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**CORROSION/DEGRADATION OF  
SOIL REINFORCEMENTS FOR  
MECHANICALLY STABILIZED EARTH WALLS  
AND REINFORCED SOIL SLOPES**



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16. ABSTRACT  This manual provide's criteria for evaluating corrosion losses when using coated or uncoated steel reinforcements, and for determining aging and installation damage losses when using geosynthetic reinforcements. Monitoring methods for in-situ corrosion rates for steel reinforcements are evaluated and remote methods using electrochemical methods are recommended. Monitoring methods for determinations of in-situ aging of geosynthetics are evaluated and protocols for implementation are recommended.			
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<b>SI CONVERSION FACTORS</b>				
APPROXIMATE CONVERSIONS FROM SI UNITS				
<b>Symbol</b>	<b>When You Know</b>	<b>Multiply By</b>	<b>To Find</b>	<b>Symbol</b>
<b>LENGTH</b>				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
<b>AREA</b>				
mm <sup>2</sup>	square millimeters	0.0016	square inches	in <sup>2</sup>
m <sup>2</sup>	square meters	10.764	square feet	ft <sup>2</sup>
m <sup>2</sup>	square meters	1.195	square yards	yd <sup>2</sup>
ha	hectares	2.47	acres	ac
km <sup>2</sup>	square kilometers	0.386	square miles	mi <sup>2</sup>
<b>VOLUME</b>				
ml	milliliters	0.034	fluid ounces	fl oz
l	liters	0.264	gallons	gal
m <sup>3</sup>	cubic meters	35.71	cubic feet	ft <sup>3</sup>
m <sup>3</sup>	cubic meters	1.307	cubic yards	yd <sup>3</sup>
<b>MASS</b>				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
<b>TEMPERATURE</b>				
°C	Celsius	1.8 C + 32	Fahrenheit	°F
<b>WEIGHT DENSITY</b>				
kN/m <sup>3</sup>	kilonewton / cubic m	6.36	poundforce / cubic foot	pcf
<b>FORCE and PRESSURE or STRESS</b>				
N	newtons	0.225	poundforce	lbf
kN	kilonewtons	225	poundforce	lbf
kPa	kilopascals	0.145	poundforce / sq in.	psi
kPa	kilopascals	20.9	poundforce / sq ft	psf

## PREFACE

Engineers and specialty material suppliers have been designing reinforced soil structures for the past 35 years. During the last decade significant improvements have been made to the design methods and in the understanding of factors affecting the durability of the soil reinforcements. This work is becoming even more important now that many reinforced soil structures are well into their anticipated design life.

In order to take advantage of recent developments, the FHWA has updated the previous version (Elias, 2000) of this manual. The primary purpose of this manual is to serve as the FHWA standard reference for highway projects involving reinforced soil structures. This manual also supports the national efforts on health monitoring of bridge foundations and asset management, providing test techniques and protocols that are being employed to collect performance data for earth reinforcements, data interpretation and preliminary information available from data that has been collected to date.

Another purpose of equal importance was to support educational programs conducted by FHWA for transportation agencies.

This Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes manual has evolved from the following FHWA reports and previous manuals:

- Durability/Corrosion of Soil Reinforced Structures; V. Elias, FHWA RD-89-186
- Testing Protocols for Oxidation and Hydrolysis of Geosynthetics, FHWA RD-97-144.
- Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil slopes: V. Elias, FHWA-NHI-00-044 and FHWA SA-96-072.

The first author of this manual is the late Mr. Victor Elias, P.E. who dedicated much of his professional career to the advancement of reinforced soil technology. As indicated above, he was the author of the previous two FHWA manuals on corrosion/degradation of soil reinforcements and was the principle investigator for much of the research supporting this manual. Mr. Elias was instrumental in the introduction and implementation of reinforced soil technology in the U.S., as a Vice President for The Reinforced Earth Company from 1974 to 1985. His work included major reinforced soil retaining walls. In addition, he expanded the applications of soil reinforcement to slabs, dams, storage facilities and bridge abutments.

Mr. Elias provided significant contributions to the design and construction of safe, cost-effective geotechnical works in highway (and private) works. He has been the Principal Investigator for several major research and/or implementation projects focused on durability of soil reinforcement materials and design specifications for foundations and retaining walls, and ground improvement methods. He provided this leadership role in eight design and construction reference manuals for FHWA, four corrosion/durability reinforced soil research projects, and the recommended revisions of the MSE wall section of the 1990 and 1994 AASHTO Standard Specifications for Highway Bridges as well as contributions to the revisions of the MSE wall sections in the 1997 and 2002 AASHTO Standards. The coauthors of this manual, his many colleagues, fellow engineers and friends within the geotechnical community will dearly miss Victor's leadership, insights, staunch opinions, and experience.

The authors also recognize the efforts of Mr. Jerry A. DiMaggio, P.E. who was the FHWA Technical Consultant for most of the above referenced publications. Mr. DiMaggio's guidance and input to this and the previous works has been invaluable.

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- Association of Metallically Stabilized Earth (AMSE)
- Geosynthetic Materials Association (GMA)
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# CHAPTER 1

## INTRODUCTION

### 1.1 OBJECTIVES OF MANUAL

The use of mechanically stabilized earth (MSE) systems for the construction of retaining wall structures and steepened slopes has gained widespread acceptance among owners, as evidenced by the many thousands of completed structures. As usage increased in the 1980s and 1990s there was, however, a desire by owners and the research community to confirm that current methods are valid and that the design models used will ensure that these structures will perform as intended for their full design life. Previous editions of this reference guide proposed test protocols including electrochemical techniques for condition assessment and corrosion monitoring of MSE soil reinforcements. Various researchers, the FHWA and state transportation agencies have implemented these test protocols to collect information and develop performance databases. These techniques are now considered mature technologies that can be implemented for asset management to document performance and practices that contribute to effective use of resources and cost savings to owners.

It is important to consider the performance of MSE structures within the context of Transportation Asset Management (TAM), as slopes and retaining walls are important components of the highway system. Their performance depends on proper selection of materials, details of construction and maintenance. These are important considerations and their impacts on cost and service life affect decisions inherent to TAM.

The design of MSE structures requires that the combination of a select soil and reinforcement be such that the interaction between the two materials produces a composite structural material that combines their best characteristics. The judicious placement of reinforcements in the select soil mass serves to restrain the deformation of the soil in the direction parallel to the reinforcement.

The most commonly used soil-reinforcing for retaining walls on transportation projects has been galvanized steel, either in strip or grid configuration (~80 to 90 percent of applications to date), connected to a precast concrete facing. Polymeric soil reinforcements were introduced in the 1970s and early 1980s. They have been used with increasing frequency in both MSE walls and reinforced soil slopes (RSS) since their introduction. Today, the majority of RSS on transportation projects use geosynthetic soil reinforcements.

A major design concern for MSE structures has been the durability of reinforcements in the soil/water environment in which they are placed. The dual aim of this manual is to provide criteria to guide design engineers in evaluating potential corrosion losses when using coated or uncoated steel reinforcements, and degradation losses when evaluating the use of polymeric reinforcements. The other aim is to guide engineers in implementing field evaluation schemes to monitor such corrosion/degradation mechanisms in constructed structures.

The monitoring of corrosion losses in these structures is addressed by implementation of non-destructive field evaluation systems using remote electrochemical measuring equipment capable of determining in-situ corrosion rates of galvanized and base steel and inferring from them the loss of section. The monitoring of degradation losses for polymeric reinforcements is addressed by implementation of retrieval protocols and destructive testing of samples to measure loss of tensile strength and changes in the polymer structure. This manual was originally developed (Elias, 1997) in support of a FHWA Demonstration Project on the design, construction and monitoring of MSE walls and slopes. The manual was updated in 2000 (Elias, 2000). The principal function of this manual is to serve as a reference source for the long-term performance of soil reinforcements used in MSE structures. This current update incorporates the most recent work in this field, the NCHRP 24-28 study (see Section 1.4, below).

The test techniques and procedures described in the manual have been researched and developed over the past several decades. These electrochemical test techniques and test protocols are mature technologies, and useful tools for asset management. Another objective of this manual is to describe how these tools can be used for asset management. The benefits of performance monitoring and asset management are demonstrated by several examples included in the manual.

## **1.2 SCOPE**

The scope of this manual includes:

- Description of the corrosion/deterioration mechanism that occurs in reinforced soil structures constructed with metallic reinforcements, leading to design procedure recommendations.
- Description of techniques and instrumentation designed to measure in-situ corrosion rates of steel reinforcements in MSE structures.

- Review of laboratory test methods for the electrochemical analysis of select reinforced fill materials used in MSE structures. Relationships between these test variables and predictions of corrosion/degradation are also discussed.
- Review of criteria to determine survivability of fusion bonded epoxy coatings.
- Identification of degradation mechanisms consistent with in-ground regimes for geosynthetic reinforcements.
- Monitoring methods and evaluation of degradation mechanisms for geosynthetic reinforcements.

### **1.3 ORGANIZATION**

Chapter 2 is devoted to the fundamentals of corrosion of metals in soil, identification of corrosive environments, and details current design approaches to account for in-ground corrosion.

Chapter 3 details monitoring methods for metallic reinforcements and their application to existing and new construction.

Chapter 4 is devoted to the fundamentals of polymer degradation and identification of in-soil regimes that may accelerate degradation.

Chapter 5 details monitoring methods for geosynthetic reinforcements, and their application to existing and new construction.

Greater detail on topics discussed in Chapters 2 and 3 are detailed fully in FHWA RD 89-186 *Durability/Corrosion of Soil Reinforced Structures* (Elias, 1990), a primary source document for this manual.

Greater detail on topics discussed in chapters 4 and 5 are detailed fully in FHWA RD-97-144, *Testing Protocols for Oxidation and Hydrolysis of Geosynthetics* (Elias et al., 1997).

### **1.4 PROJECT NCHRP 24-28 “LRFD Metal Loss and Service Life Strength Reduction Factors for Metal Reinforced Systems in Geotechnical Applications.”**

NCHRP 24-28 aims to: (1) assess and improve the predictive capabilities of existing models for corrosion potential, and for estimating metal loss and service life of earth reinforcements, and (2) to develop methodology that incorporates the improved predictive models into an

LRFD approach for the design of MSE. The project scope includes collecting data on the performance of metallic reinforcements used in the construction of MSE, developing a database for archiving performance data, statistical analysis of performance data, and reliability analysis of metal loss estimates used to ensure the specified design life. The test techniques and protocols for condition assessment and corrosion monitoring described in this reference manual were used in pursuit of performance data for NCHRP 24-28.

We expect the final report for NCHRP 24-28 to be issued for distribution before September 2010. Anticipated products from NCHRP Project 24-28 include:

1. A performance database documenting the attributes and metal loss observed from in-service MSE reinforcements.
2. Updated metal loss models considering targeted levels of confidence and various site conditions.
3. Recommended resistance factors for use in LRFD designs that account for the estimated metal loss over the service life of the structure.
4. A recommended practice that specifically addresses issues related to metal loss from corrosion including required sampling and testing, example design procedure, and commentary.

The field experience, performance data, and insights into factors affecting metal loss gained from NCHRP 24-28 contribute to information included in this manual. Retaining walls and slopes are important components of the highway system, and the database and test protocols resulting from NCHRP 24-28 serve as important tools for asset management. These data incorporate effects of climate, soil environment and site conditions, which are significant factors in terms of service-life. A major contribution from NCHRP 24-28 is to evaluate effects from reinforced fill quality, site conditions, maintenance operations and climatic factors on variance, and hence uncertainty, with respect to corrosion and anticipated metal loss.

Changes to the current AASHTO metal loss model are not anticipated; however, confidence and model reliability are assessed. The current AASHTO model only applies to galvanized reinforcements and reinforced fill meeting stringent electrochemical requirements. Data from reinforced fills that don't necessarily meet all of these requirements (e.g.,  $\rho < 3000 \Omega\text{-cm}$ ) were also collected during the fieldwork for NCHRP Project 24-28. These data will be used to evaluate parameters and other adjustments needed to estimate the impacts of marginal quality fills on service life.

## **CHAPTER 2**

### **CORROSION OF METALLIC REINFORCEMENTS**

The current design approach to account for potential corrosion losses is to add to the required structural thickness a sacrificial thickness equal to the projected section loss over the design life of the structure. To minimize the sacrificial thickness and reduce uncertainties, a select fill with controlled electrochemical properties is specified for the reinforced zone. This chapter is intended to provide a background in the fundamentals of corrosion, the identification of corrosive environments by electrochemical testing and a review of the basis for the currently used design corrosion loss rates.

#### **2.1 FUNDAMENTALS OF CORROSION OF METALS IN SOIL**

Accelerated or unanticipated corrosion of the reinforcements could cause sudden and catastrophic failure of MSE structures, generally along a nearly vertical plane of maximum tensile stresses in the reinforcements. This plane is located at a distance varying from 0 to 0.3H from the facing where H is the height of the structure. Few instances of advanced corrosion that have compromised service life of MSE structures have been documented in the United States, Europe and South Africa (Blight and Dane, 1989; Elias, 1990; Fishman et al., 1986; Frondistou-Yannis, 1985; Armour et al., 2004; Gladstone et al, 2006; McGee, 1985; Raeburn et al., 2008).

Corrosion is the deterioration or dissolution of metal or its properties by chemical or electrochemical reaction with the environment. When a large surface is affected it can be viewed as general corrosion and approximated by an assumed average, uniform rate of corrosion per year. If confined to small points so that definite indentations form in the metal surface, it is referred to as pitting corrosion and generally reported as maximum pit depth per year.

Corrosion is fundamentally a return of metals to their native state as oxides and salts. Only the more noble metals (platinum, gold, etc.) and copper exist in nature in their metallic state and are resistant to corrosion. Other metals are refined by applying energy in the form of heat. Unless protected from the environment, these metals revert by the corrosion process, which is irreversible, from their temporary state to a more natural state.

Although most chemical elements and their compounds are present in soil, only a limited number exert an important influence on corrosion. In areas of high rainfall, the passage of

time has resulted in the leaching of soluble salts and other compounds, rendering these soils generally acidic. In arid locations, soluble salts are brought to the upper soil layers through capillary and evaporative processes, causing the soils to be generally alkaline. (Romanoff, 1957)

The authoritative reference work to date on underground corrosion is National Bureau of Standards (NBS) Circular 579 (Romanoff, 1957). The corrosion mechanism of ferrous and other metals in soils is essentially electrochemical. The corrosion process releases the energy the metal gained during its refining in the form of electrical energy. Current flows because of a voltage difference between two metal surfaces or two points on the same surface in the presence of an electrolyte. Two pieces of metal or two portions of the same metal in an electrolyte seldom have the same potential. The amount of potential difference depends on the nature of the metal, the condition of the surface, the nature of the electrolyte, and the presence of different materials at the interface of the metal and electrolyte. Under these conditions, a current will flow from the anodic area through the electrolyte or soil to the cathodic area and then through the metal to complete the circuit. The anodic area becomes corroded by the loss of metal ions to the electrolyte.

In general, the most corrosive soils contain large concentrations of soluble salts, especially in the form of sulfates, chlorides, and bicarbonates and may be characterized as very acidic (low pH) or highly alkaline (high pH). Clayey and silty soils are characterized by fine texture, high water-holding capacity, and consequently, by poor aeration and poor drainage. They are also prone to be potentially more corrosive than soils of coarse nature such as sand and gravel where there is greater circulation of air. Buried metals corrode significantly by the process of differential aeration and sometimes by bacterial action. Corrosion by differential aeration may result from substantial local differences in type and compaction of the soil or variations in the oxygen or moisture content resulting thereof. Such a phenomenon is generally associated with fine-grained soils. Microbial induced corrosion is associated with the presence of anaerobic sulfate-reducing bacteria that reduce any soluble sulfates present in the soil to sulfides. The corrosion process can be slowed or mitigated by the use of coatings.

#### **a. Galvanized Coatings**

A common method to protect the base metal, carbon steel, from corrosion is to galvanize it whereby a layer of zinc on the surface is used to protect the underlying steel. Zinc layers are deposited via the hot dip process by dipping the steel member in a bath of molten zinc. Coatings of this type initially protect the underlying metal mechanically. When the continuity of the coating is destroyed by potential difference on the surface, the underlying metal may be protected either galvanically or mechanically by the formation of a protective

film of zinc oxides. The protection process is of a sacrificial nature in which zinc acts as the sacrificial anode to the bare portions of the steel until it is all consumed. However, there is a limit to the distance (throw) along an element between areas of bare steel and zinc coating to achieve an effective level of galvanic action.

**b. Metalization with GALFAN**

GALFAN<sup>®</sup> (a registered trade name of the International Lead Zinc Research Organization (ILZRO)) is a zinc-5% aluminum-mischmetal alloy that offers an alternative to galvanization. GALFAN is applied similar to the hot dip galvanization process. Some observations (CTL, 2001) suggest that GALFAN may perform as well, or better than zinc coating, for slightly acidic, normally to moderately corrosive backfill soils. However, advantages to using GALFAN are less distinct considering mildly corrosive reinforced fill soils, typical of MSE wall construction. Any advantages of using GALFAN versus zinc coating has not been demonstrated for burial conditions that represent engineered, select, granular soils specified for use as MSE wall fill. Furthermore, the performance of GALFAN has not been verified for a wide range of reinforced fill conditions including alkaline soils.

Jailloux and Anderson (1999) describe application of a protective coating consisting of a zinc aluminum alloy (85% Zn-15% Al), called “Dunois”, by thermal spraying. Dunois is proposed as an economical solution for applications with reinforced fill soils that are considered aggressive relative to the potential for corrosion of plain or galvanized steel. Similar to GALFAN, in aggressive environments, Dunois may provide better corrosion protection compared to galvanization, however data is not currently available to verify its effectiveness.

Given the added costs of GALFAN and Dunois compared to zinc, they may only be economical solutions where marginal quality materials are used for reinforced fill and the benefit of improved performance compared to zinc can be realized.

**c. Fusion Bonded Epoxy Coatings**

As another alternative to galvanized coatings, fusion-bonded epoxy coatings on (non-galvanized) steel reinforcements have been used on a number of projects. Galvanized reinforcements may also be epoxy coated. Fusion-bonded epoxy coatings are dielectric. They cannot conduct current and therefore deprive the corrosion mechanism of a path for galvanic currents to flow, essentially terminating the corrosion process.

These coatings need to be hard and durable to withstand abrasion under normal construction conditions and have strong bonding properties to the base metal to ensure long-term integrity.

Significant use of fusion-bonded epoxy protection for underground structures has been made by the pipeline industry. However, in most cases pipelines also use cathodic protection in addition to coatings.

To be effective, fusion-bonded coatings must be impermeable to gases and moisture and free of even microscopically thin gaps at the interface between the metal and the coating. The ability to withstand construction induced abrasions must be determined in order to develop design recommendations that would ensure longevity. Newer epoxy coatings including purple marine epoxy such as 3M Scotchkote 426 are also available.

These corrosion protection and mitigation systems may be beneficial, and offer improved performance, when marginal wall fills are being considered; that do not resist corrosion to the extent of those currently specified by AASHTO. Johnston (2005) describes an MSE wall in Deadwood, South Dakota where fusion bonded, epoxy coated reinforcements were used and performance was documented. Given the added cost compared to zinc, fusion bonded epoxy coatings may be economical solutions where marginal quality materials are used for reinforced fill and the benefit of improved performance compared to zinc can be realized. This is particularly true if the epoxy coating is applied on top of a galvanized surface rendering a double corrosion protection system.

#### **d. Polymeric Barrier Coating with Zinc**

Galvanized steel reinforcement may be coated with a polymeric barrier as described in ASTM A641/A (2004a) and ASTM A975 (2004a). Similar to epoxy coatings, polymeric materials are dielectric and therefore deprive the corrosion mechanisms of the needed current path. To be effective, coatings must be impermeable to gasses and moisture and free of microscopically thin gaps at the interface between the metal and coating. Coatings need to be hard and durable to withstand abrasion under normal construction conditions and bond strongly to the base metal to ensure long-term integrity. The polymeric barrier is used to increase the service-life of the steel and zinc in the MSE reinforcement. The rate of zinc and steel loss may be considerably reduced by the presence of the polymeric barrier. The design life of the reinforcement depends on the durability of the polymer, time for the zinc coating to be consumed, and amount of sacrificial steel.

HITEC (2002) describes evaluation of a system that includes PVC-coated, galvanized, double-twisted wire mesh reinforcement for use within select granular backfill. The PVC coating is extruded onto the galvanized wires before they are twisted together forming the mesh. The steel wires are galvanized to a minimum of 0.8 oz ft<sup>2</sup>, which is equivalent to a thickness of approximately 33 μm, and then PVC coated to a minimum thickness of 0.5 mm.

## 2.2 IDENTIFICATION OF CORROSIVE ENVIRONMENTS

Escalante (1989) and Fitzgerald (199) discuss the effects of soil characteristics on corrosion. General descriptions of soil corrosivity based on pedological descriptions are also possible using soil survey data (Miller et al., 1981). Soil environments that are known to be aggressive relative to corrosion of galvanized or plain steel reinforcements are summarized in Table 2-1. Aggressive soils are identified in terms of electrochemical properties including pH, resistivity, and salt content. Details of each of these soil environments and other conditions that contribute to corrosive conditions including the presence of stray currents and other environmental factors are described in the following subsections.

**Table 2-1. Aggressive Soil Environments.**

<b>Environment</b>	<b>Prevalence</b>	<b>Characteristics</b>
Acid-Sulfate Soils	Appalachian Regions	Pyritic, pH < 4.5, SO <sub>4</sub> (1000-9000 ppm), CL <sup>-</sup> (200-600 ppm)
Sodic Soils	Western States	pH > 9, high in salts including SO <sub>4</sub> and Cl <sup>-</sup>
Calcareous Soils	FL, TX, NM and Western States	High in carbonates, alkaline but pH < 8.5, mildly corrosive
Organic Soils	FL (Everglades), GA, NC, MI, WI, MN	Contain organic material in excess of 1% facilitating microbial induced corrosion
Coastal Environments	Eastern, Southern and Western Seaboard States and Utah	Atmospheric salts and salt laden soils in marine environments
Road Deicing Salts	Northern States	Deicing liquid contain salts that can infiltrate into soils
Industrial Fills	Slag, cinders, fly ash, mine tailings	Either acidic or alkaline and may have high sulfate and chloride content

## **a. Geological**

Potentially corrosive environments are usually characterized as being highly acidic, alkaline or found in areas containing significant organic matter that promotes anaerobic bacterial corrosion. In the United States, acid sulphate soils are often found in areas containing pyritic soils, as in many Appalachian regions in the Southeast and Middle Atlantic States. These soils are further characterized by a high level of soluble iron (Fe) that can produce highly aggressive biogenic iron sulphides. Generally, rock containing pyritic sulfur in excess of 0.5 percent and little or no alkaline minerals will produce a pH of less than 4.5, which has a considerable potential for producing sulfuric acid.

The predominant anion in acid sulphate soils is sulphates with concentrations ranging from 1000 to 9000 PPM (parts per million) and the predominant cation is sodium with reported concentrations of 1500 to 3000 PPM. Typically, acid sulphate soils contain significant soluble levels of iron and chlorides, although levels vary greatly. Chloride levels are reported in the range of 200 to 600 PPM. These soils and rocks are identified by the presence of noticeable yellow mottles attributable to pyrite oxidation.

Alkaline soils are described as being either salt affected (sodic) or calcareous. Sodic soils are generally found in arid and semiarid regions where precipitation is low and there are high evaporation and transpiration rates. In the United States, they primarily occur in seventeen western states.

Sodic soils are characterized by low permeability and thus restricted water flow. The pH of these soils is high, usually >9 or 9.5, and the clay and organic fractions are dispersed. The major corrosive solute comprising dissolved mineral salts are the cation Na and the anions Cl and SO<sub>4</sub>.

