Evaluation of Metal-Tensioned Systems in Geotechnical Applications, Phase I

Interim Report

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Acknowledgments

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SUMMARY

This interim report presents the findings of Phase I of NCHRP 24-13, "Evaluation of Metal-Tensioned Systems in Geotechnical Applications." The objectives of the project are to study methods for condition assessment and service life prediction of metal-tensioned systems.

Phase I activities include a survey of existing practice, evaluation of nondestructive test 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 are to be implemented during the second phase of the project.

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 covers 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 test techniques that may be used to monitor the condition of the metal-tensioned system throughout its useful service life.

Types of Metal Tensioned Systems

Geotechnical applications of metal-tensioned systems include ground anchors, rock bolts and soil nails. Table 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 160. Strand elements are manufactured from Grade 250 and 270 high strength steel. Wire tension systems, using the button head anchorage of BBRV and Prescon, have been used in some early applications but are now obsolete. They are not discussed further in this report.

Current guidance documents (PTI, 1996; Sabatini et al., 1997) 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, which are reviewed in this report, all offer systems that comply with the current standards. However, many of the older installations do not incorporate details that meet today’s standards, or may have been installed without any corrosion protection beyond the passivation of the grouted portion of the tensioned elements.

Rock bolts are installed with either 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.

Table 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, Galvanized, Grout Cover, older installations may have none
Soil Nails	Bars	Cement grout entire length	Grout cover, bars may be epoxy coated

Performance

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.

Particularly for the higher strength steel, corrosion is often localized and evident in the form of pitting. Hydrogen embrittlement and stress crack corrosion have been observed in strand wire and bar type elements used as prestressing steels in prestressed, reinforced concrete applications. Stress crack corrosion is aggravated by high tension from prestressing, which is often required for ground anchors and rock bolts.

Compared to 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 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, prestress concrete pipe and tanks, and prestress reinforced concrete for bridge and building construction. Based on a review of the performance of metal-tensioned systems we have reached conclusions that are similar to those of the Federation Internationale de la Precontrainte (FIP, 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 only been 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 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; even for aggressive ground conditions.

Based on the above conclusions the performance and service life of metal tensioned systems depend upon 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 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.

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 if corrosion protection systems are intact, if 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 requires that each tendon element be electrically isolated from the rest of the system. In practice, this 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 if corrosion is occurring, or the existing condition of the metal element. Thus, nondestructive test (NDT) techniques are needed to obtain information about the condition of the system.

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

EVALUATION OF NDT

Nondestructive test techniques with the potential for application to metal tensioned systems were reviewed. Based on 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.

Literature Search

A search of the literature was conducted 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 based on their potential for success, ease of application, cost of instrumentation and availability of needed equipment. Table 2 summarizes the test methods considered. The table identifies each method, describes previous research or application, indicates the level of expense and training required for operation of the equipment, and the relative ease by which the technique may be implemented to metal-tensioned systems (MTS).

Laboratory Evaluation of NDT for Implementation to Metal-Tensioned Systems

Based on results of the literature survey, the impact echo, ultrasonic test, half-cell potential, and polarization measurements were identified as tests which had the potential for successful implementation for condition assessment of rock bolts, soil nails or ground anchors. Contact resistance and a radio wave reflection technique (similar to GPR) were also evaluated during preliminary laboratory evaluations.

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 to metal tensioned systems, the sensitivity 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. Based on the results of the literature search and laboratory evaluation, the impact-echo test, ultrasonic test, half-cell potential and polarization measurements are recommended for implementation at selected fields sites. Implementation of the test techniques at field sites, and a study of the test results and data collected are planned for Phase II of this research.

MATHEMATICAL MODELS FOR SERVICE LIFE PREDICTION

The power law similar to that applied by Elias (1990) for service life prediction of buried steel soil reinforcements, and the approach resulting from Project NCHRP 10-46 for estimating the service life of steel pile foundations will be adopted in this study.

FIELD SITES FOR PHASE II

Nine field sites have been identified for Phase II of the investigation. Table 3 summarizes relevant information for each site. Pertinent information includes the application as rock bolts, or as tie-backs (grouted anchorage) or anchors (deadman anchorage) for a retaining wall system; the type of element which is either bar or strand; the date of installation; the existence of a corrosion protection system; the availability of soils data; whether or not the elements are prestressed; and special comments.

The ages of the elements planned for the condition assessment range from 3 to 40 years old. Not all the tendons at the sites considered were installed with corrosion protection systems that meet today's standards. At three of the sites (#2, #3 and #4), anchors will be exhumed as part of planned reconstruction.

Table 2. Summary of NDT Methods Considered for Condition Assessment

Method	Previous Application	Equipment	Suitability for Testing MTS in Geotechnical Uses	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 to previous applications, therefore, same style hammer employed by previous researchers may not be necessary.	**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 %.
Impulse Response	Evaluation of drilled shaft foundations; evaluation of tension levels in prestress rods.	Similar to impact echo except response is measured w/ velocity transducer and larger impact w/ lower frequency content applied.	Similar to impact echo	**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.
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.	**Half -Cell Potential**	Monitor condition of steel reinforcement in concrete, 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.
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 prestress 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.	**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.
Ultrasonic Test	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.	**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.
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.	**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.
Reflective Impulse Measurement Technique	Evaluation of unbonded post-tensioned 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.	• **ontact 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.
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 requires 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.	**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.

Eight of the sites are ground anchors and one site is a rock bolt site. The rock bolt site in Ellenville, NY was selected based on the presence of different types of rock bolts, the density of the rock bolt pattern, the rock conditions, and the access available at the site. In 1992, a rock-slide occurred at this location and some information is already available from a study conducted by NYSDOT after the slide.

For the sites with ground anchors, five of the tendons have bar elements and three have strand. Bars at the NGES and O-Street sites have a grouted anchorage and are prestressed, but deadman anchorages are used at the other sites with bar elements. The strand elements at sites #3 and #5 are grouted and prestressed.

At the sites where reconstruction activities will take place our schedule for the NDT, and condition assessment, must be coordinated with the construction schedule for the project. The project schedule is more flexible relative to NDT activities planned at the remaining sites. We plan to perform NDT evaluations at the field sites during the spring, summer and fall of this year (2000).

Table 3. Proposed Sites for Phase II Field Studies

No.	Site Location	Appl.[1]	Type	Install Date	Corr Prot.[2]	Soils Data	Pre-stress	Comments
1	West Valley, NY	Wall anchor	Bars	1968	No	Partial	None	Sheet-pile wall; good site for preliminary testing
2	Buffalo Inner Harbor	Wall anchor	Bars	1967	No	Partial	None	Sheet-pile quay wall; tie-backs will be exhumed
3	NYS Route 5, Sennet, NY	Tie-backs	Strand	1994	Yes	Partial	Yes	Sheet-pile bulkhead; ground anchors will be exhumed
4	O - Street, Washington, D.C.	Tie-backs	Bars	1975	No	Yes	Yes	Concrete diaphragm wall; known failures of anchors; anchors will be exhumed
5	Parking Garage, Pittsburgh, PA	Tie-backs	Strand	1972	No	Need	Yes	Sheet-pile wall; corrosion evident at wall face
6	NGES- Texas A&M Riverside Campus	Tie-backs	Bars	1991	Yes	Partial	Yes	Soldier-pile and lagging; accessible as wall built for experiment; instrumentation is installed; case study
7	Route 38, Gawanda, NY	Tie-backs	Strand	1989	Yes	Need		soldier pile and lagging wall
8	Route 23A, Palenville, NY	Wall anchor	Bars	1995	No	Need	No	soldier pile and lagging wall
9	Route 52, Ellenville, NY	Rock Bolts	Bars	1972 to 1999	No		Yes	different ages and types of bolts; previous slide area; case study

[1] Appl. is application
[2] Corr Prot. is corrosion protection

INTERIM PROJECT REPORT
NCHRP 24-13
EVALUATION OF METAL-TENSIONED SYSTEMS IN GEOTECHNICAL APPLICATIONS

1.0 INTRODUCTION

Metal-tensioned systems include prestressed ground anchors (strands and bars), soil nails, rock bolts, metal strip and bar-grid used for earth reinforcement, and micropiles. These systems have been used with increasing frequency by transportation agencies for the construction and repair of foundations, retaining walls and excavated and natural slopes. Rock bolts were first used in the mining industry, and later adopted for use by the transportation industry in the early 1960's. The use of permanent ground anchors for public sector projects became common in the United States in the late 1970's. Soil nailing is a more recent innovation, having become popular within the last decade.

Thus, some of the earlier rock bolt and ground anchor installations are approaching a service life of approximately 30 to 40 years. Since visual observation of conditions at the anchor head assemblies is often not indicative of potential problems, the condition of existing systems is uncertain. Transportation agencies, faced with the task of allocating budgets for rehabilitation of aging facilities, need a protocol for performing condition assessment and estimating the remaining useful service life. NCHRP 24-13 is an on-going study to develop a recommended practice, suitable for adoption by AASHTO, for procedures to evaluate the condition and remaining useful service life of in-place metal-tensioned systems, and to estimate the design life of new installations.

1.1 DESCRIPTION OF THE PROJECT

Project NCHRP 24-13 is divided into Phase I and Phase II. In general, Phase I is intended to assimilate existing information, evaluate existing techniques for condition assessment, and prepare a work plan for monitoring and evaluating existing and new installations of ground anchors, soil nails and rock bolts to be conducted in Phase II. This interim report presents the results from Phase I of the study.

Phase I consists of the following six tasks- :

- ***Task 1 - Review and Evaluate Existing Practice, Performance 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 its estimation. Work presently being carried out for other applications such as pretensioned concrete systems, bridge cables, buried pipelines, and other situations which involve environments having characteristics which are of interest, are studied for potential application to buried metal-tensioned elements.

- ***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.

- ***Task 3 - Evaluate and Select Viable Performance Monitoring Systems.*** Evaluate and select promising, practical methods for field measurement of the condition of metal-tensioned systems in both new and existing installations. Laboratory testing is conducted to support the evaluation and selection of viable monitoring systems.

- ***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.

- ***Task 5 - Develop 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.

- ***Task 6 - Develop 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.

- ***Prepare Interim Report.*** Summarize the results of Tasks 1-6.

1.2 DESCRIPTION OF INTERIM REPORT

This Interim Report is divided into seven sections. Section 1.0 is this introduction. Sections 2.0 and 5.0 present a summary of information gathered from the literature search prescribed in Task 1, and the Task 2 evaluation of technical information. Section 3.0 and Section 4.0 describe the evaluation of NDT methods and service life prediction models specified in Tasks 3 and 4, respectively. Section 5.0 provides case histories, and a survey of existing practice which is information that is useful for development of the work plans to meet the requirements of Tasks 5 and 6. Sections 6.0 and 7.0 describe the work plans for field evaluation of existing and new installations.

Six appendices are included at the end of the report. Appendices I-IV present detailed information and test results in support of the description of the NDT evaluation presented in Section 3.0. Appendix I presents detailed information about instrumentation employed for NDT, and Appendices II - IV are test results from evaluation of the impact echo, ultrasonic and electro-chemical tests, respectively. Appendices V and VI present details of the project questionnaire and responses to the survey of existing practice described in Section 5.2.

2.0 REVIEW OF METAL-TENSIONED SYSTEMS

A review of existing literature relevant to Tasks 1 and 2 has been pursued. Results from the literature search are compiled into a database. The database has over 300 entries and includes literature directly acquired by the project team as well as the results of searches through existing databases including Compendex and NIST. The database is compiled using Textware™, which is a DOS-based indexing program. The indexed bibliography, complete with review summaries or abstracts, is saved on a CD-ROM. The data on the CD-ROM can be searched via subject headings, keywords, authors or titles.

The review described in this section includes: 1) types of metal-tensioned systems and anchoring techniques, 2) factors that affect their useful service life, 3) recommended practice, 4) performance data, and 5) test methods that may be useful for condition assessment. Additional information related to current practice is described in Section 5.0 of this report.

2.1 TYPES OF METAL-TENSIONED SYSTEMS

2.1.1 Prestressed Ground Anchors

Sabatini et al. (1998) describe a prestressed grouted ground anchor as a structural element used to transmit an applied tensile load into the ground. The basic components include the: (1) anchorage; (2) free stressing (unbonded) length; and (3) bond length. These and other components are shown schematically in Figure 2-1. The anchorage is a combined system of anchor head, bearing plate, and trumpet that is capable of transmitting the prestressing force from the prestressing steel (bar or strand) to the ground surface or supported structure. The unbonded length is that portion of the prestressing steel that is free to elongate elastically and transfer the resisting force from the bonded length to the structure. The bond length is that length of the prestressing steel that is bonded to the primary grout and is capable of transmitting the applied tensile load to the ground.

Figure 2-1 shows an unbonded tendon in the unbonded length (stressing length). There are many ground anchors that have no permanent unbonded length. The earlier anchors had no sheath in the stressing length and were grout bonded after stressing.

The complete ground anchor assembly, excluding the grout is referred to as the tendon. The tendon includes the prestressing steel element (strands or bars), anchorage, corrosion protection, sheathings, centralizers and spacers. The sheathing is a smooth or corrugated pipe or tube that protects from corrosion the prestressing steel in the unbonded length.

Ground anchors are installed in drill holes that are subsequently grouted. Centralizers enable the tendon to be positioned in the center of the drill hole such that the specified minimum grout cover is achieved around the tendon. For multiple element tendons, spacers are used to separate the strands or bars of the tendons so that each element is adequately bonded to the anchor grout.

The grout is a Portland cement based mixture that provides load transfer from the tendon to the ground and provides corrosion protection for the tendon. Anchor grout for soil and rock anchors is typically a cement grout conforming to ASTM C150. A water/cement ratio of 0.4 to 0.45 by weight, and Type I cement, will normally provide a minimum compressive strength of 21 MPa at the time of anchor stressing. Admixtures are not generally used for most applications. Anchors may be grouted by tremie (gravity displacement) methods, pressure grouted, or post grouted.

Both bar and strand tendons are commonly used for soil anchors for highway applications in the United States. Material specifications for bar and strand anchors are codified in American Society for Testing and Materials (ASTM) A722 and ASTM A416, respectively. Bar tendons are commonly available in 26 mm, 32 mm and 35 mm diameters, and uncoupled lengths up to 18 m. Strand tendons comprise multiple or single, seven wire strands. The common strand in U.S. recent practice is 15 mm in diameter, although many anchors have used 12 mm strand.

In this report, the term “tieback” is used to describe ground anchors supporting retaining walls. Elements that support retaining walls, but are anchored to deadmen will be referred to as “wall anchors”.

2.1.2 Rock Anchors

EM 1110-1-2907 (1980) describes the use of tensioned rock bolts for rock reinforcement in underground and surface structures in civil engineering works. The desired result of a tensioned rock bolt installation is a permanently tensioned reinforcement with positive bond to the rock. Rock bolt systems are made up of wires, strands or bars. Rock bolts may employ either a mechanical or a grouted type of anchorage. Slot and wedge set bolts (Figure 2-2) and expansion shell (Figure 2-3) are examples of mechanically anchored bolts. Grouted anchorages include resin-grouted (Figure 2-4), or cement grouted (Figure 2-5) bolts. Resin grouted bolts are easier to install compared to cement grouted anchorages.

Hardware including bearing plates, bevel washers, hardened flat washers, and nuts is used to transfer load from the anchorage to the rock face. Bevel washers should be used between the bearing plate and the hardened washer to create a uniform bearing surface for the nut normal to the bolt axis. Lack of uniform contact between the nut and bearing surface will result in combined stresses (due to axial load and bending moment) in the bolt, which will reduce its strength.

2.1.3 Soil Nails

As described by Byrne et al. (1996) soil nailing is an in-situ reinforcement technique which consists of inserting long bars, or “nails”, into otherwise undisturbed natural soil to stabilize a soil mass. Nailing differs from tie-back support systems in that the soil nails are passive elements that are not post-tensioned as are the tendons in the case of tie-backs. Load is transferred to soil nails during excavation. The excavation is advanced in increments such that the soil is able to stand unsupported until installation of soil nails at that level is complete.

The sequence of excavation is as follows: 1) excavate below the first level of soil nails, 2) drill holes at each location, insert soil nails and grout along their full length, 3) apply wire mesh and shotcrete to face, and 4) advance the excavation to the next increment. Upon completion of the excavation, a permanent wall facing of reinforced shotcrete, cast-in-place, or precast concrete panels is often installed.

Drill hole diameters range from 100mm to 300mm and are spaced between 1m and 2m apart. For conventional drill and grout nail installations neat cement grout is used. Soil nails typically consist of steel reinforcement having a diameter in the range of 25 to 35 mm, and a yield strength of 420 to 500 MPa. Soil nail inclinations are generally on the order of 15° below horizontal to facilitate grouting. For permanent nails the steel bar is typically protected against corrosion damage with a heavy epoxy coating or by encapsulation in a grout-filled corrugated plastic sheathing.

2.2 FACTORS AFFECTING SERVICE LIFE

Factors affecting service life of buried metal tensioned systems include corrosion, loss of prestress, loss of anchorage, and creep of clay soils (Briaud et al., 1998). The corrosion of these systems can have serious consequences where aggressive soil, rock and groundwater conditions are present and where provisions have not been incorporated to provide adequate redundancy (e.g., protective coatings and covers, sacrificial thickness and cathodic protection) and/or redundant elements during the service life of the structure. Compared to 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 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.

2.2.1 Corrosion Mechanism

The corrosion of metal-tensioned systems for geotechnical and other applications occurs as the result of the dissolution or deterioration of metals (or their mechanical properties) by chemical or electrochemical reaction with the ground environment. Except for the more noble metals and copper, which exist in nature in their metallic state, other metals (i.e., steel) processed by application of energy revert by corrosion from their temporary or processed state to their natural state. In effect, corrosion releases the energy imparted during processing in the form of electrical energy.

Electrical flow occurs because a voltage difference exists, in the presence of an electrolyte, between two metal surfaces or two locations on the same part. If a constraint is not present to inhibit this tendency, the metal will react with oxygen and water from its environment to form oxides and/or hydroxides. During this process, electrons are transferred from the metal to form hydroxyl ions, and the metal ions migrate into the aqueous electrolyte. The sites where dissolution of the metal occurs (corrosion sites) are anodic, whereas sites where oxygen and water are converted into hydroxyl ions are cathodic. Metal areas surrounded by higher oxygen concentration form cathodic zones, and those with the low concentration are the anodic zones.

Because two metal pieces, or two portions of the same metal, do not often have the same electrical potential in an electrolyte, rates of corrosion depend on the:

- Type of metal and condition of the metal surface
- Type of electrolyte
- Presence of different materials at the metal/electrolyte interface

Whereas most chemical elements and their compounds are present in soil and rock (or dissolved in groundwater), only a few contribute a significant effect on corrosion. In areas of high precipitation, the leaching of soluble salts and other compounds with time tends to lead to generally acidic ground conditions. In arid areas, evaporation and capillary processes tend to bring soluble salts to the soil and rock layers at shallow depth leading to generally alkaline ground conditions with time. The majority of failures and problems associated with metal-tensioned systems used for walls and ground support occur within a relatively shallow depth (i.e., about 2m) of the exposed face (Cheney, 1988; FIP, 1986).

Except for areas where cracks develop during prestressing, or due to shrinkage, hydrated cement grout surrounding metal-tensioned elements usually provides adequate protection against aggressive ions. Passivity refers to the loss of chemical reactivity experienced by certain metals and alloys under particular environmental conditions. Certain metals and alloys become essentially inert and act as if they were noble metals. A surface film or protective barrier is formed that is stable over a considerable range of oxidizing power. Metals that possess an active-passive transition become passive or very resistant in moderate to strong oxidizing environments.

Service life may also be increased by use of surface treated steel. The treated steel has a thin oxide coating. Hou, et al. (1997), and Fu and Chung (1996 and 1997) describe treatment by oxidation using either water or ozone. The improved corrosion resistance resulting from the oxidation treatment of the steel is due to the increased uniformity relative to the as-received steel, which is non-uniform due to uneven rusting. Uniformity means less microscopic galvanic cells, which cause corrosion.

2.2.2 Types of Corrosion

Fantana (1986) describes the theory of corrosion and Xanthakos (1991) describes the process as it relates to ground anchors. A number of different types of corrosion are described including uniform, galvanic, pitting, crevice corrosion, stress crack corrosion, hydrogen embrittlement, microbacterialogic induced corrosion, and stray current corrosion.

As illustrated in Figure 2-6, for uniform corrosion, localized anodic and cathodic sites do not exist. With this form of attack, the metal is gradually, and uniformly changed to ferrous ions from the outer surface inwards. Consequently, the reduction of cross section is uniform and the inner portion of the metal is unaltered by the process. The rust formed on the metal surface has no cohesion, and is readily displaced by water flow.

If variations in electrode potential along the metal surface allow separate corrosion cells to develop, then localized corrosion can occur. Localized attack is generally associated with the presence and localized breakdown of a protective film on the metal surface. A mechanism of pitting or crevice corrosion will occur in the presence of aggressive ions such as chloride. A pit is formed at an area where chloride ions locally weaken the passive film that protects the steel. The anode is established where the passive film is destroyed and the surrounding steel becomes the cathode. While pitting can cause little overall metal loss, the effects can be significant due to the localized reductions in the metal cross section and increased stress on the unaltered section. It is generally accepted that pitting can be defined in relation to pit geometry, and although the distinction between pitting and localized corrosion is somewhat arbitrary, it has been suggested that this transition occurs when the pit width to depth is 4 or less, although a ratio of 1 is the most widely accepted definition of a pit. Pitting corrosion is of particular concern with prestressing steels, such as anchor tendons since they are subjected to high stresses and have small cross sectional areas, (Weatherby, 1982).

Crevice corrosion is another form of localized corrosion. Where a crevice is formed at the contact between metal surfaces corrosion is accelerated according to a mechanism causing rapid consumption of oxygen. When this results, it is possible for the local environment within the crevice, where the electrolyte solution may stagnate, to become more acidic. Corrosion initiates within the crevice where the anodic reaction is cultivated. Several locations in ground anchors may be susceptible to crevice corrosion. These would be the grout steel bond, the anchor head region, and portions of the unbonded regions.

Stress crack corrosion (SCC) is produced by the combined action of static tensile stress, and localized corrosion. Although it is not completely understood, SCC is believed to be an anodic corrosion process with the crack developing at anodic sites, often a pit or crevice that has formed. Apparently, tension forces concentrate at the tip of a narrow pit, resulting in the formation of fresh metal surfaces and further deterioration by pitting corrosion. The subsequent reduction of the metal cross section can lead to plastic yielding. Stress crack corrosion is usually associated with high strength metals ($f_y > 850$ MPa) subjected to high tensile stresses. Cold drawn steel is significantly less sensitive to SCC than quenched and tempered high strength steels with martensitic structure.

For prestressing steels, SCC is likely to take the form of hydrogen embrittlement. This involves the migration of atomic hydrogen into the metal lattice where hydrogen molecules are formed producing internal pressure in the metal. Many of the early ground anchor failures appear to be related to hydrogen embrittlement. The problem of hydrogen embrittlement is unlikely to arise if proper attention is paid to the selection of materials and workmanship (Griffin, 1981). The Post Tensioning Institute (PTI) recommendations prohibit contact between cement grout and galvanized strands because of the risk of hydrogen embrittlement (PTI, 1998).

Other forms of corrosion include bacterial attack, corrosion fatigue and corrosion involving oxygen. Corrosion due to bacterial attack is usually associated with metabolic processes of sulfate reducing bacteria under anaerobic conditions. Conditions suitable for bacterial attack occur in organic soils or sulfate-bearing clays below water. The corrosion process can be either local or general in extent depending on the ground type, depth of embedment and presence of protective coatings. Corrosion fatigue occurs due to the combined effects of cyclic loading and corrosion. However, different from stress corrosion, corrosion fatigue can occur in most aqueous environments and is not associated with any special combination of metals or ions. Corrosion involving oxygen accelerates corrosion at the cathode under alkaline environments. When enclosed in a highly alkaline grout, rust can form to protect the steel element unless the grout cracks and circulating water removes the rust coating.

Stray electrical currents may exist around direct current mass transit facilities, electrical transmission systems, waterfront structures in salt water, and welding shops. Corrosion of tendons placed in stray current fields can accelerate due to the establishment of an electric potential along the tendon. Stray currents cause corrosion at locations where current leaves the structure and enters the ground or water electrolyte. High tensile strength steel used in tendons is more sensitive to stray current corrosion than normal steel.

2.2.3 Creep of Metal Tensioned Systems

A literature search was conducted to obtain information on creep behavior of soil nail, rock bolt and tied-back retaining wall systems. According to Schnabel (1998), many tiebacks are installed in soils vulnerable to creep, and it is obvious that some tiebacks have failed by creeping. However, very little specific information is available describing creep-induced failure of in-service metal-tensioned systems. Studies are needed whereby anchor loads and anchor capacities are measured over time. This requires that load cells, strain gauges and/or provisions for lift-off testing be installed and accessible throughout the service life of the structure. Since these provisions are not routine construction practice, data on the long-term performance of facilities installed in creeping ground is limited. Some information is available on laboratory studies of creep, and correlations to field behavior, but most of the information obtained was related to guidance on soil types within which ground anchors are vulnerable to creep, and insitu creep testing as described by PTI (1996).

Creep is defined as any continuing movement under constant stress. Experience has shown that the rate of creep may decrease with time, remain constant, or accelerate. The service life of a metal-tensioned system may be affected by the inability to accurately estimate creep over the long term. Creep behavior may have an effect on the load carrying capacity of the anchorage, and on the load transferred to the element from restrained soil or rock. Briaud, et al. (1998) observed from field experiments conducted at the Texas A&M Riverside Campus NGES that tie-back load increases over time for that site were mostly related to down drag (settlement) of the retaining wall relative to the anchors. The Texas A&M study was unique in its attempt to measure the load carried by the anchor over time. Most of the information available on creep of metal-tensioned systems is related to relaxation of the anchorage and subsequent loss of anchor capacity.

Theoretically, creep may develop in three components of the anchor: creep of the ground surrounding the anchor bond zone and including the ground/grout interface, creep of the grout in the bond zone and including the steel/grout interface, and creep of the steel comprising the tendon and/or connections. Generally, the latter two factors have little significance for typical design loads, although chemical grouts frequently have time-dependant properties that must be carefully studied in relation to anchor applications. Creep of the ground surrounding the anchor zone is the primary source of creep movement in metal tensioned systems in geotechnical applications (Cheney, 1988).

Some soils and rocks are less susceptible to creep than others. Sands deform rapidly and do not undergo measurable creep. Creep is encountered in plastic strain softening soils and materials subject to progressive strength reduction such as fissured or slickensided clays (ASCE, 1997).

Past practice has been to avoid installing ground anchors or soil nails in soils, such as soft clays, that are susceptible to creep. Schnabel (1998) reports that past industry practice has been to use tie-backs only in soils where an undisturbed shear strength of 50 kPa can be reliably counted on, or alternatively, when the blow count from the Standard Penetration Test exceeds 8 blows per foot. The FHWA manual on soil nailing (FHWA, 1996) recommends that soil nails not be used in a number of soil conditions, including “organic or clay soils with a Liquidity Index greater than 0.2 and undrained shear strength less than about 50 kPa.” These restrictions result from the idea that clay soils with these properties may creep over the long term, resulting in unacceptable ground movements or pull-out failure as nails experience increasing loads with time (Oral and Sheahan, 1998).

Most approaches to designing for creep draw heavily on the theory of consolidation (Schnabel, 1982). For a bearing-type anchor, this is a very similar situation and the strength of the clay surrounding the anchor increases as consolidation occurs. This behavior leads to a stable creep response where the creep deformations decrease over time. However, for a shaft type anchor there is no increase in soil strength associated with creep deformations and it is possible that creep will increase over time. According to Cheney (1988), soil creep phenomena are related to secondary compression of the soil structure. Clays and organic soils may undergo significant secondary consolidation depending on plasticity and preconsolidation history. Theoretically, bond zone creep potential is directly related to the coefficient of secondary compression obtained from consolidation tests of soils in the bond zone. However, no direct correlation has been made between laboratory and field tests. Based on laboratory tests, Briaud, et el. (1998) conclude that anchor creep is not a problem if the shear load transferred to the clay is less than the yield strength of the clay.

Schnabel (1998) suggests that tiebacks may be installed in soils susceptible to creep using a reduced tie-back capacity and careful creep testing. Recent experience in softer clays has shown that the hollow stem tieback, which is a technique that casts a large grout body, helps to achieve desired capacity in soft clay. Time-delayed strain may not be readily apparent during short-duration tests. Creep performance testing of anchors is described by PTI (1996). According to Schnabel (1998), where creep testing has been done the performance of tied-back retaining walls has been good.

2.3 EXISTING CODES, STANDARDS, AND RECOMMENDED PRACTICE

2.3.1 Design of Metal-Tensioned Systems

Ground Anchors

Goldberg et al. (1976) is a three-volume set that provides recommendations for design, construction, and field-testing of tie-backs. Volume I (FHWA-RD-75-128) is a summary of detailed information given in Volumes II (FHWA-RD-75-129) and III (FHWA-RD-75-130), and serves as a reference guide for design and construction of cut and cover tunnels. Volume II is a detailed discussion of geotechnical engineering issues related to the design of tied-back wall systems. General information about tie-backs is provided including a comparison between the use of tie-backs and internal bracing. Recommendations are included for estimating lateral earth pressure in a variety of soil types, and the effect of wall stiffness on load distribution. Volume III presents criteria for design and construction of tie-backs, and tied-back wall systems.

Littlejohn (1980) describes empirical design methods for the estimation of ultimate pullout capacity of the grouted fixed anchor zone. The benefits of pressure grouting along the bonded zone are illustrated. Relationships between anchor capacity and grouting pressure are presented for anchors installed within a variety of soil and rock types.

Schnabel (1982) is one of the first comprehensive guides to tie-backs in the United States. The book is divided into two sections. The first section is a basic overview of the design, construction, testing, and contracting procedures related to installation of tie-backs. The second section goes deeper into specific design issues including materials, corrosion protection, performance testing, capacity to resist uplift, different wall types, and global stability.

Weatherby (1982) summarizes current tie-back technology. It contains recommendations for the design, specification, corrosion protection, and testing of permanent and temporary tie-backs. Descriptions of tie-back applications, construction techniques, load-transfer mechanisms, and creep behavior are also included.

The Naval Facilities Design Manual DM-7.2 (NAVFAC, 1986) provides design guidance for excavation support and support of permanent retaining structures. Design guidance for computation of lateral earth pressures and factors of safety to be used in the design of naval shore facilities is provided.

Cheney (1988) provides guidance to assist engineers in evaluating the feasibility of anchors in specific situations. The manual serves as an introduction to basic anchoring concepts.

FHWA-RD-89-043 Reinforced Soil Structures Design and Construction Guidelines (1989) presents a design methodology for multi-anchored retaining systems, which derive their pullout resistance from the passive resistance offered by anchored deadmen.

AASHTO (1996) provides specifications on the design, fabrication, installation, testing, and stressing of permanent cement-grouted ground anchors. Some design guidance is also provided relative to lateral earth pressures applied to multi-tiered support systems in different soil types.

FHWA Geotechnical Engineering Circular No. 4 Ground Anchors and Anchor Systems (1998) is an updated guidance document that provides information on the use of ground anchors for highway applications. Ground anchor systems and their components are described in detail. Topics include materials, applications of ground anchor systems, and corrosion protection systems.

Rock Bolts

The USACOE (1980) provides guidance on the engineering and design of rock reinforcement in EM 1110-1-2907. Design procedures and examples of successful installations are presented for guidance in the design and construction of rock reinforcement systems.

FHWA TS-89-045 Rock Slopes Design, Excavation, Stabilization, (Golder Associates, 1989) describes design methods for stabilization of rock slopes. Equations are presented for computing the factor of safety of a rock slope stabilized with rock reinforcement. Use of bar and strand elements and the advantages and disadvantages of prestressing are described. Construction techniques and methods of corrosion protection are briefly discussed.

Soil Nails

FHWA-RD-89-043 Reinforced Soil Structures Volume I. Design and Construction Guidelines (1989) is a comprehensive publication dealing with the design, construction, and evaluation of soil reinforcement methods. Included are design examples, information from manufacturers, and results from research to assist the engineer in all aspects dealing with reinforced soil structures, including soil nailing.

Recommendations Clouterre - Soil Nailing Recommendations for Designing, Calculating, Constructing and Inspecting Earth Support Systems Using Soil Nailing (1991) is a comprehensive guide dealing with issues in soil nail systems. This guide provides information on construction techniques, anticipated response of the wall system, soil testing required to determine design parameters, design considerations and methodology, construction sequence, durability of soil nails, and inspection of soil nailed structures.

FHWA-PL-93-020 FHWA Tour for Geotechnology-Soil Nailing (1993) is a summary of a scanning tour that focused on European soil nailing techniques. This book summarizes information obtained from Europe's leading experts on the technique of soil nailing. Issues addressed include: design, construction, contracting practices, and research and development.

FHWA- SA-96-069 Manual for Design and Construction of Soil Nail Walls (1996) gives an extensive description of the history of soil nailing in North America and Europe. A recommended construction sequence and details of each step are presented. Requirements for materials, construction equipment, corrosion protection, and wall drainage are described. Figures showing typical soil nail installations, and tables outlining selected case histories are available.

