acoustic couplant for this test.

7 EQUIPMENT CHECK

7.1 Check the power supply to the pulse source/receiver.

7.2 Inspect cables for damage. Cables should be surrounded with insulation and bare cable should not be exposed at any location along the length. Damaged cable should be replaced.

7.3 Be sure connections are tight including couplings. The readout should not be affected by touching or moving the cable. If the signal is affected, recheck the connection and the condition of the cable.

7.4 Apply a known voltage to the data acquisition system and confirm that the reading displayed is consistent with the applied voltage.

7.5 Performance of the measurement system should be evaluated on a bench-scale specimen representative of the elements installed at the site being evaluated. The bench-scale specimen should be 1- to 2-m long and supported on low impedance material such as expanded polystyrene or foam rubber. The ends of the specimen should be prepared, instruments attached and electrical connections made as described in Sections 8.2 to 8.4. Bench scale testing and data interpretation should follow the procedures described in Sections 9 and 10. The L_R described in Section 10.3 should correspond to the known length of the bench-scale specimen. If not, this indicates a problem with the equipment, electrical connections, or data acquisition software.

8 PREPARATION OF TEST ELEMENT

8.1 Access is required to one end of the element to be tested for placement of the transducer at the face of the element. Protective caps or grout must be removed if the element face is covered.

8.2 Good acoustic coupling between the transducer and the face of the element is required for ultrasonic testing, and the face of the each element must be flat and smooth. Grinding may be required. Care must be taken to ensure that the element faces are properly prepared for testing.

8.2.1 Full contact must be achieved between the end face of the element and the face of the transducer. The element face may need to be ground with a surface grinder to remove high edges and rust scale may need to be removed.

8.3 Make necessary connections between the transducer and the pulse source/receiver; and the output of the pulse/source receiver to the data acquisition system.

8.4 Apply couplant to the element and transducer faces. Couplants should be applied as a thin layer.

9 TEST PROCEDURE

9.1 Determine the compression wave velocity of the test element. This may be determined from the literature, or from measuring the travel time of reflected compression waves along a known length of a similar element. For reference, the compression wave velocity of low carbon steel is approximately 5,950 m/s. For elements surrounded by grout, the compression wave velocity will be less than that of an element in air, or surrounded by a low stiffness material such as grease or plastic.

9.2 Turn on the pulse source/receiver and data acquisition system. Set the pulse amplitude on the pulse/source receiver to the highest level for safe use with the transducer. Allowable voltage input to the transducer should not be exceeded.

9.3 Set the pulse repetition rate, input signal damping, and the output high and low pass filters to optimum levels for the element being tested.

9.4 Set the gain of the pulse source/receiver and the sensitivity of the data acquisition equipment to an optimum level. The optimum level is just below that at which electromagnetic noise reaches an intolerable level or triggers the data acquisition system at its lowest triggering sensitivity. The noise level shall not be greater than one tenth of the amplitude of the first peak signal received from the reflected wave.

9.5 Set the recording time and trigger level for the data acquisition system.

9.6 Apply the transducer to the face of the element.

9.7 Observe the reflected waveform from the readout.

9.8 Store the data.

9.9 Remove the transducer and repeat 9.6 to 9.8 until three repeatable signals are observed and recorded.

10 CALCULATION

10.1 Plot the time history of the accelerations.

10.2 Identify reflections apparent in the time-history.

10.2.1 Reflections may be obscured and data processing as described in this report may be necessary to enhance the signals.

10.2.2 Determine the arrival time of each observed reflection.

10.3 Calculate the location of the reflectors as follows:

$$L_R = V_P \times T_R/2$$

where:

L_R = the location of the reflector from the face of the element (m)
V_P = velocity of compression wave propagation (m/s)
T_R = the arrival time of the observed reflection (s)

10.3.1 Compute L_R for each reflection observed in the time history. Multiple reflections may be observed from the same reflector, particularly if the reflector is relatively close to the face of the element. Consider only the first arrival from each source.

11 REPORTING

11.1 The report shall include the following:

11.1.1 Identification of the test element, including element type and location (site location and relative location of element at site).

11.1.2 Details of the anchorage head, trumpet assembly, and corrosion protection such as sheathing and/or encapsulation; unbonded length; bonded length; and level of prestress.

11.1.3 Calculated locations of reflectors.

11.1.4 Comparison between calculated location of reflectors and known details of the installation; and, identification of reflectors that are not correlated with known details of the installation.

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Abbreviations used without definitions in TRB publications:

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ITE	Institute of Transportation Engineers
NCHRP	National Cooperative Highway Research Program
NCTRP	National Cooperative Transit Research and Development Program
NHTSA	National Highway Traffic Safety Administration
SAE	Society of Automotive Engineers
TCRP	Transit Cooperative Research Program
TRB	Transportation Research Board
U.S.DOT	United States Department of Transportation

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