Calcareous soils are those that contain large quantities of carbonate such as calcite (calcium carbonate), dolomite (calcium-magnesium carbonate), sodium carbonates, and sulfates such as gypsum. These soils are characterized by alkaline pH, but the pH is less than 8.5. Calcareous soils are widespread and occur in Florida, Texas, New Mexico, and many of the Western States and are generally mildly corrosive.

Organic soils are classified as bogs, peats, and mucks. Most organic soils are saturated for most of the year unless they are drained. They contain organic soil materials to a great depth. The major concentrations are found in the Everglades of Florida, the Okefenokee Swamp in Georgia, the Great Dismal Swamp in North Carolina and Virginia, and in the peat bog areas of Michigan, Wisconsin, and Minnesota. It is estimated that one-eighth of the soils of

Michigan are peats. They are, however, locally widespread throughout the United States. Dredged soils, widespread along coastal areas, generally also contain a high percentage of organic matter.

Salt concentrations may be affected during service due to contamination in marine environments. The soil along the coast often contains salt. Salt can also directly intrude into reinforced fill where structures are constructed directly on the sea front or nearby (i.e., during storm surge events). The air in coastal environments often contains a level of salt that can deposit on the ground surface and intrude into the soil over time. Salt contamination may also occur due to use of deicing salts along roadways in the northeast and other regions subject to snow and ice, as discussed in the next section.

Industrial fills such as slag, fly ash, and mine tailings may be either acidic or alkaline depending on their origin. Cinder ash or slag-cinder ash mixtures in particular are likely to be acidic and contain significant amounts of sulphates. Slag may or may not be corrosive, as the characteristics of slag vary depending on how it is cooled and processed, e.g. air-cooled blast furnace slag is alkaline; not acidic. On the other hand, cinder ash is high in salt, may be acidic and performance problems of MSE constructed using mixtures of slag and cinder ash have been documented by the NYSDOT (Moody, 1993) and Elias (1990)). Modified soils, cement, or lime treated can be characterized by a pH as high as 12. Crushed concrete may also contain un-hydrated free lime. The corrosive potential of industrial fills should be evaluated using the procedures discussed in Chapter 3 before considering the use of these materials as reinforced fill.

#### **b. Salt Water Intrusion**

MSE reinforced fill may be contaminated during service from deicing salts applied to the roadways. Sodium chloride is by far the most popular chemical deicer, although calcium magnesium acetate (CMA) has been considered as an alternative (TRB, 1991). Chemical deicers may be applied alone or mixed with sand. If the salt is able to permeate the reinforced fill and comes into contact with steel or galvanized reinforcements, accelerated corrosion can occur. The salt intrusion is not uniform and causes the potential along the surface to vary with distinctly defined areas of cathodic and anodic activity. This may lead to localized attack of the protective layers of oxides adhered to the surface leading to a pitting type of corrosion. The advantage to using CMA is that it is less corrosive to steel compared to sodium chloride.

A remedy to salt intrusion is to ensure an impervious barrier exists near the wall top that covers the reinforced fill. An impervious barrier (i.e., geomembrane) beneath the pavement and above the reinforced fill, and draining to a collection system, is recommended (Berg et al., 2009). It is unlikely that the pavement surface itself can provide the needed barrier, even if it fully covers the reinforced soil zone, as joints and cracks will inevitably allow infiltration during winter deicing salt use. An impervious barrier is particularly important where runoff water flows toward the wall and/or where snow piles will exist above the wall. A pavement surface could provide protection if the surface is graded such that the runoff flows away from the wall, joints and cracks are quickly sealed, the pavement section is well drained, and a free-draining (i.e., less than 3 to 5% non plastic fines passing the No. 200 (0.075 mm) sieve, e.g. AASHTO No. 57 stone) reinforced fill is used. Even with these precautions, a long-term corrosion monitoring program is recommended if a pavement is to provide the barrier to deicing salt runoff into the reinforced fill.

The effect of salt contamination on corrosion rates may also be minimized by the use of free draining material (i.e., less than 3 to 5% non plastic fines passing the No. 200 (0.075 mm) sieve) for reinforced fill. Timmerman (1990) studied salt infiltration from deicing salts into bridge approach embankments and the long-term effects on MSE stability due to potential corrosion of the metal system components. Reinforced fill samples were retrieved through holes cored in the wall facing of the selected walls in Ohio and tested for chlorides, pH, and resistivity. Samples were taken at approximately 3-week intervals between January 1988 and April 1989. Of the twenty-seven sampling locations at the three MSE bridge abutment test sites, only three locations had measured soil parameters, which could be considered as marginally corrosive to the MSE components; and these conditions only existed for limited time periods during the winter road-salting season. This result was attributed to the use of free-draining reinforced fill specified by the Ohio DOT for MSE wall construction.

The Florida Department of Transportation observed corrosion rates of walls subjected to tidal inundation (Sagues, 2000). Accelerated corrosion rates were not observed, which was attributed to the use of free draining backfills and corresponding flushing of chlorides during the rainy season.

### **c. Stray Currents**

In addition to galvanic corrosion, stray currents may be an additional source of corrosion for MSE systems constructed adjacent to electrically powered rail systems or other sources of electrical power that may discharge current in the vicinity of these systems, such as existing utilities, cathodically protected radio stations, etc. Stray earth currents can be caused by DC-powered transit or other rail systems. These currents are generated by the voltage drop in the

running rails, which are used as negative return conductors. This potential difference causes differences in track-to-earth potential that varies with time, load (train), location, and other factors. Earth-potential gradients are generated by stray current leakage from the rails. The magnitude of this current is a function of track-to-earth potential and resistance. The magnitude of stray earth current being discharged or accumulated by a source can be estimated by measuring earth electrical gradients in the source area. From these measurements, the probable effect of stray corrosion can be estimated by a corrosion specialist.

In general, stray currents decrease in magnitude rapidly as they move away from the source and are believed not to be a factor 100 to 200 ft (30 to 60 m) away from the source. For structures constructed within these distances, AASHTO recommends that a corrosion expert evaluate the hazard and possible mitigating features on a project-specific basis. Furthermore, it is recommended that a long-term corrosion monitoring program be integrated into the design, if steel reinforcements are used.

For direct current traction power railways, there is some indication that the effect of stray currents on the corrosion of metal strip type reinforcements depends on the orientation of the strips with respect to the current flow path (Sankey and Anderson, 1999). For many installations the strips are orientated perpendicular to the current flow path, and provided the resistivity of the backfill is sufficiently high, effects from stray currents may not be a significant concern. A corrosion expert should determine whether stray currents are not a significant concern for a specific project and a long-term corrosion monitoring program should be integrated into the wall design to confirm the recommendations.

#### **d. Other Environmental Factors**

The level of compaction and grain size distribution of backfills placed around reinforcements have an effect on corrosion and corrosion rates.

##### Soil Compaction

Compaction of soil is defined as the reduction of air voids between particles of soil and is measured by the mechanical compression of a quantity of material into a given volume. When soil compaction occurs evenly, soil resistivity is consistent and corrosivity is generally decreased. Soil permeability is reduced with compaction and provided drainage is adequate and the soil is non-aggressive (neutral or alkaline), corrosion should be decreased. However, the effect of compaction is related to soil cohesiveness. In clay soils, the corrosion rate shortly after burial increases with compaction. Well-drained, granular soils with moisture contents of less than 5 percent are non-aggressive, but drainage decreases with increasing

compaction, leading to marginal increases of corrosion. These theoretical marginal differences have not been quantified to date.

### Moisture Content

Soil structure, permeability, and porosity determine the moisture content of a soil. Where the moisture content of a soil is greater than 25 to 40 percent, the rate of general corrosion is increased. Below this value, a pitting type corrosion attack is more likely.

The corrosion of mild steel increases when soil moisture content exceeds 50 percent of saturation. This may be compared to the critical relative humidity (rh) that occurs above ground in atmospheric corrosion. Research data strongly suggest that maximum corrosion rates occur at saturations of 60 to 85 percent (Darbin et al., 1986). This range of saturation for granular materials roughly corresponds to the range of moisture content required in the field to achieve needed compaction levels.

A survey of 14 California sites found saturation levels in MSE fills to be between 30 and 95 percent, with most samples exceeding 65 percent (Jackura et al., 1987). Therefore the placement compaction requirements for MSE structures will be subject to the maximum corrosion rates consistent with all other electrochemical criteria.

## **2.3 ELECTROCHEMICAL TEST METHODS**

The design of the buried steel elements of MSE structures is predicated on the measurement of key index parameters of the reinforced fill, which govern corrosivity, the desired life of the structure, and the assessment of such basic environmental factors as location and probability of changes in the soil/water environment. Several parameters influence soil corrosivity, including soil resistivity ( $\rho$ ), degree of saturation, pH, dissolved salts, redox potential and total acidity. These parameters are interrelated but may be measured independently. The direct link between any one soil parameter and a quantitative corrosion relationship has not been fully substantiated, but a general consensus has been established based on studies of buried metals that resistivity is the most accurate indicator of corrosion potential (Romanoff, 1957; King, 1977). Current research projects (2008), NCHRP 21-06 and 24-28, are focused on developing a better understanding between laboratory measurements of index properties and in-situ corrosion rates.

The frequency and distribution of samples for assessment of electrochemical parameters needs to be given careful consideration. The number of samples required increases when evaluating more aggressive or marginal backfill materials, and when more confidence is

needed for design (Withiam et al., 2002; Hegazy et al., 2003). Existing data involving frequent sample intervals at sites with poor conditions depict a wide scatter in results (Whiting, 1986; Fishman, et al, 2006). For moderate to large sized projects, with fill sources that are expected to be relatively nonaggressive relative to corrosion (i.e. mildly corrosive soils meeting AASHTO criteria), Table 2-2 can be used to determine the number of samples that should be taken from each source and evaluated for electrochemical parameters. More samples should be retrieved if marginal quality reinforced fills are being contemplated for construction (not recommended), or when undertaking performance evaluations at sites with poor reinforced fill conditions. In addition to the mean values used for design (i.e., the mean of the minimum resistivity ( $\rho_{\min}$ ) values obtained from each test), the distribution and variability of the measurements is of significant interest from the standpoint of reliability-based design (LRFD).

Table 2-2 places restrictions on the allowable standard deviations ( $\sigma$ ) of the resistivity and salt content (see comment 3) measurements. If these standard deviations are exceeded, then the sampling should be repeated. If the standard deviation, computed using the total numbers of samples, is still outside the limits of Table 2-2, then the backfill source should not be used for MSE wall fill. If resistivity less than 3000  $\Omega$ -cm is obtained from any test, obtain additional samples in the vicinity of this sample location to identify if there are specific areas wherein the material is unsuitable.

Stockpiles should be sampled from the top, middle and bottom portions and an excavator with a bucket should be used to remove material from approximately two feet beyond the edge of the stockpile. Particular emphasis on sampling needs to be placed at sites where different reinforced fill sources and/or types are being considered; and each source should be sampled as described in Table 2-2.

Differences in the electrochemical properties of the soil fill can adversely effect corrosion rates, and contribute to more severe and localized occurrences of metal loss. In instances where more easily compacted (e.g. open graded) material is placed adjacent to the wall face, significant differences in the soil fill conditions may exist with respect to position along the reinforcements. For cases where reinforcements are not electrically isolated (e.g., metallic facing) variations of backfill types along the height of the wall may also have a significant effect on corrosion rates of metallic reinforcements.

**Table 2-2. Recommended Sampling Protocol for Electrochemical Testing of MSE Wall Fill.**

Range of $\rho_{\min}$ ( $\Omega$ -cm)	General Description	Preconstruction		During Construction	Comments
		No. Samples	$\sigma_{\text{resistivity}}$ ( $\Omega$ -cm)	Sample Interval ( $\text{yd}^3$ )	
> 10,000	Crushed rock and Gravel, < 10% passing No. 10 sieve	1 / 3 <sup>1</sup>	NA	NA	<b>1.</b> pH outside the specified limits is not allowed for any sample. <b>2.</b> backfill sources shall be rejected if $\rho_{\min}$ measured for any sample is less than 700 $\Omega$ -cm, $\text{Cl}^- > 500$ ppm or $\text{SO}_4 > 1000$ ppm. <b>3.</b> For materials with $\rho_{\min} < 5000$ $\Omega$ -cm, $\sigma$ for $\text{Cl}^-$ and $\text{SO}_4$ shall be less than 100 ppm and 200 ppm, respectively.
5,000 to 10,000	Sandy Gravel and Sands	3 / 6 <sup>1</sup>	< 2000	4000 / 2000 <sup>1</sup>	
< 5,000	Silty sands and Clayey sand, screenings	5 / 10 <sup>1</sup>	< 1000	2000 / 1000 <sup>1</sup>	

<sup>1</sup> # resistivity tests / # tests for pH,  $\text{Cl}^-$ , and  $\text{SO}_4$

The influence and measurements techniques for key parameters used in construction control can be summarized as follows:

**a. Soil Resistivity**

Soil resistivity is defined as the inverse of conductivity. Resistivity is the convention of expressing the resistance of materials in units of ohm-cm. For more practical chemical and biological usage, the scientific community uses the algebraic inverse of ohm-cm resistance for conductivity expressed in mhos. The current preferred international standard (SI) system uses the term electrolytic conductivity expressed in units of siemen per meter (S/m) in which 100 S/m is equal to 1 mhos/cm.

The electrolytic behavior of soils is an indirect measurement of the soluble salt content. The amount of dissolved inorganic solutes (anions and cations) in water or in the soil solution is directly proportional to the solution electrolytic conductivity. The major dissolved anions in soil systems are chloride, sulfate, phosphate and bicarbonate, with chloride and sulfate the most important anionic constituents in corrosion phenomena. The electrolytic conductivity (EC) of the soil solution is the sum of all the individual equivalent ionic conductivities times their concentration.

Because soil resistivity governs the effectiveness of the ionic current pathway, it has a strong influence on the rate of corrosion, particularly where macro-corrosion cells are developed on larger steel members. Corrosion of MSE reinforcements increases as resistivity decreases (King, 1977). However, if resistivity is high, localized rather than general corrosion may occur. Increased soil porosity and salinity decreases soil resistivity. The importance of and interaction between compaction, water content, and resistivity on corrosion processes has perhaps been under emphasized in many of the available studies.

Resistivity should be determined under the most adverse condition (saturated state) in order to obtain a comparable resistivity that is independent of seasonal and other variations in soil-moisture content. AASHTO has adopted Method T-288 for measuring resistivity after review and analysis of a number of available methodologies. This laboratory test measures resistivity of a soil at various moisture contents up to saturation and reports the minimum obtained resistivity. Variations of resistivity should be expected between stockpiled soils and from subgrades, especially if the soils are friable. AASHTO Method T-288 is used for soil fills, and it requires testing on the fraction of the material that passes a No. 10 sieve. This is a problem when using coarse reinforced fills that have very little, or no, material finer than the No. 10 sieve, because we are interested in the soluble salts from within the fines.

For coarse fills with a little material passing the No. 10 sieve, a sufficient amount of fines for testing might be obtained from sieving a large quantity of the material. For no, or low, amount passing the No. 10 sieve sufficient fines may be generated from the construction of a test pad using representative construction equipment and techniques. This may be applicable if the fill material will be subject to comminution during construction. Note that crushing of material to obtain the finer fraction is not appropriate, nor dictated by AASHTO T-288; unless some breakage is anticipated during placement and compaction.

Another approach, similar to that implemented by the North Carolina Department of Transportation (Medford, 1999), is to perform specialized resistivity tests on water that has been decanted after soaking the aggregate for 24 hours. Resistivity tests are performed on the supernate in accordance with ASTM D1125.

Additional work is needed to define appropriate procedures for obtaining material for and testing of coarse fills with little, to no, materials passing the No. 10 sieve. The Texas Department of Transportation is sponsoring a study that will address the proper method of measuring the electrochemical properties of coarse, reinforced fill materials (Grant # 0-6359). This research is being conducted at the University of Texas, El Paso with a scheduled completion date in 2011.

Adkins and Rutkowski (1998) compared results from in-situ resistivity measurements to laboratory measurements including AASHTO T-288 on samples of reinforced fill extracted from MSE structures at twelve different sites. Additionally, results from resistivity and chemical tests including chloride and sulfate ion concentrations were compared and evaluated. Resistivity is very sensitive to moisture content and good correlations between field and laboratory test results were only obtained if special precautions were taken to ensure that moisture content of laboratory specimens were not altered compared to in-situ conditions. The best comparisons between in-situ measurements and laboratory tests were obtained with respect to samples tested onsite (using laboratory techniques), immediately upon extraction, rather than from samples that had been transported to the laboratory. The minimum resistivity obtained with AASHTO T-288 is always significantly less than insitu measurements, however results from this test were found to be very consistent and repeatable.

Results from in-situ testing may be useful for the purpose of comparing observed corrosion rates to in-situ conditions. However, using the results from the minimum resistivity test is considered prudent for screening reinforced fills sources for MSE construction. Although the extremely wet conditions associated with the minimum resistivity test may not represent the average moisture content in free draining reinforced fills, the moisture content close to the interface between the reinforcement and the reinforced fill is more relevant. In the authors' opinion this could be higher than the average moisture content due to condensation and collection of moisture on the surface of the impermeable steel reinforcement.

The relative level of corrosiveness, commonly accepted by the engineering community as indicated by resistivity levels, is shown on Table 2-3. Based on these, resistivity ranges in the moderately corrosive to mildly corrosive ranges are generally chosen as lower bound values. From the National Bureau of Standards data (Romanoff, 1957) shown on Figures 2-1 and 2-2, a rough estimate can be made that suggests corrosion rates are roughly increased by 25 percent in each successive aggressiveness range, all other conditions being essentially equal.

**Table 2-3. Effect of Resistivity on Corrosion. (NCHRP, 1978)**

<b>Aggressiveness</b>	<b>Resistivity (ohm-cm)</b>
Very corrosive	< 700
Corrosive	700 - 2,000
Moderately corrosive	2,000 - 5,000
Mildly corrosive	5,000 - 10,000
Non-corrosive	> 10,000

A quantitative limit has been established for MSE reinforced fills when using metallic reinforcements requiring a minimum resistivity in the saturated state greater than 3000 ohm-cm. This limit has been pragmatically established in recognition that soils meeting this criteria are widely distributed and available in the United States. Further, the associated corrosion rates are moderate and would not require significant sacrificial steel for the 75-100 year design life. The New York State Department of Transportation allows resistivity to be somewhat lower than 3000 ohm-cm, provided that the chloride and sulfate ion contents are below 100 ppm and 200 ppm, respectively (Moody, 1993). The California Department of Transportation also allows soils with resistivity less than 3000 ohm-cm, but compensates by corresponding increases in sacrificial steel (Jackura et al., 1987) and reduces the design life of the structure to 50 years (Caltrans, 2003)).

#### **b. Soluble Salts**

The amount of dissolved inorganic solutes (anions and cations) in water or soil is directly proportional to the solution electrolytic conductivity. Therefore, the electrolytic conductivity (inverse of resistivity) of a soil solution is the sum of all the individual equivalent ionic conductivities times their concentration.

Most salts are active participants on the corrosion process, with the exception of carbonate, which forms an adherent scale on most metals and reduces corrosion. Chlorides, sulphates and sulfides have been identified in the literature as being the chief agents in promoting corrosion (Romanoff, 1957). The accurate determination of chloride, sulfate and sulfide portions of the total salt content is an important element in determining corrosivity. It should be noted that the level of measurable soluble salts in a borrow area or quarry can and often is, highly variable and is effected by non chemical variables such as surface area of each particle and material soundness during handling. Each of these salts are discussed further in relation to available test methods.



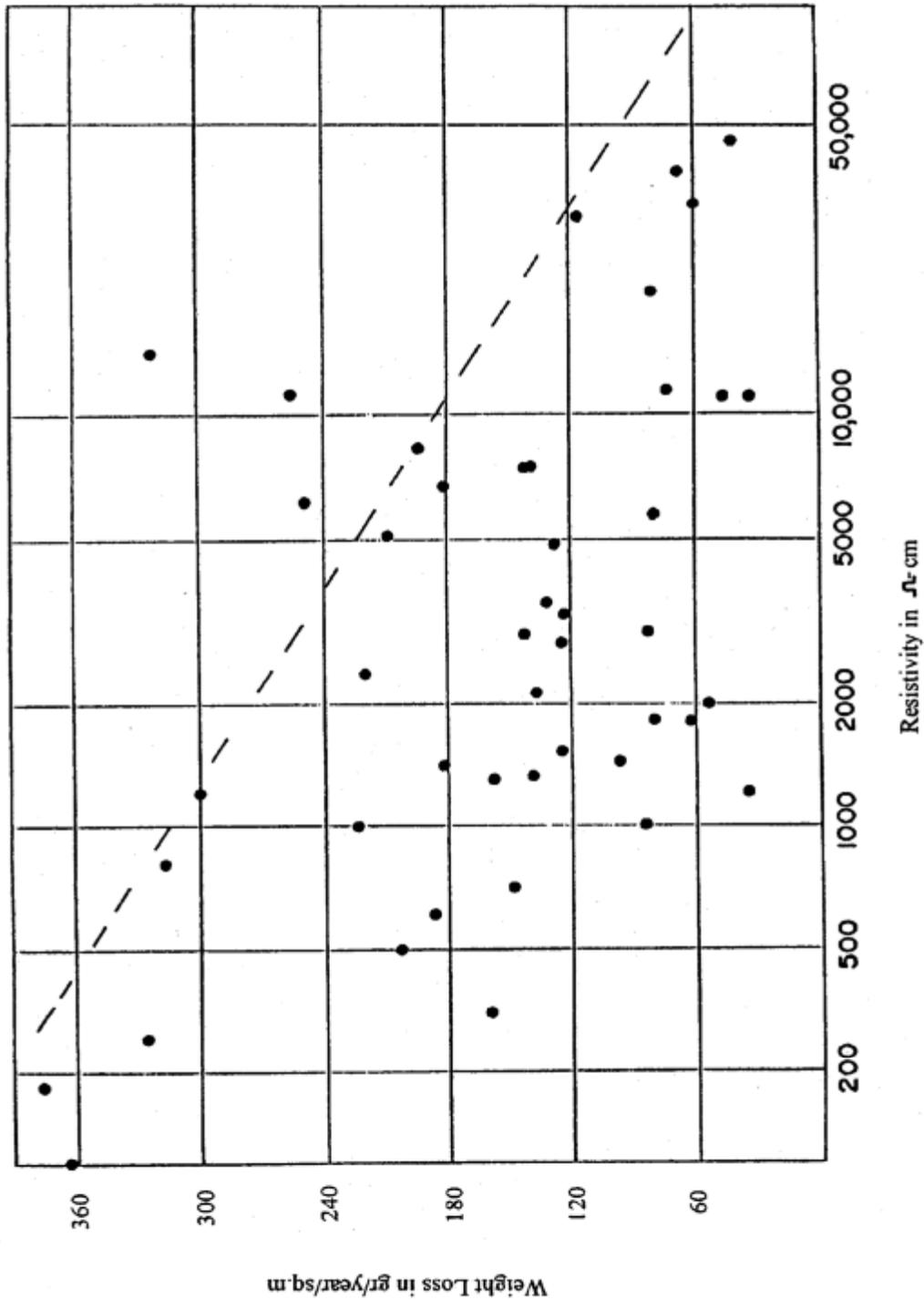


Figure 2-2. Metal loss as a function of resistivity (carbon steel). (Elias, 1990)  
 [1 gr/year/m<sup>2</sup> = 0.127 μm/year]

## Chlorides

Chloride minerals are very soluble and thus completely removed by an aqueous extract. Chloride determination methods can be categorized as electrometric or colorimetric. The electrometric methods available include potentiometric titration (i.e. Mohr argentometric), coulometric by amperometric automatic titrator, direct reading potential (i.e. selective ion electrode), or solution conductance with prior separation by ion exchange. The mercury thiocyanate colorimetric method has been devised for application for autoanalyzers.