2.3.2 Assessment of Aggressive Ground Conditions

Numerous quantitative guideline limits are used worldwide for assessing the potential aggression posed by corrosion in a subsurface environment (e.g., PTI, 1996; Merrifield, et al., 1997; Rohde and Heinz, 1997). Due to the complexity of subsurface conditions, which can cause corrosion of metal-tensioned systems, Xanthakos (1991) concluded that the design of systems can be addressed by:

- Comprehensive analysis of the environments into which the system will be installed; or

- Providing sufficient protection to ensure complete protection of metal elements.

The aggressiveness of the environment is affected by the (PTI, 1996):

- Soil resistivity
- Soil pH

- Chemical composition of soil, rock and groundwater
- Water and air permeability of ground
- Groundwater level
- External electrochemical and physical factors (e.g., stray currents)

PTI (1996) defines ground as aggressive if one or more of the following conditions are detected:

- pH < 4.5
- Resistivity < 2,000 ohm-cm
- Sulfides present
- Stray currents present
- Known chemical attack of concrete structures in vicinity of site
- Aggressive atmospheric conditions

Xanthakos (1991) identified moisture content, chloride and sulfate ion concentration and the hydraulic conductivity of the ground as the factors that most affect corrosion potential of metals in a ground environment. Table 2-1 provides general measures of corrosion potential based on the results of resistivity and redox potential testing of soil and groundwater.

Table 2-1
Corrosiveness of Soils
(Modified after Xanthakos, 1991)

Corrosiveness	Resistivity (Ω/cm)	Redox Potential (mV)
Very Corrosive	< 700	<100
Corrosive	700-2000	100-200
Moderately Corrosive	2000-5000	200-400
Mildly Corrosive/Noncorrosive	>5000	>400 for clay soil

Monfore (1968) studied measurement of the electrical resistivity of concrete. Measurements indicate that the resistivity of concrete is very sensitive to moisture conditions. Moist concrete is an electrolyte having a relatively low resistance of 10^4 ohm-cm, and dry concrete is an insulator with a much higher resistivity of approximately 10^{11} ohm-cm.

Where sulfates are present at levels sufficient to affect the grout set, measures should be taken to design a dense, low permeability grout using sulfate-resistant cements, with limits on minimum cement and maximum water-cement ratios. PTI (1996) recommends using Type II Portland cement if the sulfate content is between 0.2 and 2.0 percent, and

Type V cement if the sulfate content is greater than 2.0 percent. Flyash and/or silica with high-range water reducers can improve sulfate resistance. Due to the vulnerability of cement grouts to acidic conditions, care must be taken in designing grouts for low pH environments.

With regard to potential problems posed by groundwater, Table 2-2 from Xanthakos (1991) provides parameter limits from five tests for qualitatively assessing the potential aggressiveness of groundwater. The limits assume the groundwater is stagnant or flowing very slowly, the attack is immediate and unaffected by the presence of grout around the metal. The table 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.

Table 2-2
Parameter Limits for Aggressive Ground 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

Criteria similar to these were recommended as part of a study to evaluate the corrosion potential of steel elements in soil reinforced structures (EE&S, 1990).

Moody (1993) describes specifications, adopted by the New York State Department of Transportation (NYSDOT), that address issues of corrosion potential of backfill used in the construction of mechanically stabilized earth structures. Backfill materials must be tested for corrosion potential including resistivity and pH, and may be tested for sulfides at the department’s discretion. Backfills must meet the criteria described in Table 2-3; except as specified below.

> “Material failing to meet the resistivity criteria may be tested for sulfates and chlorides. Material meeting the criteria for both sulfates and chlorides having a minimum resistivity greater than 1000 ohm-cm will be acceptable.”

Table 2-3
NYSDOT Criteria for Mechanically Stabilized Earth Backfill
(Moody, 1993)

Parameter	AASHTO Test Method	Requirement
Resistivity	T 288-91 I	>3000 ohm-cm
pH	T 289-91 I	5 to10
Sulfates	T 290-91 I	<200 mg/kg
Sulfides		<300 mg/kg
Chlorides	T 291-91 I	<100 mg/kg

2.3.3. Corrosion Protection

FIP (1986, 1996) describe recommendations for corrosion protection of ground anchorages and prestressing steel tendons. Details of the recommendations include those for single and double corrosion protection systems, anchor head assemblies, and use of greased and sheathed tendons. Wolfel (1982) describe the 1996 FIP recommendations on prestressed ground anchors as emphasizing that the corrosion hazard for prestressing steel must not be considered as normal rust, but as brittle fracture caused by stress corrosion or hydrogen embrittlement.

The Post Tensioning Institute issued the third edition of its "Recommendations for Prestressed Soil and Rock Anchors," (PTI, 1996). The 1996 recommendations are an extensive revision of the previous version published in 1986. Nierlich and Bruce (1997) review these recommendations highlighting the most significant changes and improvements with respect to corrosion protection and acceptance testing. Recommendations related to corrosion protection underwent fundamental changes. More emphasis is put on the corrosion protection near the stressing end where statistics show by far the highest frequency of corrosion failures. For permanent anchors, single corrosion protection is only allowed in nonaggressive soils, for anchors where failure does not have catastrophic consequences, and where the increase over the cost of double corrosion protection systems results in unjustifiable and considerable extra expense.

Recommendations by PTI (1996) require the following be considered in designing a corrosion protection system for prestressed anchors:

- Service life of structure
- Aggressiveness of the environment
- Consequences of tendon failure
- Life-cycle costs
- Tendon type
- Installation details

Figure 2-7 from PTI (1996) presents the decision tree to be followed in selecting corrosion protection for particular site conditions. Corrosion protection for ground anchor tendons includes either one or more physical barriers from the corrosive environment. The barrier layers include anchorage covers, corrosion inhibiting compounds, sheaths, encapsulations, epoxy coatings, and grouts.

The requirements of an effective corrosion protection for metal-tensioned systems include:

- Effective life at least equal to the estimated service life of the element
- Environmentally inactive without adverse impacts on the element
- Should not interfere with element performance
- Compatibility with element components
- Usually can only be installed during a single application
- Should not fail during performance testing
- Sufficiently flexible (i.e., anchors) to withstand handling
- Facilitate inspection

PTI (1996) identifies two classes of corrosion protection:

- Class I - Encapsulated Tendons (often referred to as double corrosion protection)
- Class II - Grout-Protected Tendons (often referred to as single corrosion protection)

Table 2-4 presents the requirements for each class of protection.

Table 2-4
Corrosion Protection Requirements
(PTI, 1996)

Class	Protection Requirements		
	Anchorage	Unbonded Length	Tendon Bond Length
I Encapsulated Tendon	1. Trumpet 2. Cover if exposed	1. Grease-filled sheath, or 2. Grout-filled sheath, or 3. Epoxy for fully bonded anchors	1. Grout-filled encapsulation, or 2. Epoxy
II Grout-Protected Tendon	1. Trumpet 2. Cover if exposed	1. Grease-filled sheath, or 2. Heat-shrink sleeve	1. Grout

The features of Class I and II protection are presented in Figures 2-8 and 2-9.

Schupack (1998) describes a number of different sleeves that have been employed as part of corrosion protection for prestressing steels. A number of these may also have been employed for ground anchor installations. The durability of the system and the potential for corrosion is affected by the type of sheath surrounding the steel and its method of installation.

- “Stuffed or Push-Through Tendons” consist of a seven wire strand inside a loose plastic sheath. During manufacture, a greased strand is inserted (stuffed) into the sheathing. Schupack (1998) reports that often the grease does not fill the sheath, or is discontinuous along the length of the strand. This type of sleeve is vulnerable to infiltration of water. Where water is available, corrosion will likely occur with time.

- “Cigarette Wrapped Tendons” represent a slight improvement over “stuffed or push-through tendons.” The sheath is formed by folding a plastic strip around a pre-greased moving strand. The strip overlaps on itself and is then heat-sealed. Depending on the quality control during manufacture, the resulting sheathing can range from loose to tight. Corrosion seems to occur with a higher or lower frequency, depending on where it fits between these extremes and the efficacy of heat sealed joints.

- “Extruded Tendon” is formed from molten plastic around the greased strand then passing through a die and finally cooled with water. This last stage produces a measurable shrinking of the sheathing. Corrosion problems are significantly less with this type of protection. This is the most common form of sheath currently in use.

- “Encapsulated Tendons” consist of tendons with extruded sheaths and protected anchor head assemblies. A well-designed encapsulated sheath, properly protected before and during construction, provides the best assurance of long-term durability.

- “Paper Wrapped Button Head Tendons,” now practically obsolete, consist of multi-parallel wire or single strand tendon which are greased and paper wrapped.

Most systems installed today employ either extruded tendon, or encapsulated tendon sleeves (see Section 5.1.1). Some of the older systems may employ other, more vulnerable types of protection such as stuffed, cigarette, or paper wrapped tendons.

Soil Nails

Durability and provisions for corrosion protection were part of a comprehensive research study conducted by the French Ministry of Transport to develop specifications for the design of soil nail walls used for temporary and permanent support of excavations

(Schlosser, et al., 1992). In France, the most common method used to account for the effects of corrosion is to provide a sacrificial thickness to the nail cross section. On this basis, the required nail section is estimated so that at the end of the expected service life, the remaining non-corroded section is adequate to resist the imposed loads. The research study developed recommendations for the required sacrificial thickness based on consideration of the soil type, resistivity and moisture content, and other factors as summarized in Table 2-5.

Table 2-5
Recommended Sacrificial thickness for Soil Nails
(Schlosser, et al., 1992)

Class	Service Life		
	< 1.5 years	1.5 to 30 years	> 30 years
IV	0	2 mm	4 mm
III	0	4 mm	8 mm
II	2 mm	8 mm	Plastic barrier
I	Compulsory plastic barrier		

2.3.4 Recommended Test Methods

Sabatini et al. (1998) recommend that for permanent anchored systems the aggressiveness of the ground must be evaluated. Corrosion potential is of primary concern and is evaluated based on the results of tests to measure the following properties: (1) pH (ASTM G51; AASHTO T-289), (2) electrical resistivity (ASTM G57; AASHTO T-288), (3) chloride content (ASTM D512; AASHTO T-291) and (4) sulfate content (ASTM D516, AASHTO T-290).

PTI (1986) suggest testing that relies on electrical isolation of the tendons. Electrical isolation may be accomplished by encapsulating the total length of the prestressing steel elements and the anchorage assemblies. Three monitoring possibilities for an isolated tendon system include (1) detection of damage to the isolation system, (2) monitoring the polyethylene sheath for tears and holidays and (3) monitoring the integrity of the steel cable stay.

Updated recommendations described in PTI (1996) state that provided there are no catastrophic consequences, the small percentage of known corrosion failures is thought to be an acceptable construction and performance risk (Nierlich and Bruce, 1997). Electric isolation testing, as a means of confirming the integrity of the installed corrosion protection system, where the tiniest imperfection will result in rejection of the anchor is considered as too costly and impractical on a routine basis. It is required, however, in the presence of stray electric currents.

FIP (1982) describes in-service inspection procedures for prestressed concrete pressure vessels and containment structures for nuclear reactors. Testing consists of laboratory evaluations of exhumed anchors and pressure testing of vessels that incorporate in-service anchors. During pressure testing, tendon forces and elongation are measured.

Standards published by the National Association of Corrosion Engineers (NACE, 1997) provide descriptions of the measurement techniques most commonly used on underground piping. The standard contains instrumentation and general measurement guidelines. It includes methods for voltage drop considerations when making pipe-to-electrolyte potential measurements, and provides guidance to prevent incorrect data from being collected and used. Although this standard is primarily applicable to buried metallic pipelines, the measurement techniques are suitable to any buried metal structure.

The American Concrete Pressure Pipe Association (ACPPA, 1988) and Benedict (1989) describe an in-service testing protocol for buried metal pipelines. The testing protocol involves the use of half-cell potential measurements. Baseline potential measurements are established several months after installation. Approximately one year later, a second survey is conducted. If potentials more negative than 300 mV (CSE) are measured, further investigation for the possibility of corrosion is recommended.

A draft version of the upcoming ACI 222 standards was reviewed. The draft standard describes a number of procedures for identifying corrosive environments and active corrosion in concrete. The draft standard describes several electro-chemical measurements including half-cell potential mapping, linear polarization resistance, and the AC impedance technique. Special considerations related to prestressed reinforcement are also addressed in the draft standard.

2.4 PERFORMANCE DATA

Lessons learned from the use of prestressing steel strand in other applications are relevant to the performance of prestressed ground anchors and rock bolts. Much information is available from the nuclear, pipeline and bridge industries, as inspection and condition assessment of these facilities is mandated. This section starts by describing performance data obtained from these sources.

Although inspection is not often mandated, some information exists documenting the performance of ground anchors and rock bolts. Much of this information is the result of special studies, previously published by others. In this Interim Report, relevant publications are cited with a summary of pertinent findings. In the United States, soil nail installations are relatively newer; therefore, fewer performance records are available compared to ground anchors and rock bolts.

2.4.1 Nuclear Containment Structures

Irving et al. (1975) report on the results of in-service monitoring of unbonded, prestress concrete pressure vessels for nuclear reactors in the United Kingdom. Inspections of prestressing systems are conducted prior to installation and throughout the service life of the facility. Inspections include removal and visual inspection of selected strands.

At one site, water had entered the tendon ducts during construction and caused emulsification of the grease covering the tendon wires. Inspection revealed that active pitting corrosion had occurred of unusual extent and depth. The primary cause of the pitting corrosion was electrolytic attack due to impressed current from D.C. welding equipment, which had been earthed to the metal component of the vessels. Galvanic attack caused by the presence of chloride salts deposited from the atmosphere onto the tendon grease was also identified as a potential source of pitting corrosion.

A comprehensive program of research into possible causes of corrosion led to the general recommendations for the protection of prestressing material embodied in the 1973 British Standards. Regular inspections of prestressing systems, both in-service and awaiting installation, indicate that the protective measures taken have limited the incidence of pitting corrosion on wires and strands. Inspections performed at two older stations, completed before specific recommendations for corrosion protection were enforced, did not reveal corrosion approaching unacceptable limits.

Rotz (1975) reported results from surveillance and inspection at five nuclear reactors located in the United Sates. These structures are post-tensioned with unbonded, 90 wire tendons. The total number of tendons for all five facilities is approximately 4000. Tendon surveillance is typically conducted after the first year, three years following the initial structural integrity test, and at successive five-year intervals for the life of the plant. The paper presents the results from tendon surveillance at the one and three-year intervals. In all, approximately 100 wires and sheathing filler samples were inspected and tested, with liftoff measurements at approximately 100 tendon locations. Results indicate that prestress losses occurred at about the rate predicted. Tendon deterioration or wire breakage was not observed, and the physical properties of the tendons evaluated were essentially constant over time. Sheathing filler deterioration was not reported and the observed corrosion rate on wires and components was insignificant. No stress corrosion or hydrogen embrittlement was observed.

Drew (1979) reported on the surveillance of tendons and prestressing strands from the Sacramento Utility District’s Rancho Seco Unit 1 containment structure. The surveillance was conducted at one year intervals beginning in March 1975, in accordance with NRC Regulatory Guide 1.35, Surveillance Standards AP205.03. Surveillance of unbounded strand post-tensioning systems verifies not only the serviceability of the system, but also the predictability of the tendon tension with time. All corrosion surveillance strands were observed to be free of corrosion.

There have been corrosion incidents of post-tensioned unbonded tendons in several nuclear vessels. The most important is Calvert Cliffs (Poekler and Hardies, 1997), which showed both brittle and ductile wire failures near the anchorages about 20 years after construction.

2.4.2 Prestress Concrete Pipe

Prestress concrete cylinder pipe (PCCP) consists of either a concrete core encased by steel, or a concrete core with an embedded steel liner. Both types are wrapped with prestressing wire (ASTM A648) and coated with mortar.

Several factors have been identified that accelerate deterioration of PCCP including: mortar coating defects and damage, aggressive soil and groundwater conditions, and prestressing wire defects (Engineering News Record, 1990). Problems with hydrogen embrittlement may also be caused by faulty cathodic protection systems.

Clift (1991) and Hall et al. (1996) report on the overall performance of PCCP. Due to the passivating properties of the highly alkaline cement grout (pH $\approx$ 12.5), the cement slurry and mortar coating provide corrosion protection for the prestressing wire. In unusual circumstances, such as high chloride environments, the passivating properties of the cement grout may become compromised. Hydrogen embrittlement precipitated by the presence of excess chlorides and carbonation of concrete is the most common form of corrosion in PCCP.

2.4.3 Post Tension Concrete Beams and Bridge Decks

Schupack and O'Neil (1997) describe the results of exposure tests of post-tensioned beams installed in 1961 at the Corps of Engineers Severe Weather Station at Treat Island, Maine. The results of the study described in the report emphasize the importance of corrosion protection at the anchor head.

Due to the catastrophic failure of the Ynys-y-Gwas Bridge in 1985, the use of grouted duct tendons on prestress bridge decks was banned in England (New Civil Engineer, 1992a). As a result of the failure, approximately 3000 grouted duct post-tensioned bridges existing within the UK at the time were inspected. The condition of the tendons was established by delicate micro-drilling and insertion of an endoscope.

Hampejs et al. (1991) reported failure of Macalley prestressing bars within the first few days following post-tensioning on two bridges in the U.K. The failures were attributed to thread rolling defects and hydrogen embrittlement due to the zinc coating process. The results of this study demonstrate that electrogalvanising can provide a source of hydrogen embrittlement.

Tendons were evaluated from a 30-year old post-tensioned concrete viaduct in Oxfordshire, England (New Civil Engineer, 1992b). Each beam had five tendons composed of twelve, 7 mm diameter wires. Wires were extracted from 4 of the 5 tendons and severe pitting, within 10 mm of the face plate was observed in four of the tendons; and two had pitted completely through the cross section. The corroded tendons were missing grout, had bad detailing and rust at the anchor head.

West (1996) used accelerated corrosion tests to evaluate cement grouts for corrosion protection of prestressing strand. The tests used anodic polarization to accelerate corrosion by providing a potential gradient, driving negative charged ions through the grout to the steel. Cement grout with pH-values up to 12.6 proves to be one of the best means of corrosion protection of steel. Fractures of prestressing steel metal ties within a cement grouted anchorage zone have never been reported.

Henricksen et al. (1998) compiled information on the condition of post-tensioned bridges in Denmark. Corrosion of the tensioning cables was assessed to be the most important cause of deterioration. Critical parameters include the tightness of the cable joints, and, to a lesser extent, whether the cable duct is injected with grout or not. A leaky cable duct might lead to corrosion, in principle, at any location along the tendon because water penetrates along the cable duct.

Lebura and Ziobron (1998) present observations made on the prestressing cables dismantled from the beam of an undercrane bridge after 34 years of exploitation. Observations include measurements of prestressing force, and the condition of the tendons and grout. Pitting corrosion was observed on some of the tendons exhumed.

2.4.4 Rock Bolts

In the 1980's, the U.S. Bureau of Mines (USBM) carried out detailed research on the corrosion resistance of high strength, low alloy, (HSLA) steel. For nongalvanized split sets, it was determined that the best empirical predictors of corrosion susceptibility were water temperature and dissolved oxygen (these two are the most important), and the chloride sulfate and magnesium contents of the mine water. In these studies, it was also found that the HSLA steel used for manufacturing split sets is susceptible to pitting corrosion attack.

Avery (1989) studied the performance of polyester resin grouted rock bolts installed under wet conditions. Based on the results of laboratory tests, using concrete blocks to simulate the rock medium, it was concluded that the presence of water affects the resin grout by mixing with the top 300 to 350 mm to form an emulsion.

Albritton (1994) reported on the use of rock bolts in limestone and weak shale formations. During preconstruction rock bolt testing, sufficient anchorage to reach the required level of prestress was difficult to obtain in the weak shale formation. Installation and drilling techniques were identified as the source of the problem in the case of grouted anchors, and insufficient strength of the shale in the case of mechanical anchorages. Later, polyester resin encapsulated anchors were employed. This system performed well and met the design requirements. The test program provides a large amount of useful data, which revealed the system to be simple, economical, versatile, and effective.

Lokse (1992) presented a survey of research on the long-term performance of rock bolts. Also included is a summary on the type of deterioration to which rock bolts may be exposed as well as what precautions may be taken to avoid corrosion. The condition of 15-year old rock bolts installed in a special underground test room at the University of Minnesota Underground Space Laboratory was investigated. Two resin grouted rock bolts were exhumed by overcoring and examined in the laboratory. The study concluded that the passage of time had not had any profound effects on the quality of the rock bolt installation. The quality of work done during installation appeared to be satisfactory, and the resin had protected the bolts from corrosion and had not itself begun to deteriorate.

From the experiences at the Upper Occoquan Dam, Bruen et al. (1996) observed creep behavior for epoxy filled strands is greater by about an order of magnitude than bare wire strands. This excessive creep movement was attributed to strand deformation due to the presence of epoxy filling, which may initially prevent wire-to-wire contact, allowing for more elongation.

Stimpon (1998) studied the effect of corrosion on the interface shear strength along the anchorage zone of split set friction stabilizers. Increases in pullout resistance with time was attributed to increased confinement and/or frictional resistance due to corrosion. Although substantial improvements in strength from this mechanism may be counted on over the short term, it is anticipated that over longer periods of time the strength capacity will decline as the amount of corrosion increases. Split-set stabilizers are particularly susceptible to corrosion because of the longer surface area, the thin wall of the tube, and exposure to ground water and/or air.

2.4.5 Ground Anchors

Nurnberger (1980) studied data from a total of 242 failures of prestressed steel elements, reported between 1950 to 1979 mainly in connection with prestressed bridges, tanks, pipes, and constructive elements of bridges. Of these failures, nine dealt with ground anchors. The majority of failures or breaks exhibited a heavy pitting corrosion on the prestressed steel surface. Localized high stresses and bending, not accounted for in the design, played a significant role in many of the brittle failures observed. Failures occurred six times in strand elements which were made out of special alloy steel. In three cases, failures occurred in hot rolled steel bars made out of perlite steel. There were as many temporary tie-back failures as there were permanent ones. In the temporary case, often no corrosion protection was applied to the anchor assembly. In the case of permanent installations, the protection was usually insufficient and not durable. In all cases of failure, tendons were installed within aggressive environments. Three of the cases studied involved failure of strands at or near the anchor head; strand failures occurred along the free length in eight of the cases.

Based on interviews conducted during a tour of Europe, Weatherby (1980) described early experiences with ground anchors in Germany, Switzerland, France, and England. Problems related to corrosion of heat-treated steel were discussed. Studies of the condition of exhumed anchors were made and showed two very different crack patterns, longitudinal for strands and perpendicular to the axis for deformed bars.

Portier (1974) reported on failure of several 12 MN capacity tendons installed at the Joux Dam (France). Failure occurred within several months of installation and was attributed to stress crack corrosion.

Feld and White (1974) observed corrosion activity along some of the tendons during construction of the World Trade Center in New York City. The cause was determined to be aggressive groundwater conditions and the formation of sulphuric acid. A cathodic protection system was installed to increase the service life of the tendons.

Littlejohn (1992) summarized reported incidents of ground anchorage failures caused by corrosion compiled by the Federation Internationale de la Precontrainte (FIP, 1986). This summary presents 35 case histories comprising 24 failures of permanent installations. Based on the results of reported incidents compared to the number of anchors installed during the 31-year time frame covered by the survey, the incidents of prestressed ground anchor failures by corrosion are limited and generally random, with the possible exception of steel type.

Some of the relevant findings developed from this survey include:

- *Relevance of tendon type and location*: Corrosion is localized and no tendon type is immune to corrosion. Nine incidents involved bar, 19 involved wire and 7 involved strand anchorages. A total of 19 incidents occurred within 1 meter of the anchor head and 21 incidents involved the free (i.e., unbonded) length. The case histories also confirm that quenched and tempered plain carbon steels and high-strength alloy steels are more susceptible to hydrogen embrittlement than other steel types.

- *Corrosion time*: The time to failure ranged from several weeks to 31 years. Of the incidents that occurred within 6 months, 4 were permanent anchorages with some or full corrosion protection. Two incidents of permanent anchorage failure occurred within 6 to 18 months after installation. However, the majority of the incidents (i.e., 21) occurred after 18 months.

- *Fixed length*: The two incidents involving failure of the fixed length were caused by inadequate grouting in the bond zone.

- *Free length*: Failure of the anchor in the free length was caused by tendon overstressing leading to corrosion pitting or corrosion fatigue, absence of grout in aggressive ground, disruption of bitumen cover due to lack of elasticity, improper selection of protective materials, poor on-site storage conditions prior to construction, and inadequate attention to application of protection system and details during construction.

- *Anchor head*: Anchor head failure was caused by either a lack of protection or inadequate protection. For the 19 incidents occurring within 18 months after installation, inadequate protection of the anchor head was always indicated.

Parry-Davies and Knottenbelt (1997) reported on the performance of ground anchors installed at 8 sites along the South African National Road Network. Dates of construction at the sites range from 1976 to 1989 and the performance evaluation was conducted from 1992 to 1995. Tendon elements were made from 15 mm diameter, seven-wire strand corresponding to a minimum ultimate tensile strength 232 kN. Observations include visual inspection and lift-off tests. During the 1970's and early 80's, it was not generally recognized how important a study of the environmental conditions were, and this was reflected in observations reported by the authors. Some corrosion was observed due to aggressive ground conditions, stray current corrosion, and ineffective final grouting due to grout loss in jointed rock, or poor attention to detail during construction. On some projects, bare strand was observed behind the anchor heads for a distance approximately equal to the elongation from prestressing the anchors.

Briaud (1998) examined anchors that were exhumed from structures along the I-90 in Mercer Island, Washington, along the I-820 in Fort Worth Texas, and anchors that were installed and left in clay backfill for five years at the Texas A&M University National Geotechnical Experimental Site (NGES).

- In Mercer Island, anchor tendons were required to have double corrosion protection. No significant corrosion was observed on any anchor tendons. Of the 569 anchor tendons evaluated, no corrosion was observed in 560, and nine exhibited evidence of mild corrosion.

- Bar tendons were installed as tie-backs along the I-820 Glenview/Pipeline, Ft. Worth, Texas. These anchors were installed in 1983. One bar anchor was inspected and no significant corrosion was observed.

- At the Texas A&M NGES tendons were exposed to elements of nature for approximately five years. As long as the grease was in good condition, inspection of the greased and sheathed section revealed no evidence of corrosion. At some tendons the protective grease coating dried out 70 to 80 mm into the sheathing. A 100 mm section exposed to air showed clear evidence of corrosion on exterior surfaces. Uniform or general corrosion was observed to be most prevalent with possible signs of pitting.

Jones (1997) reported on corrosion of strand anchors in Kuwait due to sulfate reducing bacteria (SRB). In peeling back the PE sheathing from corroded strands a faint smell of hydrogen sulfide was noticed. By testing the visibly exposed area with hydrochloric acid, hydrogen sulfide was apparent. An SRB reaction is a reduction process. The bacteria need a free carbon source and sulfate ions to convert to sulfides. Thus, the protective grease that is applied to tendon strands may be a source of free carbon, and, in an unfavorable set of circumstances, groundwater may be a source of sulfates.

Schupack (1999) observed problems with microbiological grease degradation, leading to microbacteriological-induced corrosion (MIC) of prestressing strands. Single strand, unbonded tendons in a push through type sheath suffered severe strand corrosion, when installed in a prestress concrete structure. A bacteriological study was made of grease samples obtained from different strands under the most sterile conditions possible. Both bacteria and fungi were found in abundance. The grease degradation resulted in the formation of an organic acid (pH as low as 3) and atomic hydrogen. The cause of embrittlement of some of the strand was attributed to the organic acid.

If water is present in monostrand type tie-backs, the protective coating of grease may, eventually, undergo bacteriological degradation with associated byproducts including sulfur and organic acids. Based on this, it appears that tie-backs which employ an unbonded grease protected corrosion system for the stressing length have a particular long-term possibility of corrosion. This would be particularly true of some of the tendon “push-through” type single strand elements that have been used in many tie-backs. To determine if MIC is occurring in tie-backs it may be possible to sample the grease through the head to determine if there is bacteriological degradation.

2.5 NDT METHODS

Corrosion of a metal-tensioned element is difficult to detect from a visual inspection at the end of the element. For a strand, the worst corrosion is nearly always within the strand itself, where moisture gets in and around the central core and the central or king wire (Barley, 1997). Nondestructive test (NDT) techniques can potentially provide information about conditions along the length of a tendon.

NDT techniques are based upon principles of wave mechanics, electro-magnetic, or electro-chemical phenomena. Data generated from NDT are processed such that the properties of materials, or condition of structural elements, are inferred. Based on a search of the literature, a number of NDT techniques were identified that have been applied to configurations having features in common with elements of metal-tensioned systems. The techniques reviewed include: (1) impact echo, (2) impulse response, (3) parallel seismic, (4) acoustic emission, (5) ultrasonic, (6) ground penetrating radar, (7) reflective impulse measurement technique, (8) time domain reflectometry, (9) magnetic flux leakage, (10) fiber optic corrosion sensing, (11) electrochemical tests including half cell potential, polarization resistance, electrochemical impedance spectrometry, and contact resistance and (12) the corrosion potential method. Salient details of these tests are described in the following sections.

2.5.1 Impact Echo

Impact echo testing involves introducing a transient stress pulse into a test object by mechanical impact on the surface. The resulting stress waves propagate along the surface and penetrate into the material being tested. Sensitive displacement transducers record the arrival of compression (P) waves that penetrate the material, and are reflected by the boundary of the object, or by anomalies such as cracks and voids that may be located within the test object. The data are transformed to the frequency domain for evaluation. Equipment, and necessary software, for performing the impact echo test is commercially available.

Sansalone and Carino (1987) describe the use of the impact-echo method for nondestructive testing of concrete to locate honey combed concrete and an ungrouted metal duct embedded within the concrete. The results of experimental and finite element simulations of the test geometry and impact loading, are presented. Cheng and Sansalone (1993) described the effects caused by steel reinforcing bars on impact echo signal obtained from reinforced concrete structures. Lin and Sansalone (1994(a) and (b)) demonstrate the use of the impact echo technique for integrity testing of concrete pipe, mine shaft liners, and tunnel liners in contact with soil or rock.

Ghorbanoor (1993) reported on the use of the impact echo method to detect the presence of voids in the ducts of post-tensioned bridge structures.

2.5.2 Impulse Response

In the impulse response test the surface of an object is impacted. However, compared to the impact echo test the impact has a higher energy and a correspondingly lower frequency content. A velocity transducer is used to observe the characteristics of the reflected P-wave. Data for both the hammer force and velocity transducer are recorded in real time.

Finno and Prommer (1994) studied use of the impulse response test for evaluating the integrity of drilled shaft foundations. For this test the head of the pile shaft is impacted with a hammer. The shaft response and impact force are measured in the time domain, the signal is converted to the frequency domain and the velocity spectrum is divided by the force spectrum. These test responses can be analyzed in several ways to evaluate the length, dynamic stiffness, shaft impedance, and theoretical mass of the pile. Finno and Prommer (1994) applied four analyses as follows:

- Measure time taken for the stress wave to travel to the pile tip and back,
- Compute the pile mobility spectrum from the transfer function of the velocity and force readings,
- Calculate velocity reflectors from the Fast Fourier Transform (FFT) of velocity vs. time,
- Calculate the pile profile by the Impedance Log method.

Chen et al (1994) described the use of a dynamic characterization method applied to evaluate prestressed bars in timber bridge construction. In many respects, Chen’s method is very similar to the impulse response method. Prestressed bars are excited by an impact hammer in the transverse direction and the natural frequencies of vibration are related to their tension levels.

Dosch and Jing (1999) described the use of a Hopkinson Bar to calibrate high-impact accelerometers. The end of a long thin bar is impacted with a projectile to create an acceleration transient. The middle of the bar is instrumented with a strain gauge and an accelerometer is placed at the far end of the bar. Data is analyzed in the frequency domain and the power spectrum of reflected wave is studied.

2.5.3 Parallel Seismic

The parallel seismic test is a mechanical wave propagation technique that was developed for situations where the integrity and length of a shaft needs to be determined after the shaft head is no longer accessible. In principle, the test is similar to the impulse response method, described in Section 2.5.2. What makes the parallel seismic test unique is that the vibration response is not monitored on the shaft itself, but within the surrounding medium. A borehole is advanced adjacent to, and slightly longer than, the shaft being tested. Stress wave energy is generated on the surface of the shaft by impacting it with a hammer and the arrival time of the compression wave is monitored in the borehole by means of a hydrophone transducer.

The test was originally developed in France to evaluate the conditions of piles and drilled shafts under existing structures (Davis and Hertlein, 1993). Finno and Prommer (1994) described testing requirements, interpretation of results, limitations of the test and case histories of the method for application to drilled shafts.

2.5.4 Acoustic Emission

Stress waves that originate from fracturing of material, gas evolution and film cracking are detected by sensors placed at strategic locations along an element or structure. Typical equipment consists of an array of piezoelectric accelerometers, a pretrigger device, and a data acquisition system. Signals are filtered to eliminate background noise and subjected to signature analysis to identify fracturing of the element. Computer software, incorporating an artificial learning network, analyzes the recorded signals. By analyzing data collected from sensors at several positions, the time and location of fractures can be determined.

The mining industry is an early application where acoustic monitoring is used to detect small subaudible noises generated in the development of rock instability (CANMET, 1984; Hardy and Ge, 1986). Acoustic monitoring equipment is used to detect the acoustic emissions, and to predict times of rock bursting or collapse. Seismic sources are located in space by an array of geophones distributed around the slope or underground excavation (Hardy, 1981). A count of the number of events per interval of time has been used successfully as an index to the dynamic activity of the rock mass.