AASHTO has adopted an electrometric Method T-291 as the method for measuring chlorides concentrations for MSE walls. ASTM D4327 was adopted as a standard test to measure anions, including chloride, by ion exchange chromatography. It is the most accurate and reproducible of all methods and is well suited for automated laboratories. Most analytical labs have this equipment and prefer to run this test rather than AASHTO T-291. The test is more automated, less expensive, and provides an indication of the potential for interferences, which are not identified by AASHTO Method T-291. The use of ASTM D4327 is recommended. Furthermore, it is recommended that agencies clarify which method to use in their specifications.

## Sulfates

The extraction and quantification of soil sulfur imposes a more complex problem than chloride. Sulfate represents only one of the fractions in which sulfur can exist in the soil. In addition to different sulfur forms, the inorganic sulfate may occur as water soluble (i.e. sodium sulfate), sparingly soluble (i.e. gypsum) or insoluble (i.e. jarosite) minerals. The solubility of sulfate is also restricted in some soils by absorption to clays and oxides or by co-precipitation with carbonates. The water-soluble sulfate will not represent the total sulfate in all soils but it is an appropriate choice for quantifying the soil solution activity with regard to corrosion potential.

AASHTO has adopted Method T-290 as the method of measuring water soluble sulfate concentrations for MSE walls. This is a chemical titration method. As with chloride measurements, ASTM D-4327 methods by ion chromatography are the most accurate and reproducible of all methods. The use of ASTM D4327 is recommended. Furthermore, it is recommended that agencies clarify which method to use in their specifications.

## Sulfides

Sulfide containing soils can cause severe deterioration of both steel and concrete. Freshly exposed sulfidic materials will have no indication of acid sulfate conditions when analyzed in the laboratory. Typical pH values will be from 6 to 8 with a low soluble salt content. Once the material is exposed to aeration by disturbance or scalping of the land surface, the sulfides

oxidize chemically. Characteristics of active acid sulfate weathering include pH values lower than 3 due to free sulfuric acid generation and appearance of salt efflorescences. The occurrence of sulfidic materials is generally limited to geologic formations derived from marine sediments or strata associated with coal and lignitic geologic materials.

The pyritic sulfur of these materials is quite variable and no simple quantitative method is available. If quantitative determinations of pyritic sulfur are desired, ASTM method D-2492 may be used.

A qualitative test would involve oxidation of the sulfide with hydrogen peroxide and subsequent determination of the converted sulfate. Even though complete conversion is not likely by this treatment, sufficient increase in sulfate would be evidence for sulfides. This latter procedure is recommended where geological investigations detect the presence of pyrite. At present, no standard test method exists for this procedure.

#### Maximum Acceptable Levels

Chlorides, sulfates and other dissolved salts decrease resistivity, promoting the flow of corrosion currents and impeding the formation of protective layers. The USDA (1969) and Rehm (1980) studied the effect of chlorides and sulfates on resistivity for both theoretical considerations and controlled laboratory tests (Elias, 1989).

From these data, it can be inferred that soils in the moderately corrosive range (5000 to 2000 ohm-cm resistivity) would be limited to a range of 60 to 180 PPM for chloride ions or 90 to 280 PPM for sulfates. Where other soluble salts are present, or a combination of chloride and sulfates, these concentrations would be reduced.

Consistent with a minimum resistivity of 3000 ohm-cm, the maximum level of chlorides and sulfates in reinforced soil fills with metallic reinforcements has been established in the current AASHTO LRFD Bridge Construction Specifications (2004), as shown in Table 2-4.

**Table 2-4. Maximum Permissible Levels of Soluble Salts.** (after AASHTO, 2004)

<b>Soluble Salt</b>	<b>Maximum Level</b>	<b>Test Method</b>
Chlorides	100 PPM	ASTM D4327
Sulfates	200 PPM	ASTM D4327
Note: ASTM D4327 test method is recommended, current AASHTO calls for use of AASHTO T-290 and T-291 test methods.		

### **c. pH**

The measurement of pH represents the hydrogen ion concentration in solution. Values of soil pH represent the hydrogen activity in the soil solution and are referred to as the intensity factor. Soil pH is probably the single most informative measurement that can be made to determine soil characteristics (Thomas, 1996), and pH provides more information about a soil than merely whether it is acidic or basic. A major factor influencing pH of soils is the salt content of the soil solution, and there is often a tendency for the pH of the soil to rise in the presence of salts. Soil with a pH of 2 to 3 is usually associated with pyritic minerals, which upon oxidation forms sulfuric acid. At the other end of the scale pH is an indicator of excess calcium carbonate, and normally soils with pH values of 7.6 to 8.3 are found to be calcareous. When pH values stray towards 9, calcium carbonates are no longer controlling the system, and soluble salts such as sodium carbonates become the dominant species and normal salts of sodium including sodium chloride and sodium sulfate may be present.

The most widely accepted procedure for measuring the soil pH is by the pH glass electrode-calomel reference electrode pH meter on a 1:1 weight ratio of soil to water, which is consistent with AASHTO T-289 test method. The corrosion literature suggests that for bare steel and pH between 4 and 10, the corrosion rate is independent of pH and depends only on how rapidly oxygen diffuses on the metal surface. Resistivity tends to be higher in acid soils than alkaline soils. This effect is associated with moisture content, as highly buffered neutral and alkaline soils generally contain a significant clay fraction. This will tend to lead to a higher moisture content, the presence of which will reduce the resistivity of the soil.

Soils that are extremely acidic (pH less than 4.0) or very strongly alkaline (pH greater than 10) are generally associated with significant corrosion rates. Zinc is vulnerable to corrosion in strongly acidic and alkaline soils, whereas steel is passivated in alkaline environments (i.e.,  $\text{pH} > 10$ ) and with a pH less than 13. Therefore, the galvanized coating will have a significantly lower life expectancy in either acidic or alkaline regimes. Alkalinity tends to favor steel performance, unless it is associated with high concentrations of salts. A reasonable allowable pH range may therefore be inferred as being greater than 5 and less than 10 when using metallic reinforcements.

### **d. Organic Material**

Some soils contain a high proportion of organic material in which general microbial growth will reduce this material to organic acids which, when in contact with metals, produce pitting corrosion. The inclusion of organics in the reinforced soil fill can initiate the formation of

anaerobic pockets of soil, which could be contaminated with sulphate-reducing bacteria (SRB), thereby initiating microbial attack in the form of severe pitting.

AASHTO T-267 is normally used to determine organic content on soil fractions smaller than the #10 sieve (2 mm). This tests measures organic content by loss on ignition at 450° C for six hours. Total organic content to preclude formation of anaerobic pockets should be limited to 1 percent by weight of the total soil fraction.

Materials used for reinforced wall fill that are initially free of organics may become contaminated during service. For example, if fertilizers are used to promote vegetation along the back and/or side slopes of a retaining wall system, organics may permeate into the fill from percolation of rainwater, or from storm water run-off. Planted wall facings may also present the possibility of contaminating fill with organics near the wall face. Planted wall facings should be avoided when using metallic reinforcements. Accelerated corrosion on wire facing systems often used for plantings should also be anticipated and carefully evaluated to determine maintenance requirements Use of fertilizers should be avoided, or impervious barriers should be employed beneath the topsoil (i.e., on top of the backfill) if fertilizers are applied to back and side slopes.

## **2.4 DESIGN CORROSION RATES AND THEIR APPLICATION**

### **a. Available Data**

The most comprehensive data available in the field of underground corrosion are the results of extensive field testing on metal pipes and sheet steel buried by the U.S. National Bureau of Standards (NBS) in programs originating as early as 1910 (Romanoff, 1957). Additional data include the results of many studies conducted in the United States on the performance of metal highway culverts and buried piling. These data, generally qualitative rather than quantitative, are substantially in good agreement with the extensive burial tests conducted by NBS (NCHRP, 1978). A general conclusion of the above studies is that the rate of corrosion is greatest in the first few years of burial and then levels off to a steady but significantly lower rate.

Based on these studies, Romanoff at NBS suggested the following exponential equation to predict the amount of general corrosion at some time (t) after burial:

$$x = K t^n \quad (2-1)$$

where  $x$  is the loss of thickness or pit depth in the metal at time ( $t$ ) and  $K$  and  $n$  are constants that are soil and site dependent ( $n$  is less than unity). For low alloy and carbon steels in a number of soil burial conditions, NBS determined a " $n$ " constant varying from 0.5 to 0.6 and " $K$ " constants between 150 and 180  $\mu\text{m}$  at the end of the first year. For galvanized steels, " $n$ " constants were not evaluated, but " $K$ " constants varying from 5 to 70  $\mu\text{m}$  can be inferred.

It should be noted that the NBS data was developed from a wide range of burial conditions not necessarily reflective of select granular fills required for MSE wall structures. Various transportation departments have conducted corrosion studies with reference to metal culvert durability, summarized in NCHRP-50 (NCHRP, 1978). This summary indicated that a number of analytical methods have been proposed by transportation agencies in California, New York, and Utah that appear to be locally satisfactory. However, no method has found widespread acceptance.

Results from carefully controlled French laboratory tests, simulating field burial conditions, strongly suggests that for the range of fills utilized in MSE applications, the constant " $n$ " may be taken as 0.60 for galvanized steel while the zinc coating is still present and from 0.65 to 1 for carbon steel once significant corrosion occurs (Darbin et al., 1986). The constant  $K$  calculated at the end of the first year, for galvanized steel was found to vary between 3 and 50; with the higher values consistent with soils characterized by lower resistivities and highest concentrations of chlorides and sulfates.

The loss data as a function of time for the French studies are shown on Figures 2-3 and 2-4. The data with reference to the constant " $K$ " have been analyzed in an attempt to determine any relationship with resistivity and degree of saturation. The scatter is significant, but for resistivities greater than 5000 ohm-cm, the range for " $K$ " reduces to 8 to 45, with an average value on the order of 25. The data further suggest that once the galvanized zinc coating is depleted, the base carbon steel corrodes at the carbon steel rate.

The Association for Metallically Stabilized Earth (AMSE) has compiled performance data from observations of insitu reinforcements collected by several different researchers and state departments of transportation (AMSE, 2006; Gladstone et al., 2006). These data document performances of reinforcements that have been in service for up to 34 years at various locations within the United States and Europe. In general, the observed performance has exceeded expectations based on currently available metal loss models whenever good design and construction practices as prescribed by current AASHTO specifications were followed. The few, well documented, cases of poor performance are always correlated with a very aggressive environment; attributed to reinforced fill or to site conditions that do not conform

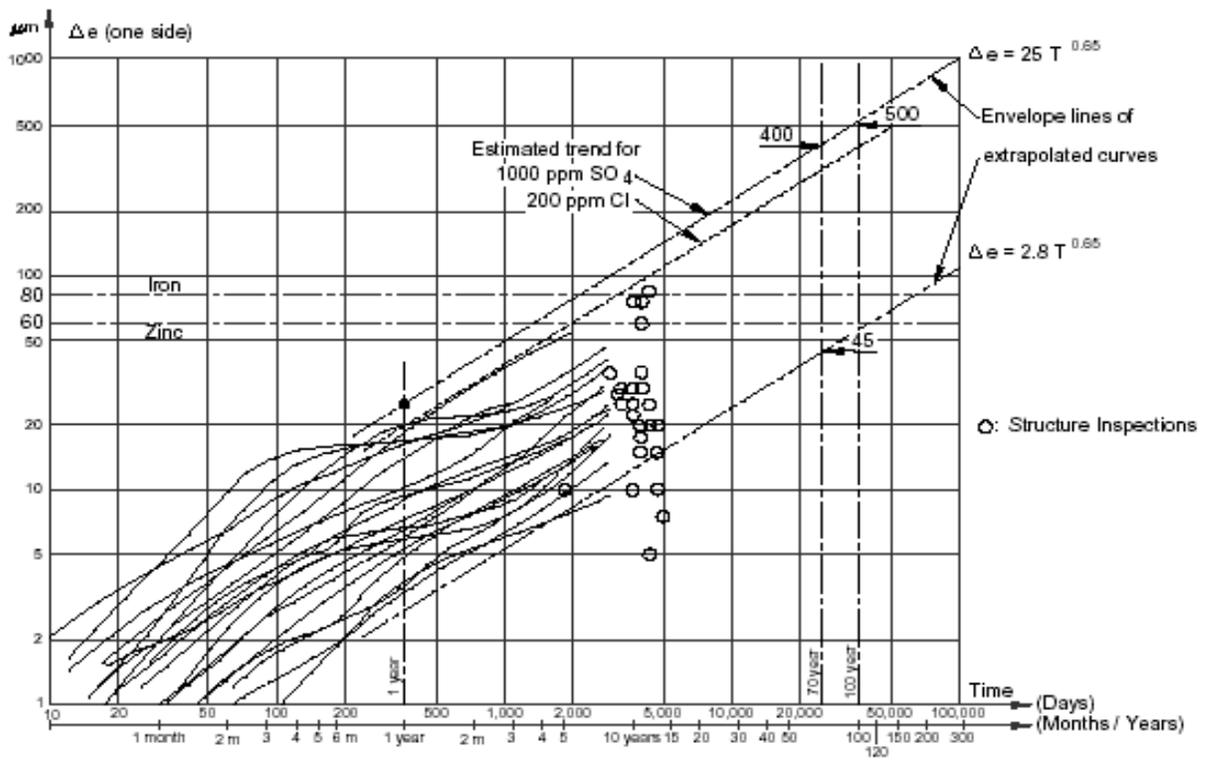


Figure 2-3. Summary of Electrochemical Test Cell Data at 25% Saturation. (Darbin et al, 1988)

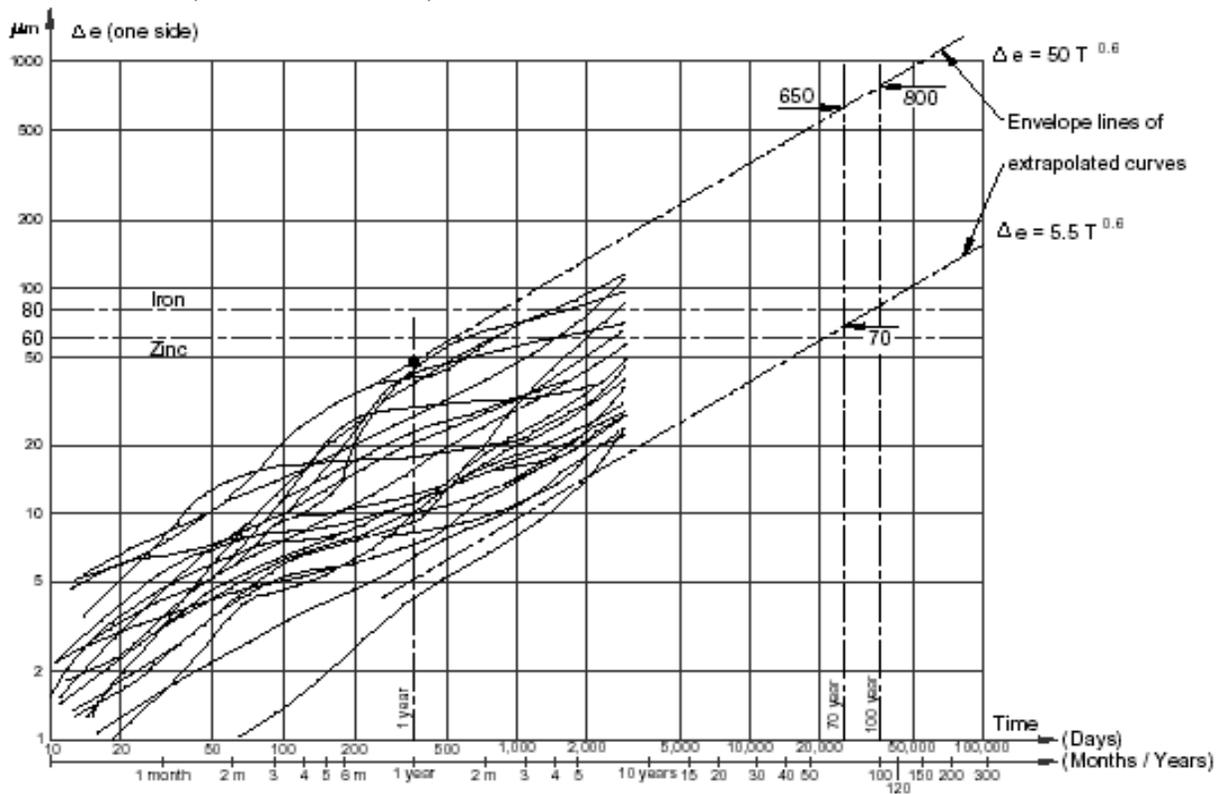


Figure 2-4. Summary of Electrochemical Test Cell Data at 50% and 100% Saturation. (Darbin et al, 1988)

to current AASHTO specifications (FronDistou-Yannis, 1985; McGee, 1985; Whiting, 1986; Blight and Dane, 1989; Elias, 1990; Deaver, 1992; Medford, 1999; Wheeler, 2002; Armour et al., 2004; Fishman et al., 2006).

Most of the walls included in the AMSE database are constructed with reinforced fills that exceed the minimum requirements described by AASHTO by a wide margin, and the data do not reflect a comprehensive geographic distribution of site locations. A currently active research project, sponsored by NCHRP (NCHRP Project 24-28), is addressing these database limitations. Results from NCHRP Project 24-28 are expected to be available by the end of 2009 and will describe the statistics and variations of observed corrosion rates and correlations with different geographies and corresponding climates. Note that no change in recommended design corrosion rates is anticipated.

Highway agencies in the European Union are undertaking a similar data collection effort. Construction of MSE walls in Europe predates that in the United States, and the ages of the oldest MSE structures in this data collection effort are approaching 40 to 50 years. These data may be compared to observations made 20-years ago on the same structures. These data will be useful, with recognition that the practices employed in Europe during the 1960's and 1970's are dissimilar to current U.S. practices; as the reinforcements do not include as much zinc and thinner base steel thickness were employed compared to current U.S. design standards.

## **b. Design Approach**

The generalized corrosion rate relationship developed by Romanoff has been found to be a reasonable predictive model to determine the range of corrosion rates for single-phase materials for the wide range of soils found in nature. The difficulty in its implementation for galvanized steels has been in determining "K" and "n" constants that might reasonably reflect the specific environment and integrate the transition in corrosion rates between a galvanized state and the subsequent bare steel phase. Such a model has not been adequately studied to date, and, therefore, extrapolations are based on limited present data. In addition, since corrosion does not occur in a uniform manner, loss of cross-sectional area will be greater where significant pitting or greater localized corrosion occurs than a loss computed by distributing corrosion losses uniformly over an element. The NBS data, substantiated by several studies (Blight and Dane, 1989; Smith et al., 1996), suggest that pitting depths could be significantly deeper than depths suggested by uniform loss. Pitting depths, however, are somewhat attenuated in uniform backfill environments for galvanized steel as evidenced by both NBS data, British studies, and results obtained in France at least in the early stages of carbon steel corrosion.

Consideration must be given to effects on tensile strength by the pitting mechanism in choosing an appropriate "K" and "n" constant when using a uniform rate of corrosion model. Alternately, the effect of non-uniform corrosion losses on the tensile strength of reinforcing members may be considered statistically, based on test results that relate the relative loss of tensile strength to relative average thickness loss. The data developed from buried samples of strip type reinforcements that had undergone significant corrosion losses strongly suggest that a factor of approximately 2 exists between average thickness loss to average tensile strength loss (Elias, 1990).

Based on theoretical considerations, a factor closer to 3, which is negatively correlated with diameter, is more applicable to single reinforcement elements with a circular cross section such as used in bar mats and welded wire fabric (Smith et al., 1996). Additional corroboration with test data from reinforcement samples collected from the field is needed to verify the appropriate factor for geometries other than flat strips. Current design specifications presume that the factor of 2, which was initially and specifically determined for flat thin galvanized steel strips, also applies to all other types of galvanized steel reinforcements, whatever their size and shape. Note that due to redundancy of bar mats, that a factor of 2 is also considered reasonable for these reinforcements.

Using the NBS model, the available data for a wide range of soils suggest that for galvanized steel loss determinations using the uniform model concept, the following equation may be applicable:

$$X = 25 t^{0.65} \quad (\text{Average}) \quad (2-2)$$

$$X = 50 t^{0.65} \quad (\text{Maximum}) \quad (2-3)$$

For carbon steels, it appears that the expression should be modified to:

$$X = 40 t^{0.80} \quad (\text{Average}) \quad (2-4)$$

$$X = 80 t^{0.80} \quad (\text{Maximum}) \quad (2-5)$$

Note that these values would be consistent for burial in a wide range of soils, many not meeting the restrictive electrochemical requirements for reinforced soil fills. The implication of using these relationships would be a predictive life for the galvanization of 7 years on average, which would considerably underestimate the results from retrieval tests conducted both in the United States and Europe (King, 1977; NCHRP, 1978; Darbin et al., 1986).

Careful study of the NBS corrosion results in soils meeting the stringent requirements for MSE wall structures suggests that an alternate simplified model can represent the corrosion losses well enough to estimate sacrificial steel requirements for design. This latter model has been adopted by AASHTO to predict uniform maximum loss rates for determination of sacrificial thickness of galvanized steel reinforcement as follows:

- |    |                                   |   |
|----|-----------------------------------|---|
| 1) | Zinc corrosion rate first 2 years | 0.58 mils/yr (15 $\mu\text{m}/\text{yr}$ )/side |
| 2) | Zinc corrosion to depletion       | 0.16 mils/yr (4 $\mu\text{m}/\text{yr}$ )/side  |
| 3) | Carbon steel rate                 | 0.47 mils/yr (12 $\mu\text{m}/\text{yr}$ )/side |

**The AASHTO model only applies when reinforcements are galvanized (i.e. not plain black steel), and soil fills meet the prescribed AASHTO requirements.** The rate of carbon steel loss includes a factor of 2 to consider the effect of nonuniform corrosion, but this factor is not applied to loss of zinc. This model should not be used if reinforced fill soils do not meet AASHTO criteria and the rate of carbon steel loss should not be applied to reinforcements that have not been galvanized, i.e. the rate of carbon steel loss considers the response of the steel surface subsequent to depletion of zinc. Applicable corrosion rates for plain steel reinforcements (i.e. not galvanized) are higher than 0.5 mils/yr (12  $\mu\text{m}/\text{yr}$ ), depending on time of exposure and increase in corrosion anticipated from non-uniform, pitting type corrosion.

**c. Current Practice**

The recommended design practice as outlined in AASHTO is consistent with the philosophy presently in effect worldwide. It considered minimum or maximum mandated long-term electrochemical limits for the backfill and the addition of a sacrificial thickness to the required structural reinforcement thickness. Potential flow of salts from the retained fill must be considered in determining the long-term regime within the reinforced backfill.

Current practice can be summarized as follows:

- Recommended Electrochemical Limits (Metallic Reinforcements)

<u>Property</u>	<u>Standard</u>	<u>Test Procedures</u>
Resistivity	$\Omega\text{-cm} >3000$	AASHTO T-288
pH	$>5 <10$	AASHTO T-289
Organic Content	1% Max.	AASHTO T-267
Chlorides	$< 100 \text{ PPM}$	ASTM D4327
Sulfates	$< 200 \text{ PPM}$	ASTM D4327

With minor variations, typically in grain size distribution and occasionally in pH limits, AASHTO specifications are used by most state DOTs. Because of significant variability of reinforced fill sources, multiple samples must be tested as previously shown in Table 2-2 to assess mean conditions. The requisite number of samples depends on the number of stockpiles, history of laboratory testing of the material source, and the volume of material required for construction.

- Required Design Life

For the purpose of determining the sacrificial thickness, the following design life is recommended in AASHTO.