Acoustic emission monitoring was performed on reinforced concrete specimens in the lab by using the steel rebar as the wave-guide (Zdunek et al., 1995). Piezoelectric sensors, used as A/E transducers, are placed on each end of a steel rebar embedded in a concrete specimen. With this technique, acoustic emission events were shown to detect film cracking, gas evolution, and micro-cracking which are correlated with the onset of rebar corrosion.

Chen et al. (1992) investigated fiber reinforced plastic (FRP) bars and FRP reinforced concrete elements with the acoustic emission technique. One of the conclusions of this research was that the AE technique is useful in detecting the time and stress intensity of debonding of FRP bars from concrete. This result may be useful if it can be applied with respect to the bond zone of rock bolts and ground anchors.

Continuous Acoustic Monitoring (CAM)

Continuous acoustic monitoring (CAM) has is used for log-term monitoring and can provide data for assessing probable long-term prestressing steel element failures. The acoustic monitoring system will detect new prestressing steel element failures, but provides no information on past or ongoing corrosion. The continuous acoustic monitoring technique has been applied to post-tensioned concrete structures including buildings and bridges. Elliot (1996) described examples of corrosion in prestress tendon strands, and discussed the need for evaluating the condition. The acoustic emission monitoring system is described, and details of a case study are given.

Paulson and Cullington (1998), and Paulson (1999), evaluated acoustic continuous monitoring as a means of detecting flaws in grouted post-tensioned members and suspension bridges. Paulson (1999) reported on acoustic monitoring, which began in October 1997, of the Bronx-Whitestone suspension bridge in New York City. The system is used to detect corrosion-induced wire failures in the main cables. As the cable was undergoing a major rehabilitation during the test, the wire wrapping was removed. This presented the opportunity to generate wire failures and other events to test the system's capability to detect and locate events and discriminate between wire breaks and other events. As is the case with post-tensioned structures, the failure of a tensioned suspension cable wire was found to produce a significant amount of energy.

Paulson and Cullington (1998) report on a study being conducted in cooperation with the United Kingdom's Transportation Research Laboratory (TRL). For the evaluation, six wires of the main cable of a suspension bridge were deliberately cut. Also, wire fractures in fully grouted, partially grouted and ungrouted tendons were produced by cutting wires either mechanically using a small grinder, or by accelerated corrosion. For the ungrouted and partially grouted cases, wire fractures induced by corrosion are easily detected. Results for the fully grouted case are less reliable. A long-term monitoring exercise on a post-tensioned bridge in the U.K. is currently in progress.

Pernica and Rahman (1998) evaluated the effectiveness of a commercially available acoustic continuous monitoring system for post-tensioned buildings. The evaluation included cutting tendons with a quiet, high-speed cutting tool. The acoustic monitoring system was considered effective in reporting the time and location of tendon ruptures. However, the monitoring systems need to be further developed to incorporate the capability of reporting the number of wires rupturing during an acoustic event; and to improve the reliability of locating where wire ruptures occur.

2.5.5 Ultrasonic Test

Similar to the impact echo and impulse response tests, the ultrasonic test is a wave propagation technique. However, for the ultrasonic test, sound waves in the ultrasonic frequency range are propagated through the material being tested. Sound waves are generated with a piezoelectric crystal that must be acoustically coupled to the test object. Often the test is performed in the pitch-catch mode where the source is at one end and the signal is received by an accelerometer placed at the far end of the object. However, the test may also be performed using the same transducer as both the source and receiver. In this configuration the arrival time and frequency content of the reflected wave is observed.

Equipment required for ultrasonic testing includes the transducer(s), a pulse generator, a receiver, and an oscilloscope or high-speed data acquisition system. The transducer includes a piezoelectric crystal with internal microelectronics, including a low pass filter and preamplifier. The pulse generator provides excitation to the crystal causing it to oscillate, and controls the amplitude and damping of the oscillations. The receiver has the ability to condition the received signal with low or high pass filters, selectable gain and phase controls. Data may be limited to arrival times for direct or reflected wave. Advanced signal processing requires more information including wave amplitude, or the time history of acceleration of the received signal.

Schupack and O'Neil (1997) described the use of ultrasonic testing to monitor the condition of post-tensioned concrete beams. The beams were part of a test program to study the durability of post-tensioned concrete exposed to a harsh marine environment. Measurements were taken by transmitting an ultrasonic pulse through the beam along its axis (longitudinally), and through the thin web section (transversely). Test data were used to study changes in the compression wave velocity of the concrete. The observed longitudinal pulse velocity values appear to be reasonably consistent and indicate some deterioration of the concrete in the beams occurred throughout 33 years of exposure.

In Sweden, an ultrasonic test device referred to as the "Boltometer" was developed to detect flaws in rock bolt installations (Thurner, 1983; Jeremic, 1987). The Boltometer is used to reveal poor quality or insufficient grouting. A transducer, which serves as both source and receiver, is pressed against the free planar end surface of a rock bolt. The end of the bolt is cut and/or ground planar, and an acoustic couplant is applied. Sound waves, transmitted from the sensor, propagate along the bolt. If the condition of the grout is good, the impedance of the surrounding grout and the rock bolt are matched such that, as the wave propagates, energy is dispersed. Wave dispersion has an effect on the amplitude of the propagating waves. If the bolt is surrounded by good quality grout, the observed amplitude of the reflected wave is less than if the grout is deficient or lacking. Thus, the amplitude of the reflected wave is a useful index of the grout condition.

Fish (1999) described use of ultrasonic testing for evaluating anchor bolts for sign bridges and light poles. The test method requires that the signature of reflected waves be studied for bolts with defects of various size and location. Based on the observed travel time and amplitude of the reflected wave, the location and severity of defects can be determined.

Niles (1995) described use of ultrasonic test equipment to measure and analyze section loss due to corrosion of anchor bars guying steel transmission poles. The anchor guy system tested consisted of a steel plate embedded in concrete connected to a steel bar buried in earth and attached to the guy wire from the pole. The test method employs the cylindrically guided wave technique. Field correlation of the method was verified by digging up and inspecting several bars.

Bruce et al (1988) performed inspection of a cable stayed bridge using acoustic pulse transmission through the steel cables. Measurements were from both ends of the cable, however measurements taken from one end of the cable may be possible. The effects of cracks in the grout on the signals remain to be studied.

Crowther (1992) described the essential findings of NCHRP Project 10-30, "Nondestructive Methods for Field Inspection of Embedded or Encased High-Strength Steel Rods and Cables." Ultrasonic testing was used to detect serious deterioration of high strength steel tendons or bars embedded in concrete. An important application is post-tensioned, prestressed concrete structures, with seven wire strand steel tendons inside metal ducts.

Chen and He (1992) demonstrated that the ultrasonic test method can be a useful NDT technique to predict the stress level of a prestressed bar. Both theoretical and experimental results indicated that ultrasonic waves traveling within a slender bar are dispersive in nature. Ultrasonic wave propagation in a circular bar, which is usually called a wave-guide, is a complex problem since the wave is propagating with group velocity. A simplified method of analysis was introduced by considering the elastic wave-guide theory and the wave speed change at different stress levels using Murnaghan's third-order elastic constants.

2.5.6 GPR

Radar propagation in nonmetallic solids is analogous to the acoustic sounding techniques except radar utilizes an electromagnetic energy source (radio waves) rather than a mechanical source. Echoes and reflections originating at the interface of differing materials (i.e., materials with differing dielectric or conductivity properties) are analyzed. Ground penetrating radar (GPR) is currently employed to assess the condition of bridge decks and is useful for locating voids, steel reinforcement, and corrosion-induced delamination.

Maser (1996) and Morey (1998) describe GPR technology as it applies to the evaluation of pavements, bridge decks, abutments, piers, geotechnical evaluations, and railroad track structures. A new technique, synthetic aperture radar (SAR), currently under development is described by Maser (1996). Synthetic Aperture Radar may be useful for obtaining 3-D images of buried or embedded objects.

2.5.7 Reflective Impulse Measurement Technique (RIMT)

RIMT is an electromagnetic wave propagation technique that uses a high frequency echo to locate anomalies such as corrosion, breakage of strands or wires, and defects along the length of a tendon. Signals are transmitted and received from the same end of the element. The time of arrival, duration, amplitude, shape, rise and fall times of the reflection are used to calculate the location and size of a defect. Applications include unbonded post-tensioned tendons, and the test may be useful for detecting the presence of corrosion pits, cracks and breaks.

RIMT measurements are affected by environmental factors such as water in the tendon sheath, nearby electrical fields and electrical contact with other tendons and metals. Pernica and Rahman (1997) evaluated use of RIMT for condition assessment of unbonded post-tensioned tendons. The technique, as tested, is not recommended as a reliable procedure for detecting defects and estimating severity. Based on the results of their evaluation, Pernica and Rahman (1997) recommended improving the technique and focusing efforts on estimating severity, but not necessarily the location of defects.

2.5.8 Time Domain Reflectometry

Time domain reflectometry (TDR) is an electromagnetic wave propagation technique used to evaluate the integrity of a transmission line that functions as a wave guide. A step generator and oscilloscope are connected to the transmission line. The step generator produces a step voltage which propagates down the line. Discontinuities in the line, such as those associated with corrosion sites, will produce returned reflections, separated in time, which can then be seen on an oscilloscope display. If the pulse returned is different in height, this gives information about the nature of the discontinuity.

TDR is a technique that has been used by the electric power industry to evaluate the integrity of electrical transmission lines. TDR may be used to detect, locate and identify the extent of corrosion. Physical defects such as abrupt pitting corrosion, general surface corrosion, and voids in grout surrounding the element will change the electromagnetic properties of the line. These defects, modeled as different kinds of discontinuities, can be detected by TDR.

Bhatia et al. (1998) applied TDR to evaluate cables used in the construction of cable stayed and suspension bridges. The transmission line model is applied to bridge cables and the electrical parameters of interest are determined by analyzing the asymmetric two-cable transmission line. The returned signal gives information about the cable. Models are developed for various types of commonly encountered cable defects including distributed surface corrosion, abrupt pitting corrosion and grouting voids. The models show what types of waveforms can be observed if these defects are present.

In modern bridge cables, a silver monitoring wire running parallel to the cable is present. This assembly represents the classic twin conductor geometry. In general, this may not be the case for ground anchors. However, the approach may be a useful monitoring technique for new anchor installations.

2.5.9 Magnetic Flux Leakage

Magnetic Flux Leakage is a magnetic based NDT concept that utilizes the ferromagnetic property of metal to detect perturbances of an externally applied magnetic field due to the presence of flaws. Necessary equipment includes an electro-magnet and a sensor array. The electro-magnet applies a magnetic field near the metal element causing it to become magnetized. Any change in the cross sectional area of the element will cause a flux leakage. Detection devices (i.e. Hall probes) are used to measure and record the perturbance of the magnetic field as an electrical charge. If the magnetic source and sensors are moved along the length of the element, the signature is then recorded as a continuous signal that can be analyzed to obtain information relevant to the location and extent of the flaw.

Ghorbanpoor and Shi (1995) studied use of the magnetic flux leakage test to assess the condition of steel in concrete structures. The technique is applied to reinforcing bars and prestressing steel cables embedded in laboratory specimens. Relative to other NDT, the technique proves to be very sensitive. Loss of cross sectional area in bars and cables of approximately 3 percent are detected.

Kusenburger and Barton (1981) described magnetic inspection equipment for detecting deterioration in the reinforcement of prestressed concrete bridges. Use of promising electronic signature enhancement and recognition methods for discriminating between steel artifacts and deterioration was investigated.

2.5.10 Fiber Optic Corrosion Sensing

Fuhr et al. (1995) described a fiber optic corrosion sensing system where optical fibers were used as light conduits, or transmission pathways, for the wave-length modulated signal. The color-shifting of incident broad-band light due to reflection from a corroded surface is transmitted back to a detection position through an optical fiber after being illuminated by a separate optical fiber. The system utilizes fiber optic chemical sensing, and reflected signals are observed by direct chemical spectroscopy. It is feasible to use a single optical fiber both as a transmit and a windowed receiver. Time-resolved analysis indicates the amount of corrosion occurring within each window segment of the fiber.

2.5.11 Electrochemical Measurement Techniques

Electrochemical measurement techniques may be used to monitor the presence, and/or rate of corrosion. The techniques take advantage of the fact that corrosion is an electrochemical process that requires a potential difference between a cathode and an anode, in the presence of an electrolyte. Electrochemical measurements include electrical resistance, half-cell measurements, polarization resistance, electromagnetic impedance spectroscopy, and electrochemical noise. Results from electrochemical tests may be useful for indicating if the process of corrosion is currently active, and possibly at what rate. However, these methods can not be used to indicate the amount of corrosion that has already taken place. Searbrook and Hanson, (1996) described half-cell potential mapping, linear polarization resistance measurements and the electrochemical noise technique, and their use in field measurements. Fantana (1986) described theory and application of corrosion measurement techniques including potential measurements, polarization resistance, and AC impedance methods.

Electrical Resistance Measurements

Measurements of electrical resistance are useful to see if the resistance of the electrolyte is low enough to facilitate corrosion, and to determine whether there is continuity such that a complete corrosion circuit can develop. Measurement of soil resistance is often used to assess the aggressiveness of the environment to corrosion as described in Section 2.3.2. This section describes resistance measurements between the metal element and the soil or rock. These measurements are useful to monitor the condition of the corrosion protection system. For an intact corrosion protection system, the observed resistance will be relatively high. Low resistance readings indicate that the corrosion protection system may be compromised.

Schneiter et al (1994) and Fischli (1997) described the use of the electric resistance method (ERM) to check the integrity of an installed corrosion protection system. The ERM is applied to strand elements that are electrically isolated with a non-conducting, watertight encapsulation such as is provided by a polyethylene sheath. During installation, the resistance between the tendon strand and the ground is measured after grouting the unstressed anchor. If the measured resistance is high, the encapsulation is presumed to be intact. A low measured resistance (below 0.1 M Ohm) indicates a problem. The ERM is so sensitive that even the smallest leaks, humid PE sheathing joints, or connections will not escape detection. If a problem is detected, an isolation plate is placed between the bearing plate and the wall face.

To check the effectiveness of the isolation plate, resistance is measured between the anchor plate and the wall face, which serves as an earth ground. The minimum resistance required at the anchor head is 100 Ohm. The ERM test technique is recommended by the Swiss Society of Architects and Engineers (1995). A maximum failure rate of 10 percent is allowed under the condition that insufficient anchorages are more or less evenly distributed over the test area of the wall.

Half Cell Potential Measurements

The electric potential of corroding metals is more negative compared to non-corroding metals. Therefore, potential measurements provide a useful index that can be correlated with the occurrence of corrosion. Potential measurements indicate if corrosion is occurring, but cannot be used to indicate the severity or rate of the corrosion process.

The half-cell measurement test is used to evaluate the potential of a metal element. The metal element being tested serves as one electrode and voltage measurements are made with reference to another standard, stable electrode. Equipment required for the test includes a high impedance voltmeter, a reference electrode, and a set of lead wires. A copper/copper-sulfate half-cell is often used as the reference electrode. Part of the element being tested must be accessible, and the common lead from the voltmeter is connected to the element. The half-cell is then the positive lead. To complete the circuit, an electrolyte is needed between the half-cell and the element being tested.

Half-cell measurements are used to measure the corrosion potential of rebar using a grid pattern over the concrete surface (ASTM C876). The basis for mapping half-cell measurements is that the corrosion potential of the steel rebar will shift in the negative direction if the surface changes from the passive (noncorroding) to the active (corroding) state.

Lokse (1992) described the use of potential measurements to compare the corrosion potential of rock bolts installed in an underground test chamber. The technique was applied to an array of rock bolts, whereby one of the bolts served as the reference electrode. Potential measurements from all the bolts in the array were compared, and bolts with higher observed electric potential were identified as potential corrosion problems. A corroding rock bolt is expected to show a high potential with respect to another, non-corroding, bolt. For the measurements to be reliable there needs to be a sufficient degree of electrical contact between the different rock bolts. As the rock itself is a poor conductor, this connection must rely mostly on the water present within fissures in the rock.

Polarization Resistance

The rate of an electrochemical reaction is limited by various physical and chemical factors. Hence, an electrochemical reaction is polarized, or retarded, by these environmental factors. Polarization potential results from reactions that take place at the electrode.

Under natural equilibrium corrosion conditions, the number of electrons released by the anodic reaction is exactly equal to that consumed by the cathodic reactions, therefore, there is no net current that can be measured. To determine the corrosion current, the system must be biased away from equilibrium and the resulting net current is then measured.

If the conductor is an electrolyte, the passage of direct current will cause polarization and the establishment of a potential at the electrode that opposes the applied potential. As described by ASTM G-59, polarization resistance, and polarization potential, are evaluated from the current corresponding to an applied potential. The metal element being tested serves as the working electrode. Although the technique has been used in electrochemical laboratories for decades, equipment and methods for field application are relatively recent developments (Scannell et al, 1996). The test requires a reference electrode, a counter electrode, potentiostat, a high impedance voltmeter, and a low resistance ammeter.

The potentiostat is the electronic instrument that carries out the electrochemical polarization experiment. It comprises a power supply for impressed current on the electrochemical cell and circuits that measure and control the potential to set values. A high impedance voltmeter measures the potential between the working electrode and the reference electrode without affecting the potential of the working electrode. A low resistance ammeter measures the current flow between the counter and working electrodes, without affecting current flow.

AUCSC (1995) describes a test method for observing the E vs. log I (potential vs. current) relationship for a buried metal element/soil system. The characteristics of the E vs. Log I relationship may be related to the condition of the existing corrosion protection system.

Alternatively, the polarization resistance (PR) can be determined from the corresponding measurements of applied voltage and corresponding current as:

$$R_p = (E_a - E_p)/ I; \text{ when } (E_a - E_p) \approx 0.$$

Here,

I = current corresponding to E_a (amp)
E_a = applied potential in (volt)
E_p = polarization potential in Volts (back emf)
R_p = polarization resistance (ohm).

From the polarization resistance, the corrosion current density can be calculated as

$$i_{corr} = B/R_p$$

where, B is a constant that depends on the material type being tested. Using Faraday's Law, corrosion current density can be related to corrosion rate as described in ASTM G-102. Calculation of corrosion rate depends on knowledge of the area of material being corroded. Often this must be estimated, such that the rate of corrosion is an average rate, acting over an assumed area.

Elias (1990; 1997) describes polarization resistance measurements applied to soil reinforcing elements used in the construction of reinforced earth retaining walls. Instrumentation to produce an E versus I plot at $\pm$ 20 millivolts of the free corrosion potential is required to measure the polarization resistance. Instrumented sections must be isolated from the wall, and plastic stand pipes are used for potential measurements on deep instrumented reinforced members. Corrosion measurements are practical, only at shallow depths for existing structures. Corrosion rates obtained from the measurements yield average corrosion rates for the whole reinforcing length being measured.

Soil resistance may have a significant effect on the measurement of polarization resistance. This effect may be neglected if considerable experience has been gained on actual structures to show it to be justified. Alternatively, the soil resistance can be measured with a soil resistance device and subtracted form the measured polarization resistance. Fully automatic equipment for making polarization resistance measurements, soil resistance measurements and integrating all data are commercially available.

Lawson et al.(1993) describes methods for measuring polarization resistance of steel reinforcing strips used in MSE walls using the Model 4500 PR Monitor manufactured by Cortest Inc. The method accounts for the resistance of the aqueous soil, R_s when measuring $R_{p,}$ similar to that described by Elias (1990).

Electromagnetic Impedance Spectroscopy (EIS)

Electromagnetic Impedance Spectroscopy, or AC Impedance methods measure corrosion resistance, R_{corr}. The electrical connections are similar to that for the polarization resistance (PR) measurement. Rather than direct current, as in the PR measurement, a sinusoidal voltage perturbation about the free corrosion potential is applied between the working and counter electrodes. The perturbation is of sufficiently small amplitude that the response of the system is a linear function of the amplitude of the input function. The test measures the impedance, or admittance spectra, of the corrosion system, whereby the transfer function is determined as a function of frequency (Nyquist Diagram). Analysis of data obtained with AC impedance methods requires analysis of an electrically equivalent circuit. It is generally possible to deconvolve R_{corr} from the remainder of the equivalent circuit by taking account of the frequency ranges in which the various terms contribute most to the measured impedance, and mathematically extracting impedance elements from the measured test data. In this manner, the contribution from solution or soil resistance can be separated from the measurement of polarization resistance. The advantages of the AC impedance method are principally the precision with which a small sine-wave perturbation can be measured in an electrically noisy environment.

Elias (1990) used an AC Impedance technique to observe soil resistance during measurement of polarization resistance of metal reinforcements used in the construction of reinforced earth retaining walls. At the end of the polarization resistance measurement cycle, an AC signal is applied between the working and counter electrodes. A high frequency signal permits measurement of soil resistance, independent of polarization resistance.

Briaud et al (1998) performed EIS tests in the laboratory on samples consisting of a steel plate mounted at the bottom of a cylindrical mold. The mold was filled with clay or grout and the steel plate was either bare, epoxy, acrylic, or fusion-bonded epoxy coated. With this test arrangement, the effect of grout cracking and grout thickness on the measured corrosion rate was studied. They proposed a test protocol for screening ground anchor systems that evaluates the potential for anchor corrosion at a given site. The proposed protocol includes the EIS test with flat steel specimens and soil from site. The use of round specimens would be possible with a modification of the test set-up.

Andrade et al. (1986) and Monteiro et al (1998) applied polarization resistance measurements and AC Impedance methods to determine the corrosion rate of steel embedded in concrete.

Electrochemical Noise Technique

Unlike the other electrochemical techniques, electrochemical noise measurements do not rely on any "artificial" signal imposed on the element. Rather, natural fluctuations in the corrosion potential and current are measured to characterize the severity and type of corrosive attack. A three electrode system is required for simulating potential and current noise mass. Noise data can be particularly useful for identifying the initiation and propagation of corrosion pits.

Searbrook and Hanson (1996) described how three nominally identical rebar probe elements can be embedded in the concrete for electrochemical noise measurement. Extremely sensitive instrumentation is required with minimum current and potential resolution levels around 0.1 mV and 0.1μA, respectively.

Olek and Ferrel (1998) describe testing which combines electrochemical noise with electromagnetic impedance spectroscopy. They present an example of data analysis using Fast Fourier Transform (FFT) and the Nyquist plot.

Contact Resistance

The contact resistance gives information on the structure of the interface between different materials. Contact resistance measurements have been used to assess the surface condition of steel reinforcement embedded in concrete (Fu and Chung, 1995). Since oxidation product is a poor electrical conductor, it is assumed that contact resistivity is related to the amount of oxidation product at the rebar-concrete interface.

Fu and Chung (1995, 1996 and 1997) applied contact resistance measurements to specially prepared reinforced concrete specimens in the laboratory. They used the four-probe method similar that that described by ASTM G57, and silver paint as electrical contacts. Two contacts, one for applied current and one for voltage measurement, were placed around the circumference of the steel rebar. The other voltage and current contacts were on the concrete embedding the rebar, such that each of these contacts was around the whole perimeter of the concrete in a plane perpendicular to the rebar. The resistance measured between the two voltage probes corresponds to the sum of the rebar volume resistance (the resistance down the length of the rebar), the steel concrete contact resistance (the resistance across the interface), and the concrete volume resistance (the resistance radially outward from the interface to the vertical sides of the concrete). In comparison to the contact resistance and the concrete volume resistance, the rebar volume resistance is negligible. Thus, the volume resistance of the concrete, obtained from a separate measurement, is subtracted from the measured resistance in order to obtain the contact resistance.

2.5.12 Corrosion Potential Method

The corrosion potential method is applicable for measuring the probable extent of corrosion for unbonded tendons in concrete members. The protocol for the corrosion potential method involves measuring the moisture content of air within the tendon sheaths. The method involves drilling two small holes into the tendon sheath, one at each end and passing dry air at low pressure into the tendon sheath. Using a calibrated relative humidity and temperature (RH&T) sensor the relative humidity and temperature of the escaping air is measured. Corrosion is estimated based on a relationship between moisture content and state of corrosion.

Pernica and Rahman (1998) evaluated the use of the corrosion potential and concluded that the method is a practical technique for estimating the probable extent of corrosion of unbonded tendons provided representative tendons are selected for the correlation.

2.5.13 Summary of NDT

Table 2-6 is a summary of the test methods that could potentially be implemented for metal-tensioned systems. The table identifies each method, describes previous research or application, indicates the level of expense and training required for operation of the equipment, and the relative ease by which the technique may be implemented to metal tensioned systems (MTS).

Based on the information collected in the literature search, the impact echo, ultrasonic test, half cell potential, polarization measurements, contact resistance and radio wave reflection technique (similar to GPR) are identified as tests which have the potential for successful implementation for condition assessment of rock bolts, soil nails or ground anchors. These test methods were further evaluated using test facilities at the University at Buffalo (U.B.). Details of the evaluation are described in Section 3.

Table 2-6
Summary NDT Methods Reviewed

Method	Previous Application	Equipment	Suitable for testing MTS in Geotechnical Applications
1. Impact Echo	Evaluation of plate type elements, honey combing in concrete, voids in ducts	Commercially available at reasonable cost; instrumented and modally tuned impact hammers are expensive	Need to apply impact and measure response at the same end of the element; will use impact with lower frequency compared to previous applications, therefore, same style hammer employed by previous researchers may not be necessary.
2. Impulse Response	Evaluation of drilled shaft foundations; evaluation of tension levels in prestress bars.	Similar to impact echo except response is measured with a velocity transducer and larger impact with lower frequency content is applied.	Similar to impact echo
3. 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.
4. Acoustic Emission	Used by mining industry to detect rock instability.	Equipment is commercially available. Sensors are relatively inexpensive but signal conditioning and data acquisition equipment is expensive.	Has potential for application to MTS, but capability of system is limited.
4 (a). Continuous Acoustic Emission	Monitoring of post-tensioned concrete structures, cables of suspension and cable stayed bridges and prestress concrete pipe	Specialized software is required and signals must be processed and interpreted by trained expert.	Current technology is used to listen and indicate when failures occur. Does not indicate condition prior to failure.
5. Ultrasonic Test	Used to evaluate plate type elements, weld quality, anchor bolts, anchor bars, bridge cables.	Equipment is commercially available. Cost of the equipment is moderate.	Can be readily adapted to MTS. Need access to anchor head, but can perform the test with access to only one end of the element. Much of the signal may be lost due to dispersion, and there is a limit to the length of bar that can be tested.

6. Ground Penetrating Radar	Evaluation of pavements, bridge decks, subsurface investigations.	Equipment is commercially. Costs are moderate although many agencies may already own this equipment	Not sensitive enough to detect defects in elements that are not relatively close to the ground surface. Method may be more useful if a technique can be found for penetrating bar elements with radio waves.
7. Reflective Impulse Measurement Technique	Evaluation of unbonded post tensioned cables.	Equipment is commercially available at moderate cost.	Implementation is possible since it may be applied to only one end of an element. Technique is not proven to be effective.
8. Time-Domain Reflectometry	Used to locate discontinuities in electrical transmission lines. Recently studied for application to bridge cable.	Equipment commercially available at moderate cost. Data requires 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.
9. Magnetic Flux Leakage	Condition assessment of reinforced concrete structures.	Equipment is commercially available at moderate cost.	Not directly applicable to MTS since the entire length of the element is must be accessed. The technique is interesting because of the claim that it may measure loss of cross section as low as 3 %.
10. 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.	Equipment is commercially 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.
11. Half -Cell Potential	Monitor condition of steel reinforcement in concrete, rock bolts	Equipment is readily available and relatively inexpensive.	May be applied to MTS. Electrical connection to one end of the element is required. Half-cell must be make electrical connection through electrolyte. May be difficult to get meaningful readings from elements that are not electrically isolated.
12. 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
13. AC 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.

14. 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 measurement. May be useful to identify severity and type
15. 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.
16. 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.

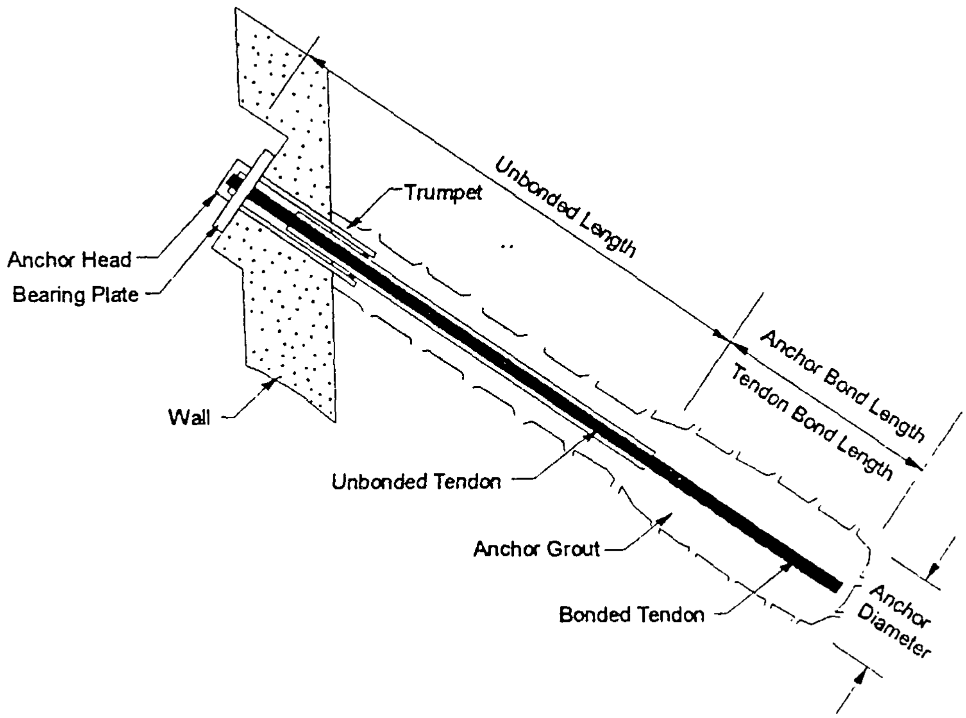

Figure 2-1. Prestress Grouted Ground Anchor.

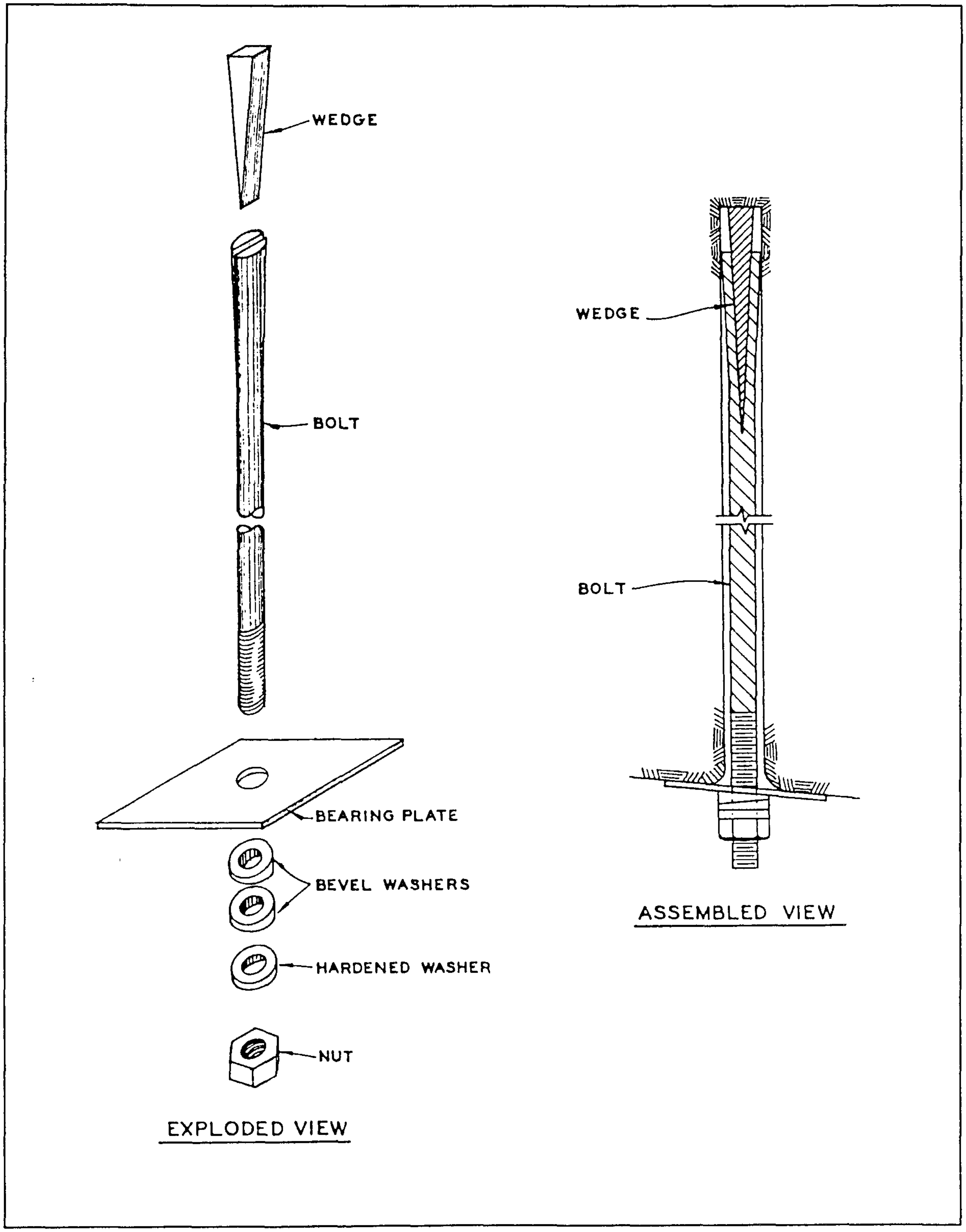

Figure 2-2. Slot and Wedge Rock Bolt.