<u>Structure Classification</u>	<u>Design Life (yrs)</u>
Permanent structure	75
Abutments	100

- Sacrificial Thickness Requirements

For MSE structures constructed with select and tested backfills to ensure compliance with the electrochemical requirements, the maximum mass presumed to be lost per side due to corrosion at the end of the required service life may be computed by assuming a uniform loss model that considers the following loss rates:

-	Zinc corrosion rate first 2 years	0.58 mils/yr (15 $\mu\text{m}/\text{yr}$ )
-	Zinc corrosion to depletion	0.16 mils/yr (4 $\mu\text{m}/\text{yr}$ )
-	Carbon steel rate	0.47 mils/yr (12 $\mu\text{m}/\text{yr}$ )

The resulting sacrificial thickness for a 75-year life based on initial galvanization of 3.4 mils (85  $\mu\text{m}$ ) is approximately 0.06 in. (1.5 mm) for a steel strip. Since this is a maximum loss rate, it is presently assumed that the reduced minimum thickness remains proportional to tensile strength and therefore no further reduction is necessary.

Some systems use ties strips embedded in precast concrete panels to connect the reinforcements at the wall face. The tie strips have top and bottom plates that sandwich the reinforcements; constituting a double shear connection. Within the tie-strip/strip interface there is very little loss of thickness; negligible compared to that of surfaces exposed to the soil. Thus, loss of thickness and sacrificial steel requirements are applied the gross cross section.

Bolts, nuts and washers at the connection should be galvanized in accordance with AASHTO M 232. In general, the design shear stress expected to be transferred to the bolt cross section is well below the allowable shear stress. Therefore, section loss due to corrosion of the bolt is often not an important consideration.

- Use of Resin Bonded Epoxy Coatings

Use of epoxy coatings for routine corrosion environments provide no greater degree of design confidence than galvanization. Where used the following coating standards should be followed:

<u>Reinforcement Type</u>	<u>Coating Standard</u>
Strip	AASHTO M-284
Grid	ASTM A-884

The coating thickness should be on the order of 18 mils (450  $\mu\text{m}$ ), and the design life should be considered as equal to that of a galvanized reinforcement with a coating thickness of 85  $\mu\text{m}$  (16 years).

Therefore, for a 75-year design life, a sacrificial thickness of 0.06 in. (1.5 mm) of a structural steel strip section is presently recommended. The sacrificial thickness required for bar mat wire reinforcement is a function of its initial diameter.

- Use of Polymeric Barrier Coating

The effective life of a polymeric barrier coating may be evaluated based on the time it takes for the polymer to become brittle and to expose the underlying metal to the reinforced fill. A protocol suitable for determining the functional life of polymeric coating is described by HITEC (2000). The protocol considers the potential for construction-induced damage, chemical compatibility of the polymer, degradation due to oxidation in the soil environment, or due to UV radiation if the reinforcement is exposed to the atmosphere (i.e., used as facing). Field tests, as described in ASTM D5818 (2004b), may be used to evaluate construction-induced damage of polymer coatings. Aging tests, conducted at elevated temperatures as described by UL 746B (Underwriters Laboratories, 2000) are used to simulate the long-term effects of oxidation. Degradation rates are then computed using conventional Arrhenius modeling employing time and retained strength or elongation change

from at least three temperatures as the variables. The sensitivity of polymers to UV exposure is examined by exposing the material to UV radiation in a controlled test chamber as described in ASTM D1499 (2004c) and ASTM D4355 (2004b).

- Use of Welded Wire Steel Facing

Steel facing is sometimes employed, particularly for walls constructed with welded wire facings (WWF) or bar mat reinforcements. Often bar mats or WWF is bent at about a 90° angle at the wall face and a backing mat is applied to control the spacing between wires or steel bars at the wall face. Corrosion of the steel facing could cause maintenance issues, and corrosion rates will be affected by atmospheric conditions. Typical values of corrosion rates applied to steel (i.e., non-galvanized that is used in temporary walls (design life of 3 years or less)) sheets in contact with soil on one side is 1.0 mil/yr (25 µm/yr) per side. And these rates should be applied to the case of steel faced walls. Substantially higher rates should be used if the wall face will be vegetated, where road salts are used, if atmospheric conditions are corrosive such as marine environments or when air quality may be compromised by nearby industrial activity. Corrosion potential can be reduced by using open graded stone in the facing.

Steel facings should be galvanized consistent with the use of galvanized reinforcements. Hot dip galvanizing of at least 2 oz/ft<sup>2</sup> is expected to protect the steel in atmospheric conditions for between 20 and 50 years, depending on atmospheric conditions (AGA, 2004). Forty to fifty years are expected in rural and suburban environments, 25 to 30 years in coastal areas, and approximately 20 years if located in proximity to industrial areas where the atmosphere may be acidic. These zinc lives are considerably higher compared to those anticipated for galvanized elements buried in soils.

Hardware cloth that is sometimes used with welded wire facings to contain fill material may be vulnerable to degradation from UV radiation. The durability of these polymeric materials should be considered as described in Chapters 4 and 5.

## **2.5 MARGINAL FILLS**

In some areas of the country, it is becoming more and more difficult to locate fill that meets the AASHTO electrochemical requirements. In addition, there is also a national effort to allow for increases in the amount of fines used for reinforced fill due to the rising cost of select borrow materials. There are fewer data available to document performance and

formulate appropriate metal loss models for design in the case of marginal wall fill soils that do not meet the stringent AASHTO specifications for electrochemical properties.

Based on the results from limited field studies, Caltrans (Jackura et al., 1987) has proposed design guidance for a wider range of reinforced fill conditions than those considered by AASHTO. Higher rates of metal loss are specified for computing sacrificial steel requirements when reinforced fills that are more aggressive relative to corrosion are considered during design. These metal loss rates are based on limited data collected from MSE wall sites in California (Jackura et al., 1987), and using data available from the earlier NBS studies. Interim design guidance considered corrosive conditions with minimum resistivity less than 1000  $\Omega$ -cm. However, current specifications used by Caltrans do not allow use of reinforced fill with minimum resistivity less than 2000  $\Omega$ -cm.

Specifically, Caltrans current specification allows for backfill with a resistivity of greater than 2000  $\Omega$ -cm, a pH between 5.5 and 10, and maximum chloride and sulfate concentrations 250 ppm and 500 ppm, respectively. California considers these conditions by using a higher rate of metal loss in determining sacrificial steel and reducing the design life of the MSE wall to 50 years. Caltrans assumes that the zinc coating provides 10 years of service life for the specified minimum coating thickness of 2 oz.ft<sup>2</sup> (85  $\mu$ m per side). This is less than the 16 years of zinc life inherent to the AASHTO metal loss model. A corrosion rate of 1.10 mils/yr (28  $\mu$ m/yr) is considered to affect the base steel after the zinc has been consumed, and used to compute the sacrificial steel requirements. These corrosion rates account for the potential for localized corrosion and pitting; i.e. a factor of two relating the loss of tensile strength to idealized uniform corrosion rates is included.

Caltrans specifications provide incentives to use select granular fill, which is a better quality fill with less than 5% fines and with PI <6. Caltrans reduces the steel corrosion rate to 0.50 mils/yr (12.7  $\mu$ m/yr) for backfill meeting additional requirements for select granular fill. For select granular fill, lower resistivity and higher salt concentrations are allowed, but the allowable fines content is less compared to current AASHTO requirements.

## **2.6 TEMPORARY WALLS**

Temporary structures have a design service life of less than 3-years. The temporary reinforcements should be galvanized if contact between reinforcements from temporary structures and those from permanent structures with galvanized reinforcements is possible. Otherwise, the presence of the plain steel could have an adverse effect on the durability of the permanent galvanized reinforcements. If plain steel reinforcements are used, and

reinforced fill is not corrosive or only mildly aggressive, a corrosion rate of 28  $\mu\text{m}/\text{yr}$  should be applied to assess sacrificial steel requirements for temporary walls (i.e., design life of 3 years or less). This includes a factor of 2 to consider the effects of nonuniform corrosion. Higher corrosion rates need to be considered for reinforced fills that are moderately aggressive or corrosive, and a corrosion specialist should be consulted to assess the sacrificial steel requirements or other possible corrosion protection measures.

– see section on Current Practice – sacrificial thickness requirements

– see section on Current Practice – use of welded wire steel facing

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## CHAPTER 3

### MONITORING METHODS, METALLIC REINFORCEMENTS

The primary objectives of monitoring techniques are to assess corrosion rates as an adjunct to design and to provide data to assess the integrity of MSE structures constructed with steel or galvanized reinforcements. It is important to consider the performance of these systems within the context of Transportation Asset Management, as slopes and retaining walls are important components of the highway system. The test techniques and procedures described in this chapter have been researched and developed over the past several decades. They are mature technologies, and important tools for collecting performance data that is an integral part of asset management. Two techniques have been used to monitor corrosion rates of buried metallic elements for these purposes:

- **Retrieval of buried coupons** with measurement of weight loss and section thickness, and/or loss of tensile strength at each retrieval interval. This is a destructive test method that requires excavation, or extraction through the wall face, for each retrieval. Further, the number of assessments is limited by the number of coupons buried and is restricted to the measurement of corrosion at the coupon only. Coupon corrosion is often not representative of the corrosion of full sized elements, which are affected by the variations of density, moisture content and dissolved salts inherent in contact with fill soils, which are never totally homogeneous.
- **Remote electrochemical methods** using potential and polarization resistance measurements to obtain instantaneous, average, in-situ corrosion rates of reinforcement elements while in service, i.e., in their stressed state. This method is nondestructive and measurements can be taken at any time (e.g. different seasons, age) to more closely monitor performance.

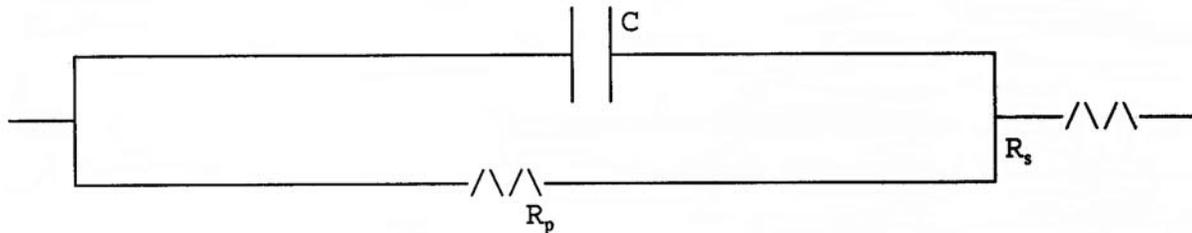
Given the advantages, utilization of remote electrochemical methods is highly recommended with at least some coupons buried for retrievals to confirm results. These techniques can be implemented on both existing structures and during construction and equipment to perform these measurements is commercially available. This chapter will develop the theoretical background for the use of potential and polarization resistance measurements, recommend protocols for implementation on both existing and new construction, and review data obtained since 1988 from corrosion monitoring programs that have been implemented by many state DOT's and from research sponsored by FHWA and NCHRP.

### 3.1 CORROSION MONITORING FUNDAMENTALS

Corrosion is an electrochemical process. In underground corrosion of steel, the electrochemical reaction responsible for corrosion is the oxidation of iron from the steel;



Because it is an electrochemical process, there is an associated current (flow of electrons). Numerous methods exist to measure the current or the rate of corrosion (Tait, 1994). If the current is properly measured and the area of the specimen involved in the reaction is known, the local corrosion rate can be calculated directly. The following equivalent circuit idealizes the specimen surface involved in the corrosion process (Lawson et al., 1993):



where  $R_p$  is the polarization resistance, which is the resistance of the surface to the corrosion process and is inversely proportional to the corrosion rate;  $C$  represents the capacitance of the surface. The electrolyte or soil resistance is represented by  $R_s$  on the circuit.

If the electrochemical potential of the specimen is shifted, a slight amount (less than 0.02V) from its rest (or corrosion) potential by the use of a power supply and a remote electrode, then the current necessary to cause the shift can be measured and the value of  $R_p$  per unit area can be calculated. When normalized for area, which involves multiplying the measured polarization resistance by the electrode area, the polarization resistance of the material ( $R_p$ ) results and has the units of ohm-cm<sup>2</sup>.

The classic description of the relation between  $R_p$  and corrosion rate was derived by Stern and Geary (1957) and is a modification of the fundamental equation for electrochemical kinetics, the Butler-Volmer Equation. The classic equation of Stern and Geary is:

$$i_{CORR} = \frac{B_A B_C}{2.3(B_A + B_C) R_p} \quad (3-2)$$

where  $i_{CORR}$  is the corrosion current density of the surface of the specimen, and  $B_A$  and  $B_C$  are the anodic and the cathodic Tafel slopes, respectively. A Tafel slope is the rate of change of voltage (in volts) per decade of current as the voltage of the specimen is shifted away from its rest or corrosion potential (Fontana, 1986). These Tafel slope values can be determined by experiments or estimated from literature data for soil corrosion. Further, because the expression involves the product of the two Tafel slopes divided by the sum, and because Tafel slopes are typically limited to a relatively small range, the calculation of corrosion rate (corrosion current density) is not highly effected by errors in the values of the Tafel slopes.

The term

$$\frac{B_A B_C}{2.3(B_A + B_C)}$$

typically varies between 0.08 and 0.02 volts and is hereafter referred to as the conversion constant. Stern and Weisert (1959) showed that the corrosion rate calculated from  $R_p$  correlated well with actual corrosion rates determined from weight loss measurements for a variety of steels in aqueous environments.

Electrochemical Impedance Spectroscopy (EIS) or AC Impedance is another electrochemical technique that can be used to determine the values of  $R_p$ ,  $R_s$ , and  $C$  in the equivalent circuit (Andrade et al, 1986; Scully, 2000). Although more information about the corrosion process may be obtained from the results of EIS, the test data are more difficult to interpret. The presence of factors such as metal phase (e.g. both zinc and steel present on surface) and passive film layers affect the results and needs for sophisticated techniques for data analysis and interpretation.

Corrosion rates determined from polarization resistance measurements require that:

- the exposed area of the component to be analyzed be known or estimated accurately. An accurate estimate can be made for reinforcing strips or grids in reinforced soil structures because the geometry is well defined.
- the polarization resistance be determined independently of the ohmic resistance (soil resistance). This can be accomplished by using a spectrum of AC measurements, or can be neglected when considerable experience has been gained on actual structures that show it to be justified. Alternately, it can be measured separately by direct measurement with a soil resistance device and subtracted from the measured polarization resistance.

- the conversion constant necessary to convert the polarization resistance to corrosion rate be known. Estimates of this constant suffice in most applications. Data from Tokyo Gas suggest a value of 0.021 volts (V) for this constant represents the average of all corrosion systems. Pipeline companies routinely use a constant of 0.035 V for steel pipe. Montuelle and Jailloux (1979) suggest a conversion constant on the order of 0.050 for galvanized steel in numerous soils. Therefore, it can be assumed that conversion constants range between 0.020 V and 0.050 V.
- the composition of the surface being analyzed be known. In the early life of MSE structures, it can clearly be assumed that the surface is galvanized. Corrosion potential monitoring can be used to determine subsequent metal phases as the reinforcement loses zinc, ultimately down to the carbon steel base.

The use of polarization resistance measurements in general compliance with ASTM-G59, corrected for soil resistance by separate measurements, are applicable for determination of corrosion rates of reinforcing elements in MSE structures. ***While the conversion constants for steel and galvanized surfaces may vary (within a factor of 2), use of 0.035 V for steel and 0.050V for galvanized steel to obtain an average corrosion rate should be considered where exact data for a particular soil are not known.*** Potential measurements that will distinguish existing surface composition should be used as a basis for selecting the conversion constant. The basis for potential measurements is outlined below.

The interpretation of potential measurements considers that four distinguishable layers of zinc coating are formed as a result of the hot dip process used to galvanize MSE reinforcements. The outside layer is nearly pure zinc, and the succeeding inner layers are essentially zinc-iron alloys. Progressively higher iron contents prevail as the interface with the base steel is approached. Therefore, as zinc consumption progresses towards the base steel interface, the half-cell potential is consistently shifted towards values inherent to iron. Ultimately, measurements of the half-cell potential reflect the presence of steel after all four layers of the zinc coating are exhausted and bare steel is exposed; at least in some areas.

The primary purpose of potential measurements in MSE structures is to establish when significant portions of the reinforcements have lost zinc coverage and steel is exposed to the soil environment. The presence of rust does not necessarily indicate that zinc has been consumed. Rust color due to the presence of ferrous hydroxide may be visible on the surface during corrosion, however, some zinc is still present in the layer. Therefore, comparisons between the potential of the reinforcements and the potentials of buried zinc and carbon steel coupons are used to distinguish when a significant surface area of bare steel is exposed to the

soil. Once bare steel is exposed to the soil environment, the corrosion rate of the sacrificial zinc (galvanization) at other locations on the surface may be accelerated due to the galvanic couple with the steel.

*If the potential of the reinforcing element is near that of the zinc coupon, the soil reinforcement remains well galvanized. As the potential of the reinforcement element becomes more positive and begins to approach that of the steel coupon, the galvanizing is being lost and more bare steel is being exposed.*

The corrosion potential is the voltage of a reinforcement element of interest measured with respect to some suitable reference electrode. The common reference electrode in underground corrosion studies is Copper/Copper Sulfate (Cu/Cu<sub>2</sub>SO<sub>4</sub>). For a given material in a given environment, the potential is an indication of the corrosion activity. The more positive the potential, the greater, in general, is the corrosion. Potential measurements are therefore only qualitative indications of corrosion activity and should only be used to determine the composition of the surface.

*Typical values for a galvanized reinforcement would be between -1.10 to -0.65 V and for carbon steel in the range of -0.60 to -0.20V. Transition stages would yield intermediate values.*

## **3.2 IMPLEMENTATION OF FIELD CORROSION MONITORING PROGRAMS**

### **a. Plan Development**

Corrosion monitoring programs and monitoring of in service reinforcements should be implemented with the following objectives:

- Assess the validity of the design corrosion rates.
- Evaluate how service life and sacrificial steel requirements may be affected by using reinforced fills that do not meet the minimum electrochemical criteria specified in current AASHTO specifications. Use of materials for reinforced fill that do not meet AASHTO electrochemical specifications is not recommended. If materials fall outside of the specified values, a long-term monitoring program is strongly recommended.

- Identify the impact of changing site conditions on service life and performance, e.g. if there is a suspicion that the groundwater regime established in the structure during service is acidic, highly alkaline, or contaminated with salts.
- Provide a means to warn of impending failure in response to some visible distress in the MSE structure, and for structures of critical importance.
- Identify how particular climatic conditions may affect service life, e.g. structures in or adjacent to warm marine environments.
- Evaluate the effects from extreme events on service life, e.g. hurricane induced tidal surges may inundate structures constructed in estuary environments.

The benefits of condition assessment and corrosion monitoring are to provide better estimates of expected service life, and to identify conditions where service life may be compromised. In this manner agencies can be proactive with respect to asset management and be better prepared to meet future obligations with respect to maintenance, rehabilitation, retrofit or replacement.

Agencies need to develop an inventory of constructed MSE facilities as part of an asset management strategy. The inventory should include details of the construction, location and character of the reinforced fill. A list of priorities and a sampling plan can be developed from the inventory to establish a basis for performance monitoring. In general, approximately thirty sites should form the basis of the sampling plan. This basis should include a representative cross section of walls incorporating different wall height, function, wall supplier, contractor, fill sources, regions, and climates within an agency's jurisdiction. The thirty-site sampling should be from conditions expected to correspond to good performance to serve as a baseline. Sites with special concerns can be added to check for the possibility of poor performance, based on issues that may be related to backfill quality, particularly adverse site conditions, or poor construction practice. Corrosion monitoring and performance data for asset management will be discussed in more detail in Section 3.4.

The measurement concept recommended for monitoring MSE structures includes both potential measurements and polarization resistance (PR) measurements. Both measurement techniques must be performed on buried coupons, as well as the actual reinforcement members. The buried coupons will include coupons made from carbon steel, zinc, and galvanized steel.

## **b. Monitoring Programs**

The number of monitoring locations in each MSE structure is a function of the length and anticipated variability of the in-situ regime. As a rule of thumb, two locations spaced at least 200 ft (60 m) apart should be considered for MSE structures 800 ft (250 m) or less in length and three locations for longer structures. At each location, corrosion should be monitored at a minimum of two depths from the surface or preferably at depth intervals of 10 to 13 ft (3 to 4 m) because differences in oxygen content, moisture content, and salt concentration can produce different corrosion behavior. One critical location (center of structure) should be selected for establishing test locations at both shallow and deep positions. Higher oxygen and salt content are anticipated near the surface, and higher moisture contents or free water near the base of a structure. Prior field programs have indicated that where groundwater intrudes at the base of the structure, higher corrosion rates should be anticipated. Where this condition is not likely, representative estimates may be obtained from shallow-depth monitoring. The shallow depth stations should be approximately 5 ft (1.5 m) in depth, and the deep position should be approximately at one-fourth of the structure height from base level. Figure 3-1 shows the location of the coupons and instrumented reinforcement members.

In general, more monitoring locations should be established for structures where poor performance is anticipated or known to exist (Withiam et al, 2003; Hegazy et al, 2003). Particular attention should be given to monitoring near drainage inlets or other areas that may be subject to fluctuations in moisture content, high moisture content, or inundation. However, monitoring at locations with “normal” conditions is still necessary to serve as a baseline, and to ensure that the sample statistics are not skewed.

Ideally, three types of coupons should be placed at each location and depth; zinc, steel and galvanized. For monitoring, it is desirable to have one-zinc, one-steel and up to four galvanized coupons at each depth. The multiple galvanized coupons can provide opportunities for periodic removal. Coupons each have two leads to provide back-up in case one connection fails.

At each location, access to the test leads should be housed within a water-tight lockable box. The front panel should have the capacity to provide isolated test lead connections from all depths. The total number of connections will depend on the number of coupons buried. All leads should be encased in conduit to prevent breakage.

Similar location selection criteria apply for existing and new structures. However, it is realized that for existing structures access to deeper reinforcements may only be possible by

advancing core holes through the wall face, and coupons may only be placed in proximity to the wall face.

The rationale for measurements can be explained by viewing the corrosion process as having 3 Stages as shown on Figure 3-2.