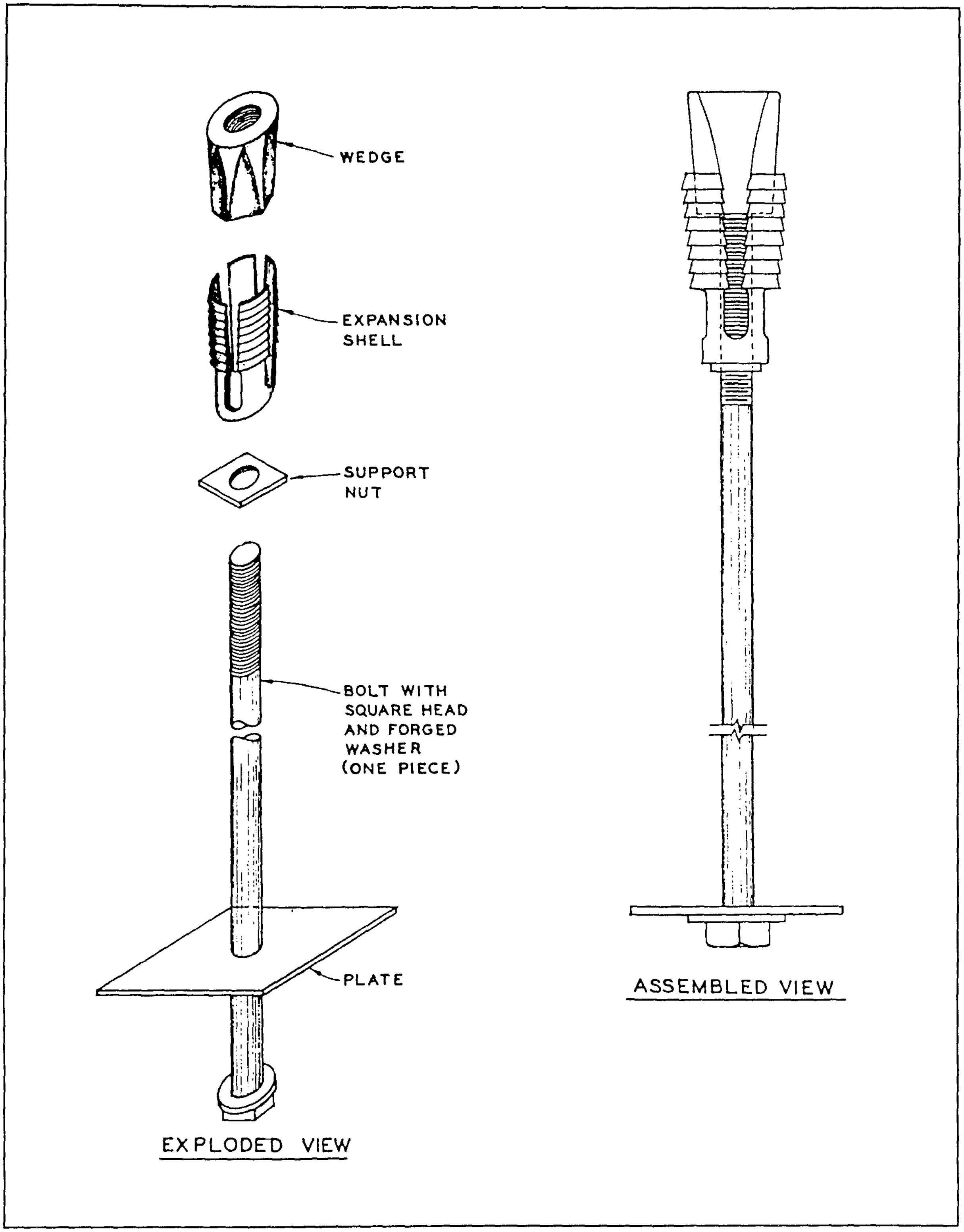

Figure 2-3. Expansion Shell Rock Bolt.

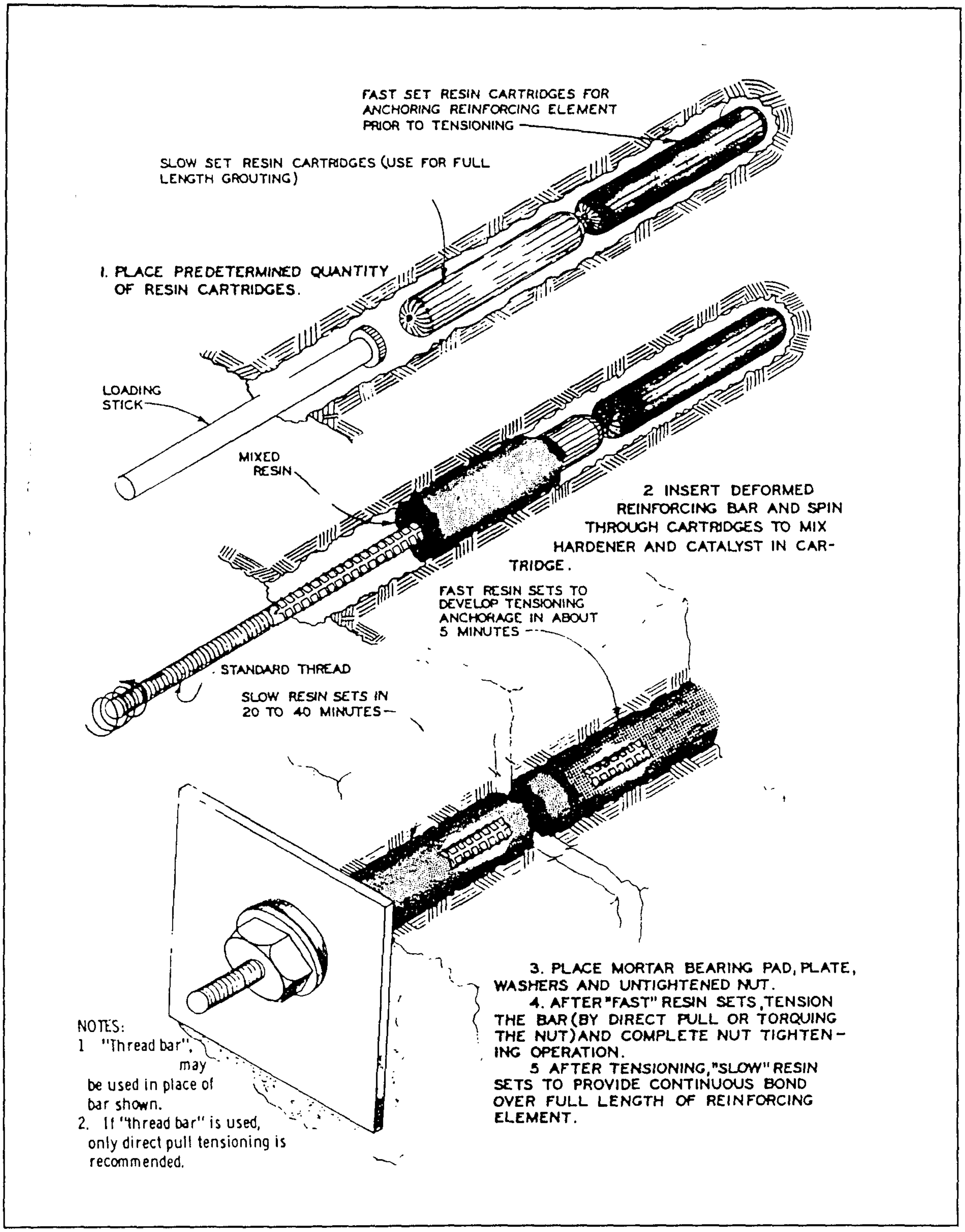

Figure 2-4. Resin Grouted Rock Bolt.

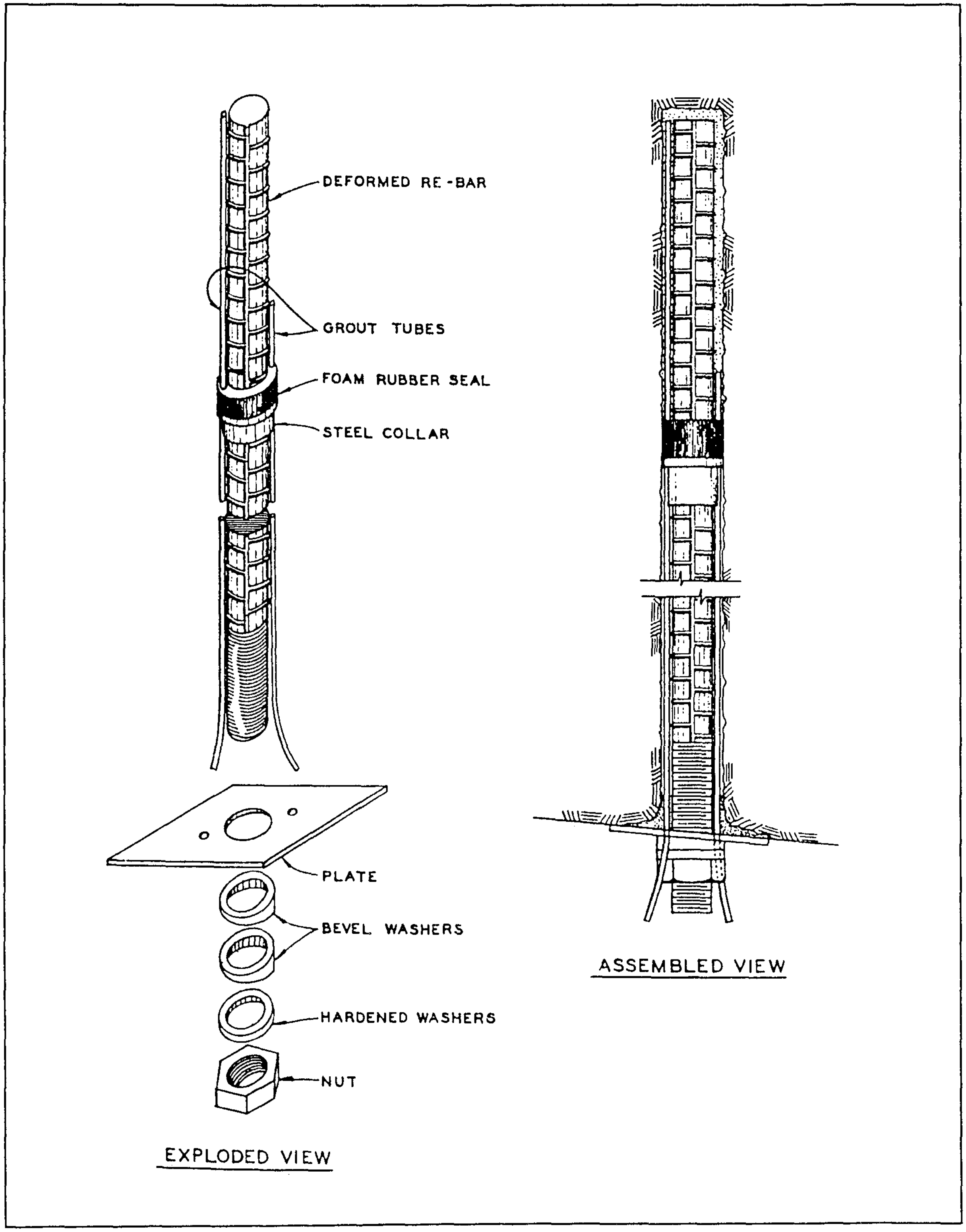

Figure 2-5. Cement Grouted Rock Bolt.

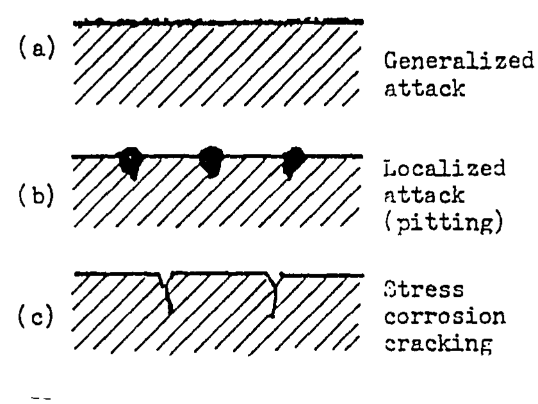

Figure 2-6. Primary Types of Corrosion (Xanthakos, 1991).

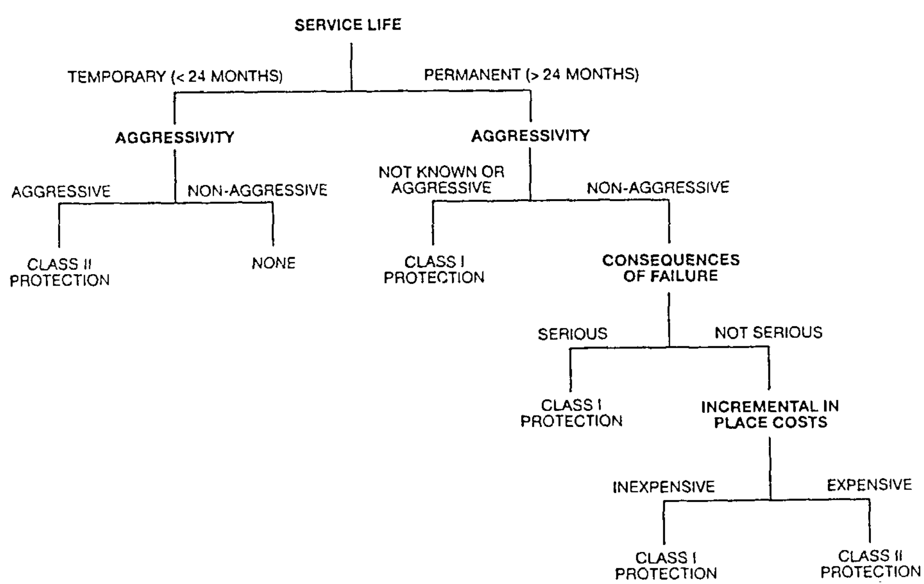

Figure 2-7. Corrosion Protection Decision Tree (PTI, 1996).

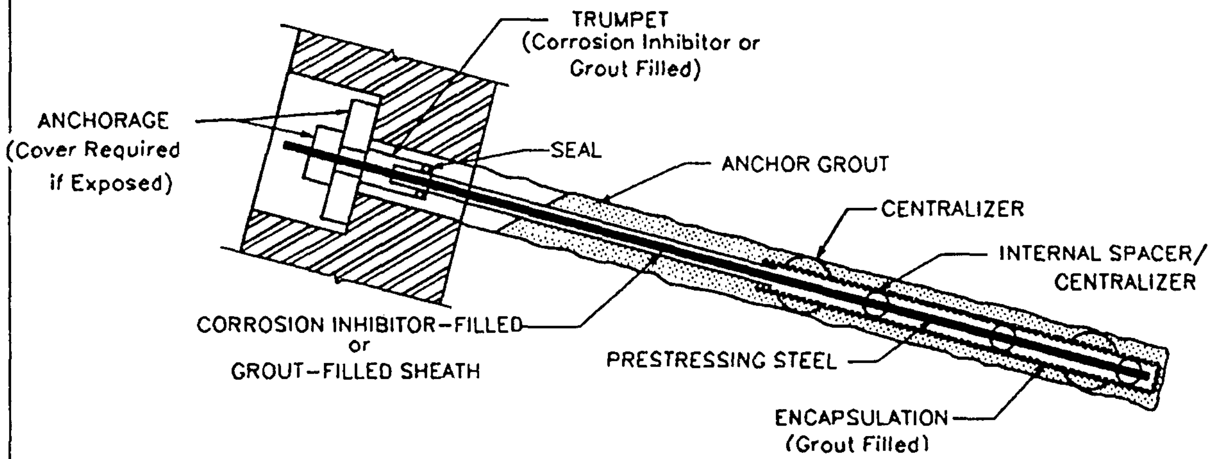

Figure 2-8. Class I Protection - Encapsulated Anchor (PTI, 1996).

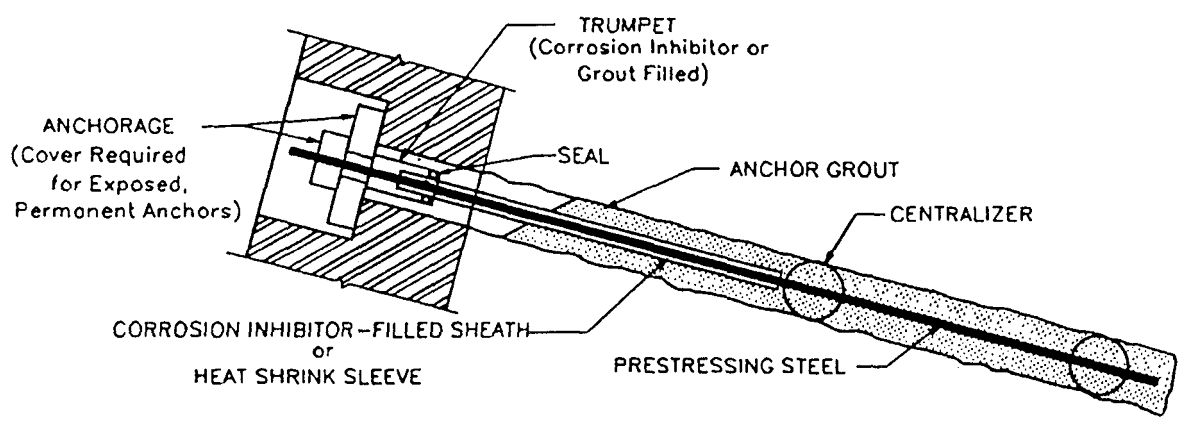

Figure 2-9. Class II Protection - Grout Protected Anchor (PTI, 1996).

3.0 EVALUATION OF CONDITION ASSESSMENT TECHNIQUES

This section describes our evaluation of several NDT techniques with respect to their specific application to condition assessment of ground anchors, rock bolts or soil nails. NDT methods, similar to those described in the preceding section, need to be studied with respect to the particular requirements for testing of metal-tensioned systems. For existing systems however, some of the NDT methods will have to be modified as access to the element will be limited to the portions which extend from the wall or slope.

A variety of methods were evaluated for in-situ condition assessment of metal-tensioned systems in both new and existing installations. While most of the equipment considered for NDT can be obtained commercially, its specific application to buried metal tensioned elements has not been studied. Thus, NDT system development and evaluation was necessary for the proposed application. Laboratory and some in-situ field testing were conducted at the University at Buffalo (U.B.) to support the evaluation and selection of viable monitoring systems and development of recommended protocols for the field study planned for Phase II of the project.

To evaluate the viability of candidate NDT test methods, a series of tests were conducted using bench-scale, and in-situ test facilities developed for the study. The tests allowed several NDT methods to be evaluated and calibrated under known conditions, and then implemented and studied under approximate field conditions. Initially, bench-scale tests were performed to determine the success of selected methods using simple testing configurations which provided quick and cost-effective means to evaluate different test parameters including element type, confinement, and the location of simulated corrosion defects. Where favorable results were obtained from bench-scale tests, NDT methods were further evaluated using in-situ test specimens. In-situ test specimens were installed in a soil under environmental conditions that simulate possible field conditions.

The following sections describe details of the testing and data acquisition systems employed, evaluation of the NDT methods studied, and the plan used to maintain quality control of the testing program.

3.1 TESTING AND DATA ACQUISITION

3.1.1 Test Specimens

Industry suppliers donated all the samples that were tested in the laboratory and in-situ test facilities.

Samples of 12 and 15 mm diameter seven-wire strand used for permanent ground anchor applications were supplied by Polystrand, Inc. and VSL Inc. The strands were manufactured from high strength steel alloy, Grade 270, in conformance with ASTM A-416. Samples were delivered in two 9 m and one 18 m lengths. Corrosion protection of the strands consisted of corrosion inhibitor grease surrounding the strands, and a seamless polypropylene sheath extruded over the grease. In addition, samples of 32 mm diameter bars, also used for permanent ground anchor applications, were supplied by Dywidag Inc. The bars were manufactured from high strength steel, Grade 150, in conformance with ASTM A-722. Four 9 m long bars were supplied; two were uncoated, and two were epoxy coated.

3.1.2 Test Facility

The bench-scale arrangement included testing several Dywidag bars, approximately 2 m long and 32 mm in diameter. The tests were conducted using undamaged bars and bars with simulated corrosion defects. This testing offered a means of developing experimental arrangements and providing a baseline against which tests on bars with defects could be compared. Initially, simple tests were conducted on specimens with free boundary conditions at each end. Subsequently, the bars were installed in a 1.2 m long, 30 cm diameter Sonotube. The tubes were fitted with end plates with a hole to align the test bar in the center of the tube. The tests were conducted with test bars installed in an empty Sonotube and with the Sonotube filled with dry sand, to completely surround the bar with soil. The laboratory set-up and some of the specimens are shown in Figure 3-1. Subsequent test arrangements included bars having defects simulating the loss of cross-section due to corrosion. The bars with simulated corrosion defects included defects having a length of 75 mm or 150 mm, which were located at the middle of the bar and 0.6m from one end (Figure 3-2).

To verify the accuracy of the instruments, tests were conducted using mechanically isolated bars. The observed dynamic response was compared to that predicted with equations based on one-dimensional wave propagation along a slender bar. Bars were suspended using light wire supports or isolated with expanded polystyrene supports to minimize the effect of any restraint or added mass along the bar, which could represent a source of reflected wave energy. Metallic supports for the Sonotube and the attachment of various metallic pieces to the bar (nuts or clamps) affect the specimen response.

After bench-scale testing, further evaluation of the candidate test method was performed using in-situ specimens. The in-situ anchor specimens were installed at a test facility located adjacent to the structural-testing laboratory.

The anchor elements were installed in vertical test holes. The services of a subsurface exploration and drilling company were employed to install the specimens (Figure 3-3). Auger borings with a diameter of 150 mm were advanced to a depth of 2.75 meters at each element location. The specimens were placed and centered within the hole and the auger borings were backfilled with native soil. For the first boring, soils were sampled continuously using a split-barrel sampler, and the Standard Penetration Test (SPT) resistance was obtained at 0.6m intervals in general conformance with ASTM D1586. The soil samples were tested in the laboratory for moisture content, Atterberg limits and grain size. Chemical analysis of the soil samples is currently underway and includes pH, sulfates and chloride content.

The in-situ specimen test facility is pictured in Figure 3-4. Eight elements, each with a length of approximately 3 m, were installed. Specimens were placed along two rows, and separated by approximately 4.5m as shown in Figure 3-5. All of the elements have a 0.3m 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 U.B. Test Facility (Figure 3-6) including:

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

Two specimens were installed for each type of element: one intact without any defect, and the second with a defect. Bar element defects were constructed as a notch placed about one meter 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 75mm length of the HDPE sheath and exposing the strand to the subsurface environment.

3.1.3 Data Acquisition System

The data acquisition (DAQ) equipment used for this project is manufactured by National Instruments and Dell Computer Corporation. The system utilizes a multi-function data acquisition board (National Instruments Model # PCI-6023E) with 16 single-ended analog inputs, which have 12-bit precision each. 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 200 kHz (200k samples/sec). Additional DAQ board specifications are provided in Appendix I. The DAQ board was incorporated into a Dell desktop computer, which has a 550 MHz processing speed, a 20 GBytes hard disk, and 128 MBytes of RAM.

A software package called *VirtualBench* was utilized for data acquisition and data processing. *VirtualBench* is a product of the National Instruments Corporation. A version of the program, available at U.B., was upgraded to meet the requirements of the new generation of DAQ boards. *VirtualBench* software allows the computer system to acquire, display, process and store data. The computer 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, record lengths ranging from 550 to 660,000 points, time base ranging from 10ns/div to 100ms/div and a sensitivity range of 2 mV/div to 10 V/div. 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.

Recorded data was analyzed in the time-domain (time-history representation) and/or frequency domain (Fourier, amplitude power and phase spectrum representation**)** using the *VirtualBench* package, as well as custom-made software developed with the *Matlab* software package (MathWorks product family).

Additional details regarding the technical characteristics of other equipment used to acquire data for the impact echo, ultrasonic, electric-chemical, radio wave reflection and Contact resistance test methods are presented below.

3.2 EVALUATION OF NON-DESTRUCTIVE TESTING METHODS

Based on the literature search of NDT test methods, the following NDT test methods were selected for evaluation relative to their capability to detect defects, and/or the rate of corrosion of metal-tensioned systems:

- Impact echo
- Ultrasonic
- Electro-chemical measurements
- Radio wave reflection (similar to GPR)
- Contact Resistance

3.2.1 Impact Echo Test Measurements

From the results of our review of potential NDT methods, the impact echo method is a good candidate for evaluating cracking of grouts, fracture of tendons and loss of element section. For this test method the specimen is impacted using a hammer or ball device, which generates elastic compression waves with relatively low frequency content. The recorded signals from wave arrival times, and the timing and patterns of the wave energy that is reflected at specimen ends, discontinuities or changes of material density is transformed into the frequency domain. By inspection of the frequency response or by utilizing signature analysis, it is then possible to detect changes of cross section, cracks in tensioned system and/or characteristics of the surrounding element/grout interface.

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 multi-channel signal conditioner that also includes a power supply with necessary excitation. Components of the test were connected with standard coaxial cables and BNC connectors as shown in Figure 3-7.

PCB Piezotronics, Inc. (PCB) of Depew, NY was contacted to discuss equipment needs and to acquire specialized equipment needed for this study. PCB specializes in developing and applying instruments that employ piezoelectric crystals as the sensing element, such as accelerometers and force or pressure sensors. After describing the details of project NCHRP 24-13 to PCB engineers, research scientists and sales representatives, equipment was observed which is used to calibrate PCB's high shock load sensors. The equipment employs a Hopkinson bar, whereby, via an air-pressurized barrel, an impact is applied to one end of a solid round bar. The response of the bar is observed with a set of strain gauges attached to the center of the bar and an accelerometer at the far end. Signals acquired from the test are analyzed in the frequency domain. This technique is similar to that proposed for evaluating metal-tensioned systems with the exception that, for these systems, application of the impact load and anchor response must be observed at the exposed element end.

PCB donated two accelerometers for conducting further evaluation of impact tests in the lab at U.B. One of the accelerometers is a high sensitivity sensor (Model #U353B34) that has a sensitivity of 106 mV/g, a frequency range of 1 to 4000 Hz, a resonant frequency of 26kHz, and a measurement range ±50 g, and which may provide stronger signal levels and higher resolution for low amplitudes of vibration. The other accelerometer is a high shock sensor (Model #U350A14) that has a sensitivity of 9 mV/g, a frequency range of 1 to 7000 Hz, a resonant frequency 56 kHz, and a measurement range ±5000 g, and which is capable of measuring the response of high accelerations associated with metal-to-metal impacts. The accelerometers may be fixed on a mounting base, which is attached to the specimen by special adhesives or magnets.

The signal conditioner is a PCB product (Model # 483M76) that can simultaneously process six channels, and which has a range of selectable gain from 0.1 to 1000. This unit was already available in the Civil Engineering Laboratory at U.B. The acquisition system is placed on a wheeled cart to facilitate access to specimens and testing sites (Figure 3-8).

A number of impact devices were evaluated for introducing the stress wave into the metal-tensioned systems. Small and medium size instrumented hammers, and an in-house manufactured pendulum with a ball-peen hammer attached to the end were used.

The instrumented hammers are PCB products, and were already available at U.B. The medium size hammer is a modally tuned device (Model # 086C05) that has a frequency range of 1 to 5 kHz, a sensitivity of 1 mV/lbf, a resonant frequency of 28 kHz, a mass of 454 grams, and a head/tip diameter of 25/6.3 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 size hammer is equipped with an impact force sensor, 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 Appendix I.

The bench-scale arrangement included several Dywidag bars with the characteristics presented in Table 3-1. Tests were conducted on mechanically isolated bars, and on bars installed in Sonotubes filled with sand. The bars have a variety of configurations: intact without defect or with notches of different sizes at various locations.

Table 3-1
Bench-Scale Specimen Characteristics

Element Type	Length (m)	Diameter (mm)	Defect		
			Size		Location from one end (m)
			Length (mm)	Depth (% section)	
Plain Dywidag bar	2	32	-	-	-
Plain Dywidag bar with defect	2	32	75	25	1
Plain Dywidag bar with defect	2	32	150	25	1
Plain Dywidag bar with defect	2	32	75	25	0.6
Plain Dywidag bar with cut	2	32	5	25	0.6
Plain Dywidag bar with cut	2	32	5	50	0.6
Plain Dywidag bar with cut	2	32	5	75	0.6
Plain Dywidag bar with defect in Sonotube filled with sand	2	32	75	25	1

The impact test procedure used involved, (1) striking the bar 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 bar. Tests were conducted in the pitch-catch mode (i.e., impact and receiver placed at opposite ends of the bar), and with the impact and receiver placed at the same end of the bar. A special sleeve was placed at the end of the bar to allow impact and placement of the accelerometer at the same end, and to permit the necessary wiring to pass through the sleeve.

The impact force and the acceleration were digitally recorded at a sampling rate of 200k samples/sec. Initially data were collected over an interval of 100 msec, but later in the test program shorter sampling intervals of 2, 5, and 10 msec were employed. The *VirtualBench-Scope* and *VirtualBench-DSA* program settings allowed the impact hammer to function as a trigger, so that the response of the specimen could be captured within a single frame on the CRT screen. Appendix II presents detailed results from impact testing. The responses of the instrumented impact hammer and the bar are presented in the time and frequency domains. For the medium sized, rubber tipped, modally-tuned hammer, the duration of the impact force was in the order of a few milliseconds. The corresponding frequency spectrum has significant frequency components up to approximately 1 kHz.

Impact test results on intact bars, presented in Appendix II, indicate the effect of changing bar end conditions (plates or nuts attached) on the response of the bar (natural frequencies).

The impulse waveform and its frequency content are affected by properties of the impact device including head stiffness, mass and impact velocity. Small and medium-sized rubber, or plastic tipped, hammers were evaluated in this study. Similar bar response was observed when comparing results from tests performed with different hammers. However, the amplitudes of the acceleration depicted in the Fourier spectra were sensitive to the impact energy.

In an effort to control impact energy, an in-house pendulum was manufactured to apply controllable and repeatable impact forces (Figure 3-9). The pendulum was mounted on an adjustable stand, and the arm of the pendulum could be set at three selected angles of inclination corresponding to different levels of impact energy. The impact force was applied with a metal tipped, ball-peen hammer. The high sensitivity accelerometer was used during the initial part of the testing program. This accelerometer worked well with the rubber-tipped impact hammers. However, when a metal tipped hammer was used the acceleration record was “clipped” for the incipient part of the time history when peak accelerations exceeded 50g. Therefore, the high impact shock accelerometer was used with the metal-tipped hammer to allow higher impacts without the adverse effects of “clipping” prevalent in the observed response.

Typical bar response from impact with a hard metal tip is shown in Figure 3-10. The first three natural frequencies of an approximately 2 m long bar, calculated based on one-dimensional compression wave propagation along a cylindrical bar with free-free ends, are approximately 1.4, 2.8 and 4.2 kHz. Results presented in Figure 3-10 are in agreement with the predicted response. This provides some verification of the instrumentation, hardware, and test technique used for the impact echo test evaluation.

Impact tests were performed on specimens with defects introduced along the length of the bar to simulate the loss of cross-section due to corrosion. Figure 3-10 shows a typical comparison of results from bars tested with and without defects. The vibrational response of isolated bars (bars suspended or placed on polystyrene pads), and bars installed in Sonotube filled with sand, is presented in time and frequency domain in Appendix II. Changes in response due to the nature, size and location of various defects in the bar are evident in the results. A perturbation of the frequency response was observed when comparing results from tests conducted on unaltered cross sections versus those with a defect. Values of the natural frequencies of the specimens detected during the impact tests are presented in Table 3-2.

The time history and amplitude spectrum for an intact specimen (isolated bar placed on polystyrene pads) and for a specimen with a defect located in the middle, or at 0.6m from one end of the bar were compared. A shift in resonant frequencies, of up to 10 percent is the result of a decrease in stiffness of the material created by the introduction of various notches along the length of the bar. A shift in resonant frequency of 10 percent to 15 percent was observed for a bar with a 75 percent reduction of cross section, located at 0.6m from the bar end; compared to one with 25 percent reduction of cross section along a 150 mm notch cut in the middle of the bar. Results from tests conducted on bars placed in the Sonotube indicated that the presence of soil surrounding the bar also affects the natural frequencies of the bar.

Table 3-2
Natural Frequencies for Bench-Scale Tested Specimens

Specimen	Modal frequency (kHz)			Damping %
	I	II	III	I
Intact bar installed in empty Sonotube w/end plates	1.38	2.74	4.14	-
Intact bar installed in empty Sonotube w/end plates and nuts	1.15	2.48	3.84	2.3
Intact suspended bar installed in empty PVC tube w/end plates and nuts	1.15	2.32	3.54	2.4
Bar with defect (25 % section reduction and 75 mm long notch in the middle) installed in empty Sonotube w/end plates	1.29	3.05	3.78	3.6
Bar with defect (25 % section reduction and 75 mm long notch in the middle) installed in empty Sonotube w/end plates and nuts	1.05	2.51	3.80	2.6
Intact isolated bar on polystyrene pads	1.36	2.71	4.07	6.6
Isolated bar with defect (25 % section reduction and 75 mm long notch in the middle) on polystyrene pads	1.33	2.77	4.02	8.8
Isolated bar with defect (25 % section reduction and 75 mm long notch at 0.60 m from one end) on polystyrene pads	1.29	2.61	4.2	-
Isolated bar with defect (25 % section reduction and 150 mm long notch in the middle) on polystyrene pads	1.31	2.77	4.0	-
Isolated bar with defect (75 % section reduction cut at 0.60 m from one end) on polystyrene pads	1.12	2.52	4. 1	-
Bar with defect (25 % section reduction and 75 mm long notch in the middle), installed in Sonotube filled with sand	1.51	2.44	3.78	-

Following the bench-scale evaluations, tests were performed on an in-situ specimen. The characteristics of each specimen are presented in Table 3-3.

Table 3-3
In-Situ Specimen Characteristics

Specimen No.	Element Type	Length (m)	Diameter (mm)	Defect		
				Size		Location from the far end (m)
				Length (mm)	Depth (% section)	
1	Plain Dywidag bar	3	32	-	-	-
2	Plain Dywidag bar w/defect	3	32	75	25	1
3	Epoxy Dywidag bar	3	32	-	-	-
4	Epoxy Dywidag bar w/defect	3	32	75	25	1
5	Seven-wire strand w/ corrosion protection	3	15	-	-	-
6	Seven-wire strand w/ corrosion protection and defect	3	15	75	0	1
7	Grouted Dywidag bar	3	32	-	-	-
8	Grouted Dywidag bar w/defect	3	32	75	25	1

Appendix II present results in both the time and frequency domains for the in-situ specimens tested. Results indicate that the length of the bar can be inferred. Typical results comparing results observed for in-situ specimens, with and without defects, are presented in Figures 3-11(a) and 3-11(b). As in the case of bench-scale tests, the placement of a notch along the length of the bar introduces perturbations in the dynamic response of the bar.

Future laboratory work on impact tests is expected to identify the sensitivity of these perturbations to the characteristics of defects (size, depth, shape, orientation and location) on bare and epoxy bars. Several bench-scale tests will be done on bars installed in plastic pipes filled with grout, and on bars having various grout cracks placed along their length.

3.2.2 Ultrasonic Test Measurements

From the results of our review of NDT methods, the ultrasonic test method is another good 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. Compression waves consisting of alternating regions of compression and dilatation propagate along the axial direction of a bar. Transducers can be selected offering a wide range of pulse frequencies. Through careful selection, a wave length can be selected which is most suitable for the particular application. For slender bars, plane-wave conditions apply for all distances from the source. For this case, the compression wave speed is given by $c = (E/\rho)^{1/2}$, where E is the modulus of elasticity and ρ is the material density. Typical values of acoustic properties of some commonly used materials are presented in Table 3-4.

Table 3-4
Acoustic Properties of Materials

Material	Density (kg/m^3)	Speed (m/s)		
		Longitudinal waves	Shear (transverse) waves	Surface (Rayleigh) waves
Air	1	3300	-	-
Cast iron	7700	4500	2400	-
Mild steel	7830	5950	3200	2790
Stainless steel 302	7800	5660	3120	2780
Stainless steel 347	7890	5790	3100	…
Stainless steel 410	8030	7390	2990	2160
Water	1000	1490	-	-

The pulse technique is the simplest of all ultrasonic test methods. It consists of measuring the time taken for a short train of sound waves to move through a given distance. Travel time, which is indicated by the position of maximum amplitude in the waveform, depends on the specimen dimensions, and the sound wave velocity. Both of these quantities depend on external temperature and pressure. Ultrasonic waves propagating in a slender element are dispersive in nature. The amplitude of vibration at some distance from the source depends on the attenuation coefficient of the waves, and the divergence of the beam relative to distance from the source. The use of high frequency sound waves, which have small wave length compared with the cross-sectional dimensions of the specimen, has an advantage of an increase of sound beam directivity.

With the pulse-echo method (single-probe operation), the time taken for sound pulses, generated at regular intervals, to pass through the specimen and return, is measured.

Return pulses may be either from a single reflection at a discontinuity, or from multiple reflections between a discontinuity and the end of the specimen. The patterns of the received pulses can provide valuable information about the nature of a defect, and of the structure of the material being tested. The advantage of the pulse-echo method is that only one side of the specimen needs to be accessed for transducer placement. To achieve a direct reflection, the far surface of the specimen must be parallel to the surface where the transducer is placed.

A preliminary evaluation of the ultrasonic test technique was made during the summer of 1999. Mr. Tom Ott from EMCO Intertest, Inc. visited the project team in Buffalo to demonstrate the use of ultrasonic test equipment for evaluating the condition of long bars and strands. The equipment demonstrated was the Panametrics high-voltage pulser-receiver, Model #5058PR, a V1011 piezoelectric transducer, and an oscilloscope to view the resulting wave form. The pulser-receiver unit has capabilities of generating pulses with selected pulse repetition frequency rate of 20, 50, 100, 200, 500, 1000 and 2000 Hertz, 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, 0.1, 0.3 and 1MHz or high frequency cutoff points at 0.5, 1.0, 3 and 5 MHz. The equipment features a shock-excitation voltage of approximately 900V, which was necessary to meet the high-energy requirements for generating acoustic waves in the bar system. The pulser-receiver unit allows selecting either the pulse-echo or the through-transmission mode as testing operation.

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, and although this is a relatively low frequency 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 for 32 mm diameter Dywidag bars was necessary due to the unit's ability 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 is provided in Appendix I.

The initial evaluation of the pulse-echo technique was made using a 7m long, 32 mm diameter, Dywidag bar in the laboratory at U.B. The amplitude of the returned signals were affected when a technician held onto the end of the bar (hand damping method), which confirmed that returned signal were propagating from the opposite end. The ultrasonic test equipment was also evaluated in the field at the site of the 10th St. Parking Garage in Niagara Falls, NY. The reinforced concrete deck beams for this structure had been retrofitted recently with 13 mm diameter, seven-wire monostrand that was easily accessible (Figure 3-12). The strands were approximately 18 m long and were pretensioned to a level of approximately 150 kN. The major difficulty encountered at this site was the uneven surface of the strand ends, which limited the coupling efficiency of the transducer.

A more detailed evaluation of the equipment was conducted from November 30, 1999 to January 6, 2000. Panametrics, Inc. loaned ultrasonic test equipment which was used for bench-scale and in-situ ultrasonic tests using the specimen arrangement mentioned in the previous section.

As for the impact echo technique, computer controlled DAQ equipment was used to record and analyze data. An oscilloscope, available at the U.B. laboratory, was also used for quick data visualization. Figure 3-13 shows a schematic of the ultrasonic test arrangement.

Use of a variety of acoustic couplants was evaluated in the laboratory including: Valvoline grease, Panametrics gel, and dielectric tune-up grease. Couplants were used to provide a suitable sound path between transducer and test surface to facilitate the transmission of the ultrasonic pulse energy into the test piece. Couplants need to be kept as thin as possible, because a thick layer affects the direction of the ultrasonic beam in the specimen and can produce spurious reflections in the layer.

The ultrasonic procedure consists of the following steps:

- The transducer receives its shock excitation, and generates a short ultrasonic pulse. A large (i.e. transmission) peak appears on the computer or oscilloscope display.

- The probe receives echoes of the pulses after reflection. The peaks might correspond to the first back-end echo, or an echo from an internal defect. Each reflected echo represents a return transit through the material so that the acoustic path length is equal to twice the thickness of the sample. The return of the leading edge of the first echo can be easily detected by visual means.

- Ultrasonic attenuation may be measured by comparing the heights of consecutive back-end echoes. Spurious peaks may appear as a result of (a) generation of a fresh pulse before its predecessor had died down, (b) echoes from a delay medium, or (c) lateral reflections giving rise to compression to shear wave conversions, and vice versa.

Identification of spurious peaks can be made by testing a known defect-free specimen. There is no simple relationship between echo and defect size. Signal amplitude depends on a wide range of different factors, including metal microstructure, grain size, defect distance, shape, orientation, as well as acoustic impedance differences. Echo amplitudes are calibrated by comparison with signals from standard sizes of ideal-shaped defects over a range of distances.

Ultrasonic testing has been applied in mechanical engineering where smaller size specimens are encountered with a known nature of defects. Usually, the evaluations assume that the material is isotropic and the stress level across the entire cross-section is homogeneous. Comparator specimens are often utilized to compare the echo from a flaw with that from an artificial defect. Thus, the application of ultrasonic testing for damage assessment of metal-tensioned systems due to corrosion is a straightforward task. The large diversity of defects (flaws) encountered in soil nails, ground anchors or rock bolts (various size, depth, shape, location, and orientation), makes defect sizing using a comparator prototype difficult. If a wide range of defect sizes and depths need to be considered, a large population of specimens with known defects must be evaluated. Because the echoes take on a complex pattern, the acquisition system must be "trained" to identify the signatures associated with a particular type of failure or deterioration.

The same specimens used for impact echo testing were also used to evaluate the ultrasonic test technique. The bench-scale tests conducted provided an opportunity to observe the effect of the test parameters including amplitude, damping, repetition rate of the pulse, and conditioning of the signal received from the reflected wave. A sampling rate of 200,000 points per second was used for digitally recording data over time intervals of 2 to 5 msec. The traveling time and the sound wave forms were recorded and the test results were processed in time domain, such that natural frequencies of vibration could be observed. The observed response of the bar compared well with the theoretical predictions. Bench-scale testing results demonstrated that both the end of the bar and the presence of a defect can be detected with the ultrasonic equipment. Results from evaluating the ultrasonic test with bench-scale specimens are presented in Appendix III. Figure 3-14 presents a typical result from testing a Dywidag bar with no defect.

Ultrasonic tests were performed on bars with defects located at various positions along the length of the test specimen. The responses of mechanically isolated bars, and bars installed in the Sonotube filled with sand, are presented in Appendix III. The amplitude of the back-end echo decreased exponentially with distance in the far field, such that only two to four back-end reflections were captured during the test. Additional echoes present in the results were characteristic of discontinuities within the specimen. Compared to back-end echoes, low amplitude echoes were received from discontinuities. In addition, attenuation measurements could be used for damage evaluation. The presence of a defect gives rise to a reduction in the amplitude of the received signal, which can be correlated to the characteristics of the defect. A change in both amplitude and frequency takes place, compared to the values for an intact specimen.

The ultrasonic test was also evaluated using the in-situ specimens. Typical results from testing epoxy coated Dywidag bars with and without defects are shown in Figures 3-15(a) and (b). More results from testing in-situ specimens are presented in Appendix III. Results from testing buried bar elements were consistent with the results obtained from bench-scale testing.

A previous attempt to implement the ultrasonic test at the site of the Niagara Falls Parking Garage was not successful because the wires at the end of the strand were not cut evenly, which created a problem in coupling the transducer with the strand. For this subsequent effort to apply ultrasonic testing to evaluate the condition of strands, the ends of the strand anchor specimens were cut flat such that a good coupling was achieved between the transducer and anchor end, making possible the detection of a reflection with the strand–type anchor element. Therefore, the finding that the test may be applied to strands is significant.

Based on the successful experience realized up date (March 2000), the project team is considering acquiring the ultrasonic test equipment for use during Phase II of the project. The team is discussing the cost of purchase versus the cost of rental, and also the possibility of purchasing a demonstration model (used equipment) at a prorated price. The expense associated with acquiring the ultrasonic equipment is well within the project budget allocated for equipment costs.

The ability of the system to probe buried slender bars having a length of at least 10 meters must be verified. During bench-scale testing, at least five recognizable reflections of the sound wave from the end of the 2 m long bar were observed. The fifth reflection corresponds to a travel path of approximately 20 m. In addition, an approximately 7.5 m long bar was successfully tested in the laboratory, providing some verification of the ability of the ultrasonic test to obtain reflections from longer bars. Therefore, it is expected that at least one discernible reflection can be realized when the method is applied to bar lengths of up to 10 m or more. To further verify the ability of the system to probe longer elements, the preliminary testing of existing tie-backs at a field site is recommended.

More extensive laboratory work of the ultrasonic test method is necessary to identify the sound waveform sensitivity to perturbations introduced by various defects on bare and epoxy bars, and seven-wire strands (different notch size, shape, orientation and location). In addition, bench-scale tests are planned on bars installed in plastic pipes filled with grout, and on bars having various grout cracks placed along their length. Careful consideration will be given to the signal-to-noise ratio to ensure the detection of small defects.

3.2.3 Electro-Chemical Measurements

Electro-chemical measurement techniques were evaluated to document the type of information that can be generated regarding possible deterioration of buried metal-tensioned systems. The use of half-cell, and polarization measurements were studied. The results indicate that these tests will be very useful for evaluating the integrity of corrosion protection systems, and for detecting the presence of active corrosion mechanisms.

Half-cell measurements are typically used to detect corrosive activity. However, use of the method generally requires a distinct electron flow path between each half-cell making up the system. Thus, the conductivity of soil or grouts needs to be carefully evaluated to determine their effect on such measurements. In some cases there may not be a sufficient level of conductivity without the use of an embedded probe. The use of half-cells may be particularly useful in evaluating the performance of soil nails, rock bolts and solid bar anchor systems not protected by a plastic sheath along the unbonded zone to determine if corrosive activity is taking place. This approach will also be useful to determine whether a corrosion protection system, such as a sheathed tendon, has been compromised.

The gas pipeline industry uses the observed E vs. Log I relationship for assessing cathodic protection requirements for buried metal pipe (Pinto, 1999). The method involves installing a common ground at some distance from the measurement location, applying a known voltage between the pipe (or individual metal element) and the ground bed and observing the E vs. log I relationship for the pipe with the cathodic protection system deactivated. The method is easily adaptable to ground anchors, rock bolts and soil nails.

The equipment needed for the test includes a power supply with a rheostat, an ammeter, a high impedance voltmeter and a reference electrode. This equipment is standard, relatively inexpensive and the components are readily available but must be specially incorporated into a test facility for evaluating metallic anchor systems. Results from the test can provide a comparison between metallic elements at different locations at the same site, as well as evaluations of specific metallic elements.

The project team evaluated the test method using the buried specimens at the U.B. Test Facility. At the time the measurements were taken the air temperature was approximately 2°C and the ground was saturated. Initially, half-cell resistance measurements were made between the buried specimens and a copper/copper sulfate reference electrode to evaluate the conductivity of the soil and to obtain reference potentials. If the element is perfectly insulated and there are no compromised areas in the protective system, the potential readings should be very low. As no element protective systems are perfectly insulated, the half-cell measurement will produce a specific base reading. If a field of similar elements is measured, higher potential readings on some of the specimens indicate increased electrical activity with the soil and an increased probability of corrosion activity taking place. Observed half-cell potentials correlated well with the condition of the buried areas of the specimens and provided a distinct indication of level of specimen protection and intact, or impaired corrosion protection systems. A schematic of the test arrangement, and half cell potential measurements of buried specimens are presented in Figures 3-16 and 3-17.

To observe the relationship between impressed current and the surface polarization potential of the anchor specimens, a ground bed was established approximately 30 m away from the closest anchor specimen. The ground bed consisted of three copper plated rods driven in the ground to a depth of approximately one meter. A known current was impressed on the system such that the direction of current flow was from the anchor specimen towards the ground bed (i.e. the positive connection of the power supply connected to the ground bed) with the soil/pore water serving as electrolyte. Current flow from the specimen was controlled by a rheostat system that permitted fine adjustments to the flow of current. As a result of the current flow, the surface of the anchor specimen was polarized after a fixed amount of time (1 to 2 minutes in the initial tests). At this time the impressed current was turned off and half-cell potential measurements were made between the reference electrode and the anchor specimen. For these measurements, an electrical connection was made from the exposed top of an anchor to the half-cell and the half-cell was then placed in firm contact with the ground. During the measurements, the half-cell was in the range of 0.5 to 1.0 m from the specimen. Measurements were manually recorded using a high impedance multi-meter. The current was then increased incrementally and the test repeated so that a relationship between the half-cell potential and the current can be established. A schematic illustration of the test arrangement and typical measurements made on the buried specimens are shown in Figures 3-18 and 3-19(a) and (b). Appendix IV contains a complete set of results of polarization measurements for all the buried specimens.

The observed relationship is unique to the type of anchor and whether or not the anchor corrosion protection system is impaired. Also, based on the levels of current involved, it is possible to estimate the area of the element that is vulnerable to corrosion as there is a relationship between the amount of current required to polarize the specimen and the area of surface exposed. Repeated measurements taken over a period time can be used to determine if changes in corrosion activity are occurring.

Initially, measurements were made with manually operated electronic equipment. However, it became apparent that electronic control of the test and automatic data acquisition was desirable. Therefore, technicians at U.B. constructed an electronic interrupter switch connected to a digital timer, which will automatically shut off the impressed current. A computerized data logger will be used to measure the half cell potential of the bar in a polarized state and the potential as the bar returns to it's unpolarized state. This will provide more control and assure that the test conditions are more precisely repeated.

The electrochemical measurements have great potential for condition assessments of metallic anchor systems in the field. There are, however, a number of potential problems that will have to be evaluated in actual field tests. One problem could be whether, or not, each anchor in a multiple anchor installation is electrically isolated. In some cases the anchor system may be restraining a steel sheet pile wall that would form an electrical path between the anchors. For this case, tests would have to be made to evaluate the minimum distance from the wall which would have to be used so that the potential and current from an anchor would be measured rather than the current from the wall system.

3.2.4 Radio Wave Reflection Technique (GPR)

The radio wave reflection technique uses radio waves to penetrate the material, similar to GPR. In an attempt to minimize the direct reflection from the end of bar type elements, radio waves were transmitted at an angle with respect to the exposed end of the bar. We anticipated that more energy would be refracted into the bar using an acute angle of attack. Waves transmitted along the bar would be reflected by discontinuities and the reflected wave intensity measured at the same angle of attack. The reflected wave intensity would be affected by the extent of corrosion along the bar.

Since the technique uses frequencies in the gigahertz range, it has potential for detecting small-scale imperfections that are present at early stages of corrosion such as pitting and scaling. Data analysis similar to that applied for impact echo and ultrasonic testing can be applied to the radio wave reflection technique to identify different wave signatures representing conditions along the length of the metal-tensioned element.

The radio wave reflection technique requires the use of a wave-guide to direct the radio waves into the steel bars. In general, most of the energy of a radio wave directed at the end of a bar will be reflected, and very little will propagate along the bar. A study was conducted to observe the effect of varying the angle of incidence of the wave to avoid direct reflection from the end of the bar. Results from the investigation reveal that reflections from the front face of the steel still dominate and overshadow reflections from other faces.

3.2.5 Contact Resistance Method

In this application, the contact resistance between adjacent wires in a strand-type anchor element is measured. The initial contact resistance along the surface of individual wires was measured and found to be low. This indicates that initially, a good electrical connection exists between the wires. After removing the sheathing, the strand was immersed in a salt-water solution and corrosion of the strand/wires was observed. After the onset of corrosion, the contact resistance between the wires was observed to increase, but was observed to be unstable. Results from testing indicated that the contact resistance does not vary systematically as corrosion occurs. The contact resistance remains low throughout the corrosion process.

Based on the results obtained from tests with the Contact Resistance and Radio Wave Reflection techniques these test methods will no longer be pursued and are eliminated from the list of candidates for field evaluation.

3.3 QA/QC

All laboratory test procedures, and calibrations of instrumentation have been documented. A QA/QC Plan to document methods to be followed for testing, data acquisition and analysis, and equipment calibration will be developed. The QA/QC Plan will be developed and implemented by representatives from Geotechnics certified as an ISO 9000 Quality Lead Auditor.

Figure 3-1: Bench-Scale Test Facility.

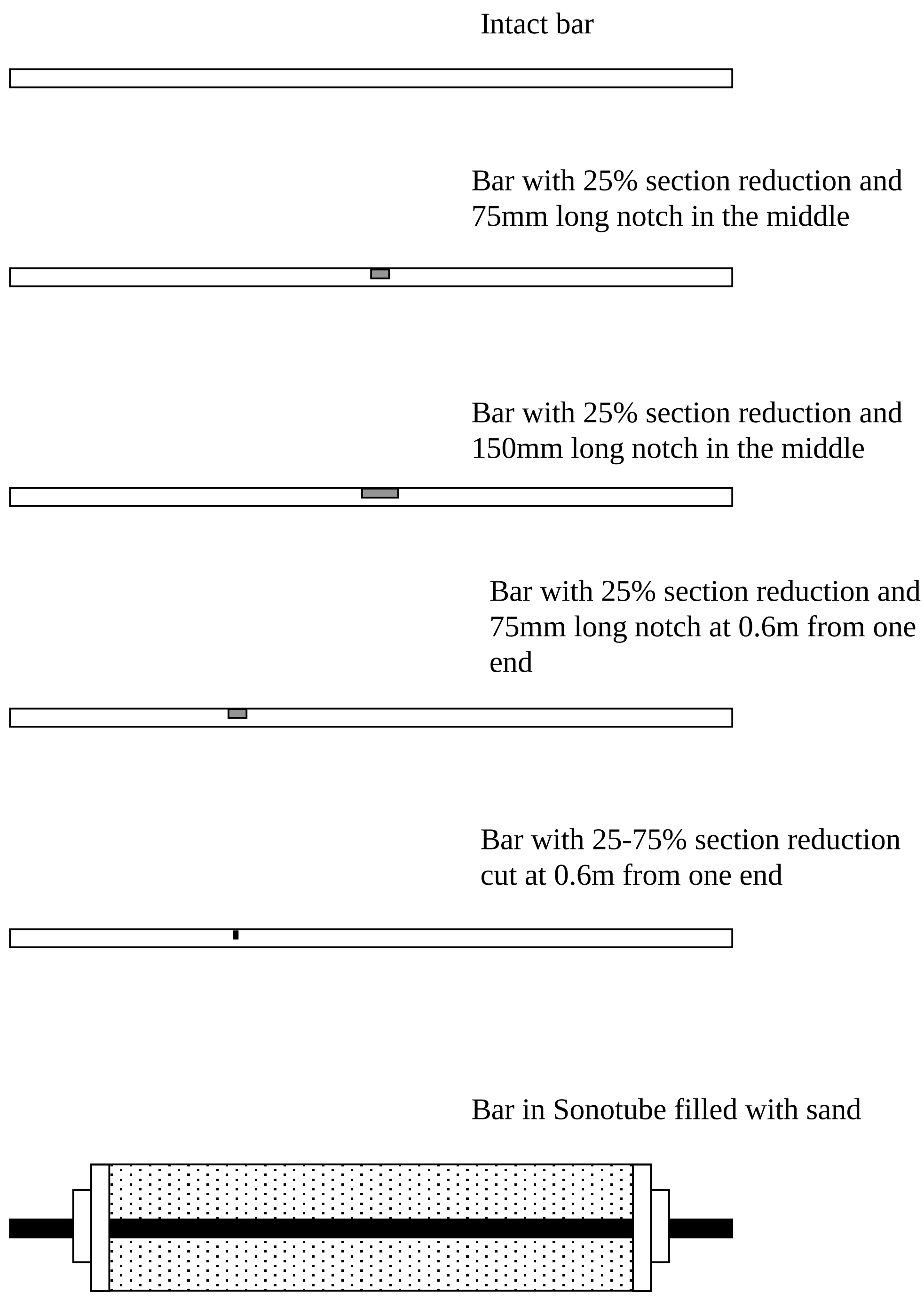

Figure 3-2: Bench-Scale Specimens- 2.0 m Long Plain Dywidag Bars.

Figure 3-3: Installation of In-Situ Specimens.

Figure 3-4: In-Situ Metal Specimen Testing Facility at U.B.

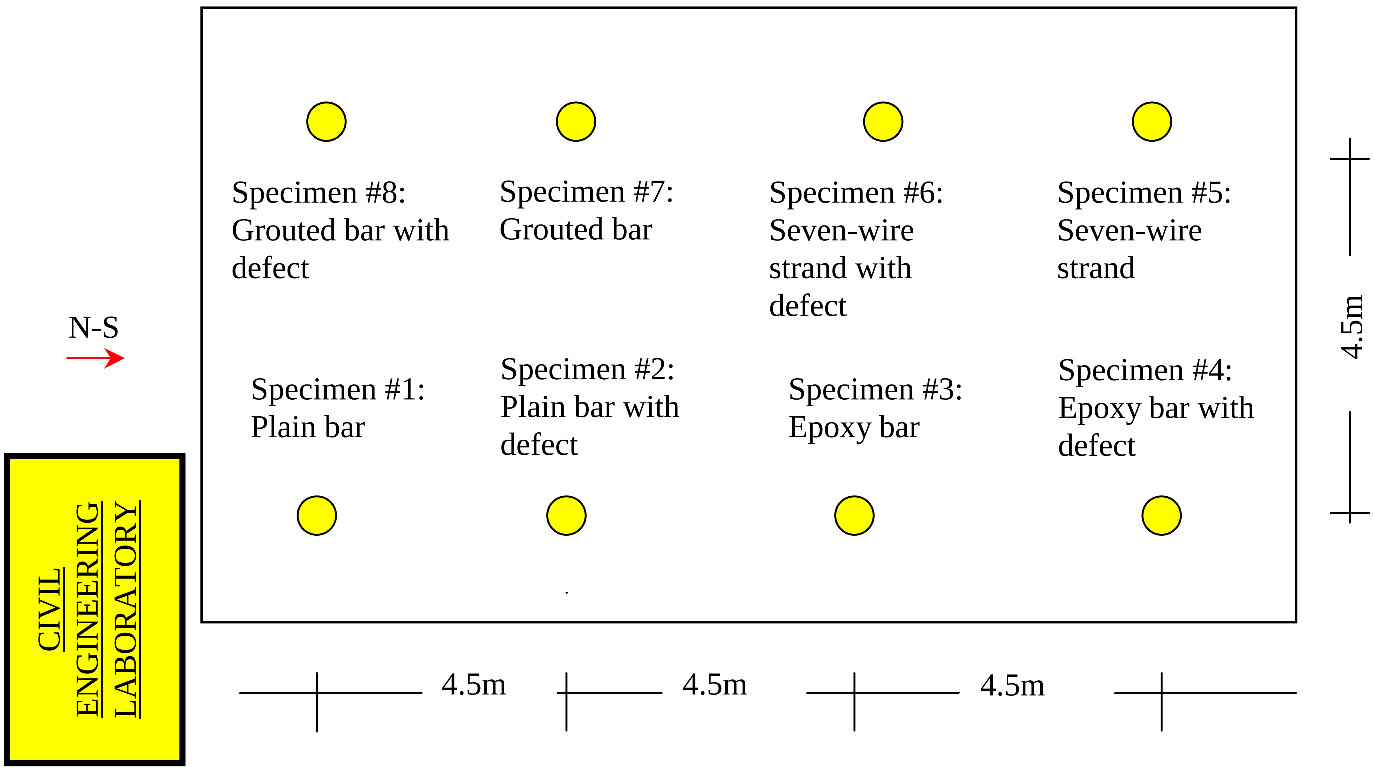

Figure 3-5: Plan View of In-Situ Specimens at U.B. Test Facility.

Figure 3-6(a): Installed In-Situ Specimen #1: Plain Bar.

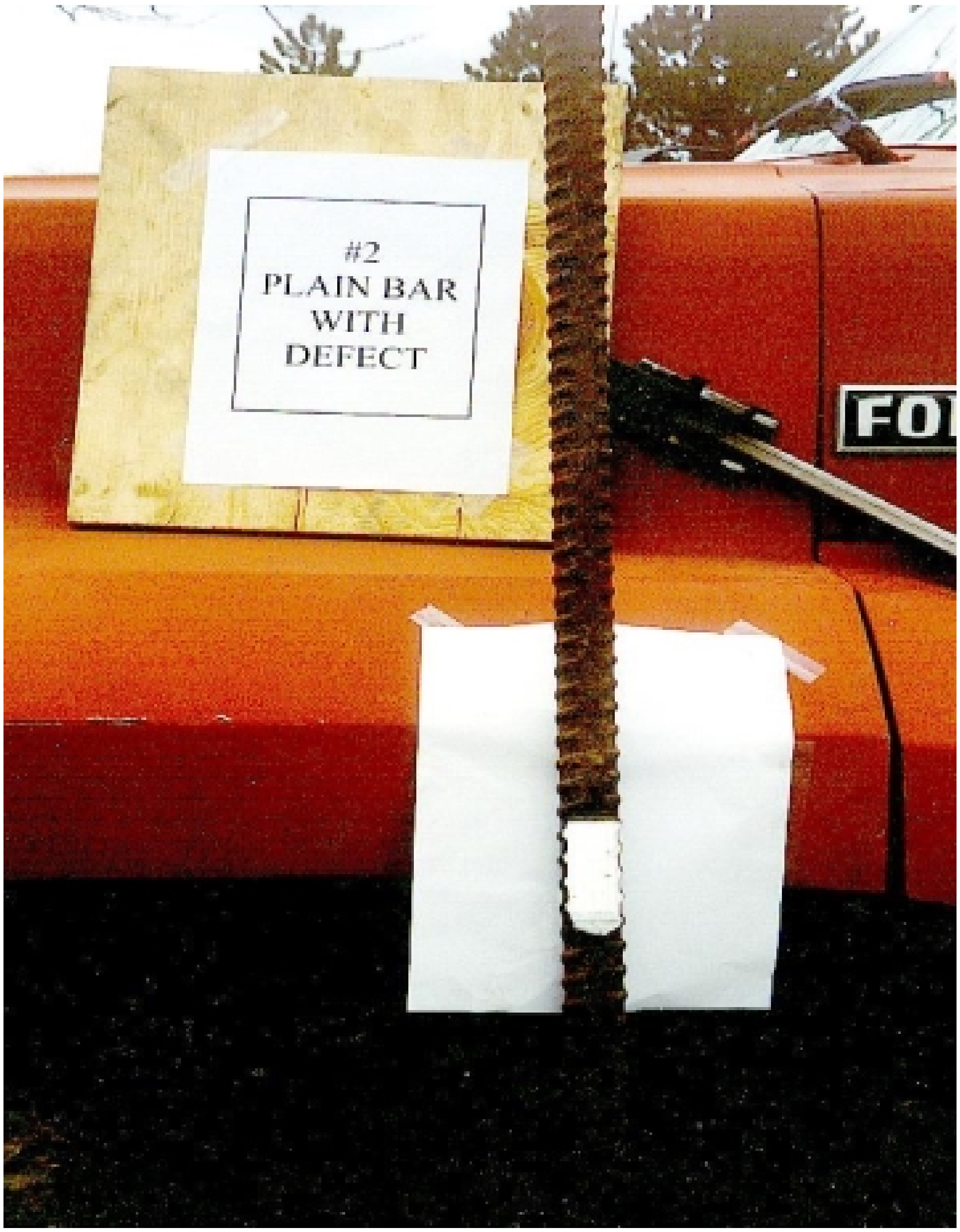

Figure 3-6(b): In-Situ Specimen #2: Plain Bar With Defect.

Figure 3-6(c): In-Situ Specimen #4: Epoxy Bar With Defect.

Figure 3-6(d): In-Situ Specimen #6: Seven-Wire Strand With Defect.

Figure 3-6(e): In-Situ Specimen #8: Grouted Bar With Defect.

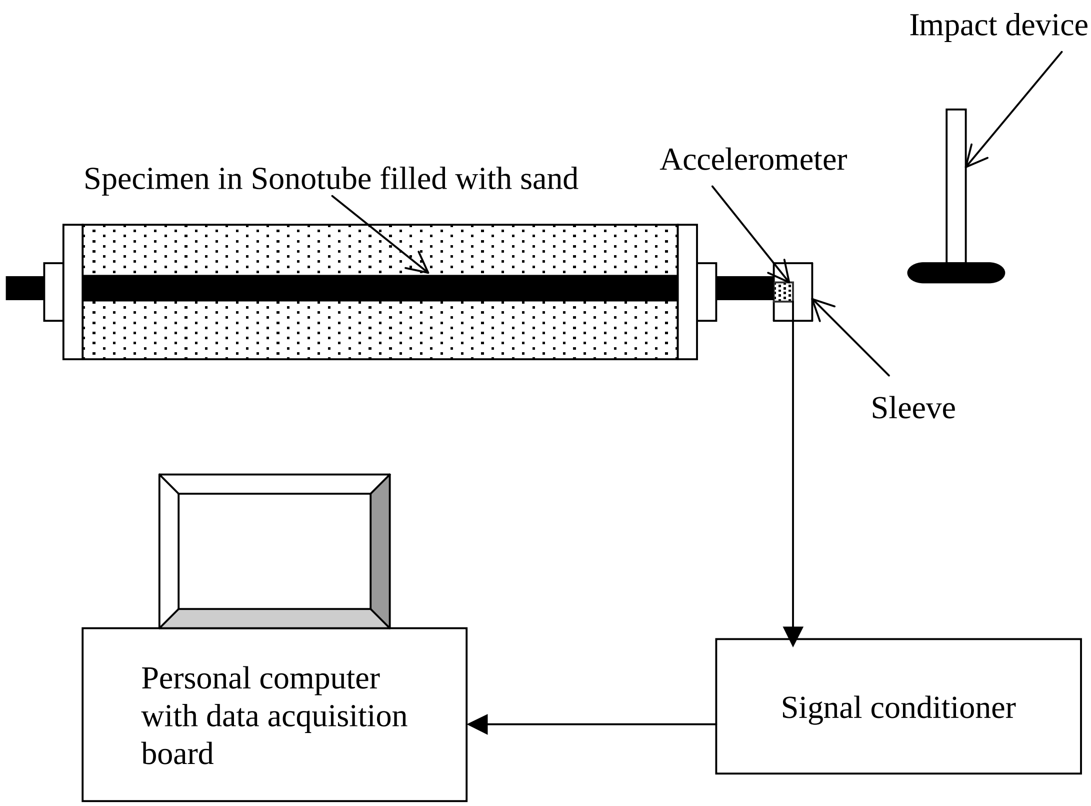

Figure 3-7: Schematic of Impact Echo Test.