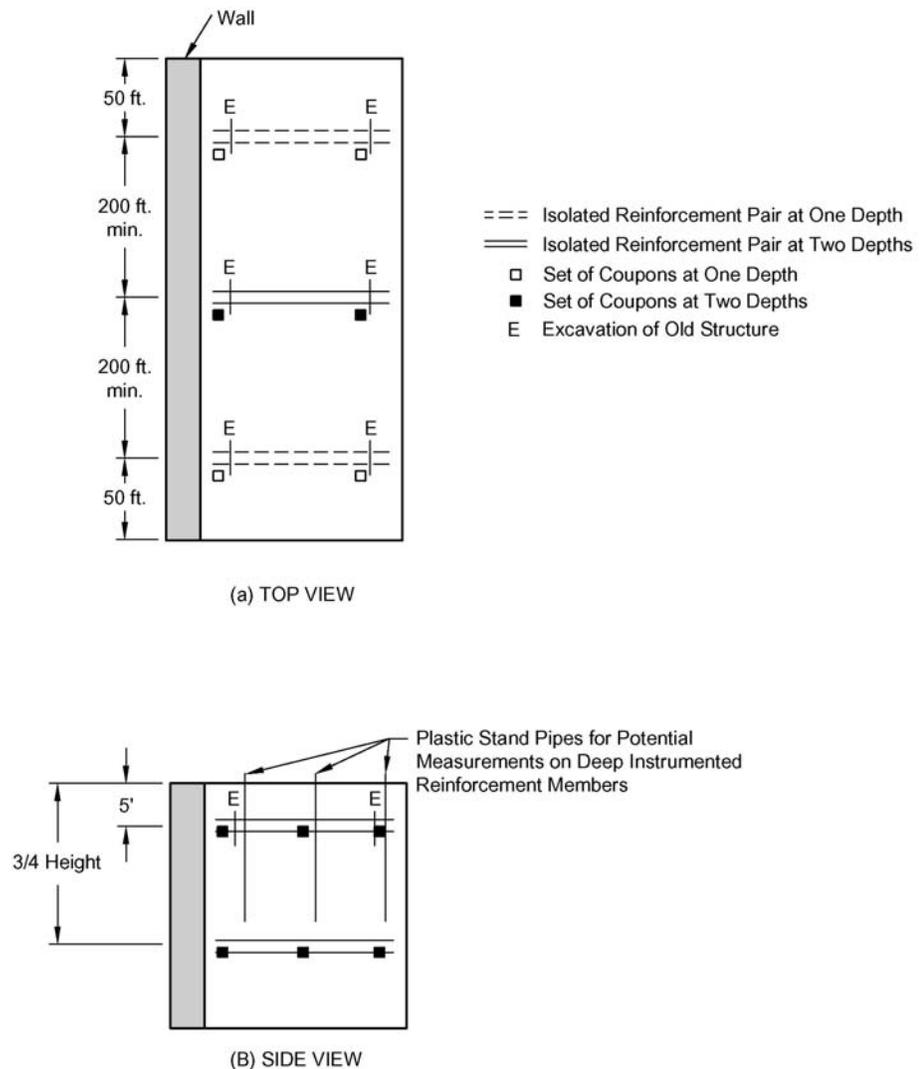


Figure 3-1. Schematic diagram showing locations of coupons and instrumented reinforcement members.

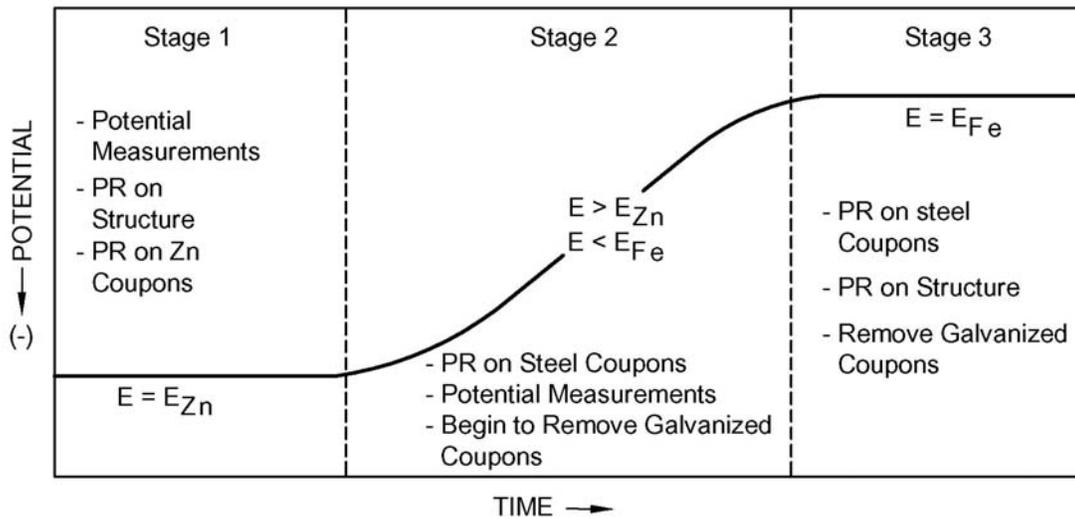


Figure 3-2. Stages of galvanized structure performance.

During Stage 1, the galvanizing is intact and either no steel is exposed to the soil or the steel is well polarized and completely protected. The potential of the structure is the same as that of zinc  $E = E_{Zn}$ . In Stage 2, increased steel is exposed as galvanization is lost, and the potential of the structure becomes more positive and approaches that of bare steel ( $E > E_{Zn}$  and  $E < E_{Fe}$ ). In Stage 3, essentially all the galvanization has been lost and the potential of the structure is the same as that of carbon steel ( $E = E_{Fe}$ ). During the different Stages, the monitoring procedures change, as summarized schematically in Figure 3-2. Real-world data depicting the trends described in Figure 3-2 are presented in Chapter 3.5 from the North Carolina Department of Transportation (NCDOT). NCDOT routinely monitors half-cell potentials of steel and zinc coupons and in-service reinforcements in an effort to evaluate zinc life for galvanized reinforcements.

The details of the monitoring program differ from new construction to existing structures, as measurements for existing structures may begin in either Stage 1, 2 or 3, while for new construction, they always begin in Stage 1.

### c. New Structures

For new structures, steel, zinc and galvanized coupons should be buried at each depth selected. At each monitoring station, sets of coupons should be placed both near the front and back of the reinforcements, i.e. close to the wall face and further back within the

reinforced fill. This provides an opportunity to evaluate the spatial distribution of corrosion. Spatial variations along the reinforcement lengths are due to variations in moisture content, salt content and availability of oxygen. Also, many times a more open graded reinforced fill is placed within approximately 3 feet (1 m) from the wall face (i.e. less fines) compared to the backside of the reinforced fill to facilitate compaction. Coupons placed near the wall face may also be extracted through core holes cast into the wall face or cored during construction to facilitate removal of coupons for inspection without the need for more costly excavation. Direct physical observations of coupon condition are useful to verify and confirm results from nondestructive, electrochemical tests.

In addition, actual reinforcement members must be instrumented for measurement purposes. The three stages of MSE structure life will be determined by comparing steel and zinc coupon potentials to those measured for the structure. During Stage 1 (reinforcement potential = zinc coupon) the following monitoring should be performed:

- Potential measurements on each coupon and selected reinforcement members to establish the change from one stage of the corrosion process to another.
- Polarization Resistance (PR) measurements on zinc and galvanized coupons and on the structure to estimate rate of zinc corrosion (zinc loss).
- PR measurements on carbon steel coupons to characterize changes in the corrosion rate with time.

Stage 2 is established when the potential of the reinforcement becomes more positive than the zinc coupon and approaches the potential of the steel coupon. During Stage 2, the following monitoring should be performed;

- Potential measurements on each coupon and reinforcement to establish the change from one stage to another.
- PR measurements on steel coupons to provide a conservative (high) estimate of the corrosion rate of the reinforcement. PR measurements on the reinforcement would yield mean values, as the actual exposed area is largely unknown, and the conversion constant is therefore uncertain or variable.
- Begin retrieval of the galvanized coupons to quantitatively evaluate the condition of the structure.

Stage 3 is established when the potential of the structure becomes similar to the potential of the steel coupon. At this stage, little or no galvanized coating remains. For mildly corrosive conditions inherent to reinforced fill and site conditions meeting current AASHTO specifications, Stage 3 is not anticipated until at least 16 years of service and may not be reached before 30 or more years of service (Gladstone et al., 2006). During Stage 3, the following monitoring should be performed.

- PR measurements on steel coupons and the reinforcement to estimate the corrosion rate (metal loss).
- Potential measurements on galvanized coupons for comparison to the reinforcement, until the potentials of the steel and galvanized coupons are similar.
- Continue periodical retrieval of the galvanized coupons to evaluate the condition of the reinforcement.

Each reinforcement element must be isolated from the rest of the MSE structure to permit accurate polarization resistance measurements. Otherwise the polarized area is unknown and the corresponding corrosion rate calculation is erroneous. This excludes the possibility of monitoring individual in-service reinforcements for steel faced walls (e.g. wire facing), but coupons and “dummy” reinforcements may be used to infer performance of in service reinforcements for these systems. Reinforcements that are attached to precast concrete facing are generally isolated since current specifications require that reinforcement tie-strips embedded in the concrete be isolated from concrete reinforcing steel. The potential for electrical contact between reinforcements and other steel elements exists for MSE walls that serve as bridge abutments; near the corners of abutment facings and wing walls, and where steel piles protrude into the reinforced fill. Affected reinforcements may be isolated during construction by placing a dielectric (e.g. plastic wedge) to avoid electrical contact.

Instrumentation of the reinforcement strips should be performed in pairs, such that two parallel strips are instrumented. Each member of the pair should have the same length (or the element serving as the counter electrode should be longer) to ensure that the entire surface area of the test element is polarized during the LPR test. On new structures, "dummy" pairs of reinforcement members may be instrumented and placed in the structure at the desired location to be sure that no contact is made to the working members on the structure or to the concrete panels. The instrumented members should be positioned such that 1 to 2 ft (0.3 to 0.6 m) separation exists between members of the pair. The instrumented pairs will be exposed to the same conditions as the working members except that the stress profiles along

the instrumented members will be somewhat different since the instrumented members will not be connected to the facing.

The isolated strips will become test strips for making PR and potential measurements. The procedure for attaching test leads will be similar to that described for galvanizing coupons as described under Materials. The connection is shown schematically in Figure 3-3; red colored lead wires should be used to distinguish leads from the reinforcements and the steel coupons (black leads), zinc coupons (green leads) and galvanized coupons (white leads). Two connections will be made on each instrumented strip for redundancy.

For MSE structures that have grid-type reinforcing members, the procedures for wiring the members will be similar to those previously described for reinforcement strips. If the connections to the front wall panels for grid systems are not tied to the rebar cage and the grid members are electrically isolated from each other, there is no need for "dummy" members to be placed in the wall.

#### **d. Existing Structures (Retrofit)**

Excavations can be made from the surface to reach upper level reinforcements to be monitored, and readings can be made at the top of the wall for those reinforcing elements. This method offers some advantages relative to data quality, but will be difficult to implement if pavement and impact barriers exist above the wall; and then only for shallow excavations.

If excavation is performed from the top of the structure isolation should be accomplished by removing a 1 to 2 ft (0.3 to 0.6 m) section of the reinforcement strip or grid at a point 5 to 10 ft (1.5 to 3 m) from either end. This removed section will also permit the condition of the reinforcement to be determined. The remaining section at either end of the reinforcement should be periodically excavated and 1-foot (0.3-m) sections cut and removed for evaluation. The 1-foot (0.3-m) sections removed from existing structures serve the same purpose as the retrievable galvanized coupons used for new structures. Care should be exercised because in many instances taking reinforcements out of service near the top of the wall may compromise the stability of the facing, e.g. considering truck impacts to parapet walls or Jersey barriers supported at the top of the MSE.

If coupons are to be buried, they should be installed in the least disturbed soil along the side cuts of the excavation in order to obtain similar conditions to the existing fill. Care should also be taken to replace and compact the removed soil to a similar density as the original

structure so as to avoid a differential condition that could potentially influence results as discussed in Chapter 3.2c.

Instrumentation and lead connections on existing structures are made in a similar manner as described for the reinforcement strips and are shown on Figure 3-3.

Alternately to minimize excavation, more limited but generally sufficient data can be obtained by accessing the reinforcements by drilling multiple 4-inch (100 mm) diameter holes thru the facing adjacent to reinforcement locations. Procedures using this alternate method have been developed and demonstrated (Berkovitz and Healy, 1997; CC Technologies, 1999; Wheeler, 2002; Beckham et al. 2005; Fishman et al., 2006). Figures 3-4 and 3-5 illustrate the necessary field setup.

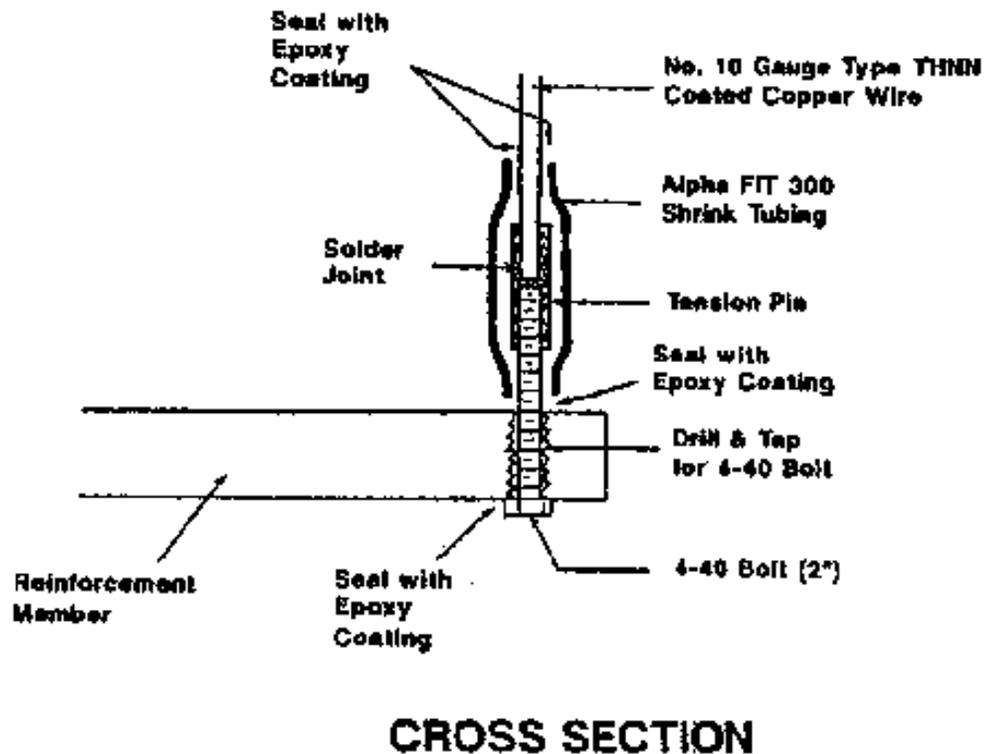


Figure 3-3. Schematic diagram for connection to reinforcing members.

Electrical isolation of the reinforcements must be assured by testing prior to any potential or polarization measurements. For MSE walls that serve as bridge abutment, reinforcements may be in contact near the wing wall corners, and for reinforcements that are splayed around piles that are installed to support bridge seats. Older MSE walls, constructed before the early 1980's, may incorporate a U-shaped tie strip that is embedded into the precast facing such that the upper and lower reinforcing strips in a precast concrete panel are electrically connected through this connection. If electrical continuity exists, the working reinforcing element must be detached or isolated from the facing system, or another reinforcement should be selected for monitoring.

Connections to in service reinforcements as shown in Figure 3-3 may not be possible. In this case connections may be achieved by soldering directly to the reinforcement after exposing bare steel, or attachments can be made with special c-clamps as described by Sagues et al. (1998). It is very important to ensure that connections are durable and made waterproof. The coupons may be inserted into the access holes, which also provide an opportunity to retrieve samples of reinforced fill. A smaller, 2-inch (50 mm) diameter hole is advanced near the center of the sample location, i.e. between the wired reinforcements, for contacting the reference electrode with the reinforced fill and subsequent potential and polarization measurements.

Placement of the half-cell directly against the surface of the wall face will not provide sufficient contact with the reinforced fill and potential measurements are best achieved through additional core holes advanced through the wall face. A two-inch diameter core hole with a moist sponge at the base facilitates good electrical contact between the half-cell probe tip and the reinforced fill. The access hole should be located near the center of the sample location, i.e. in the middle of the reinforcements that are wired for monitoring. Another alternative depicted in Figure 3-4 is to place the half-cell in contact with soil at the base of the wall.

#### **e. Materials**

##### Carbon Steel Coupons

Carbon steel coupons are used to estimate the rate of corrosion on the reinforcement members once the galvanized coating is lost (or partially lost) i.e. Stages 2 and 3. The carbon steel coupons will also provide the potential of steel for comparison to the potential of the reinforcement members in establishing when all galvanization has been lost (Stage 3). The carbon steel coupons should be made of similar materials as the reinforcement members.

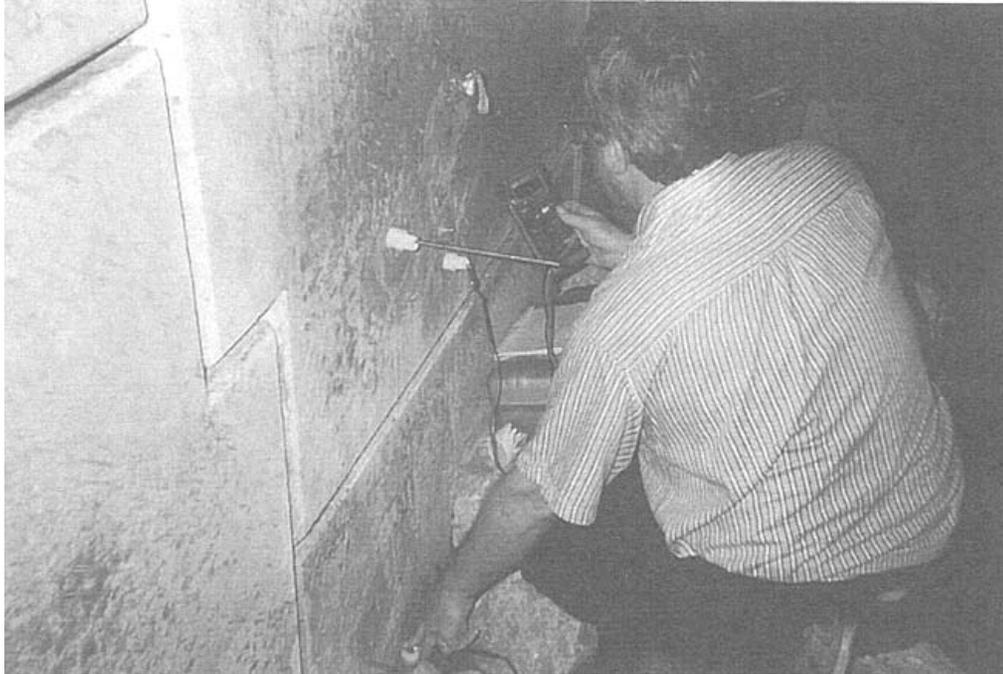


Figure 3-4. A portable Copper/Copper Sulfate half-cell is hand held on the soil at the base of the wall as a reference electrode for multi-meter testing of electric potential.

Exact duplication is not necessary since small concentration variations do not typically have a significant effect on general corrosion of carbon steel in soil.

Coupon installation is described below and shown in Figures 3-6 and 3-7:

- Coupon size is 100 by 100 by 10 mm minimum.
- The top edge is drilled and tapped at two locations.
- Coupon surfaces are finished to a 600 grit finish (320 to 600 is acceptable).
- A 4-40 bolt is threaded into the top of the coupon and the head cut-off.
- A No. 10 gauge type THNN coated copper wire test lead (black) is soldered to the 4-40 bolt using a tension pin to provide support to the solder joint.
- A solder joint should be sealed with Alpha FIT 300 shrink tubing and the ends coated with Carboline coal tar epoxy coating including a 25 mm area of coupon around the connection (use two coats of epoxy, with each coat increasing in area covered).

Isolating the lead wire from the environment is critical for obtaining the desired life of the coupon. Manufacturers' cleaning procedures for epoxy application should be followed closely.



Figure 3-5. PR Monitor evaluation of a test location. Note horizontal adjacent access holes for cross testing.

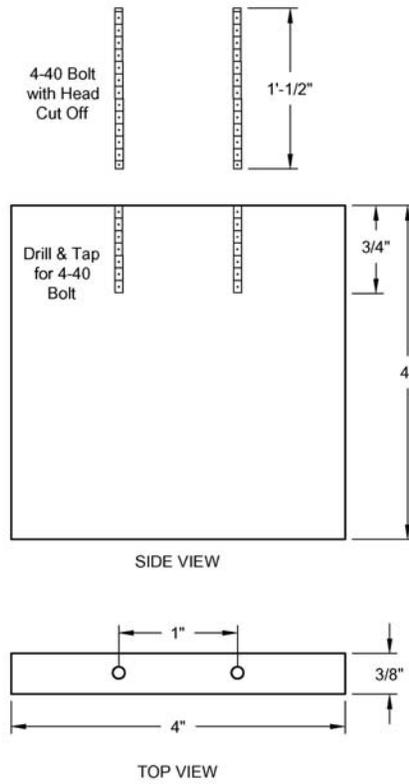
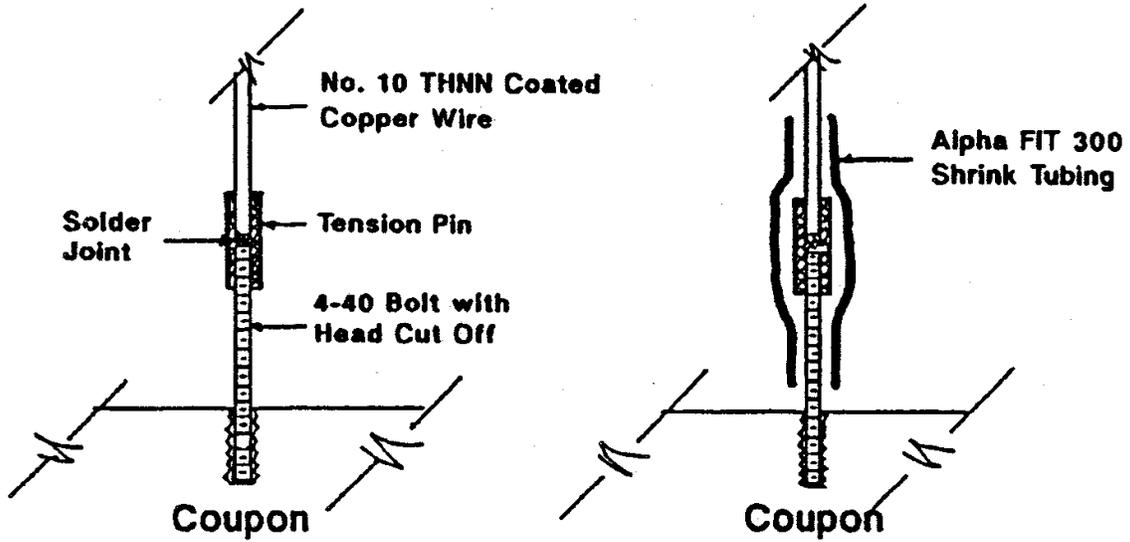
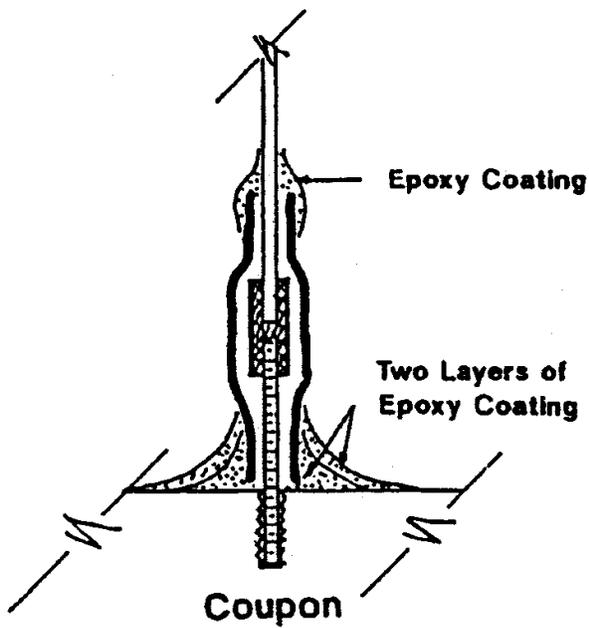


Figure 3-6. Schematic diagram for coupons.



**(a) Thread 4-40 Bolt into Coupon and Cut Off Head. Solder No. 10 Wire, Using Tension Pin for Support.**

**(b) Shrink Tubing Down to Top of Coupon**



**(c) Apply Epoxy Coating**

Figure 3-7. Schematic diagram illustrating coupon connection.

### Zinc Coupons

Zinc coupons are used to determine the rate of zinc loss, which is an estimate of the rate of galvanization removal from the in service reinforcements (Stage 1). The zinc coupons should also provide the potential of zinc for comparison to the potential of the reinforcement members for establishing the transition from Stage 1 to Stage 2. The zinc coupons should be made of solid zinc and should have a typical composition of zinc used for galvanizing.