Figure 3-8. Computerized Data Acquisition System Placed on Cart.

Figure 3-9: In-House Manufactured Pendulum With Ball Peen
Hammer.

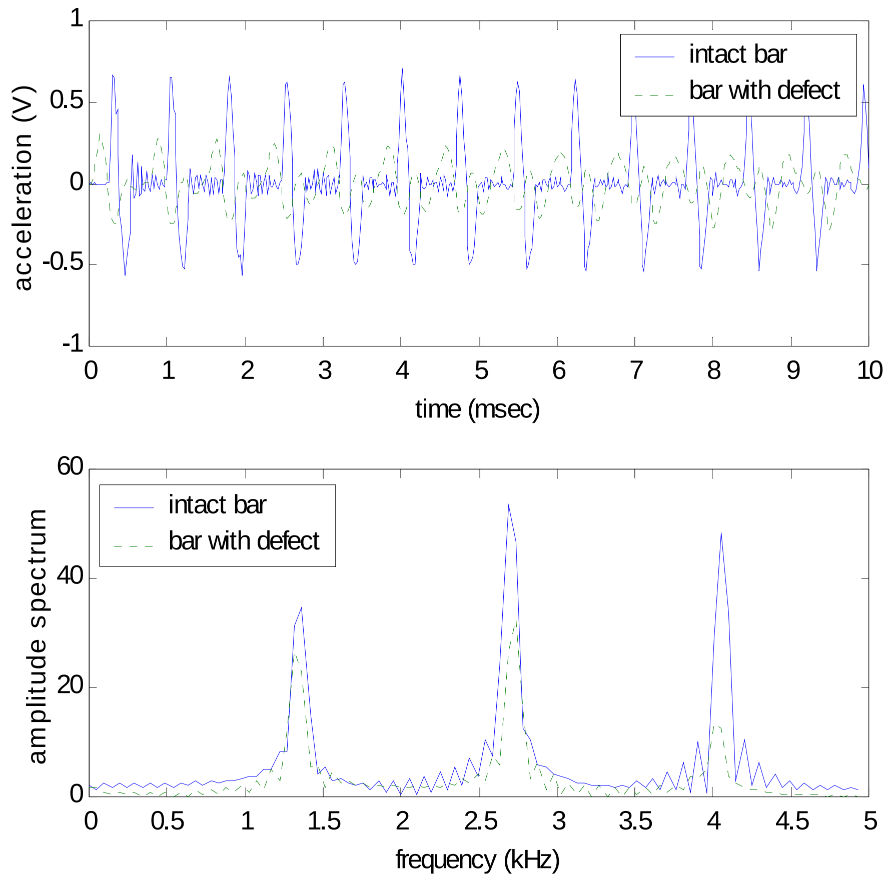

Figure 3-10: Time And Frequency-Domain Response for Impact Test on 2.0m Long Intact Bar and Bar with Defect (25% Section Reduction and 75mm Long Notch in the Middle) - Ball Peen Hammer Impact Mechanically Isolated Bars.

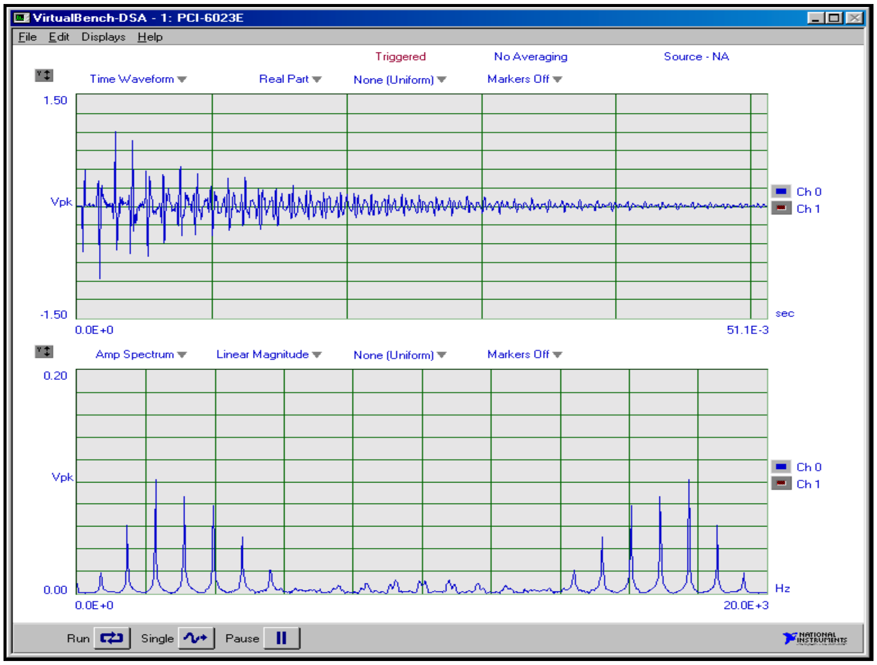

Figure 3-11(a): Impact Test Response for In-Situ Specimen #1- 3.0m Long Plain Bar; Impacted with Ball Peen Hammer.

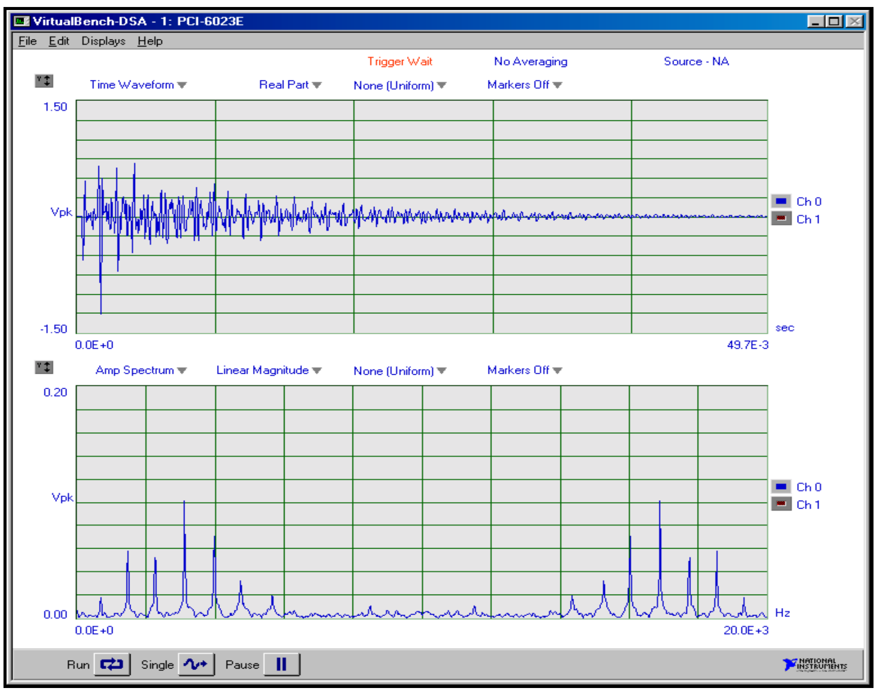

Figure 3-11(b): Impact Test Response for In-Situ Specimen #2- 3.0m Long Plain Bar with Defect; Impacted With Ball Peen Hammer.

Figure 3-12(a): Retrofitted Beam at Niagara Falls Parking Garage.

Figure 3-12(b): Tendon Anchorage at Niagara Falls Parking Garage.

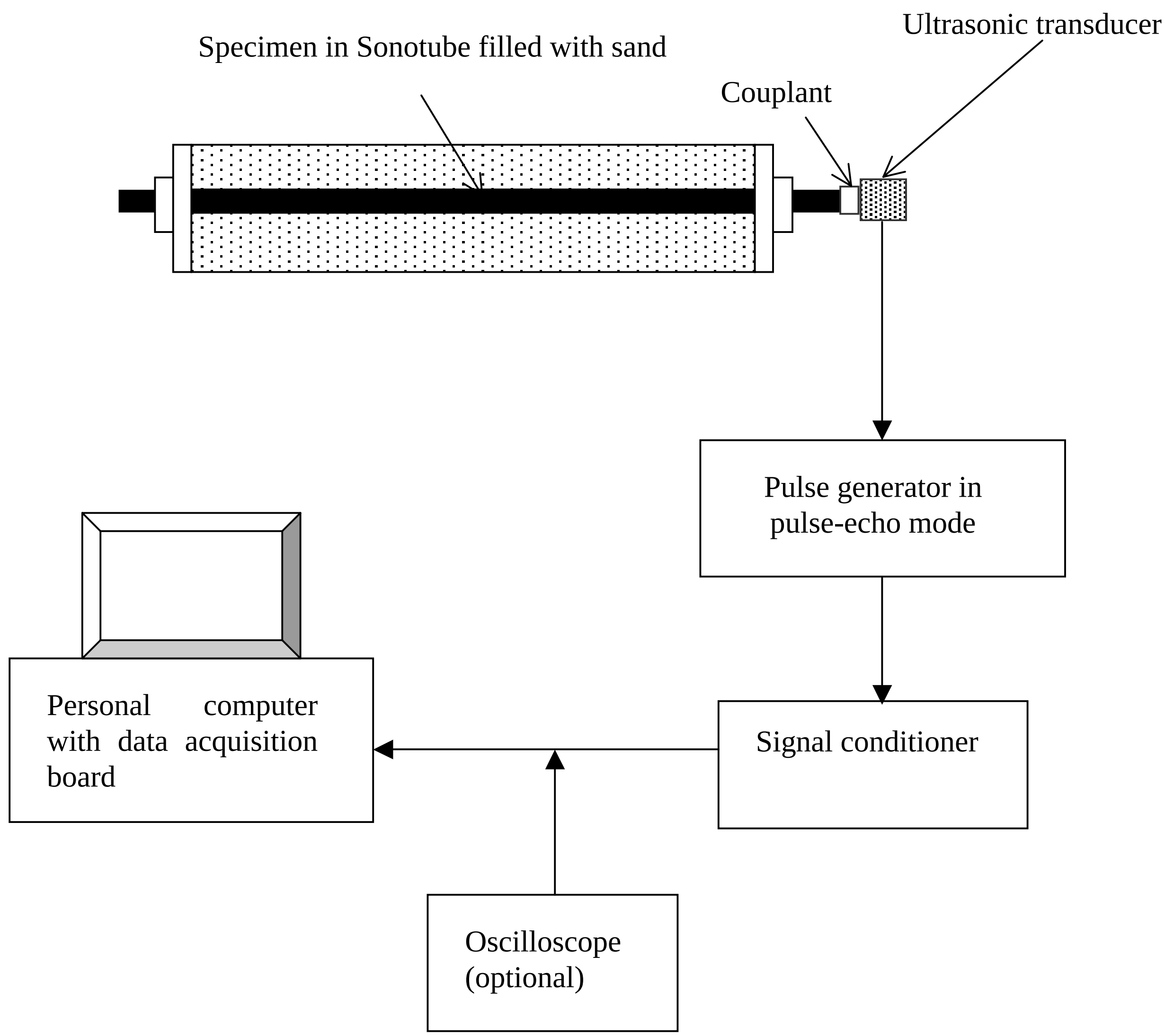

Figure 3-13: Schematic of Ultrasonic Test.

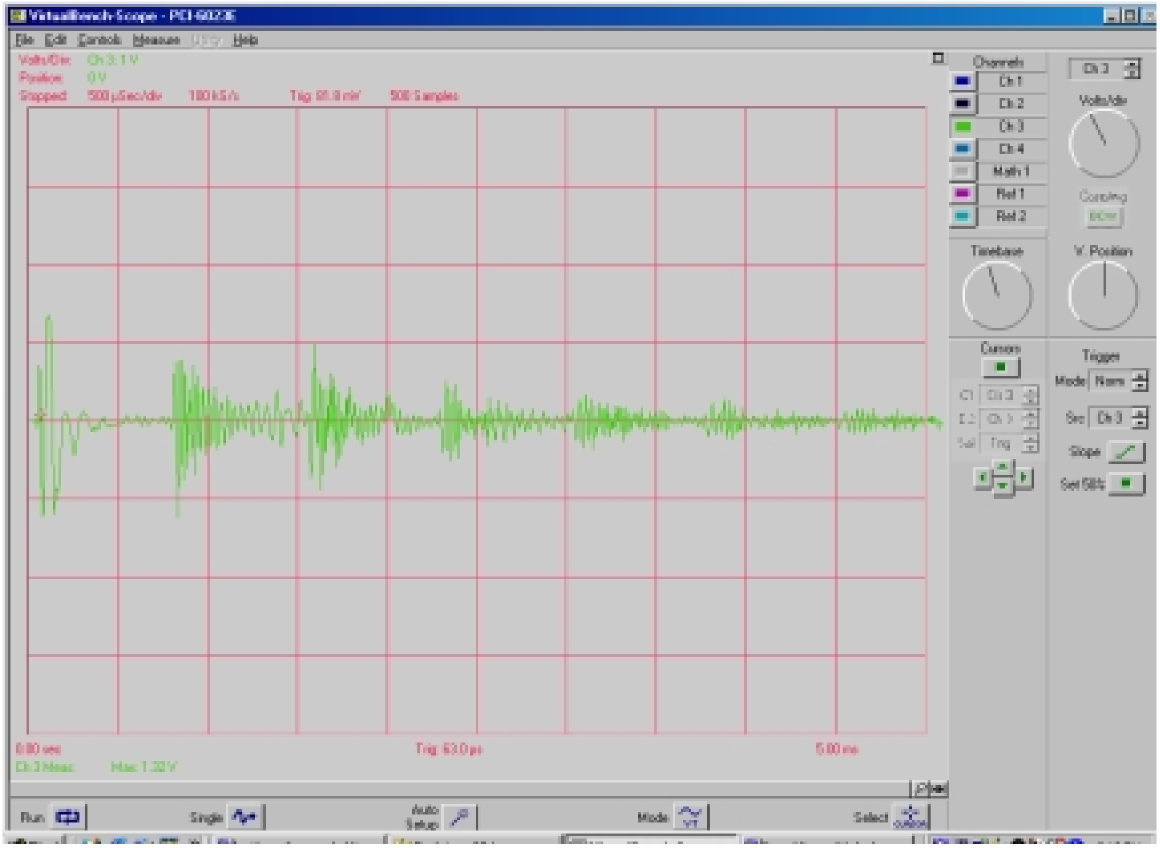

Figure 3-14: Ultrasonic Test Results (100 kHz Transducer) For Bench-Scale 2 m Long Plain Bar Without Defect.

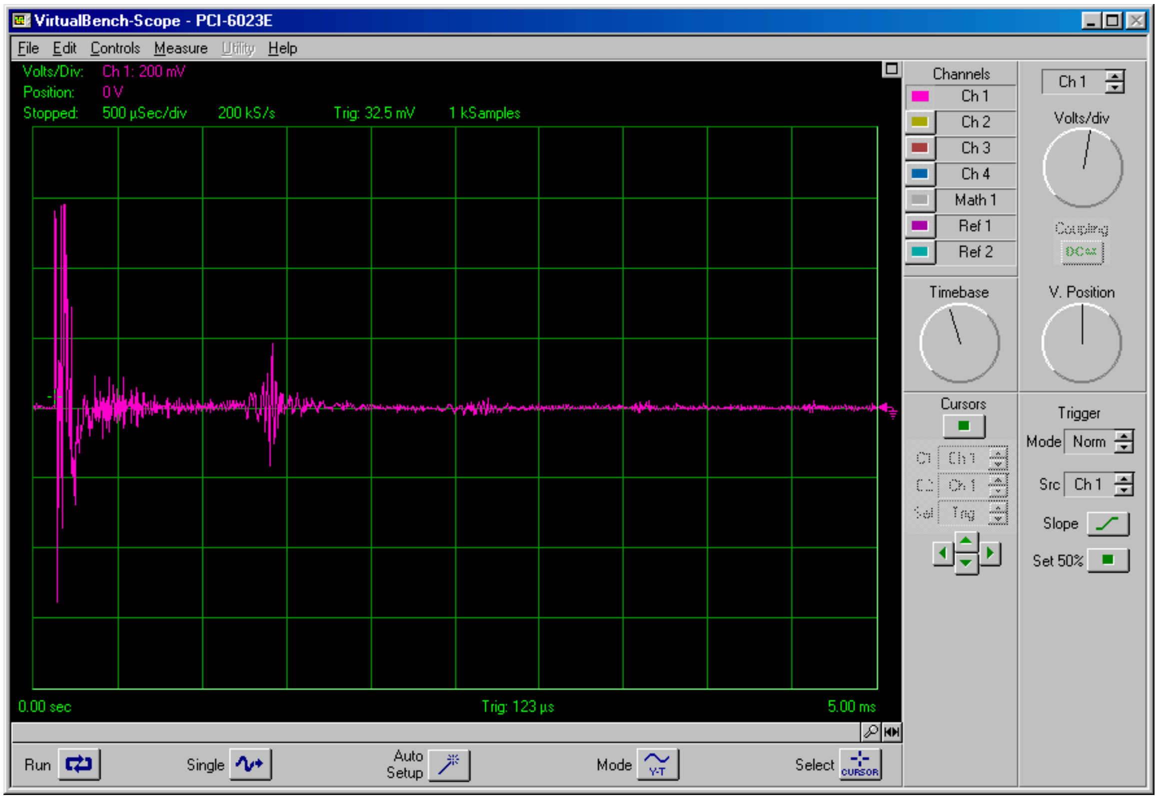

Figure 3-15 (a): Ultrasonic Tests Results (100 kHz Transducer) for In-Situ Specimen # 3; Epoxy-Coated Bar Without Defect.

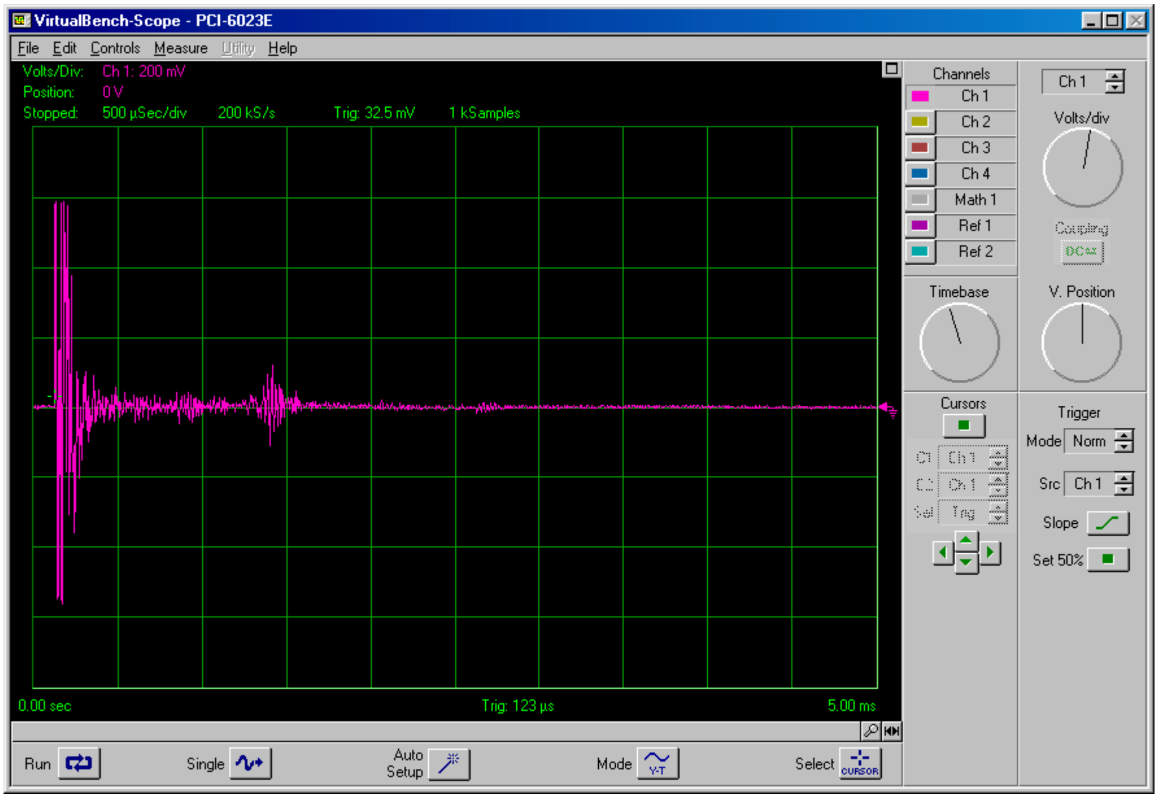

Figure 3-15 (b): Ultrasonic Test Results (100 kHz Transducer) for In-Situ Specimen #4; Epoxy-Coated Bar With Defect.

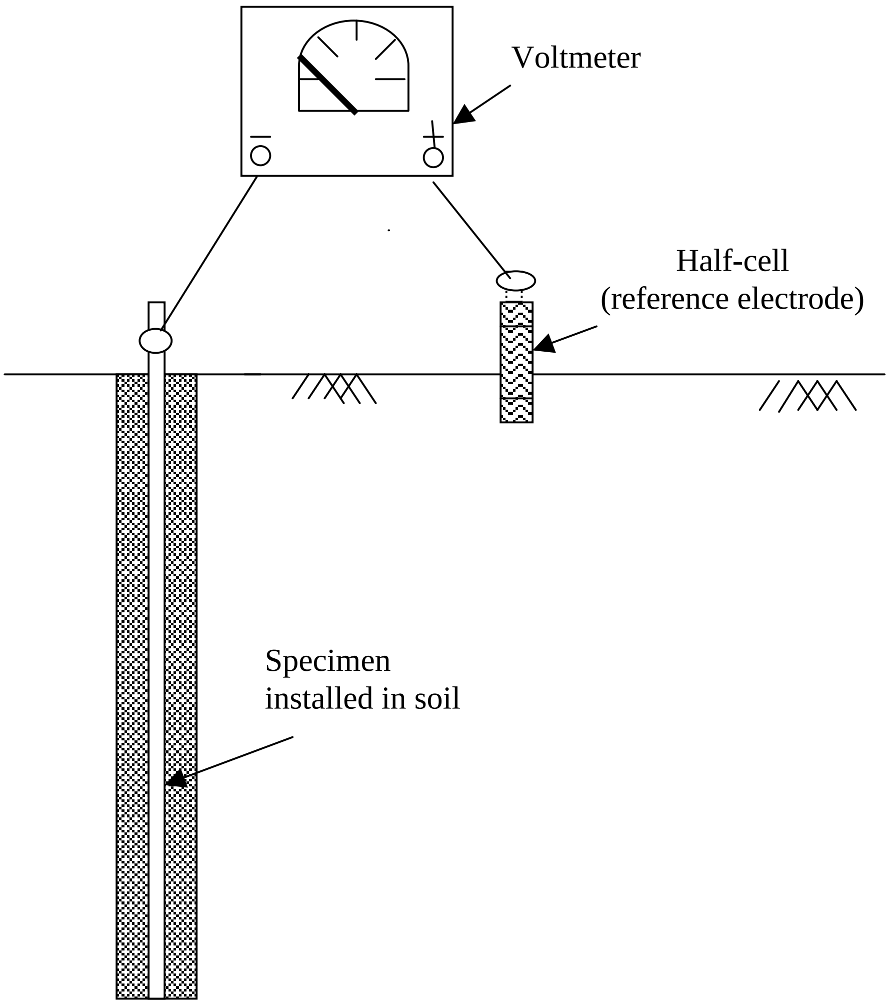

Figure 3-16: Schematic of Half-Cell Potential Measurements.

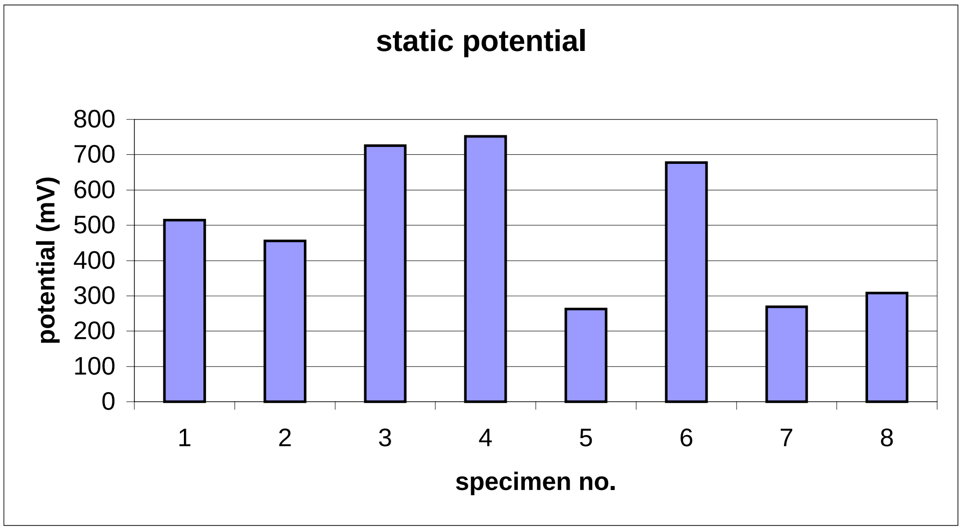

Figure 3-17: Half-Cell Potential Measurements for In-Situ Specimens.

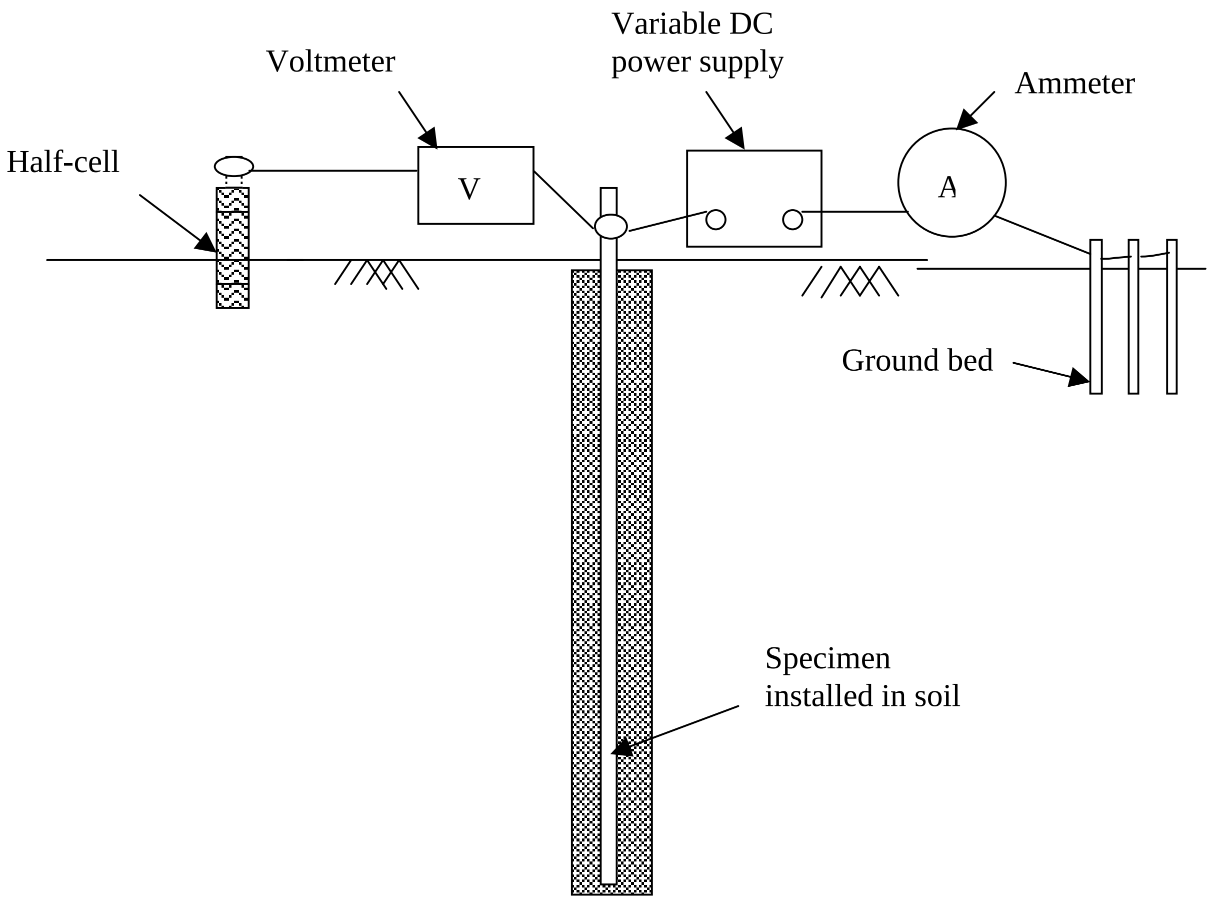

Figure 3-18: Schematic of E vs. Log I Measurement.

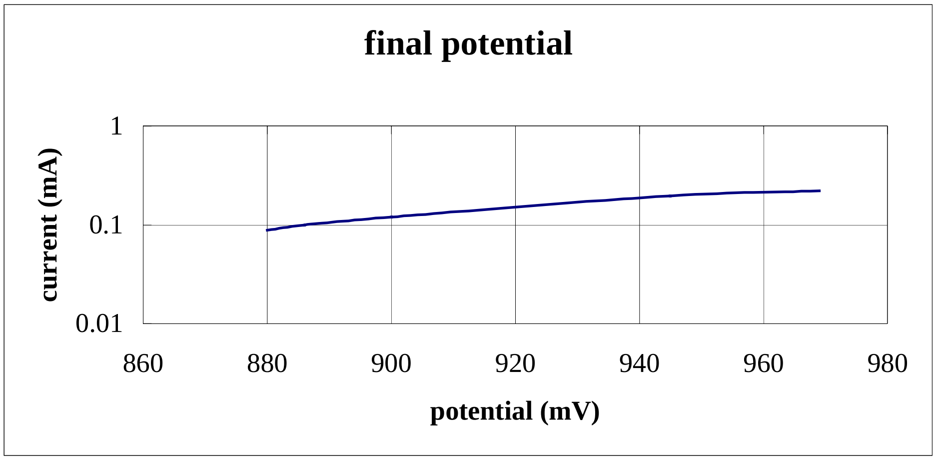

Figure 3-19(a): E Vs. Log I Measurement for In-Situ Specimen #3; Epoxy Bar Without Defect.

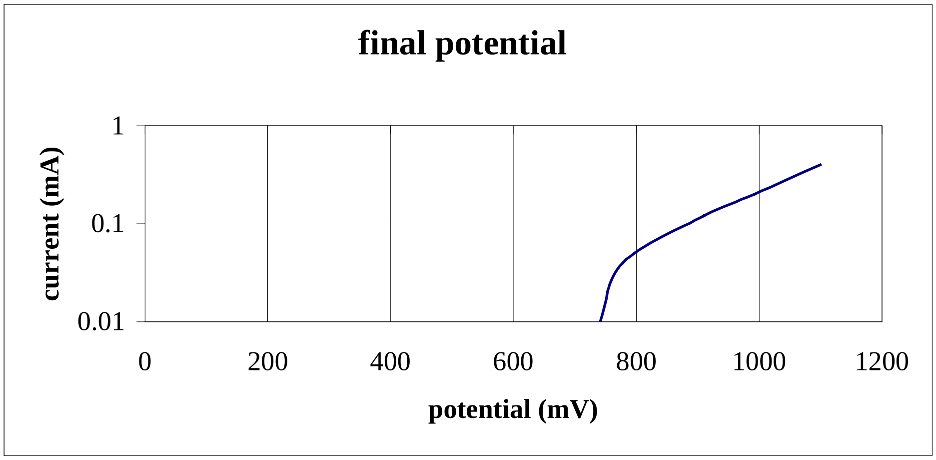

Figure 3-19(b): E Vs. Log I Measurement for In-Situ Specimen #4; Epoxy Bar With Defect.

4.0 MATHEMATICAL MODEL FOR SERVICE LIFE PREDICTION

The power law proposed by Romanoff (1957) for estimating the corrosion rate of buried metal elements, and the approach resulting from Project NCHRP 10-46 (Beavers and Durr, 1997) for estimating the service life of steel pile foundations will be adopted for this study. In their current form, these models do not incorporate all of the parameters that may affect the corrosion rate of buried metal-tensioned elements. However, details of the approaches, and the form of equations employed by the above referenced models will form the basic framework for the service life prediction model used for this study.

Romanoff (1957) proposed a power law to predict the rates of uniform corrosion of buried metal elements. Based on the power law, Elias (1990) proposed equations to predict loss of cross section due to uniform corrosion of buried galvanized and carbon steel soil reinforcements. Briaud et al. (1998) applied the equations of Elias (1990) to predict the service life of ground anchors that were not surrounded by grout. This model will also be applied to study the service life of existing systems, and to compare with field data that is compiled during the Phase II research effort.

Beavers and Burr (1997) used a statistical modeling approach to develop a regression model for predicting the corrosion of steel piling in soil environments. The following parameters are used to assess the corrosiveness of a site:

- Percents of sand, gravel and fines in soil profile
- Cation exchange capacity
- Soil pH
- Resistivity as received and saturated
- Soluble chloride
- Soluble sulfate
- Moisture content
- Corrosion rate calculated from galvanic current measurements
- Corrosion rate calculated from polarization resistance measurements
- Corrosion potential

The models proposed by Romanoff (1957) and Beavers and Burr (1997) do not consider, (1) localized or environmental cracking, (2) alternative types of corrosion protection applied to the tendon, and (3) the potential for bending of the tendon near the anchor head. These additional factors need to be regarded to adapt the models to ground anchors, rock bolts and soil nails.

The level of prestress applied to the tendon will be used as an index to evaluate the effect of localized or environmental stress cracking on the service life of buried metal tensioned elements. The vulnerability of tendons to factors that tend to compromise corrosion protection systems will be considered, and used to examine the observed level of performance in comparison to what would be expected without the benefit of corrosion protection.

Service life prediction models, that only consider the effects of corrosion, do not directly describe effects of bending of the tendon bars or strands near anchor head. When assessing the effects of cyclic loading, or the potential for overstressing, on the service life of the system, bending stresses must be estimated and added to the lock off load applied to the system. Anchorage efficiency and possible damage to the prestressing steel by the mechanical anchorage device also need to be taken into account.

5.0 SURVEY OF CURRENT PRACTICE

Details of recommended practice related to design, assessment of ground conditions, corrosion protection systems and current test requirements for metal-tensioned systems are described in Section 2.3. Section 5.0 presents additional details related to current practice including 1) product information and descriptions of metal-tensioned systems and anchoring techniques that are now installed or currently available for new installations, 2) results from a survey of highway agencies soliciting information on current practice, and 3) case studies that document the design, construction and performance of metal-tension systems.

Section 5.0 provides background information that is useful for developing the work plans described in Sections 6.0 and 7.0. The field investigations described in Section 6.0 and 7.0 should be performed at sites having installations and performances that are representative of the current practice described in this section.

5.1 PRODUCT INFORMATION

5.1.1 Prestressed Ground Anchors

Hanna (1982) and Xanthakos (1991) describe different ground anchor systems in use throughout the world. In what follows products available from manufacturers and distributors to the US markets will be described. The information is obtained from the manufacturer's literature.

Strand Tendons

There are three major manufacturers of the strand type tendons: (1) Lang Tendons, Inc. manufactures the Polystrand System, (2) Dywidag-Systems International (DSI), and (3) VSL Corporation (VSL). All three systems employ seven-wire strand conforming to ASTM A416 and steel grade with a minimum ultimate stress of 1860 MPa.

The Polystrand System is the most popular on the market. The Polystrand System is available in three nominal strand diameters including 9.5 mm, 13 mm, and 15 mm corresponding to a minimum guaranteed ultimate tensile strength (GUTS) of 102 kN, 182 kN, and 258 kN, respectively. The 15 mm diameter, 258 kN capacity, strand is recommended for ground anchors.

In partial fulfillment of the requirements for Class II protection (PTI, 1996), the strand free length is coated with corrosion inhibitor grease and surrounded with a seamless extruded HDPE sheath. The HDPE sheathing has a wall thickness of 1.5 mm. When Class II protection is employed, Lang Tendon, Inc. recommends electrical isolation at the anchor head.

For aggressive soil conditions, Class I protection is recommended. Class I protection is achieved with the addition of a grout filled corrugated plastic sheath around the bonded length. Also, mastic coating is applied to the anchor head assembly, which is covered.

Dywidag's multi-strand soil and rock anchor system uses a 15 mm nominal diameter strand corresponding to a GUTS of 260 kN. A variety of options for corrosion protection is available, considering either temporary or permanent applications. For temporary applications, all the criteria of the Class II protection (PTI, 1996) are met. The free length of each strand is coated with a layer of corrosion preventative grease over which is an extruded, seamless, layer of polyethylene. For permanent applications double corrosion protection is employed meeting the requirements for Class I protection (PTI, 1996). The bond length is either, (1) encapsulated in grout or (2) protection is provided by epoxy coating the individual strands, both externally and internally. If the free length is not surrounded with grout, the epoxy-coated strands are coated with a lubricating grease and encased in a seamless extruded polyethylene sheath.

VSL Corporation offers a 13 mm diameter strand, corresponding to a GUTS of 182 kN. The standard anchor system offered by VSL employs 52 strands with an ultimate capacity of 9550 kN. Anchor systems with less than 52 strands per anchor are also available.

For temporary applications, or in environments that would be "non-aggressive" to the anchor, VSL recommends use of a single corrosion protection system. Corrosion protection is achieved by surrounding the strands in the bonded and unbonded zones with grout. This system does not meet the criteria for class II protection (PTI, 1996) since there is no grease-filled sheath, or heat-shrink sleeve, surrounding the strand along the free length.

For permanent applications in aggressive soil environments VSL recommends use of a double corrosion protection system that satisfies the requirements of Class I protection (PTI, 1996). In addition to the requirements for Class I protection, the grease filled sheath along the unbonded portion of the anchor is surrounded by grout. VSL further recommends that the anchor hole is watertight before the system is put into place.

VSL offers a pressure bulb soil anchor for use in loose sands and gravel when sustaining an open hole is difficult or impossible. A casing is used to keep the hole open that is subsequently removed as the hole is pressure grouted with the strands in place. If necessary, this type of anchor can be double corrosion protected with the addition of a plastic sheath and tube.

Bar Tendons

There are a number of systems available that feature the use of prestressed bar anchors, including: 1) Dywidag-Systems International (DSI) Threadbar and Tie bars, 2) Williams Anchors manufactured by Williams Form Engineering Corporation (Williams), 3) Chance Anchors manufactured by A.B. Chance Company (Chance), and 4) the Manta Ray System manufactured by Foresight Products, LLC.

Bar tendons manufactured by DSI are either Threadbar Anchors, conforming to ASTM A722, or Grade 60 or 75 tie bars conforming to ASTM A-615. Both systems have a continuous rolled-in pattern of deformations along the entire length of the bar, which allows anchorage hardware or couplers to thread into the bar at any point. Threadbar is hot rolled high strength steel with two flat sides in the thread pattern. Threadbar Anchors are available in 26 mm, 32 mm, and 36 mm diameters. Threadbar is most commonly manufactured with a grade of steel corresponding to 1030 MPa ultimate stress, but 1100 MPa grade steel is also available. The possible selection of bar diameter and steel grade corresponds to ultimate tensile strengths between 567 kN and 1125 kN per anchor.

For temporary applications, DSI recommends that threadbar may be installed without corrosion protection. Unprotected anchors may be subject to corrosion, however, DSI states that the relatively large diameter and solid cross section of threadbar offers more corrosion resistance than smaller diameter, high strength, prestressing steel strands with a relatively larger surface area. For temporary applications, or for applications with nonaggressive ground conditions, single corrosion protection is available. Single corrosion protection satisfies the requirements for Class II protection (PTI, 1996), and, additionally, the unbonded length is surrounded by grout.

Double corrosion protection satisfying the requirements of Class I protection (PTI, 1996) is recommended for anchors with a long service life, for aggressive ground conditions, or where stray currents are expected. A corrugated high strength PVC sheathing is installed over the full length of the anchor with plastic end caps. The annular space between the threadbar and the PVC is fully grouted before the anchor is installed. To accommodate the bar elongation during stressing, a short length of threadbar is left free of the corrugated sheathing under the stressing anchorage. A steel pipe welded to the anchor plate and filled with corrosion preventive compound, or grout, protects the free end of the bar against corrosion. A smooth plastic sheathing is installed over the corrugated sheathing along the free length. This allows the tendon to elongate during stressing.

Tie bars are recommended by DSI for construction of heavy marine bulkheads, as an alternative to large diameter A36 tie bars with upset threads and turnbuckles. The DSI Grade 60 tie bars have a minimum yield stress of 414 MPa and are available with diameters ranging from 19 mm to 25 mm. For higher strength requirements, Grade 75 tie bars having a minimum yield stress of 517 MPa are available with diameters ranging from 19 mm to 57 mm. A larger, 63 mm diameter, Grade 80 tie bar with minimum yield stress 552 MPa is also available. Thus, tie bars are available with yield strengths ranging from 118 kN to 1747 kN. DSI offers several levels of corrosion protection for their tie bar systems, including: epoxy coating, galvanizing, heat shrink sleeves, cement grouting, and tar coating.

DSI also offers other special products, including a glass fiber bolt, and a self-drilling hollow core anchor. As an alternative to the steel bar tendon, DSI manufactures a threadbar from a resin mixture containing glass fiber. The glass fiber bolt offers several advantages over the use of steel bar. It is lightweight and has no propensity to corrode under adverse soil conditions.

The DSI self-drilling hollow core anchor (SDA), Type MAI, is a fully threaded hollow core bar that initially serves as a drill-rod with a lost bit, and later, once grouted in, becomes an anchor tendon that can be stressed and locked off with a hex nut. In certain soils, this system eliminates the need for casing. During construction, use of the SDA can save steps since advancing the hole and inserting the bar are not separate operations, and the SDA provides a conduit for grouting into soil or rock. The Type MAI anchors are manufactured with steel grade having an ultimate tensile stress of 600 MPa. Bars with outside diameters of 25 mm, 32 mm, and 38 mm are available corresponding to ultimate load capacities of 200 kN, 380 kN, 360 kN and 500 kN.

Williams anchors utilize all-thread-bar (Williams R71) manufactured in conformance with ASTM A-722, and steel grade corresponding to 1034 MPa ultimate stress. Continuous thread deformations, conforming to ASTM A-615 requirements, allow the bar to be field cut, coupled, or anchored at any location along its length. All-thread-bar is available with 26 mm, 32 mm, 36 mm, and 45 mm diameters. This corresponds to bars with ultimate strengths of 567, 834, 1054 and 1779 kN, respectively.

Williams offers three levels of corrosion protection designated as MCP I, MCP II and MCP III. Multiple Corrosion Protection (MCP) I and II satisfy requirements for Class II and Class I protection (PTI, 1996), respectively, except that an end cap is not included at the anchor head assembly. For MCP I the free length of the bar is surrounded by a grout filled sleeve. MCP II is similar to MCP I with added protection of grout filled encapsulation within the bonded zone. The highest level of protection that Williams offers is in their MCP III system, which is similar to DSI’s system for Class I protection.

The Chance anchor system uses screw type anchors that are advanced without the need of a predrilled borehole. By monitoring the torque required for installation, the capacity of the anchor system can be estimated using empirical formulas. The system uses a square central shaft with one to four rounded helices welded near the end. These helices are available in diameters of 152 mm, 203 mm, 254 mm, 304 mm, and 356 mm. Anchors are available with ultimate tension capacities that range from 312 kN to 668 kN. The shaft is advanced into the soil such that the load bearing helices are located beyond the potential failure plane associated with active conditions behind the wall face. Extension bars are then used to connect the helix portion to the retaining structure. In case of aggressive soil conditions, anchors are galvanized for corrosion protection.

The Manta Ray system is typically used to resist relatively light loads. Less equipment is required for installation and no grouting is necessary. The Manta Ray anchor system involves pushing a bar into the ground with a flat plate on the end. During advancement of the anchor tendon, the plate is parallel to the bar. After the anchor is set, the plate is adjusted into its final position, which is normal to the axis of the bar. In the in-service position, the plate acts as a deadman to develop passive resistance.

5.1.2 Soil Nails

Dywidag-Systems International (DSI) is one of the leading manufacturers of soil nails. They offer three types of bars including tie bars, DCR form ties, and Type MAI self-drilling hollow core anchors.

Dywidag offers tie bars for soil nailing, similar to those described in Section 5.1.1. Nails are available in either Grade 60 or Grade 75 steel. Grade 60 soil nails are offered with diameters of 19 mm, 22 mm and 25 mm; and grade 75 are offered with diameters of 19 mm through 35 mm. This selection results in a range of possible nail capacities at yield between 117 kN and 520 kN per nail.

The DCR form tie steel is cold rolled, high strength and fully threaded. The bar is manufactured from 965 MPa Grade steel and is available with either 16 mm or 22 mm nominal diameters corresponding to nail capacity at yield of 148 kN or 259 kN.

Details of the Type MAI self-drilling hollow core anchor are described Section 5.1.1. Given the nominal diameters available, nail capacity at yield is either 150 kN, 230 kN, 280 kN, or 400 kN.

A variety of corrosion protection systems are available for DSI soil nails. Often grout alone is a sufficient means of corrosion protection. Double corrosion protection is available for the tie bar and the DCR bar nails, and involves pre-grouting the nails in corrugated PVC sheathing before placing them in the excavation. This provides two layers of protection against corrosion. Epoxy coating is another method of protecting the nails from corrosive environments. For Type MAI bars, hot dip galvanizing is employed to restrict corrosion in non-aggressive ground, or for temporary applications.

Williams offer the R71 Grade 75 all-thread-bar described in Section 5.1.1, for soil nailing. Bar diameters ranging from 19 mm to 35 mm are available giving nail capacities at yield ranging from 150 kN to 520 kN. Simple corrosion protection relying on grout cover is available for temporary or non-aggressive soil applications. Multiple corrosion protection systems with the bar pre-grouted into a corrugated plastic sheath with end caps is available for more permanent applications, or in more corrosive soils.

5.1.3 Rock Bolts

Weerasinghe and Adams (1997) review developments in rock anchorage practice from 1976 to 1996. Strand tendons are the most popular choice in Europe; in the United States, epoxy coated strand and bar are more popular. The choice of corrosion protection depends ultimately on the specific anchorage system and its environment. Corrugated duct encapsulation is a common method of corrosion protection in the fixed anchor, whilst polyethylene sheathing and lay-flat sheathing are generally used in the free length.

Dywidag-Systems International manufactures a resin anchored rock bolt system. This system uses a two-component system of resin and catalyst to bond the bolt to the rock. Using cartridges separating the two components, the cartridges are placed into the drilled hole. After the anchor is inserted the resin and catalyst are mixed as the anchor is rotated. The fast gelling portion, placed within the bonded length, is allowed to set and the bar is prestressed, if desired, until the slow gel locks the load in place. Tendons are either threadbar or tie bars similar to those described in Section 5.1.1. Passive corrosion protection is provided by the resin grout surrounding the full length of the tendon.

Williams offers an extensive product line of rock bolts including three types of bars: hollow deformed bars, solid deformed bars, and solid smooth bars. Hollow core steels are used when cement grouting is the method of corrosion protection. Hollow deformed bars are anchored mechanically. The hollow core provides for full grout encapsulation, and allows grout flow from the lowest possible point. Solid deformed steel bars are used in situations where high tensile loadings are required. Anchors are either mechanical, resin or multiple grouted. Solid smooth steel bars may be anchored mechanically and used for temporary non-grouted applications, or for permanent grouted anchorage.

Hollow deformed bars are manufactured from steel having a minimum ultimate stress of 620 MPa or 855 MPa. The low-grade steel bars are available with an outside diameter of 51 mm corresponding to an ultimate strength of 890 kN. The high grade steel are available with outside diameters of 25 mm, 35 mm, or 51 mm corresponding to ultimate strengths of 294 kN, 614 kN, and 1334 kN, respectively. Hollow core all-thread-bar is also available with an ultimate stress of 855 MPa and nominal outside diameter of 25 mm corresponding to an ultimate strength of 356 kN.

Solid deformed bars are manufactured from Grade 60 rebar having a minimum ultimate stress of 620 MPa, or from 1034 MPa high ultimate strength steel. The solid rebar anchors are available in a range of diameters from 12 mm to 45 mm corresponding to average ultimate strengths ranging from 67 kN to 846 kN. The high ultimate strength bars are available in diameters ranging from 25 mm to 48 mm corresponding to average ultimate strengths between 400 kN and 1600 kN.

Solid smooth bars are available with steel grades having a minimum ultimate stress of 827 MPa or 861 MPa. A range of diameters from 12 mm to 51 mm is available, corresponding to an average ultimate strength between 80 kN and 1467 kN.

Williams offers several methods to bond the bolt to the rock. Their spin-lock mechanical anchorage system uses a device screwed onto the bonded end of the bolt, which spreads as the bolt is spun locking the bolt to the rock by friction. Their polygrout system is a mixable two-component grout that is placed into the hole in stages to allow for prestressing if desired. Resin anchoring is also available and follows the same practice as that offered in the DSI system.

5.2 PROJECT QUESTIONNAIRE

The project team conducted a survey of owners, designers, contractors, and researchers to solicit information regarding the design, construction, and performance of metal-tensioned systems. A questionnaire prepared for the survey is presented in Appendix V. The questionnaire was mailed to the appropriate representative at each state department of transportation, as well as knowledgeable designers, contractors and researchers throughout the United States. To further encourage responses, a project web site that includes the questionnaire was created on a server located at UB. The web site address is **http://www.civil.buffalo.edu/soilanchor**.

The following active committees, agencies, and industry groups were also contacted: (1) PTI Committee on Prestressed Soil and Rock Anchors, (2) US Nuclear Regulatory Commission, (3) ACI Committee 423-Prestressed Concrete, (4) United States University Council for Geotechnical Engineering Research (USUCGER), (5) ASCE Geo-Institute, (6) Association of Drilled Shaft Contractors - Earth Retaining Wall Committee, (7) American Rock Mechanics Association (ARMA). Each group was asked to distribute information to its members about the project, the questionnaire, and the web site.

To date a total of twenty-three responses to the questionnaire have been received. A summary and analysis of the responses is presented in Appendix VI.

5.3 CASE STUDIES

5.3.1 Rock Bolts

Underground Excavations

Sixty-eight case histories describing the design and installation of rock bolts are researched and summarized in EM 1110-1-2907 (1980). These case histories describe experience with roof bolting used in underground excavations completed since 1950.

Tie-Downs for Dams and Pumping Stations

Thompson (1969) described the use of post-tensioned anchors to strengthen the John Hollis Bankhead Dam in Alabama. Each anchor consists of 90 parallel, high-tensile strength (1654 MPa ultimate strength), 6.35 mm diameter, cold-drawn, stress-relieved wires typical of the BBRV system. The BBRV system, as described by Hanna (1982) and Xanthakos (1991), uses button-heads at the anchor head assembly. The use of the buttonhead assembly allows simultaneous development of forces in each wire during prestressing. This system is popular when large anchor forces must be attained.

Anchors were installed through the spillway into the foundation. Prior to construction, six test anchors were installed and loaded to 70% of their ultimate strength. Following installation, anchor corrosion was studied by extraction of several anchor wires that had been in-service for approximately one year. Some corroded areas were observed just below the anchor head, which was attributed to flowing water within the ungrouted free length portion of the anchor. The free length of the anchors was subsequently grouted.

During anchor installation strain gauges were placed along the anchor length and the distribution of load along the bonded and unbonded lengths of the anchor, was observed.

Cavill (1994) described installation of post-tension rock anchors to strengthen existing gravity-dams in Australia. The anchors use high strength prestressing strands fully encapsulated within polyethylene sheathing. All anchors have anchor heads with external screws to permit load measurement at any time using portable hydraulic cells.

Dehoux and Schupack (1984) described the installation of corrosion protected, prestress rock anchors to counteract buoyancy forces at the site of a pumping station. Anchors, up to 30 m in length, were installed, each with an ultimate capacity of 5 MN. For design, the ultimate bond stress between the grout and the anchor was assumed as 1.4 MPa. Each anchor consisted of 19, seven-wire strands having a diameter of 16 mm. The double corrosion protection system consisted of a watertight plastic sheath surrounding each anchor. The sheath was corrugated in the anchorage zone and smooth along the unbonded length of the anchor. Grout surrounded the steel strands inside the sheath, and filled the annulus between the outside of the sheath and surrounding rock. Spacers were used to centralize the anchors inside the tube, and the outside of the tube with respect to the surrounding rock.

Rock Slope: Ellenville, NY

On February 24, 1992, a rock-fall occurred on Route 52 in Ellenville, Ulster County, New York. The rock-fall involved 475-650 m^3 of rock that had been stabilized with rock bolts in 1972. Rock bolts were 19 mm nominal diameter, expansion shell anchorage type, manufactured by Bethlehem Steel and conforming to ASTM A306 (now A675) grade 80 standards.

As reported by Johnston (1996), twenty-two rock bolts were recovered from the slide area. Laboratory measurements performed on each included photographs, length, diameter, and dimensions of visible pits (diameter and depth). 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% to 27%. None of the bolts exhibited a yield point, indicating that the notch effect of the corrosion or some alteration of the steel crystalline lattice structures had embrittled the steel.

No direct correlation was found between loss of material (cross section) and loss of load capacity. No service life prediction was described other than manufacturer’s recommendations.

5.3.2 Ground Anchors and Tie-Backs

UK Ministry of Public Works

Wilcox (1981) described some experiences of the Ministry of Public Works in the UK with the use of permanent anchors. Two of the more interesting cases described are the Lower Shotover Bridge and a retaining wall along the Auckland Urban Motorway.

Rock anchors at the Lower Shotover Bridge experienced problems due to loss of grease cover. The grease tended to melt in the sun and leaked out through minor cracks during transportation of tendons to the job site. Exhumed rock anchors were not completely straight and some were bent 56° with respect to the horizontal.

Along the Auckland Urban Motorway, tie-backs in a retaining wall were reported to have failed due to corrosion near the anchor head assembly. The BBRV tendon system was employed, which consists of 7 mm diameter parallel wire tendons. The steel wires were cold drawn, and had an ultimate stress of 1500 MPa. Each greased wire was covered by an extruded polypropylene sheath. Reportedly, 5 wires failed out of a total of 3100. One wire buttonhead, protruding 12 mm from the anchor head, was pulled out and found to have broken at a corroded area about 8 m from the end of the anchor. Evidence indicated that the polypropylene sheathing was damaged about 8 m from the end, and the grease coating failed to protect the exposed wires. The grease coating was found to be hygroscopic. Corrosion also occurred within one meter of the anchorage. All samples of wire showed the same general pattern of corrosion, although the extent varied. Lab tests revealed the possibility of hydrogen embrittlement.

WVDOT - Wall Failure in Charleston, WV

WVDOT (1983) described the construction and failure investigation of an approximately 30 m length of retaining wall along the North bound lane of I 77 in Charleston, West Virginia. The failure occurred at 1:00 p.m. on Thursday November 25, 1982.

The approximately 10 m high retaining wall was a tied back structure having two levels of tie-backs, which were mechanically anchored into rock. The wall face was reinforced concrete with a thickness of 610 mm. An aggregate backfill (AASHTO No. 57 stone) was placed between the backface of the wall and the nearly vertical rock face. The width of the backfill was approximately 0.61 m at the base of the wall and 3m near the top.

The rock anchors were Williams Hollow Core Groutable Rock Bolts (Type US-8-HC-LCSF-175). These bolts have a 25 mm diameter, a yield load of 165 kN, and an ultimate load capacity of 223 kN. The rock bolts were installed at a 30° angle to the horizontal and anchored into rock. After grouting and prestressing to the design load of 111 kN, approximately 13 mm of grout cover surrounded the bolts. At the wall face, the rock bolts were coupled to AISC C 1050 steel bars, which were unstressed.

The report describes the cause of the failure as an overload of the tie-backs near the bottom of the wall. Possible loads on the tie-back were due to lateral earth pressure from the aggregate backfill, relaxation of the rock face, and cyclic loads from effects of temperature and varying moisture content within the aggregate. Due to the effect of cyclic loading, the bolts failed when the estimated load was greater than the yield strength, but still less than the ultimate strength of the bolts. Although some corrosion of the bolts was observed, it was not thought to be a contributor to the cause of failure. Rock bolt failures were near the rock face; either in front of, or behind the stressing plate, or within the extension. Not all rock bolts exhibited evidence of fracture at the time of failure, but may have fractured some time before failure of the wall.

Tunnel Portal at Elko, Nevada

A case study, or report, documenting this failure was not prepared. The following information was obtained from a conversation with Mr. Dave Weatherby (1998) and correspondence with the Nevada DOT. Nevada DOT provided a copy of the specifications that applied to the original installation of rock bolts in 1972 and installation of replacement bolts in 1982.

In 1972, rock bolts were installed to support the abutments to a tunnel portal along Interstate 80 in Elko Nevada. A concrete retaining wall was constructed in front of the rock face, and gravel backfill was placed between the wall and the rock face. The gravel backfill reportedly contained soluble salts and, when wet, became highly aggressive with respect to corrosion. The rock formation that provided the anchorage for the bolts is sedimentary rock with steeply dipping bedding planes.

The original rock bolts used on the project were 25 mm diameter, A306, Grade 80 bars with an expansion shell anchorage. The bolts were prestressed to 50% of their ultimate strength and grouted within the rock after post-tensioning. No other corrosion protection was employed. The original bolts were replaced in 1982 after corrosion failure. The bars were replaced with high-strength steel bars conforming to ASTM D722. A grouted anchorage was employed, and the bars were greased and sheathed along the free length. A trumpet assembly was installed at the anchor head, after post-tensioning of the bolts the free length was grouted.

No problems with this wall have been reported since the new rock bolts were installed.

Anchored Concrete Diaphragm Wall: Washington, D.C.

In 1978, a concrete diaphragm wall was constructed in Southeast Washington, along O Street SE from Carpenter Street to Branch Avenue. The wall was constructed by ICOS Corporation of America to restrain a gradual landslide. The wall is 366 m long and ranges in height from approximately 5.2 m to 9.1 m, and supports a sloping ground surface behind the wall. The base of the wall is embedded to a depth ranging from 4.9 m to 5.8 m. 100 mm diameter weep holes 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.

The subsurface conditions near of the wall consist of interbedded and irregular layers of sand, clay, and gravel to an elevation of approximately 39.6 m. Also, in this layer are somewhat irregular, wet, medium dense sand lenses. Directly beneath this layer is a clay, with slickensided zones.

The reinforced wall face is 610 mm thick, with 25.4 mm diameter Dywidag tie-backs (1 MPa). Each tie-back was designed to carry 445 kN, and was reportedly installed at angles ranging from 0° to 16° from horizontal. Panels 10, 11, and 12 each consist of two rows of tie-backs, totaling 7 tie-backs per panel. Panels 13 through 20 each have one row of three tie-backs. The installed tie-backs were reportedly 9 m to 34.5 m long and were anchored in a silty, sandy clay stratum.

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.

Shortly after construction several anchors failed by stress corrosion cracking. High tensile stress due to bending was present in the bars due to misalignment of the anchor nuts and lack of a bearing washer with a seat to accept the anchor nut. This problem was remedied by replacing several of the original anchors and retrofitting the bearing washers on the remaining anchors.

No corrosion protection was applied to the anchor head assemblies. The free length of the anchor bars are greased, sheathed and encased in grout with approximately 86 mm of cover.

The wall began to show evidence of failure in 1995, and there is evidence of wall movement and signs of distress at several anchor locations. The impending collapse threatens several homes located behind the wall. Seven SPT borings were advanced during a 1996 subsurface investigation of the site. Four of these borings were installed as monitor wells, and three have inclinometers installed in them. In addition, survey points were installed. Resupport of the wall is scheduled for the summer of 2000 by construction of a new anchored wall structure in front of the existing wall. At that time, some of the tie-back anchors will be exhumed.

Coeur d'Alene Idaho

Representatives of the Idaho DOT (IDOT) were contacted about the construction and observed behavior of a tied-back wall along I-90 East in the Coeur d'Alene district. The approximately 30 meter high wall is the first major tied-back wall constructed within Idaho. The wall is a soldier pile and lagging system whereby the soldier piles are toed into a jointed rock mass. The tie-backs are grouted tendon anchors that employ a double corrosion protection system. During construction, seeps along the jointed rock and excessive wall movements were observed. In response to this observation, a drainage system, additional tie-backs, and secondary post-tensioning were introduced at the site. Load cells were installed and have been monitored at regular intervals since construction of the wall. Although some results were presented at the 1987 Highway Geology Symposium, a comprehensive report has not been published. Bill Capaul from IDOT thinks site this will be a good candidate for our Phase II evaluation, and load cell data could be made available to the project team.

British Columbia

Representatives of the British Columbia Department of Transportation (BCDOT) were contacted about the failure of a suspension bridge cable for a bridge crossing the Peace River. The bridge was constructed shortly after the Second World War and the steel used at the time may not meet existing standards. Failure of the cable occurred in the 1950's and it is not known if this was a corrosion related problem. There are no known reports that document the performance of the bridge.

NGES: Texas A&M Wall

In 1991, a 7.5 m high, 60 m long, tied-back soldier pile wall with wood lagging was constructed at Texas A&M University's National Geotechnical Experimentation Site (NGES) in College Station, Texas. The wall was divided into two symmetric sections: one half of the wall is supported by a single row of ground anchors while the other half is supported by a two rows of ground anchors. Each section was further divided into drilled shaft and driven pile sections.

Soils at the site consist primarily of alluvial sand deposits. Soils were classified as loose silty-sand 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. Groundwater was observed at a depth of 9.5 m below the top of the wall.

Piles were spaced at 2.4 m center-to-center and embedded to a depth of 1.65 m below the base of the excavation. Wide-flange and H-pile sections were driven to the required embedment depth using an air hammer. Drilled shafts piles were constructed using 457 mm or 610 mm diameter casing. Casings were placed using a vibratory pile hammer. Wide-flange beams or H-piles were placed in the center of each drilled shaft, which were backfilled with Class A structural concrete, or a lean-mix backfill material. Casings were pulled during backfilling.

Generally, excavation was performed in 1.2 m thick lifts. The wood lagging was used to support the soil between the soldier piles during excavation is 76 mm thick. Dywidag tendons, manufactured from Grade 150 prestressing steel, having diameters of 32 or 35 mm were used as wall anchors.

A tendon length of 12 m with a bonded length of 7.3 m was required for the single-tier wall. The two-tier wall required a total tendon length of 12.8 m with a bond length of 7.3 m for the uppermost set of anchors, and 12 m total with a bond length of 7.4 m for the lower. All anchors were installed at 30° to horizontal, and were pressure grouted in the bonded zone.

The ground anchors were installed by driving an 89 mm diameter closed-end casing to the required depth. The tendon was placed in the casing and the closure point driven off. Cement grout was pumped into the shaft as the casing was pulled. All of the anchors were prestressed and proof load tested. No special corrosion protection measures were used for anchor construction as the intent of the study was to monitor relatively short term performance of the wall sections.

The wall was instrumented with strain gauges, load cells, and inclinometers. Geokon Model VSM-4000 vibrating wire strain gauges were attached to the soldier piles. Inclinometer casings were installed at each instrumented soldier pile, between instrumented piles, and behind the wall. Geokon Model 400 vibrating wire load cells were used to measure anchor loads.

5.3.3 NYSDOT Case History of Soil Nail Wall Project

Denniston (1993) describes the design, construction, and monitoring of a soil nail wall along Route 23A in the town of Hunter, Greene County, New York. The wall supports a cut on the North side of NYS Route 23A. This project represents the first soil nailed wall constructed by the NYSDOT. A value-engineering alternative was proposed to replace an anchored soldier pile and lagging wall with a soil nail wall.

Soil at the site is a glacial till described as silty gravel, sandy with occasional silty clay. The wall height ranges from 3.65 m to 8.54 m. The internal stability of the soil nailed mass was analyzed using a limit equilibrium stability analysis, and the external stability was analyzed by treating the reinforced mass as a gravity wall. In general, external stability considerations controlled the nail lengths.

Soil nails were epoxy-coated, #9 to #14 reinforcing steel bars conforming to ASTM A615, with yield strength of 413 MPa. The soil nails were inserted into 140 mm diameter drill holes. Verification tests (tension tests) were performed on production anchors and on special test anchors. During construction, as the excavation advanced beneath the previously placed wall facing, problems with sloughing of soil were encountered.

The performance of the wall was monitored with two inclinometers placed 3 m behind the top of the wall. Inclinometer readings were taken before, during, and after construction. After completion of wall face, the following instruments were installed: extensometer (2), piezometer (2), ceramic tilt plate and micro-level (2), brass tilt plate and micro-level (2). Observations indicate that wall movement occurred during construction, but subsequently no significant wall movements have been documented.

6.0 WORK PLAN FOR EVALUATION OF EXISTING SYSTEMS

A detailed work plan is described in this section for the field investigation of existing metal-tensioned systems representing a range of system types, subsurface conditions, and ages to validate the measurement techniques described in Section 3.0 and the models for estimation of remaining useful life identified in Section 4.0. Preference is given to selection of sites where subsurface conditions are well defined, construction details are known, access to the metal-tensioned elements is reasonably available, evidence of good or poor performance is apparent, and travel expenses are reasonable. For each potential site the following information is documented to the extent possible; 1) type of metal tensioned system, 2) corrosion protection installed, 3) age of metal-tensioned system, 4) soil types, pH, chloride and sulfate content, 5) climate data, 6) grout type, 7) level of prestress (where applicable), 8) presence of stray ground currents, and 9) availability of performance data. If the above information is not yet available for a site selected for the study, it will be obtained as part of the field investigation conducted during Phase II.

The project team explored opportunities at sites where exiting ground anchors, soil nails or rock bolts are available for NDT, monitoring and evaluation. Sources of information include: (1) responses to the questionnaire described in Section 5.0 of this report, (2) the NYSDOT rock-bolt inventory and NYSDOT data on soil nails and ground anchors, (3) discussion with Nicholson Construction Company, (4) discussions with representatives of PADOT, (5) discussions with representatives of the Idaho DOT, (6) the NGES data base, and (7) on-going projects that the Project PI and CO-PI are familiar with.