The configuration of the coupon should be similar to that of the carbon steel coupon, with the following exceptions:

- The shape of the coupon may be round instead of square depending on the availability of the zinc. Size should be 2¾ to 4¾ in. (70 to 120 mm) in diameter.
- The test leads are green.
- The 4-40 bolt should be galvanized.

### Galvanized Coupons

Galvanized coupons should be buried in new structures for periodic extraction to determine the condition of the galvanized coating and the steel substrate. The coupons should be made from reinforcement members; preferably from the supplier. The cut edges of the coupons should be re-dipped in a zinc bath to provide a galvanized coating at the cut edges, or else the cut edges should be coated with a dielectric (e.g., liquid tape and epoxy). Coupons should be both of the retrievable type and instrumented type. The retrievable type should be placed behind openings cut in the face panels and may be up to 4 ft (1.2 m) long.

The instrumented coupons should have the following configurations:

- Coupon size is 1 ft (0.3 m) long (thickness or diameter will be the same as the reinforcement element).
- A single 4-40, 2-inch (50 mm) long bolt should be threaded through a drilled and tapped hole in the end of the coupon prior to regalvanizing the edges so that the rod can also be galvanized.
- The No. 10 Gauge Type THNN coated copper wire test lead (white) is soldered to the bolt using a tension pin to provide support for the solder joint.
- The connection and down to the top of the coupon is sealed with Alpha FIT 300 shrink tubing, and the ends and the solder connection is coated with the carboline epoxy coating (use two coats of epoxy with each coat increasing in area covered).

## Reference Electrode

The reference electrode is used as the low end reference to probe reinforcing elements or coupons to measure the voltage corrosion potential. The purpose of the reference probe is to provide a stable, known, voltage serving as a datum. As previously indicated in this section, the common reference electrode in underground corrosion studies is Copper/Copper Sulfate (CSE). Several other types of reference electrodes are available (Fontana, 1986). However, the CSE is a popular choice for making measurements in soil because its accuracy and stability is adequate; it is relatively inexpensive and readily available. The disadvantage of the CSE compared to other reference electrodes is the need for maintenance at least once per month. Therefore, the CSE is not a good choice when a reference electrode must to be left in place for making measurements over an extended period. For the applications described herein, the CSE is only placed in contact with the soil for making discrete measurements, and there are opportunities for maintenance between monitoring activities.

## **f. Measurement Procedures and Equipment**

After the installation of leads to the coupons, reinforcing element, and filling of at least one lift, initial measurements of potential and polarization resistance can be made. Figure 3-8 shows a typical installation prior to backfilling.

**Potential measurements** are relatively simple direct measurements that can be performed with a minimum of equipment, application time and experience. Recommended equipment is a high impedance voltmeter (10 M $\Omega$  or greater) and a copper/copper sulfate (Cu/Cu<sub>2</sub>SO<sub>4</sub>) electrode. Multiple measurements (every 6 $\pm$  ft {2 $\pm$  m}) may be made along the length of the buried reinforcement by moving the electrode and developing a profile of the potentials along the reinforcement. The details of the calibration and measurements procedures are fully discussed in FHWA RD 89-186 (Elias, 1989).

**Polarization resistance** measurements require the application of a potential (current) to the specimen being monitored, with simultaneous measurement of the potential and current. The acquired data is processed to calculate the polarization resistance and to estimate the corrosion rate of the specimens. Standard methods for calibration of equipment and verification of the technique for performing polarization resistance are outlined in ASTM G-59. Soil resistance measurement equipment is typically needed for correcting polarization resistance measurements for  $R_s$ . Equipment for performing polarization resistance measurements should be checked prior to making measurements in the field by applying a known resistance between the leads or with a “dummy” cell as described in Section 5 of ASTM G59.



Figure 3-8. Typical installation.



Figure 3-9. Automated polarization resistance measurement equipment.

Fully automatic equipment is available that can apply current, make polarization and soil resistance measurements, and integrate all data. (See FHWA RD 89-186 (Elias, 1989)). The use of this fully automated equipment is recommended. It consists of a self-contained unit in which the power source (battery) lead connector unit, and interactive computer are integrated as shown on Figure 3-9.

The operation consists of connecting the leads from the coupons and reinforcements to the lead connector unit and activating the computer. The interactive programming then leads the operator through sequential steps in which the current is applied, measurements of polarization resistance and soil resistance made, and the instantaneous corrosion rate calculated and displayed.

The following suggestions are offered to ensure the quality of the data and to identify measurements that may be subject to error. These suggestions are intended to supplement information included in the operating manual supplied with the LPR test equipment.

1. Maintain the copper-sulphate reference electrode, as described by ASTM C876, prior to the site visit. The porous tip needs to be damp, so after the electrode is maintained, let the tip soak in about 1 in. (25 mm) of water for a half an hour or so before the test. Be sure to replace the cap on the porous tip when not in use to prevent drying.
2. After you arrive onsite connect the power supply and power-up the equipment for making LPR measurements. Most electronic equipment needs to warm-up for approximately 20 minutes before taking readings.
3. Place the reference electrode in the access hole, or other suitable location at a monitoring location. Direct contact with the concrete face will not achieve suitable contact with the reinforced fill. When contacting the reinforced fill through an access hole in the wall facing, place a moist piece of sponge into the hole and in contact with the reinforced fill, remove the plastic cap from the end of the reference electrode, and press the tip firmly against the sponge. Good contact must be maintained, which may be accomplished by wedging the half-cell into place (wooden shims work well). If the reference electrode is placed in contact with soil at the base or top of the wall, loosen and wet the soil before pushing the probe tip into place. If the ground surface is paved, core or chip a hole in the surface to allow direct contact with the underlying soil. Attach a short length of wire (#12 gage) to the exposed end of the reference electrode for making connections to a digital voltmeter.
4. Record the half-cell potential of each element at the test station using a digital voltmeter. The voltmeter should have an impedance of at least  $10\text{M}\Omega$  and a readout that includes at least four significant digits. Connect the common (black) lead from the voltmeter to the reference electrode, and the red lead from the voltmeter to the steel coupon. The voltage should be negative and in the range of  $-600$  to  $-200$  mV. Record the reading and then move the red lead from the voltmeter to the next element. The common lead remains attached to the reference electrode for each measurement. Record the reading and then read the remainder of the elements and record in a similar fashion. Zinc coupons normally read in the range from  $-1100$  to  $-650$  mV and in-service reinforcements and galvanized coupons will be in between the half-cell potentials recorded for the zinc and steel coupons; depending on age and condition. These reading also provide an opportunity to check that elements are electrically isolated from each other. Half-cell potentials that are within  $\pm 5\text{mV}$  between elements may indicate electrical contact. As a second check, measure the resistance between different elements and a reading less than  $5\Omega$  is indicative of contact (i.e. not electrically isolated).
5. Connect the cable bundle supplied with the LPR test equipment. The cable bundle has three leads marked WE, CE (e.g., red lead) and Ref (e.g., black lead). Connect the black

lead to the reference electrode, the CE lead to the counter electrode (usually this is the second dummy or in service element wired for monitoring), and the WE (working electrode) lead to the test element, which could be one of the coupons or the other in-service or dummy element. Use the same CE for monitoring all the other elements and, then switch this to the WE and use the other wired reinforcement or dummy element as the counter.

6. Observe the initial half-cell potential recorded by the LPR equipment. This is the initial reading taken before current is impressed between the WE and CE. This measurement should be within  $\pm 20$  mV of the half-cell potentials measured with the voltmeter as described in item #4. If this is not the case, check the cable connections (this is usually the problem) and placement of the half-cell. In some instances problems may be due to the presence of stray currents or interference from nearby magnetic disturbances. These interferences affect the cable connections and it may be necessary to move the equipment or power sources to a new location to avoid interferences.
7. Get an initial reading of soil resistance. This is often offered as a separate measurement with units specifically designed for making measurements in a high resistance electrolyte such as soil or concrete. For a given environment, the soil resistance is inversely proportional to the surface area of the test element; i.e. soil resistances surrounding in-service reinforcements should be less than those surrounding the relatively smaller coupons. If this is not the case there may be problems related to connections; e.g. the buried connections between the leads and the elements may not be sealed properly. If extremely low resistances are measured with respect to in service reinforcements, this may be an indication that more surface area is involved than anticipated; i.e. the element is not electrically isolated and is in contact with other metal elements such as piles or other reinforcements.
8. Proceed to measurement of corrosion rate in accordance with the operating manual provided with the LPR equipment. Be sure to input the appropriate conversion constant, surface area, and metal type for the test element (i.e. zinc or steel). In some systems Tafel slopes rather than conversion constants are input and metal type is in terms of density and equivalent atomic weight. Other parameters include the potential shift, step size and step duration used to control the LPR test. For MSE reinforcements typical values are  $\pm 20$  mV, in 5mV steps, applied in a 15 second duration.
9. Assign a file name for the test. A good naming scheme that allows data to be located in a search includes ***“state-day-month-year-locale-site name-wall name-WE#-CE#.”*** Also include fields of information that apply to weather and temperature as comments to the data record.

10. Run the LPR test. Check the applied step sizes during the test to see if it is consistent with the input controlled step size. If this is not the case there may be a problem with drift and possible interference. Check connections and look for sources of interference.
11. Create and review a chart (software supplied with the LPR equipment will often produce this chart automatically) of applied current versus over potential (half-cell potential). The chart should be linear or at least include a linear portion in proximately to the open circuit potential. If this is not the case, repeat the test. If linearity cannot be achieved the data is suspect.
12. Compare the measured soil resistance ( $R_s$ ) to the polarization resistance (PR-corrected for soil resistance). Preferably the ratio ( $R_s/PR$ ) is less than one, but ratios between 1 and 4 are acceptable. If the ratio is greater than 4, move the half-cell to a position closer to the test element, and repeat the test. If a ratio less than 4 cannot be achieved, the measured corrosion rate is subject to a relatively high error.
13. If previous measurements have been made at this site be sure to have them available for comparison. Discrepancies should be noted and these tests should be repeated and checked for consistency.
14. LPR tests impart minimum disturbance to the system, and therefore a test may be repeated as a check on precision and repeatability. A wait period of approximately 10 minutes between tests is advised to allow the half cell potential of the test element to return to the open circuit potential, and to be sure that any stray currents have been discharged. The LPR test is not accurate for measuring very high corrosion rates, and, in these cases, an average from three measurements should be achieved.

#### **g. Frequency of Measurement**

Potential measurements of each coupon and instrumented reinforcement must be made at the time of installation to check lead connections and establish initial measurement values. Subsequent measurements are recommended monthly for the first 3 months, bi-monthly for the next 9 months (to determine seasonal variations, if any) and annually thereafter at approximately the same date.

Polarization resistance measurements should be made at the same intervals and schedule. For long-term monitoring schemes, four galvanized coupons should be buried; the first coupon should be removed at the midrange of Stage 2, the second at the beginning of Stage 3, and the remaining two at intervals established by the predicted metal loss from PR measurements.

For existing structures, shallow excavations can be made for observations of the reinforcement and thickness measurements. Reinforcements should be cleaned with a wire brush and all surface corrosion removed before measurements are made. For a more thorough evaluation, isolation of the reinforcement member can also be accomplished by removing a 1-foot (0.3-m) section at a point 5 to 10 ft (1.5 to 3 m) from either end of the reinforcement member. Of course, any reinforcements that are removed must be replaced and the excavation should be sufficiently large to accommodate this process. The sections removed during initial excavation are used for evaluations to establish the starting condition for monitoring. The remaining sections at either end can serve the same purpose as the galvanized coupons used for new structures and the middle section can be instrumented. For long term monitoring schemes, excavations can be performed and a 1-foot (0.3-m) section removed from the remaining sections at either end of the reinforcement members. These excavations should be performed at the midrange of Stage 2, at the beginning of Stage 3, and thereafter at intervals established by the predicted metal loss from PR measurements

The above procedure is impractical when accessing reinforcements thru the facing.

### **3.3 EVALUATION OF CORROSION MONITORING DATA**

The outlined concepts, methods, and equipment to determine corrosion rates on both new and existing structures have been evaluated at five existing and two new structures and reported in FHWA RD 89-186 *"Durability/Corrosion of Soil Reinforced Structures"* (Elias, 1989).

The data obtained at Site 4, lower level of a newly constructed wall, for 26 months is shown on Table 3-1. At this site the reinforced fill soils were granular with minimum resistivities of 15,000 ohm-cm, which would suggest a non-corrosive regime. Measurements for both the coupons and actual reinforcements (Reinforcement 1,2) are shown.

The free corrosion potential measurements,  $E_{cor}$ , for the steel coupons, using a copper sulfate reference electrode,  $Cu/Cu_2SO_4$ , after an initial stabilization period are on the order of -0.4 v, which is within the range of potentials for carbon steel (from -0.75 v to -0.35 v depending on steel composition and activity). The corrosion rate as measured by polarization measurements is initially low at 2 to 3  $\mu\text{m}/\text{year}$ , then increases to 10-11  $\mu\text{m}/\text{year}$ , which is below the maximum predicted rate of 12  $\mu\text{m}/\text{year}$  used to establish sacrificial steel thickness.

**Table 3-1. Summary of Field Corrosion Data, Site 4, Lower Level.**

Test Specimens	Initial Data	Exposure Times			
		2 Months	8 Months	15 Months	26 Months
$E_{cor}^{(a)}$ , Steel, in Volts	-0.548	-0.458	-0.400	-0.329	-0.351
Corrosion Rate <sup>(b)</sup> , Steel, in $\mu\text{m}$	6.5	3.2	1.9	10	11
$E_{cor}$ , Zn, in Volts	-0.915	-1.000	-0.921	-0.830	-0.963
Corrosion Rate, Zn, in $\mu\text{m}$	55	2.5	0.4	1.2	1.1
$E_{cor}$ , Galvanized, in Volts	-0.938	-0.708	-0.775	-0.781	-1.014
Corrosion Rate, Galvanized, in $\mu\text{m}$	43	1.0	0.4	1.6	2.7
$E_{cor}$ , Reinforcement level 1, in V.	-0.976	-0.681	-0.870	-0.654	-0.710
Corrosion Rate, Reinf. 1, in $\mu\text{m}$	7.6	1.2	0.9	1.8	0.9
$E_{cor}$ , Reinforcement level 2, in V.	-1.005	-0.793	-0.920	-0.663	-0.756
Corrosion Rate, Reinf. 2, in $\mu\text{m}$	4.5	0.8	0.6	1.3	0.6
(a) Free-corrosion potential, V(Cu/CuSO <sub>4</sub> ). (b) Corrosion rate estimated from polarization resistance corrected for soil resistance, $\mu\text{m}/\text{yr}$ unless otherwise noted. (c) $\mu\text{m}/25.4 = 1 \text{ mil}$					

The free potential for zinc coupons, galvanized coupons and reinforcing strips are all more negative than that for steel. They range from -0.75 to -1.1 v, which is typical for zinc or well galvanized steel. After an initial stabilization period, the corrosion rate as measured by polarization measurements, varies from approximately 0.5 to 2.7  $\mu\text{m}/\text{year}$ , which is considerably less than the maximum design rate of 15  $\mu\text{m}/\text{year}$  for the first 2 years for zinc or galvanized steel. This lower rate is reasonable when considering the resistivity of the fill at 15,000 ohm-cm, which is considerably greater than the minimum required 3000 ohm-cm for MSE reinforced fills. This typical data confirms the suitability and practicality of the monitoring techniques using electrochemical principles. Complete confirmation of the suitability of the design corrosion rates previously recommended can only be made after at least a few years of measurements.

Additional confirmation of the suitability and practicality of techniques using potential monitoring have been made by a number of state transportation agencies through MSEW corrosion assessment programs as reviewed in Section 3.5.

### **3.4 APPLICATION OF CORROSION MONITORING TO ASSET MANAGEMENT**

Asset management is an important issue facing highway operations, and forecasting the needs for maintenance, retrofit or replacement of existing facilities is an important component of Transportation Asset Management (TAM). MSE structures should be included in a TAM program along with pavements, bridges, ancillary structures, etc., to help ensure optimal usage of limited available funding (FHWA, 2008). Properly defining the existing inventory and development of a performance database are important components of asset management. Relatively rapid, non-intrusive, and nondestructive test techniques are needed to collect data necessary for corrosion monitoring and condition assessment of MSE. Results from condition assessment and corrosion monitoring indicate when, or if, accelerated corrosion is occurring, and can help transportation agencies decide on the most appropriate course of action when subsurface conditions are unfavorable and service life is uncertain. Agencies can also use these data to evaluate the variance associated with the performance of an inventory; this is valuable information for those with an interest in making reliability-based decisions. This chapter describes the framework of a performance database useful for asset management, test techniques and protocols that are being employed to collect performance data for earth reinforcements, data interpretation and preliminary information available from data that has been collected to date.

#### **a. Document Inventory and Prepare Performance Database**

As part of a TAM strategy, data are collected to document the performance of MSE structures including the condition of metallic reinforcements and corresponding rates of metal loss. A performance database is under development (NCHRP Project 24-28) wherein the attributes of the MSE inventory are documented, as are results from corrosion monitoring and condition assessment of metallic reinforcements. TAM programs are data driven and require databases that (a) identify and name components, (b) describe locations, c) define and describe data, and d) explain performance. Several existing databases developed by the: NYSDOT (Wheeler, 2002), Colorado Department of Transportation (CDOT [Hearn, et al., 2004]), Association for Metallically Stabilized Earth (AMSE, 2006), KTRC (Beckham, et al., 2005), and ODOT (Timmerman, 1990) were reviewed. In general, these databases follow a format and protocol consistent with that employed by the FHWA mandated Bridge

Management System (Hearn, et al., 2004). These databases were considered and used as a basis to develop the framework for an MSE performance database that can be a useful resource for asset management.

In general, the database is self-contained yet structured such that it can be ported to other existing databases. The database is formatted using MicroSoft Access<sup>®</sup>, which is linked to a GIS (ArcView) platform to provide visual and spatial recognition of data. The organization and structure of the various tables and data fields are updated, as necessary, to accommodate different types of information that are identified from available data sets. For example, observations of reinforcement performance/condition may be based on nondestructive testing (NDT), direct physical measurement, or visual observations, and these data types are archived accordingly. Drop down lists and check boxes are implemented to facilitate mining/querying of the database.

Information within the shell of the database is distributed amongst seven distinct tables comprising a total of 150 data fields. The tables are divided into categories of information similar to that employed in other databases that are based on the FHWA Bridge Management Inventory. The database includes the following tables:

- Project
- Walls/Structure
- Reinforcements
- Reinforced fill/Subsurface
- Observation Points
- NDT Results (e.g., potential and polarization resistance measurements)
- Direct Observations (e.g., retrieval of buried coupons)

Data forms were created to facilitate data entry. Tables are related using a one to many relationship using “project number” as a key parameter. Other relationships may also be created, but currently all other tables are considered to be a sub-form to the project form, which serves as the master form. Thus, a project may have a number of walls and/or reinforced fills. A wall may have numerous observation points; and a number of observations, including NDT or direct physical observations may be associated with each observation point. For example, the project in Las Vegas, Nevada described by Fishman et al (2006) includes three walls; Wall #1 has 15 monitoring locations, Wall #2 has six, and Wall #3 has four. Each monitoring location includes two in-service reinforcements wired for monitoring (NDT) and at least two steel coupons; one plain, and one galvanized. Also, direct physical measurement of section loss is performed on 18 samples retrieved from six of the

monitoring stations (i.e., three reinforcements exhumed from six of the stations). These data are all organized into separate tables that are linked to the Las Vegas, Nevada entry from the Project Table. Relationships are also defined between reinforced fill, wall, reinforcements, monitoring stations and results tables.

Each project is associated with a point that is displayed on a map within ArcView. ArcView mapped points are also linked to the Microsoft Access tables so pertinent information for each project can be displayed next to each point when selected by the user. In this way, the geographic distribution of performance data, as well as specific attributes for each site can be displayed within a GIS platform. Thus, the user may associate the data with geographic location and view all of the performance data and pertinent information associated with that point.

The database provides an archive for information and can be used as a reference to compare measurements from sites that may represent similar conditions. As such the database is an asset and a valuable resource. Agencies implementing condition assessment and performance monitoring programs can use this information to help interpret their results within a broader context than a site-specific study may provide. Interpretations based on past experience can be useful for refining metal loss models, understanding conditions leading to end of service life, and gaining a better understanding of the various factors and their effects on corrosion rates.

## **b. Data Needs and Data Analysis**

Several studies (e.g., Anderson and Sankey, 2002) involve exhuming reinforcements from in-service facilities, but the original dimensions of the elements are uncertain because corrosion monitoring was not anticipated during installation. In these cases, the original dimensions or weights of the elements must be estimated to compute metal loss from observations of remaining thickness or weight. Nominal dimensions may be used with confidence for the steel thickness, however the original thickness of the galvanized layer is more uncertain. A minimum thickness of zinc coating is specified, however the hot dipped galvanization process specified by ASTM A123 (ASTM, 2007) results in galvanized layer thickness that may exceed the minimum by a wide margin. (For example, Rossi (1996) reports thickness measurements as high as 280  $\mu\text{m}$  from elements with a specified minimum zinc coating thickness of 86  $\mu\text{m}$ .)

Various estimates of initial zinc thickness are used as a basis to evaluate zinc loss of exhumed reinforcements in the absence of initial measurements including (1) observing the zinc thickness from a sample retrieved from the connection to the wall face where the reinforcement is sandwiched between two plates and assuming that zinc loss is negligible due to the fact the reinforcement sandwiched within the connection is not in direct contact with the reinforced fill, and (2) assuming that zinc oxide adhered to the surface is equivalent to the loss of zinc. Both of these assumptions can lead to large errors, and uncertainty with respect to initial conditions remains. However, often one or both of these methods may be used to obtain a conservative estimate of zinc loss, i.e., overestimate the initial zinc thickness.

Alternatively, electrochemical corrosion monitoring test techniques may be applied for monitoring earth reinforcements as described in Section 3.1 and 3.2. With these techniques, a large number of frequent samples may be obtained, and the initial thickness of the zinc is not needed to interpret the measured corrosion rate. Because the tests are nondestructive, reinforcements are left intact and in service after testing, and available for future monitoring. As discussed in Section 3.2, when measurements are taken throughout the service life of a wall, these techniques can quantify the relationship between rate of metal loss and time. Ideally, protocols for condition assessment and corrosion monitoring should include both direct physical observations (i.e., weight loss, remaining thickness, pit depth) and electrochemical tests such as LPR and half-cell potential measurements. However, very few of the documented studies currently include such complete data (see Section 3.5 for examples of existing data).

Collecting information on site conditions and reinforced fill character are very important components of data collection activities. Better and more consistent data on reinforced fill are needed. In particular, samples need to be taken in proximity to reinforcements that are being monitored, and at the time that measurements are being taken. Current data on reinforced fills are often from pre-construction samples taken from stockpiles that may not represent conditions prevailing during corrosion monitoring.

The performance database includes thousands of measurements of element condition and corrosion rate from more than 150 sites distributed throughout the United States and Europe. The large sample domain allows evaluation of sample statistics, distributions of element conditions and corrosion rates, and corresponding probability based analyses. These issues are related to reliability of metal loss modeling, quantification of the effect of construction practice on performance, and understanding the cost benefits of using different materials. All of these are important components of asset management. For example the database can be used to:

- Study the mean and variance of corrosion rates for data sets grouped according to different climate, site conditions, and reinforced fill conditions
- Quantify performance for marginal reinforced fills
- Evaluate the performance of different material, i.e. steel vs. zinc, other forms of metallization, and the use of polymeric coatings

**c. Estimate Future Needs for Maintenance, Rehabilitation and Replacement; or Extended Design Life**

Issues that can address future needs for maintenance, rehabilitation, retrofit or replacement include:

- Spatial variations of element condition and corrosion rate; e.g. top vs. bottom of wall
- Special areas that may deserve increased maintenance; e.g. in proximity to drainage inlets
- Quantify effects of different climates, use of deicing agents, etc.