Based on information obtained from these sources we are planning field studies at <u>rock bolt sites</u> in New York; <u>ground anchor sites</u> in New York, Pennsylvania, the District of Columbia, Coeur d'Alene Idaho, Pittsburgh, PA, Buffalo, NY and at the NGES site at Texas A&M's Riverside Campus; and <u>soil nail sites</u> located in Pennsylvania and owned by PADOT**.** These sites are representative of the range of systems, subsurface conditions, and ages typically encountered. We will confirm the availability of each potential site by calling contacts for each site and securing written confirmation for the availability of the site at the time the field work is anticipated.

6.1 ROCK BOLT SITES

The NYSDOT provided the project team with an inventory of rock bolt sites in New York State. The list includes 213 sites, mostly located along the north-south Transportation Corridor at the eastern side of the state. Details for 2 255 rock bolts are documented in the inventory, representing approximately 13,100 linear meters of bolts. Based on the information contained in the inventory, approximately 22% are expansion shell anchorages, 42% are polyester resin grouted end anchorages, and 36% are slot and wedge anchorages. The ages of the rock bolts included in the inventory ranges from six to 37 years. The majority of the rock bolts installed after 1985 are resin bonded.

A study of the NYS rock bolt inventory revealed that along several highways in particular there are different rock bolt installations featuring a range of installation dates from 1969 to 1994 and a variety of anchorage types. On January 22 and 23, 2000, Dr. Fishman conducted a reconnaissance survey of selected rock bolts installations in the eastern part of NYS with Doug Hadjin, a geologist from the NYSDOT, who assisted in locating and providing background information about each site. The goal of the reconnaissance trip was to identify a section along one alignment that may provide ease of access and a good cross section of different rock bolt parameters that may be included in the field study.

Sites of existing rock bolt installations were visited in three general areas. The first area was in NYSDOT Region 1, North of Albany, along NYS Route 22, in the Taconic Mountain Range. The second area was in NYSDOT Region 8, located along NYS Route 52 in Ellenville New York, in the Shawangunk Mountain Range. This site is described as a case study in Section 5.3.1. The third area is in the Hudson Highlands, near West Point Academy and Bear Mountain State Park, which is also in NYSDOT Region 8.

6.1.1 NYS Route 22- Taconic Mountain Range

Three rock bolt installations were observed along NYS Route 22. According to the NYSDOT database, these bolts were installed in 1980. Rock bolts were installed in this area to stabilize outcrops of phyllite with steeply dipping bedding planes. The density of the rock bolt patterns was not high, and usually two to three rock bolts were observed for each block of rock being stabilized.

Bolts diameters were approximately 16 mm diameter with mechanical anchorages. Bearing plates, nuts and washers were observed at the anchor heads. The use of spherical seats between the plate and the nut was not observed. Leveling pads were not employed, and bearing plates were in direct contact with the rock face. Bending of the bearing plates was observed at some locations. Based on a visual observation, the NYSDOT inventory rates the condition of these bolts as good. At the time of our reconnaissance trip the rock surface was covered with snow, however, for those bolt heads that could be observed rust stains were not evident.

Route 22 is a narrow, two-lane, highway with small shoulder width. Many of the rock bolts observed were accessible with a ladder, and some of the rock bolts were within reach from the ground surface. Figure 6-1 shows some of the rock bolts observed along NYS Route 22.

Access to rock bolts for NDT could be achieved with a single lane closure. Doug Hadjin indicated that during the summer, a high volume of traffic uses Route 22; therefore lane closures during peak travel times may impact the level of service along the highway.

6.1.2 Ellenville, New York, Shawangunk Mountain Range

Rock bolt installations were observed at five milepost locations along NYS Route 52 in Ellenville, New York. Rock bolts were installed to stabilize highway cuts in sandstone conglomerate with steeply dipping joints, infilled with mylonite. The ages of the bolts in this location are variable. Some of the bolts were installed in 1972, which is prior to the rock-slide, described in Section 5.3.1. These older bolts were approximately 16 mm diameter with mechanical anchorages. Additional rock bolts were installed after the rock-slide between 1992 and 1999. The newer rock bolts have a diameter of 32 mm and are resin grouted. In some locations older rock bolts are observed alongside new rock bolt installations. The observed rock bolts patterns are relatively dense, and more than 100 bolts have been installed along an approximately 1.5 km stretch of the highway.

The older installations employ hardware similar to that described in Section 6.1.1. New installations employ nuts and plates with spherical seats, and in some locations grout leveling pads are used. During the site visit, ice formations were observed at some bedding joint locations indicating the presence of groundwater. The rock face had rust stains at some of the bolt head locations.

NYS Route 52 is a two-lane highway with wide shoulders and ditches along the side for catching falling rock. A few of the bolts can be accessed from the ground. However, climbing equipment would be necessary to access bolts in many of the locations. Figure 6-2 shows some of the rock bolts observed along NYS Route 52.

A partial lane closure would be required to insulate workers from traffic while performing NDT. The traffic volume on this roadway is moderate, and a partial lane closure would cause some delay to traffic.

6.1.3 West Point-Bear Mountain State, Hudson Highlands

Rock bolts near West Point Academy and Bear Mountain State Park were observed. The sites are located along, or a short distance from NYS Route 9W, which follows the Hudson River.

Several sites were observed along NYS Route 9W, near milepost 1079. Rock bolts were used to stabilize very deep (> 30 m), steep cuts in gneiss with nearly vertical joint sets. Bolts are approximately 25 mm diameter with resin grouted anchorages. The condition of the bolts appeared to be good, although rust stains were observed near the bolt heads at some locations. Pre-spilt blasting was used to stabilize the face of the rock and the bolting pattern was not very dense. Bolts were not located close to the ground surface and access to the bolts would not possible without the use of a crane or mountain climbing techniques. A lane closure would be required to gain access to the bolts. Route 9W is a very busy, four-lane highway with narrow shoulders. A lane closure would severely affect the level of service along the highway.

Rock bolts were also observed along a very high, steep rock cut along the NYS Route 9W to NYS Route 218 access ramp (Figure 6-3). These bolts were also resin grouted with a diameter of approximately 25 mm. Conditions were similar to those observed along Route 9W, but access to the bolts was relatively easier. A partial lane closure would be required to gain access to the bolts. However, the ramp has a parking area, and traffic volume is relatively low compared to Route 9W.

Rock bolts within Bear Mountain State park were observed (Figure 6-4) that had more favorable access conditions compared to those along Route 9W. Rock slopes were not as high as along Route 9W and the rock face was not stabilized with pre-split blasting. Several of the observed bolt locations were within reach of the ground surface. The density of the bolting pattern was relatively high such that a number of bolts clustered within a small area could be studied. Access to bolts would require a partial lane closure. At some locations, a moderately wide shoulder is available and at other locations there is a guard rail and little, or no shoulder. In the summer months, a partial lane closure may affect the serviceability of the highway during peak hours.

Older rock bolt installations were observed near Bear Mountain Bridge along NYS Routes 9D and 6, respectively. They are approximately 19 mm diameter expansion shell anchorages. According to the NYS database, these rock bolts were installed around 1972 and the condition of the rock bolts ranges from fair to excellent. Rust stains were observed on the rock face at some locations. The density of the rock bolts patterns was not high, although several bolts were within reach of the ground surface. A lane closure would be required to gain access to the bolts. Routes 6 and 9D are very narrow, winding, two-lane highways with narrow shoulders. The traffic volume is high, and a lane closure would have a significant impact on the level of service along the highway.

6.2 GROUND ANCHOR SITES

6.2.1 West Valley, NY (Bar Elements)

An existing anchor sheet pile wall located at the site of the West Valley Nuclear Waste Reprocessing Center in Ashford, NY is a good candidate for preliminary testing. The approximately 32-year old retaining wall serves as a wing wall to an existing 5.5 m diameter culvert that passes through a railway embankment (Figure 6-5). The sheet-pile wall is supported with two rows of anchors. The bottom row is approximately 1 m from the ground surface and the top row is over the crown of the culvert. Anchors are approximately 11 m long and 44 mm diameter, continuously threaded bars, which are anchored to sheet-pile deadman. At the wall face, anchors extend through a wide-flange beam waler, and are secured with a bearing plate, nut and washer assembly. The anchor head is not encapsulated and no corrosion protection was installed with the tendon. Eight anchors are accessible at the wall face.

Soils data are available from test borings that were advanced along the embankment. Samples were retrieved and standard soils tests performed including grain size analysis, moisture content, and Atterberg limits. At this time, chemical analysis of the soils has not been conducted.

The site is attractive due to its proximity to the Project Team, and ease of access to the anchor heads. Representatives of the West Valley Facility have granted permission for access to the site. The team made a reconnaissance trip to the site to gather necessary information. A gravel road allows cars and trucks to travel within approximately 0.25 km of the culvert invert. Equipment must be carried the remaining distance down the railway embankment to access the culvert wing-walls and the tie-bars. We plan to perform preliminary testing at this site before evaluations are conducted at other sites that may be more difficult to access, or where testing time is limited due to construction activities.

6.2.2 Buffalo Inner Harbor (Bar Elements)

The Buffalo Inner Harbor Development Project involves relocation of an existing quay wall. The existing quay wall will be demolished and a new quay will be constructed approximately 15 m behind it. During construction of the new quay wall, backfill behind the existing wall will be excavated and the anchor bars will be exhumed.

The existing sheet-pile, quay wall was installed in 1967, and is approximately 10 m high. The wall is anchored at the top by a single row of anchors. Bar elements are used as anchors, which are anchored to sheet pile deadman. The elevation of the tie bars is above the high water mark, and a corrosion protection system was not included in the design of the tendons. The anchors are attached to walers on the backside of the sheet piles. Direct access to the anchor heads is not possible without cutting a hole through the sheet pile wall face.

A site investigation was performed in support of the design of the new sheet pile wall. A number of test borings were advanced, and soil samples were retrieved for laboratory testing. Laboratory testing included standard soil tests for moisture content, grain size analysis, and Atterberg limits. Chemical analysis of soil samples was also conducted including pH, and measurement of trace compounds typical of those conducted for environmental assessment.

Based on the test boring logs, the site soils are described as granular fill which extends from the ground surface to a depth of approximately 3 m, followed by a layer of very loose silty sand to a depth of approximately 6 m. Beneath the silty sand, dense sand was observed between approximate depths of 6 to 12 m. Hard, competent, limestone bedrock was observed below a depth of approximately 12 m.

The construction manager for the project is Ciminelli Cowper Co., Inc. and the contractor responsible for demolition of the old wall, and installation of the new one, is Herbert F. Darling, Inc. (Darling). Both companies have agreed to allow site access to the project team. Darling owns a barge at the Inner Harbor that can be used to access the face of the existing wall. They have agreed to allow the project team use of the barge, and will cut holes in the front to provide access to the existing anchor heads. During excavation of backfill behind the wall, anchor elements may be retrieved to verify the results of NDT, and for further laboratory evaluation. Construction is currently underway with testing activities tentatively scheduled during the spring of 2000.

The Buffalo Inner Harbor job is also a good candidate for monitoring a new metal-tensioned system installation, which is discussed in Section 7.0.

6.2.3 Parking Garage in Pittsburgh, PA (Strand Elements)

A tied–back sheet pile wall in Pittsburgh, PA provides grade separation for an adjacent parking garage. Tie–back elements are strand type anchors installed by Nicholson Construction Company in 1972. Based on discussions with representative of Nicholson, we understand the anchors were installed without any corrosion protection. This site is of interest to the study because of evidence of corrosion observed at the wall face and anchor heads, as shown in Figure 6-6. The wall can be accessed from the inside of the parking garage. The owner of the parking facility will be contacted for permission to access the wall. We are not aware of any current construction activity, and there are no obstructions, or traffic considerations at this location. Therefore, timing of our NDT condition assessment activities, and securing a work area, are not problems at this site.

6.2.4 Washington, D.C., O Street Wall (Bar Elements)

D’Appolonia is currently working on a project in the District of Columbia that involves stabilization of an approximately 25-year old, anchored, concrete diaphragm wall. The project is described as a case study in Section 5.3.2. As shown Figure 6-7, the anchors at this site are solid bars and the anchor heads are readily accessible for testing. This site is also attractive to the study because some of the existing anchors will be exhumed as part of repairs, which will be undertaken beginning during the Spring of 2000. D’Appolonia is performing engineering design services for reconstruction of this wall, and will coordinate activities for the NDT evaluation prior to demolition of the existing wall.

6.2.5 NYSDOT Walls

NYSDOT provided the project team with a list of sites with permanent tie-backs. A total of 24 sites are included on the list. Two of the listed sites are soil nail walls, 10 are dams that employ anchor tie downs, and the remaining 12 sites are retaining walls with tie-backs. The dam sites are not owned by NYSDOT but are owned and operated either by the New York State Canal Corporation, the NYS Power Authority or NYS Gas and Electric. The dam sites may be useful for Phase II evaluations due to the ease of access because many times anchor ties downs at dam sites are placed within a gallery to facilitate their inspection and lift-off testing. Three retaining wall sites were short listed for consideration due to the ease of access and the possibility that the walls would be demolished in the near future, allowing visual inspection of exhumed anchors for validation of NDT results.

Palenville (Bar Elements)

The site is located in the Catskill Mountains, along NYS Route 23A in NYSDOT Region 1 near Palenville, NY. A reconnaissance trip was made to this soldier pile and lagging wall on January 23, 2000. We were accompanied by Paul Bailey, a structural engineer from the NYSDOT, who assisted in locating and providing background information about the wall. The wall, shown Figure 6-8, supports the highway embankment along NYS Route 23A. The wall was constructed in 1995. Steel H-piles serve as soldier piles for the wall and precast, reinforced concrete panels are used as lagging. The wall is supported with a single row of wall anchors near the top of the wall, which are anchored to deadman. Tendon elements are galvanized steel, approximately 57 mm diameter bars that extend through a double channel waler at the face of the wall. The anchor head is secured with a bearing plate, nut and washer. The anchor head is not encapsulated.

During the reconnaissance visit, no evidence of corrosion was visible at the anchor head. Approximately 0.3 m of ground settlement was evident in front of the soldier piles.

The terrain in front of the wall face is very steep and access to the anchor heads will require scaffolding that may be suspended from the top of the wall. A lane closure will be necessary to provide a work area for the NDT evaluation. NYS Route 23A is the access road to Hunter Ski Area, and is a very narrow winding road carrying a high volume of traffic.

Gowanda, NY
The site is located along NYS Route 38 and is in NYSDOT Region 5 near Gowanda, NY. A reconnaissance trip was made to this site on December 15, 1999. The soldier pile and lagging wall, shown in Figure 6-9 is used to support a highway embankment. The wall was constructed in 1989 after a landslide near a culvert headwall consumed part of Route 38. Steel H-piles serve as soldier piles which are spanned by timber lagging. A single row of tie-backs is used to support the wall. Tie-backs extend through a double channel waler at the face of the wall. The anchor head is covered with a metal cap.

The terrain in front of the wall is very steep and access to the anchor heads will require scaffolding that may be suspended from the top of the wall. A partial lane closure will be required to provide adequate work space for NDT evaluation. NYS Route 38 is a two lane highway with moderate traffic. Shoulders are very narrow in the vicinity of the wall. A partial lane closure will have a moderate effect on the level of service provided by the highway.

NYS Route 5 (Strand Elements)
The site is located in the town of Sennet, NY along NYS Route 5 and is in NYSDOT Region 3. A reconnaissance trip was made to the site on January 24, 2000. The wall, shown in Figure 6-10, supports a railway embankment that serves as the approach to the elevated crossing of Conrail over NYS Route 5. The wall was constructed in 1994. Current plans include reconstruction of the highway and changing to a grade crossing for the railroad. According to the construction schedule, the embankment will be excavated and the wall demolished in the Spring of 2000.

The height of the sheet-pile ranges from approximately 1 m to 4 m, and it is supported by a singe row of ground anchors near the top of the wall. The tendon elements are seven-wire strands, which are grouted in the anchorage zone. At the wall face, the strands are surrounded by a metal sheath that extends through a wide flange beam waler. The metal sheath has a flange at the end. The anchor head with lock-off wedges fits inside of a flange plate that mates with the metal sheath. The anchor head is covered by a metal end cap that is welded to the flange plate.

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. At this time, no chemical analysis of the soils has been performed.

We scheduled an appointment and met with the project engineer at the NYSDOT Region 3 Construction Office in Syracuse, NY. The job has been bid and the prime contractor is Economy Paving, Inc. We have spoken with representatives of Economy Paving and they have agreed to allow us access to the site. Part of their work involves a lane closure and we will coordinate our activities to coincide with the scheduled lane closure. A single lane closure will provide the working space needed for the NDT evaluation.

After NDT measurements are made we will return to the site during excavation of the ground anchors and observe their condition. Some strands may be collected for further laboratory evaluation.

6.2.6 NGES

The NGES database indicates that ground anchors are installed at both the sand and clay sites of the Texas A&M Riverside Campus. Details of the wall at Texas A&M are described in Section 5.3.2. We also pursued information about ground anchors installed at the UMASS, Amherst site. After communication with Dr. A. Lutenegger, we learned that the specimens were representative of the grouted length of bar anchors, had a vertical orientation, and were approximately 1.5 to 3 m long. Therefore, this site will not be considered further as it offers no significant advantage over the U.B. Test Facility.

Prof. J.L. Briaud is the manager of the Texas Riverside NGES site. He has been contacted and agreed that the site may be used for this study. A soils laboratory and other facilities in support of the field work are available at Texas A&M. We are in the process of preparing the required paperwork for requesting and obtaining access to an NGES site. The site offers many advantages for our test program including the fact that load cells and other instrumentation are currently installed. In addition, the anchors were installed without corrosion protection (except for grouting), and thus provide an opportunity of evaluating the performance of anchors installed in soil for a period of about 8 years.

6.3 SOIL NAIL SITES

Detailed plans for evaluation of a soil nail wall have not been completed at this time. Discussion are on-going with several state agencies including PADOT. Since nail heads at most soil nail sites are not exposed, plans will most likely necessitate cutting into an existing wall face to gain access to soil nail heads.

6.4 SUMMARY OF WORK PLAN

At this time, nine field sites have been identified for Phase II of the investigation. Table 6-1 is a summary of sites with pertinent information. Pertinent information includes the application (Appl.) as rock bolts (RB), tie-backs (TB, grouted anchorage) or wall anchors (WA, anchored to deadmen); the type of element which is either bar or strand; the date of installation (DOI); the existence of a corrosion protection system (Corr. Prot.); the availability of soils data; whether or not the elements are prestressed; and special comments.

The ages of the elements planned for the condition assessment range from 3 to 40 years old. Not all the tendons at the sites considered were installed with corrosion protection systems that meet today's standards. At several of the sites (e.g., #2, #3 and #4), anchors will be exhumed as part of planned reconstruction.

Eight of the sites are ground anchors and one site is a rock bolt site. The rock bolt site in Ellenville, NY site was selected based on the presence of different types of rock bolts at the same site, the density of the rock bolt pattern, the rock conditions at the site, and access to the rock bolts at the site compared to the other rock bolt sites considered. The site is of particular interest because of the recent rock slide, and existence of data from a study of the slide.

For the sites with ground anchors, six of the tendons have bar elements and two have strand. Bars at the NGES site have a grouted anchorage and are prestressed, but deadman anchorages are used at the other sites with bar elements. The strand elements at sites #3 and #5 are grouted and prestressed.

At the sites where reconstruction activities will take place, our schedule for the NDT and condition assessment must be coordinated with the construction schedule for the project. The project schedule is more flexible relative to NDT activities planned at the remaining sites. We plan to perform NDT evaluations at the field sites during the spring, summer and fall of this year (2000).

6.5 PROTOCOL FOR PHASE II EVALUATIONS

6.5.1 Existing Installations

The purpose of the evaluations of existing installations proposed for Phase II is to demonstrate the application of nondestructive test techniques in the field, and to correlate results of the nondestructive tests with subsurface conditions, details of the installation and expectations based on service-life prediction models. For each site, information will be collected on the regional climate, subsurface conditions, and details of the installation;

followed by performance of the nondestructive tests on selected metal tensioned elements.

Regional Climate. Relevant climatic information includes the annual amount of precipitation, depth of frost, severity and duration of freeze periods, and average number of annual freeze/thaw cycles.

Subsurface Conditions. Soil and rock samples will 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. Preferably, soil samples with minimal disturbance will be retrieved using AASHTO Methods T206 ("Standard Specification for Penetration Test and Split-Barrel Sampling of Soils"), or T207 ("Standard Specification for Thin Walled Tube Sampling of Soils"). In the event that a minimally disturbed soil sample cannot be obtained, disturbed samples will be obtained by hand auger (AASHTO Method T 203, "Standard Specification for Soil Investigation and Sampling by Auger Borings") or test pit excavation. Soil samples will be tested for physical and chemical properties as described below.

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 T91)	

If possible, the soil resistivity test shall be performed in-situ.

Rock outcrops representative of rock bolt or ground anchor installations shall be located, and the rock type identified by visual inspection. If no outcrops are available, rock samples shall be obtained by diamond core drilling techniques as described in ASTM D 2113, "Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigation". Groundwater present within the rock mass shall be sampled and tested for pH, sulfate and chloride content. Rock joints shall be observed, and for rock joints which daylight at the face of an outcrop, infill materials, if present, will be sampled. Samples of joint infilling will be subjected to the soil tests described above.

Installation Details. Information about the installed metal-tensioned system shall be obtained including date of installation, element type, size and length, corrosion protection measures, level of prestress and visual evidence of corrosion. Much of this information can be obtained from shop drawings and/or construction record plans, and databases maintained by the various state departments of transportation. Depending on access to the element head, lift-off tests may be necessary to establish existing levels of prestress.

If relatively easy access to a grease-protected sheath in the free length of the tendon is available, an attempt will be made to obtain grease samples as sterilely as possible. Grease samples will be tested for bacteria content, which may be correlated with biologic activity. An attempt will also be made to obtain grease samples from the vicinity of areas where corrosion is identified.

Performance of NDT. Nondestructive testing will be conducted at each site to evaluate the condition of existing metal-tensioned systems. The electrochemical tests described in Section 3.2.3, including measurement of the half-cell potential and E vs. Log I relationship, may be useful in assessing the integrity of the corrosion protection system, and may indicate if, and at what rate, corrosion is occurring. Wave propagation techniques, i.e. impact and ultrasonic tests as describe in Sections 3.2.1 and 3.2.2 can be used to assess the existing condition of elements, i.e. severity of corrosion. The advantages and disadvantages of each test, which will be considered in developing site-specific testing protocols are identified below.

TEST TYPE	ADVANTAGES	DISADVANTAGES
Electrochemical	• Inexpensive • Simple to perform • Proven technology • Data analysis is simple	• Affected by: a) dielectric properties of the ground b) lack of electrical isolation • Cannot determine type, or severity of corrosion
Impact	• Proven technology • Provides information on severity of distress • Simple to perform • Can test relatively long elements	• Not applied to strand elements • Accelerometer needs to be firmly attached to end of element • Limited sensitivity
Ultrasonic	• Can be applied to strand elements • Same transducer is source and receiver • Very quick test to perform	• Requires high speed data acquisition • Transducer is delicate • High attenuation • Limited sensitivity

The proposed NDT's require access to the end of an element for attachment of necessary wiring and/or instrumentation. Protective caps, if present, will need to be removed from the end of the elements, and for encapsulated anchorages, grout may need to be chipped away from the end of the element. Special considerations for making connections and attaching instrumentation and wiring at the end of the element for each test method are summarized below:

> *Electrochemical Tests.* Electrical connections involve attaching a wire to the end of the element. Scale or rust, if present shall be cleaned to achieve good electrical contact.
>
> *Ultrasonic Test.* Surface preparation of the end of the anchor may be required to achieve good acoustic coupling with the transducer. Surface preparation may include squaring the end with a metal cutting saw or grinder to achieve a relatively flat, smooth surface.
>
> *Impact Test.* The accelerometer may be attached with a specially design base plate glued to the end of the bar. Better results may be achieved if the end of the bar is drilled and tapped to receive the threaded fitting at the base of the accelerometer directly. A hand-held punch, or other device, may be required to achieve a direct impact at a prescribed location without interference from the accelerometer.

NDT results are qualitative in the sense that data obtained from a number of different elements at a given site are compared to one another, and to signatures that represent the response of typical elements. Elements will first be screened with the electrochemical tests to determine locations where corrosion is likely to occur based on agency records or visual observations made at the site. The results from electrochemical tests will be supplemented with results from the impact response and/or ultrasonic tests to identify and locate defects along the length of the element. Performance tests, i.e. load tests, will be recommended at elements where signs of distress have been identified. The service life at the site may be predicted using techniques described in Section 4.0 and compared with results from NDT.

The NDT test protocol is described as follows:

BARS	STRANDS
• Perform Electrochemical Tests	• Perform Electrochemical Tests
• Perform Both Impact and UT Tests	• Perform UT Test
• Correlate Performance with Service-life Prediction Model	• Correlate Performance with Service-life Prediction Model

Results from the electrochemical tests require limited data reduction. Measurements of half-cell potential are reported directly in terms of the measured voltage. Data from the E vs. log I relationship is plotted using a semi-log format and the shape of the resulting curve is studied. If a break in the curve is located, polarization of the element surface is indicated, which may be due to a compromised corrosion protection system. Comparison with results from a number of elements at the same site indicates where anomalies are located.

For both the impact and ultrasonic tests, vibrations measured at the end 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.

Calibration Requirements. Instruments used in the NDT need to be calibrated including the half-cell utilized in the electrochemical tests and the ultrasonic transducer and accelerometer used in the wave propagation techniques. The copper-copper sulfate half-cell may be calibrated as described in ASTM C876 ("Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete"). The accuracy of voltage and ammeters can also be checked by comparing measurements obtained with different units.

The calibration of both the accelerometer and transducer can be checked using reference standards as described by the instrument manufacturers. Also, many of the instrument suppliers have in-house calibration facilities, and, as a customer service, offer periodic calibration of the transducers. In the field, an approximately 2 m long bar of known properties with flat smooth ends, and isolated on foam pads, can be used as a reference to check the accuracy of the instrumentation and high speed data acquisition system.

An approximate check on the calibration of the accelerometer can be performed by taking readings before and after the accelerometer is inverted. The corresponding readings should correspond to twice the acceleration due to gravity. The inversion technique may not work well for transducers intended for measurements in the high frequency range, since their accuracy is often diminished within the lower frequency applied as the transducer is inverted.

For all tests, the integrity of wires and electrical connections must be continually monitored to check for abnormally high levels of resistance, and/or loss of, or intermittent contact.

Table 6-1
Proposed Sites for Phase II Field Studies.

No	Site Location	Appl	Type	DOI	Corr. Prot.	Soils Data	Pre-stress	Comments
1	West Valley, NY	WA	Bars	1968	No	Partial	None	Sheet-pile wall; good site for preliminary testing
2	Buffalo Inner Harbor	WA	Bars	1967	No	Partial	None	Sheet-pile quay wall; anchors will be exhumed
3	NYS Route 5, Sennet, NY	TB	Strand	1994	Yes	Partial	Yes	Sheet-pile bulkhead; anchors will be exhumed
4	O - Street, Washington, DC	TB	Bars	1975	No	Yes	Yes	concrete diaphragm wall; known problems with anchors; anchors will be exhumed; case study
5	Parking Garage, Pittsburgh, PA	TB	Strand	1972	No	Need	Yes	Sheet-pile wall; corrosion evident at wall face
6	NGES- Texas A&M Riverside Campus	TB	Bars	1991	Yes	Partial	Yes	Soldier-pile and lagging; ease of access due to use as experiment; instrumentation is installed; case study
7	Route 38, Gowanda, NY	TB	Strand	1989	Yes	Need		soldier pile and lagging wall
8	Route 23A, Palenville, NY	WA	Bars	1995	No	Need	No	soldier pile and lagging wall
9	Route 52, Ellenville, NY	RB	Bars	1972 to 1999	No		Yes	different ages and types of anchorages; previous slide area; case study

Figure 6-1: Rock Bolts Observed Along NYS Route 22.

Figure 6-2: Rock Bolts Observed Along NYS Route 52 in Ellensville, NY.

Figure 6-3: Rock Bolts Observed Along NYS Route 9W, Hudson Highlands.

Figure 6-4: Rock Bolts Observed in Bear Mountain State Park, NY.

Figure 6-5: Tied-Backs Observed for Culvert Wing Wall, West Valley, NY.

Figure 6-6: Corroded Sheet Pile Section of Anchored Wall, Pittsburgh, Pennsylvania.

Figure 6-7: Anchored Concrete Diaphragm Wall, Washington, DC.

Figure 6-8: Soldier Pile and Lagging Wall, Palenville, NY.

Figure 6-9: Soldier Pile and Lagging Wall, Gawanda, NY.

Figure 6-10: Tied-Back Wall for Embankment, Conrail Over NYS Route 5.

7.0 WORK PLAN FOR MONITORING OF NEW INSTALLATIONS

This section describes field investigations of new metal-tensioned systems to permit measurement of their condition throughout their service life.

The VETEK V2000 Corrosion Monitoring System is recommended for monitoring new installations. The V2000 monitoring system consists of a silver-silver chloride wire that is installed with the metal-tensioned element and serves as a reference electrode. 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 electro-chemical measurements being evaluated for existing systems. Therefore, much of the hardware and data acquisition equipment needed for the tests are in common with that described in Section 3.2.3.

Plans are underway for field investigations of new metal-tensioned systems to permit measurement of their condition throughout their useful service lives. For each potential test site the following information will be documented; 1) anticipated schedule for system installation, 2) type of metal tensioned system, corrosion protection to be installed, 3) soil types, pH, chloride and sulfates content, 4) climate data, 5) grout type, 6) level of prestress (where applicable).

McMahon & Mann Consulting Engineers (MMCE) is working with Herbert F. Darling, Inc. on several design alternatives that involve the anchor systems for the new quay wall, and for mooring structures to be constructed in the water at the site of the Buffalo Inner Harbor Development Project. Use of both bar and strand type tendon elements are planned as tie-backs for the quay wall, and as tie-downs for the mooring structures. Strand will be anchored into rock; and some of the bar elements will be anchored to deadmen and others will be grouted into rock. Construction is anticipated during the Summer of 2000.

Based on discussions with representatives of the North Carolina DOT, new installations of ground anchors are planned as part of the construction of the Route I–26 extension. We are currently investigating location of installations and the contract schedule.

8.0 CONCLUSIONS

The focus of Project NCHRP 24-13, "Evaluation of Buried Metal-Tensioned Systems in Geotechnical Applications," is condition assessment and service-life prediction of existing and new metal-tensioned systems. This report describes Phase I of the project which includes a study of types of buried metal-tensioned systems, performance of metal-tensioned systems, test methods for condition assessment, models for service life prediction, and development of work plans for evaluating viable test methods for condition assessment and models for service life prediction for existing and new installations.

Applications for buried, metal-tensioned systems include rock bolts, soil nails and permanent ground anchors. The service-life of these systems is affected by their tendency to corrode and the potential for loss of prestress and tensile load capacity over time. Recommendations for single and double corrosion protection systems are described by PTI (1996). Even in aggressive ground conditions, if a corrosion protection system is employed that satisfies requirements for double corrosion protection, including encapsulation and electrical isolation at the anchor head, the service life of the anchor will not be substantially affected by corrosion. However, attention must be paid to details of the corrosion protection system. Care must be taken during transportation and installation of tendon elements such that sheathing is not damaged and grease or other corrosion inhibitor compounds remain in contact with the element. Selection of corrosion inhibitor compounds must consider the affinity of the compound to water, and the potential for micro-bacteriologic activity.

Many early ground anchor installations (before about 1986) were not designed to the same standards in use today. Older existing systems may not incorporate adequate details, or may have no corrosion protection other than single stage grouting. Some of the older mechanical rock bolt anchorage systems were installed without corrosion protection.

Based on a survey of the literature, conclusions regarding the performance of buried metal-tensioned systems are consistent with those described by FIP (1986). In the past, the majority of corrosion problems have occurred at, or near, the anchor head region, and some are observed along the free length. Most documented corrosion problems for buried metal-tensioned systems have been correlated with aggressive ground conditions or the presence of stray currents.

Standards are available for assessment of aggressive ground conditions. If aggressive ground conditions are present, the condition of existing metal-tensioned systems should be considered suspect. For these cases, further testing is needed to check if corrosion protection systems are intact, if corrosion is occurring, and to assess the current condition of the metal-tensioned element. There is a standard for electric resistance testing of grouted ground anchors, but this requires that each tendon element be electrically isolated from the rest of the system. Results from the electrical resistance test indicate whether the corrosion protection system has been compromised, but do not indicate if corrosion has or is occurring, nor any information regarding the existing condition of the metal element. Some NDT test 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 are either available, or are currently under development.

A search of the literature was conducted to collect information on NDT that could potentially be implemented for condition assessment of buried metal-tensioned systems. Mechanical and electromagnetic wave propagation techniques and electro-chemical type tests were studied. Tests employing these techniques were evaluated based on their potential for success, ease of application, cost of instrumentation and availability of needed equipment. Several test methods were further 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 to metal-tensioned systems, the sensitivity 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. Based on the results of the literature search and laboratory evaluation, the impact-echo test, ultrasonic test, half-cell potential and polarization measurements are recommended for implementation at selected fields sites. Implementation of the test techniques at field sites, and a study of the test results and data collected are planned for Phase II of this research.

Nine field sites were recommended for Phase II of the evaluation. The sites represent a range of different conditions and types of existing metal-tensioned systems. Based on the NDT evaluations performed at these sites a protocol will be developed for condition assessment of existing systems. Results from the field evaluation will help to verify the accuracy of the test methods, and of the available service life prediction models described in Section 4.0.

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