*Improved knowledge of spatial variations and special problems can lead to improved allocation of resources.* For example, in some cases extended service life may be best achieved by retrofitting areas surrounding drainage inlets, or the benefits of improved maintenance of drainage inlets may be realized in terms of increased service life. In areas where deicing slats are used, corrosion monitoring can demonstrate the need to maintain pavements, improve drainage or install impervious barriers.

**d. Update Experience with Different Reinforced Fills**

An example of the experience gained from collecting and analyzing data relates to the use of reinforced fills that may or may not meet AASHTO specifications for electrochemical properties. The available database was divided into two primary groups including data from reinforced fill conforming to AASHTO criteria and from reinforced fill not conforming to AASHTO criteria. The AASHTO corrosion model was applied to estimate reinforcement corrosion rates and compare them to measured corrosion rates. The following observations were made from the existing database. These observations are preliminary and may be updated as more data become available.

- For reinforced fills conforming to AASHTO criteria, the AASHTO corrosion model overestimates steel corrosion rates for 98% of the data. It should be noted that most of the data in this group are associated with reinforced fills that meet AASHTO requirements by a wide margin.

- For reinforced fills conforming to AASHTO criteria, marine environments have minor to no effect on measured corrosion rates of galvanized reinforcements, but marine environments accelerate corrosion rates of plain steel reinforcements.
- For reinforced fills that do not satisfy AASHTO criteria, marine environments are associated with relatively high corrosion rates.
- Reinforced fills that do not meet AASHTO criteria, i.e. soil resistivity values  $\rho < 3000 \Omega\text{-cm}$  and pH values  $< 5$ , can significantly affect steel corrosion rates, which tend to dramatically increase beyond rates estimated by the AASHTO corrosion model.
- Based on available data, organics content, chlorides, sulfates and relatively high values of pH have much less effect on measured corrosion than do relatively low resistivity and pH.
- Review of the latest research information confirms the safety of the electrochemical requirements for fill and associated metal loss rates in the current AASHTO standards.

### **3.5 STATE CORROSION MONITORING PROGRAMS**

As of 2008 nine states including California, Florida, Georgia, Kentucky, North Carolina, New York, Nevada, Ohio and Oregon have implemented MSEW corrosion assessment programs. Corrosion monitoring practices vary including (1) installation of coupons/inspection elements for extraction and direct physical observations at planned intervals, (2) wiring of coupons and in service reinforcements for electrochemical testing and corrosion monitoring, (3) a combination of 1 and 2, and (4) monitoring reinforced fill conditions during service. Few agencies have comprehensive programs that include both direct physical observations and electrochemical testing. Table 3-2 is a summary of the programs, a brief description of the practices, and references that provide details of the program and results from corrosion monitoring. Detailed descriptions of corrosion monitoring activities in California, Florida, New York and North Carolina are presented to demonstrate different corrosion monitoring practices and implementation of corrosion monitoring programs.

- Results presented from Caltrans demonstrate how climate and fill quality can impact performance; and the benefits of using inspection elements that can be probed by electrochemical techniques and later extracted for inspection.
- Results presented from The Florida Department of Transportation (FDOT) describe how the impact from saltwater intrusion is evaluated with a corrosion-monitoring program.

FDOT implemented electrochemical techniques similar to those describe in this reference.

- Results collected from two sites wired for monitoring by the The New York State Department of Transportation (NYSDOT) demonstrate how corrosion rates measured with the LPR technique can vary throughout the course of a year, and with respect to different seasons. NYSDOT also implemented electrochemical techniques similar to those describe in this reference.
- Results presented from the North Carolina Department of Transportation (NCDOT) are used to demonstrate how half-cell potential measurements can be useful to assess zinc life for galvanized reinforcements.

Details from other state corrosion monitoring programs are available from the references cited in Table 3-2

### Caltrans

Based on the recommendations by Jackura et al. (1987), CalTrans began installing inspection elements within MSE walls constructed since 1987. Ten-foot long rods were inserted as inspection elements within walls using steel grid reinforcements; 10-foot (3 m) long strips were inserted within walls using strip-type reinforcements. Caltrans extracts the inspection rods to make visual examinations of surface pitting and loss of section, and laboratory testing to assess remaining zinc and loss of tensile strength ((Initial-Remaining)/Initial). Because initial tensile strength was not measured prior to installation, it is estimated from the intact sections of exhumed inspection elements. The inspection elements are accessible from the wall face and can also easily be wired for electrochemical testing.

Caltrans collected data, in cooperation with NCHRP Project 24-28, including (1) fieldwork and LPR testing at 7 sites in the southern part of the state, (2) comparison of corrosion rates collected by different operators and equipment, and (3) laboratory testing and evaluation of inspection elements extracted from the field. Table 3-3 describes the sites. Eight walls were selected for monitoring (two walls were monitored at the site of Bridge #53-2819). All of these walls support highway embankments and are approximately 10 years old. The Riverside wall is located in a desert environment, two locations in San Bernardino are in a residential area and a Mediterranean climate, and the five walls in Long Beach are in a coastal/marine environment.

**Table 3-2. Summary of State DOT MSEW Corrosion Assessment Programs.**

<b>State</b>	<b>Description</b>	<b>References</b>
California	Have been installing inspection elements with new construction since 1987, and performing tensile strength tests on extracted elements. Some electrochemical testing of in service reinforcements and coupons has also been performed. LPR and EIS tests were performed on inspection elements at selected sites as part of NCHRP Project 24-28 and results compared with direct physical observations on extracted elements.	Jackura et al. (1987), Elias (1990), Coats et al. (1990), Coats et al. (2003- Draft Report)
Florida	Program focused on evaluating the impact of salt-water intrusion, including laboratory testing and field studies. Coupons were installed and reinforcements were wired for electrochemical testing and corrosion monitoring at 10 MSE walls. Monitoring has continued since 1996.	Sagues et al. (1998, 1999, and 2000), Berke and Sagues (2009)
Georgia	Began evaluating MSE walls in 1979 in response to observations of poor performance at one site located in a very aggressive marine environment incorporating an early application of MSE technology. Exhumed reinforcement samples for visual examination and laboratory testing. Some in situ corrosion monitoring of in service reinforcements and coupons at twelve selected sites using electrochemical test techniques was also performed.	McGee (1985), Deaver (1992)
Kentucky	Developed an inventory and performance database for MSE walls. Performed corrosion monitoring including electrochemical testing of in service reinforcements and coupons at five selected sites.	Beckham et al. (2005)
Nevada	Condition assessment and corrosion monitoring of three walls at a site with aggressive reinforced fill and site conditions. Exhumed reinforcements for visual examination and laboratory testing; performed electrochemical testing on in service reinforcements and coupons. A total of 12 monitoring stations were dispersed throughout the site providing a very good sample distribution.	Fishman et al. (2006)
New York	Screened inventory and established priorities for condition assessment and corrosion monitoring based on suspect reinforced fills. Two walls with reinforced fill known to meet department specifications for MSE construction are also included in program as a basis for comparison. Corrosion monitoring uses electrochemical tests on coupons and in service reinforcements.	Wheeler (1999, 2000, 2001, 2002a and 2002b)
North Carolina	Initiated a corrosion evaluation program for MSE structures in 1992. Screened inventory and six walls were selected for electrochemical testing including measurement of half-cell potential and LPR. This initial study included in service reinforcements but coupons were not installed. Subsequent to the initial study, NCDOT has installed coupons and wired in-service reinforcements for measurement of half-cell potential on MSE walls and embankments constructed since 1992. LPR testing was also performed at approximately 30 sites in cooperation with NCHRP Project 24-28.	Medford (1999)
Ohio	Concerned about the impact of their highway and bridge deicing programs on the service life of metal reinforcements. Performed laboratory testing on samples of reinforced fill but did not sample reinforcements or make insitu corrosion rate measurements	Timmerman (1990)
Oregon	Preliminary study including 1) a review of methods for estimating and measuring deterioration of structural reinforcing elements, 2) a selected history of design specifications and utilization of metallic reinforcements and 3) listing of MSE walls that can be identified in the ODOT system.	Raeburn et al. (2008)

**Table 3-3. Selected Sites in Southern California.**

<b>Bridge #</b>	<b>Location</b>	<b>Intersection</b>	<b>Reinforcement Type</b>	<b>Year Built</b>	<b>Height (ft)</b>
56-0794M	Riverside	I10E & Union Pacific RR	Strip	1996	25
54-1093M	San Bernardino	30th St. & Pico Road	Strip	1988	30
54-1094M	San Bernardino	30th St. & Miramonte Dr.	Strip	1988	18
53-2819M	Long Beach	Rt. 47 Naval & Marine Base	Grid	1997	36
53-2821M	Long Beach	Rt. 47 & Navy Way	Grid	1997	38
53-2823M	Long Beach	Rt. 47 & Navy Way	Grid	1997	40
53-2822M	Long Beach	Rt. 47 & Navy Way	Grid	1997	28

1 ft = 0.3 m

Data collection focused on testing all available inspection elements with electrochemical test techniques including half-cell potential and linear polarization resistance measurements. In general, a cluster of 18 inspection elements was available for monitoring along each wall. The LPR test results were used as a guide to select two or three elements for extraction and physical observation. In this manner, elements suspected of a low level of corrosion, as well as those suspected to exhibit a higher level of corrosion, could be extracted and the utility of the LPR technique to identify moderate to severely corroded, or mildly corroded elements could be evaluated. This is a very important exercise because the LPR technique also provides an opportunity to study the spatial and temporal variability of corrosion rate.

Caltrans prepared special spools of wire for making the electrical connections necessary for electrochemical testing of each element. Alligator clips were used to attach a spool of wire to each inspection element. The spools were used to extend the wires from each element to a junction platform with a set of holes to accommodate the end of each spool as shown in Figure 3-10. One person, aided by an aerial lift, could make spool connections to all elements in the cluster, which greatly facilitated production.

LPR tests were performed with the 3-electrode configuration consisting of a working electrode, counter electrode and reference electrode. The element directly above or below each test element (working electrode) was used as a counter electrode. Reference cells (CSE) were placed in wetted soil at the base of the wall directly beneath each element, and connected to a spool of wire that also led to the spool junction.



Figure 3-10. Junction platform and wire spools used to organize connections to inspection elements for electrochemical testing.

**Table 3-4. Summary of Laboratory Data From Caltrans and Comparison with Field Observations.**

Inspection Element			Reinforced fill		Condition			CR ( $\mu\text{m}/\text{yr}$ )	
Bridge No.	Locale	Location, Type	w %	$\rho_{\text{min}}$ ( $\Omega\text{-cm}$ )	$E_{\text{corr}}$ (mV)	Zinc ( $\text{oz}/\text{ft}^2$ )	Pitting	Loss	LPR
53-2819	07-LA-47	R13, Rod	-	1610	-793	1.5	N	NO	0.4
53-2821	07-LA-47	L14, Rod			<b>-474</b>	-	Y		<b>40</b>
53-2821	07-LA-47	L16, Rod			<b>-484</b>	-	Y		<b>32</b>
53-2822	07-LA-47	L13, Rod	8.5	3580	-740	1.4	N	NO	0.9
53-2822	07-LA-47	L14, Rod			<b>-537</b>	-	Y		<b>104</b>
53-2822	07-LA-47	L15, Rod	10.7	5849	-714	1.5	N	NO	1.0
53-2823	07-LA-47	L17, Rod			<b>-511</b>		Y		<b>42</b>
53-2823	07-LA-47	L15, Rod			<b>-475</b>		Y		<b>25</b>
54-1093	08-SBD-30/215	L2, Strap	2.9	6223	-540	5.5	Y	9.5 <sup>1</sup>	0.7
54-1093	08-SBD-30/215	L12, Strap	2.1	12705	-594	4.7	N	NO	2.0
54-1094	08-SBD-30/215	L8, Strap	-	-	-581	4.8	Y	6.0 <sup>1</sup>	1.2
54-1094	08-SBD-30/215	L14, Strap	-	-	-610	1.3	Y	7.5 <sup>1</sup>	1.3
56-0794	08-Riv-10	L7, Strap	-	-	-356		Y	break	80
56-0794	08-Riv-10	L 11, Strap	-	-	-567		Y	40	50
56-0794	08-Riv-10	L 13, Strap	0.4	377	-550	5.2	Y	28 <sup>1</sup>	3.7

<sup>1</sup> pit involves a small surface area on strap

 High moisture content, low  $\rho_{\text{min}}$ , and corresponding higher corrosion  $\mu\text{m}/25.4 = 1 \text{ mil}$

Reinforced fill samples were extracted near inspection element locations and Caltrans completed lab testing on samples of reinforced fill and inspection rods. Table 3-4 is a summary comparing reinforced fill and inspection rod conditions, and corrosion rates measured via direct observation (measured pit depth) and from linear polarization resistance (LPR) measurements. Rod location is identified from a cluster at the left, L, or right, R, side of the wall; and from near the top (#1-#6), middle (#7-#12), or bottom (#13-#18), i.e. R13 is from the bottom row of the right cluster.

Corrosion rates computed from observations of remaining diameter/pit depth shown in Table 3-4 compare qualitatively with measurements from LPR. In cases where relatively high corrosion rates were measured via LPR ( $> 20\mu\text{m}/\text{yr}$ ), pitting and corresponding loss of section were also observed along the inspection rods. The corrosion rates at the point of maximum section loss may be four times higher than the average rates measured via LPR, which is consistent with expectations considering the geometry of the rod type inspections elements (Smith et al., 1996). In a couple of instances, corrosion rates measured via LPR are higher than direct observations, however, these LPR measurements are anomalous, and when repeated with different equipment such high values of corrosion rate are not consistently observed (Fishman and Reis, 2008). Pitting observed for strap type reinforcements covered small areas that did not have a significant impact on tensile strength, and relatively low corrosion rates are indicated via LPR.

Correlations of corrosion rate and loss of zinc are particularly interesting because the reinforced fill samples were retrieved from the same locations as the inspection elements. This is not usually the case, and most often reinforced fill data is derived from samples taken at stockpiles or from random locations within the reinforced fill. Higher corrosion rates and lower remaining zinc ( $< 2 \text{ oz}/\text{ft}^2 \{0.605 \text{ g}/\text{m}^2\}$ ) measurements are consistently correlated with reinforced fill samples that simultaneously exhibit relatively low minimum resistivity ( $\rho_{\text{min}}$ ) and high moisture content. This trend is illustrated in Figures 3-11 and 3-12 where higher corrosion rates are not always observed in Figure 3-11 when  $\rho_{\text{min}}$  is low; or in Figure 3-12 when moisture contents are higher. However, a comparison of points with  $\text{CR} > 20 \mu\text{m}/\text{yr}$  reveals that both of these conditions are met in these instances. This comparison demonstrates the value of obtaining reinforced fill samples and corrosion rate measurements at the same location and at similar times. The data shown in Figures 3-11 and 3-12 help to explain why higher corrosion rates are not always observed from sites with poor quality reinforced fill (e.g., low  $\rho_{\text{min}}$ ).

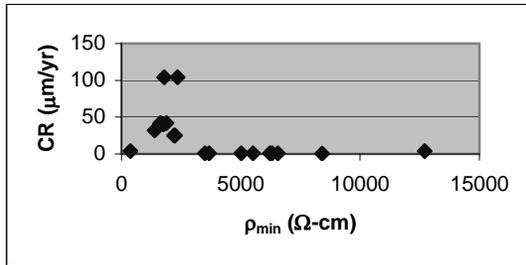


Figure 3-11. Minimum Resistivity ( $\rho_{min}$ ) vs. corrosion rate (CR) for corresponding reinforced fill and inspection rod locations.

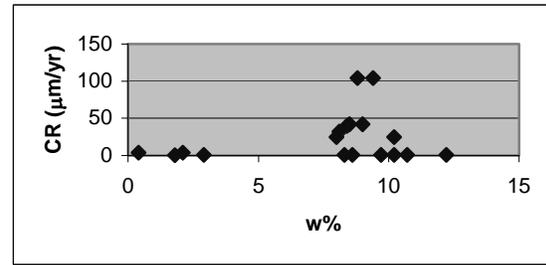


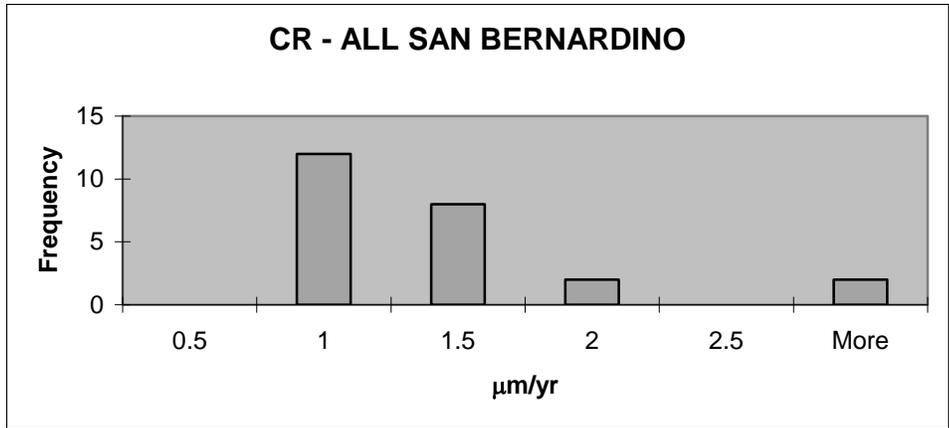
Figure 3-12. Moisture content (w%) vs. corrosion rate (CR) for corresponding reinforced fill and inspection rod locations.

Inspection elements that exhibited high corrosion rates appeared to break at a reduced cross section during extraction. Therefore, a lot of the data on remaining tensile strength is not with respect to the locations with the most severe section loss. Tensile strength data is useful to document the remaining strength of less corroded sections (that did not break upon extraction), and to study inherent variation of material strength.

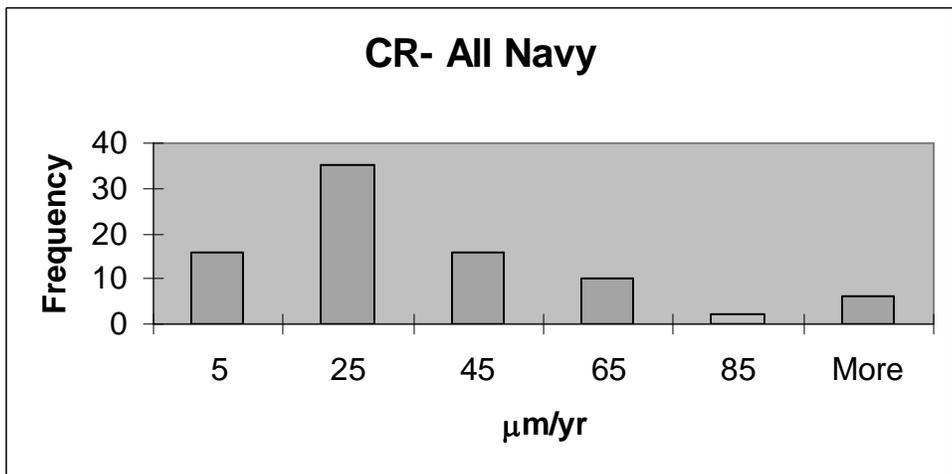
Data from LPR measurements are useful for screening sites relative to level of corrosion. Figure -13 shows the distribution of corrosion rate measurements from sites in Riverside, San Bernardino, and Long Beach/San Pedro (coastal). These data clearly indicate that corrosion is more severe at the coastal sites, and at the Riverside site, compared to San Bernardino.

### Florida

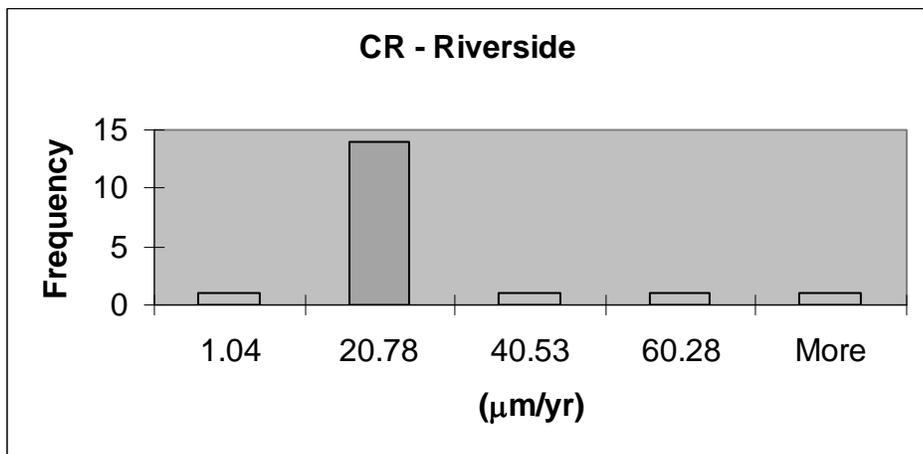
In 1995, the FLDOT implemented a corrosion-monitoring program to assess the condition of in-service galvanized reinforcements used in the construction of MSEW. Due to the coastal locations of many of the MSEWs within the FLDOT inventory, the department placed particular emphasis on the effects that saltwater inundation may have on the performance of metallic reinforcements. Due to the occurrence of episodic flooding (e.g. hurricanes), saltwater inundation may occur for sites located in tidal regimes or estuaries. Laboratory tests were conducted at the University of South Florida (Sagues, et al., 1998) to evaluate rates of corrosion for controlled environments and include effects of saltwater inundation and development of macro cells. Field studies involved NDT (LPR and  $E_{corr}$ ) performed on a set of structures incorporating a range of ages and environments, and including installation of coupons, and wiring of in-service or active reinforcements. Visual observations and results from direct physical testing of exhumed components from selected sites were also documented.



a) San Bernardino – Foothills



b) Long Beach/San Pedro Area- Coastal



c) Riverside- Arid

Figure 3-13. Distribution of corrosion rates from sites in Southern California.

The FLDOT-specified reinforced fill for MSE construction must meet a minimum set of electrochemical criteria similar to that described by AASHTO. The FLDOT reinforced fill selection guidelines relative to soil chemistry and particle-size distribution were met in virtually all of the test locations and structures tested. The mean resistivity and pH of reinforced fill data collected from all the Florida sites is 25,000  $\Omega$ -cm (range 11,000 to 60,000  $\Omega$ -cm) and 8.2, respectively. Thus, in the absence of episodic salt water flooding, these reinforced fills are in the range of low aggressivity relative to corrosion. Two of the structures monitored by FLDOT were in a marine environment where the reinforced fill was susceptible to inundation with seawater. At one of these locations elevated chloride levels (457 ppm) and correspondingly low resistivity (445  $\Omega$ -cm) were observed. However, these low resistivity levels were transient due to flushing of salts from the free draining fill.

Ten walls were selected for corrosion monitoring as described by Sagues, et al. (1998). The walls were constructed between 1979 and 1996. Baseline readings were taken in 1996. At each site, the measured half-cell potentials (CSE) of the galvanized reinforcements were lower (more negative) than those of the embedded carbon steel samples. On average, the difference was approximately 200 mV. This indicates that a significant amount of zinc coating was still present on the reinforcements. Tables 3-5 to 3-8 summarize statistics of the observed corrosion rates, which are based on LPR measurements taken between 1995 and 1998. The data are described in terms of (a) the two sites subjected tidal fluctuations (Table 3-5); (b) the oldest site located in Tallahassee (Table 3-6), where the possibility exists for zinc to be consumed along some of the galvanized reinforcements that were monitored; (c) the newest sites, Brickell Ave, in Miami used crushed limestone reinforced fill rendering a unique mineralogy compared to the other sites, high in carbonates (Table 3-7); and (d) the remaining five sites considered typical coastal or inland locations (Table 3-8). Each of the 10 walls incorporates between two and four clusters of measurements. A test cluster consists of electrical connections to two close, but electrically independent reinforcing strips or grids, and a reference electrode. A buried plain steel bar is usually included in the cluster. In general, the test cluster is identified by the number of precast panels with respect to the abutment or some other specified point or reference (i.e., Station R3 is the third panel from abutment), and usually the elevations are within approximately five feet from the base of the wall to facilitate access. Data are presented in terms of  $\mu$ , COV and range of measurements from each monitoring station or measurement cluster.

For the five walls that are considered “typical,” (Table 3-8) the mean corrosion rates range from 0.5 to 1.15  $\mu$ m/yr with a COV ranging from 21% to 38%. The mean corrosion rate for the oldest wall (Tallahassee, Table 6 is 0.9  $\mu$ m/yr, which is within the range for the typical walls, but the COV of 82% is much higher, which may reflect the variation inherent to loss

of zinc coating. After the zinc is consumed, higher corrosion rates of the underlying steel are anticipated, and due to variation of zinc consumption between samples, the corrosion rate measurements are also variable. Similar statistics are realized for the new walls (Table 3-7) and the walls subject to tidal fluctuations (Table 3-5), whereby the mean corrosion rates are slightly higher, or within the range measured for typical walls, but the COV is much higher. These higher COV's may be due to the variable effects of time on corrosion, and variable conditions within the reinforced fill subsequent to saltwater inundation.

**Table 3-5. Florida Corrosion Rate Statistics for Sites Subjected to Tidal Fluctuations.**  
(Sagues et al., 1998)

Site	Built (Year)	Station	$\mu$ ( $\mu\text{m}/\text{yr}$ )	COV	Range ( $\mu\text{m}/\text{yr}$ )
St. Lucie	1992	R3	2.0	15%	1.7-2.4
		R7	2.6	46%	1.3-4.1
		<b>Site</b>	<b>2.3</b>	<b>38%</b>	
Palm City- NW	1991	R3	1.2	65%	0.25-2.4
		R7	1.3	80%	0.48-2.7
		<b>Site</b>	<b>1.25</b>	<b>73%</b>	

**Table 3-6. Florida Corrosion Rate Statistics for 17 Year Old Tallahassee Site.**  
(Sagues et al., 1998)

Station	$\mu$ ( $\mu\text{m}/\text{yr}$ )	COV	Range ( $\mu\text{m}/\text{yr}$ )
R17	1.6	50%	0.9-3.0
R23	1.3	96%	0.4-3.1
R44	0.4	35%	0.3-0.5
R62	0.3	29%	0.2-0.3
<b>Site</b>	<b>0.9</b>	<b>82%</b>	

**Table 3-7. Florida Corrosion Rate Statistics for New Brickell Ave. Site.**  
(Sagues et al., 1998)

Station	$\mu$ ( $\mu\text{m}/\text{yr}$ )	COV	Range ( $\mu\text{m}/\text{yr}$ )
NW-Bottom	1.0	59%	0.4-2.3
NW-Top	0.7	1.7%	0.69-0.71
SE-Bottom	1.8	130%	0.01-9.8
SE-Top	0.7	67%	0.6-0.7
<b>Site</b>	<b>1.1</b>	<b>112%</b>	

**Table 3-8. Florida Corrosion Rate Statistics for Typical Coastal and Land Sites.**  
(Sagues et al., 1998)

Site	Built (Year)	Station	$\mu$ ( $\mu\text{m}/\text{yr}$ )	COV	Range ( $\mu\text{m}/\text{yr}$ )
Howard Franklin	1992	R7	0.7	33%	0.5-1.0
		R11	0.6	20%	0.4-0.7
		R15	0.4	37%	0.2-0.5
		R17	0.5	39%	0.1-0.7
		<b>Site</b>	<b>0.5</b>	<b>36%</b>	
Ocala	1983	R6	0.3	20%	0.2-0.4
		R5	0.7	20%	0.6-0.8
		<b>Site</b>	<b>0.5</b>	<b>21%</b>	
Jacksonville	1990	R9	0.5	40%	0.3-0.7
		R21	0.7	18%	0.6-0.9
		<b>Site</b>	<b>0.6</b>	<b>28%</b>	
Tampa	1997	R16	1.2	20%	1.0-1.5
		R23	1.1	23%	0.8-1.3
		<b>Site</b>	<b>1.15</b>	<b>21%</b>	
Palm City-NE	1991	R1	0.5	29%	0.3-0.8
		R5	0.7	29%	0.5-1.0
		R14	0.6	32%	0.4-0.8
		R28	0.6	54%	0.4-1.1
		<b>Site</b>	<b>0.6</b>	<b>38%</b>	

New York

In 1998, The New York State Department of Transportation established corrosion monitoring stations at two walls in Amherst, NY as part of its MSE wall corrosion assessment program. The walls included embankment walls for the Lockport Expressway (I-990) over Sweet Home Road and I-990 over Dodge Road. These MSE walls were constructed in 1980 and 1981 and were filled with a mixture of lightweight fill containing blast furnace slag and cinder ash. Due to the nature of the reinforced fill and the fact that other MSE walls backfilled with a lightweight industrial waste product in the area had to be replaced due to severely corroded metal reinforcing straps, the occurrence of accelerated corrosion was suspected at these sites. Results from chemical testing indicated that the reinforced fill provided an aggressive corrosive environment. Table 3-9 presents a summary of the chemical properties of the slag/cinder ash reinforced fill. All of the reinforced fill parameters described in Table 3-9 are outside the limits recommended by AASHTO.

**Table 3-9. Electrochemical Properties of Slag/Cinder Ash Reinforced Fill at 1990 over Dodge and Sweethome Road Sites in Buffalo, NY**

Test	Results	AASHTO Specified Limits
Resistivity	426 to 963 $\Omega$ -cm	> 3000 $\Omega$ -cm
pH	10.9 to 11.4	5 to 10
Chlorides	6 to 499 ppm with 4 of 8 samples > 100ppm	< 100 ppm
Sulfates	523 to 742 ppm	< 200 ppm

Each site includes four monitoring stations that are at least 100 ft (30 m) apart. Each station includes measurements from five in service reinforcements that are within 2 to 3 feet (0.6 to 1 m) of each other, and from steel, zinc and galvanized coupons. NYSDOT collected corrosion monitoring data from these sites subsequent to installation of corrosion monitoring stations in 1998 (Wheeler, 2002); and then in May 2002 and November 2005. Corrosion monitoring includes measurements of half-cell potential and corrosion rates from LPR. Observations from coupons and reinforcements support the following conclusions:

1. The corrosion rates of the zinc coupons are significantly higher than those of the steel and galvanized coupons and of the in-service galvanized reinforcements. This may be due to the fact that the reinforced fill at this site includes slag and cinder ash that is relatively alkaline (pH > 11), and the corrosion rate of zinc is adversely affected by high alkalinity.
2. In general, the observed temporal variations of corrosion rates are negatively correlated to the temporal variation in measured soil resistance.
3. In general, the half-cell potentials of the steel and galvanized coupons and the in-service reinforcements are in the same range of -400 mV to -500 mV, but the zinc coupons exhibit a distinctly different range of -800 mV to -900 mV. This indicates that the zinc is nearly consumed on the galvanized coupons and in-service reinforcements.
4. In general, corrosion rates observed for galvanized coupons and in-service reinforcements are similar, and less than the corrosion rates for the plain steel coupons.

As part of NCHRP 24-28 readings were taken at approximately monthly intervals between June 2007 and December 2008 in an effort to document the seasonal variations of corrosion rate. Results from these monthly measurements are summarized in Figure 3-14 including a

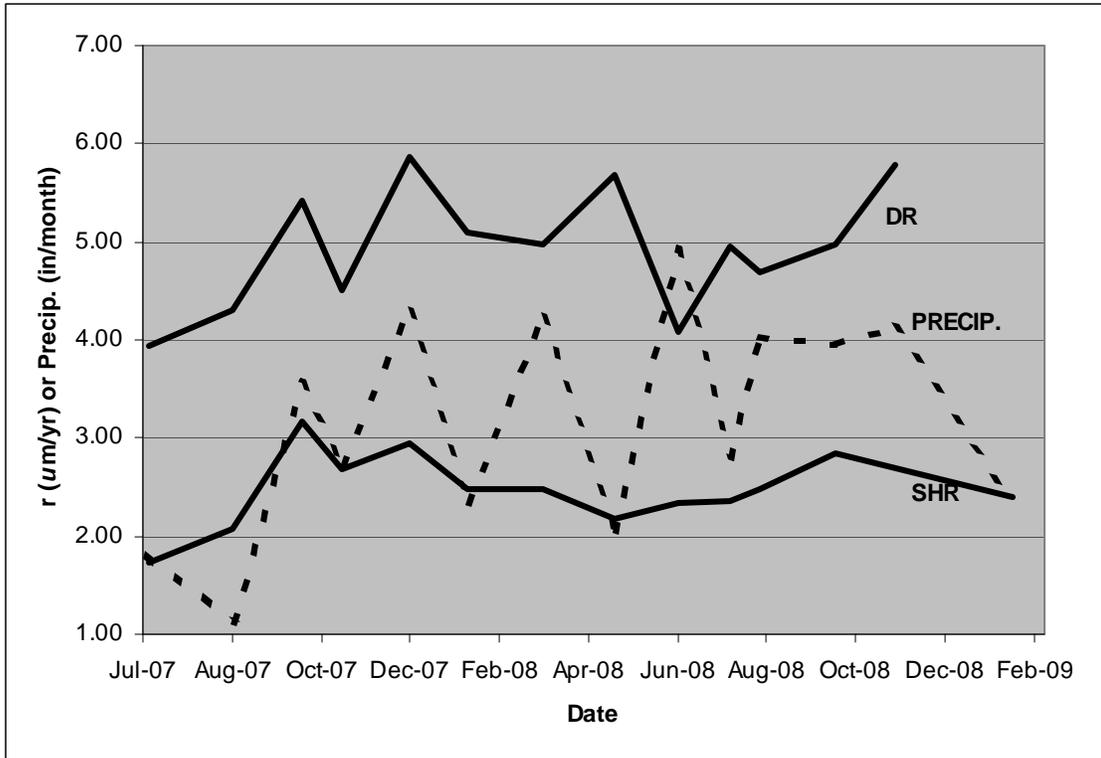
comparison between corrosion rates and monthly precipitation; and measurements of soil resistance. The data in Figure 3-14 represent the means of the measured corrosion rates from reinforcements at the respective sites. These means are associated with coefficients of variance between 10 and 40 percent. These data indicate that corrosion rates appear to correlate well with monthly precipitation and corrosion rate measurements at this site vary by a factor of 1.5 considering seasonal fluctuations.

### North Carolina

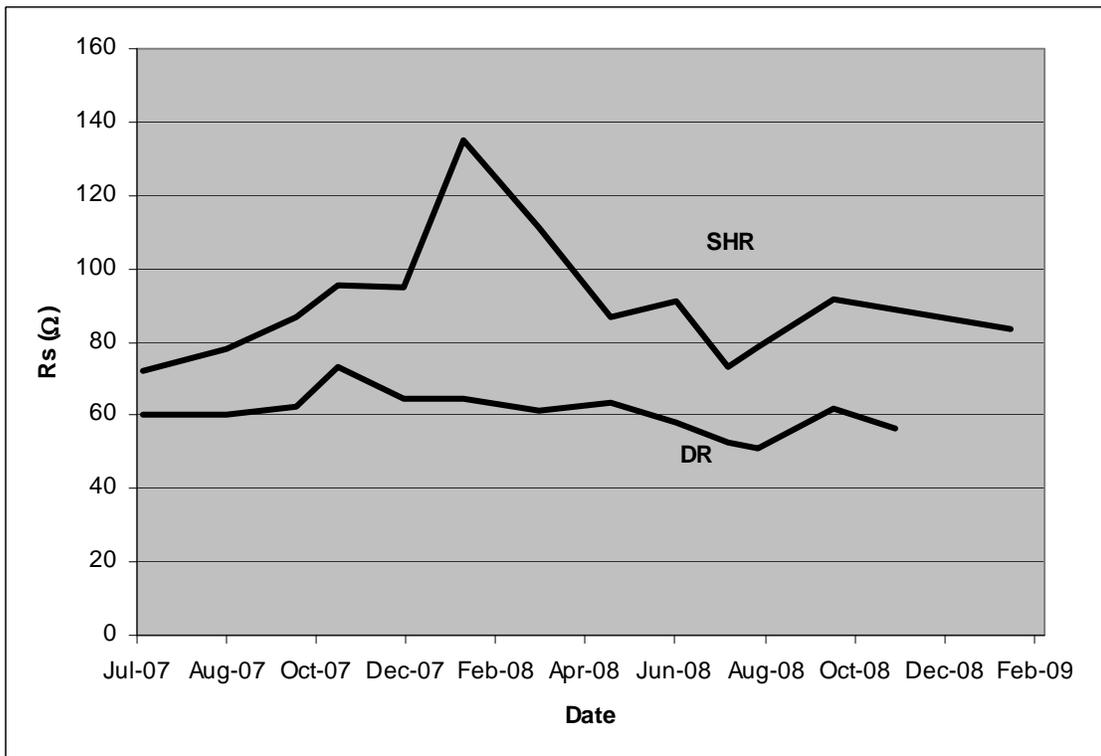
In 1990, the North Carolina DOT (NCDOT) implemented a corrosion-monitoring program for MSE walls, and proceeded to monitor half-cell potentials of reinforcements and coupons at regular intervals (Medford, 1999). Corrosion monitoring stations were established during wall construction including zinc and steel coupons, and in service reinforcements were wired for monitoring. Initial half-cell potential measurements were taken immediately after wall construction, with subsequent readings taken approximately once a year thereafter.

Figure 3-15 depicts the distribution of measurements obtained from 48 monitoring stations distributed among 18 project locations and 12 counties within North Carolina. All of the MSE walls in Figure 3-15 incorporate grid-type reinforcement, galvanized in accordance with ASTM A123 (ASTM, 2002), corresponding to at least 2 oz/ft<sup>2</sup> (0.605 g/m<sup>2</sup>) of zinc. Walls are backfilled with material meeting the requirements of North Carolina designation No. 57 stone, with resistivity greater than 5000 Ω-cm, pH between 4.5 and 9.0, and less than 1% organics; i.e. these reinforced fills are noncorrosive. Walls included in Figure 3-15 were constructed between 1992 and 1999, and records of half-cell measurements are available from the date of wall construction.

The data in Figure 3-15 are useful to evaluate how the potential of the galvanized reinforcements approach those of steel coupons; presumably as the sacrificial zinc coating is consumed by corrosion. These data present a real-world example of the concepts described in Section 3.2 and Figure 3-2. Initially, the half-cell potentials of the reinforcements and zinc coupons are relatively close. After ten years the half-cell potentials of the galvanized reinforcements have shifted slightly towards those of the steel coupons and away from those of the zinc coupons. However, in general, they are still closer to zinc than steel. This supports the conclusion that much of the zinc coating remains after ten years, which is to be expected given the noncorrosive nature of the reinforced fills included in the study.

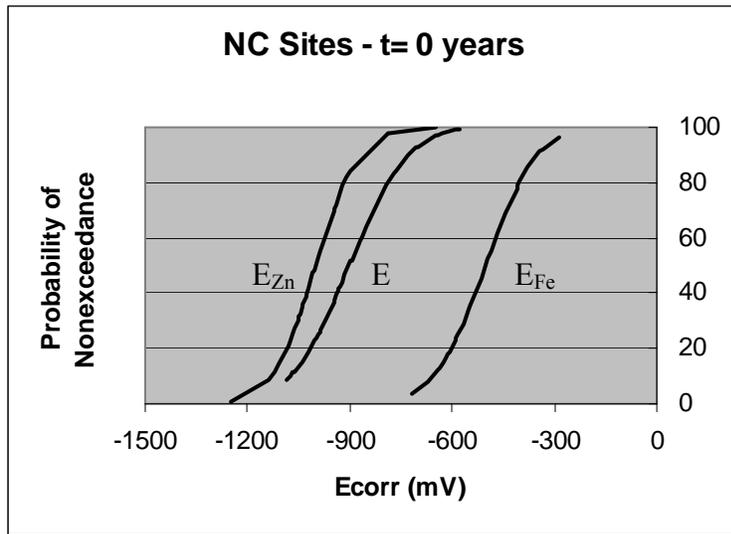


a) Mean Monthly Corrosion Rates Compared to Monthly Precipitation

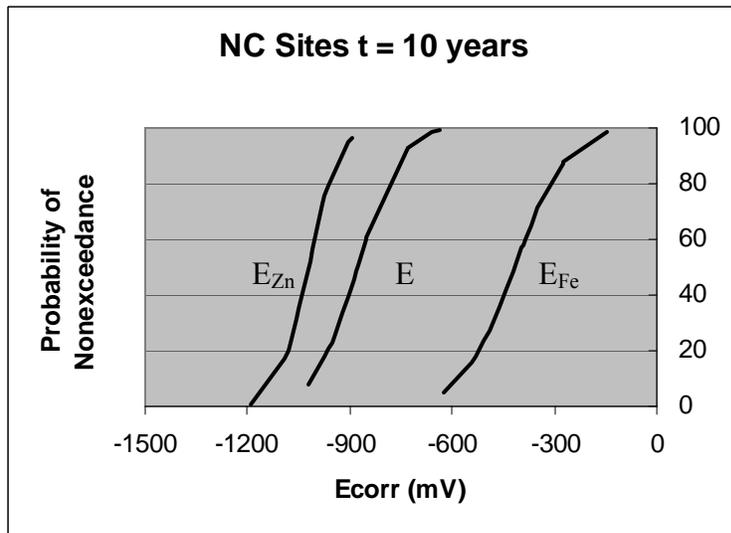


b) Mean Monthly Soil Resistance

Figure 3-14. Transient response of reinforcements at Dodge and Sweet Home Road sites.



a) initial measurements (taken within several months after construction)



b) measurements taken 10 years after construction

Figure 3-15. Half-cell potentials from coupons and in service reinforcements from corrosion monitoring of MSE walls constructed in North Carolina.

We can obtain another “picture” of the performance of the galvanized reinforcements if we define the relative change of half-cell potential in terms of the zinc index (ZI), which is computed at a given time (age of reinforcement) as:

$$ZI = \frac{\Delta E}{\Delta E_{Zn}} = \frac{E - E_{Fe}}{E_{Zinc} - E_{Fe}} \quad (3-3)$$

where  $\Delta E$  and  $\Delta E_{Zn}$  are the differences, compared to the half cell potential of the steel coupon ( $E_{Fe}$ ), for the reinforcement ( $E$ ) and zinc coupon ( $E_{Zn}$ ), respectively, at a given time. The value of the zinc index ranges from one to zero. A value of one corresponds to 100% zinc remaining, and a value of zero corresponds to 100% zinc consumption. Figure 3-16 shows the evolution of ZI as a function of time for the North Carolina data.

Figure 3-16 graphically illustrates that the rate of zinc consumption (decreasing zinc index) is greatest during the first two years; i.e. the zinc index decreases more in the first two years compared to the five-year time interval between years five and ten.

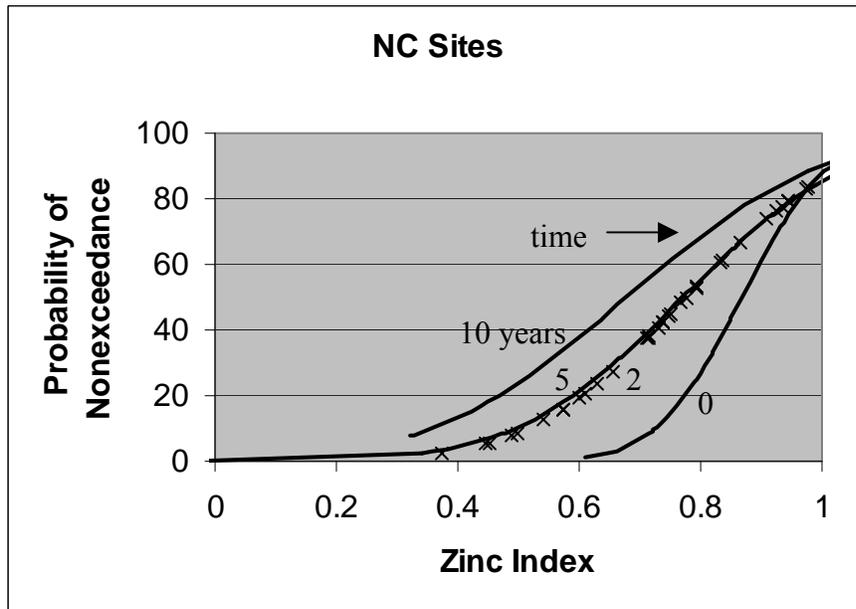


Figure 3-16. Zinc Index (ZI) distributions at increasing reinforcement age from corrosion monitoring of MSE walls constructed in North Carolina.

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## CHAPTER 4

### DURABILITY OF GEOSYNTHETIC REINFORCEMENTS

#### 4.1 INTRODUCTION

Due to their economic advantages and relatively inert state, the use of polymeric reinforcements in MSE walls and RSS embankments has significantly increased over the past three decades. Because of their relatively short period in use, there are some uncertainties as to their durability with respect to maintaining tensile strength properties after exposure to construction stresses and during exposure to an in-soil environment over the anticipated design life. Potential degradation of polymeric reinforcements with time (aging) will depend on the specific polymer, configuration of the reinforcements, the environment to which they are exposed, and the level of stress to which they are subjected.

The current design approach to account for construction damage and long-term degradation strength losses is to apply to the ultimate tensile strength ( $T_{ult}$ ) reduction factors. The nominal strength,  $T_{al}$  is then obtained from:

$$T_{al} = \frac{T_{ult}}{RF_{CR} \times RF_{ID} \times RF_D} = \frac{T_{ult}}{RF} \quad (4-1)$$

where:

- $RF_{CR}$  = Creep reduction factor
- $RF_{ID}$  = Installation damage reduction factor
- $RF_D$  = Durability reduction factor
- $RF$  = The product of all applicable reduction factors, dimensionless
- $T_{ult}$  = Ultimate geosynthetic tensile strength, lb/ft (kN/m)

This chapter is intended to provide a background on geosynthetic materials for reinforcement applications. It addresses their structure and manufacturing process as it may affect durability, identify degradation mechanisms and environments, and provide the basis for selecting appropriate polymers. It also provides the basis for selecting aging ( $RF_D$ ) and installation damage ( $RF_{ID}$ ) reduction factors, consistent with the in-ground regime and the corresponding nominal strength used for design, in lieu of using an overall default reduction factor.

### a. Overview of Available Products

The main polymers currently used for MSEW and RSS structures include the polyolefins (i.e., polypropylene and polyethylene) and polyester. The final form of the polymer and its corresponding reaction to its environment may vary considerably, depending on the polymer formulation, additives used in the composition, and the methods of processing the polymer into its final form (fibers, filaments, and subsequent fabric for geotextiles or joined drawn strands in the case of geogrids). The method of manufacture for geosynthetics may be a factor for short-term construction durability.

Based on 2003 market information (Koerner, 2005), the total worldwide geosynthetics market was over 1500 million square yards. Estimates by the authors indicating the U.S. market exceeds 600 million square yards for geotextiles and geogrids alone. Elias (2001) found that the U.S. market was divided on a raw polymer material basis as shown in Table 4-1. The U.S. market as further divided on a geosynthetic structure basis is shown in Table 4-2.

The U.S. geosynthetic market based on an application basis, is shown in Table 4-3.

*Geogrids, woven geotextile, and nonwoven geotextiles, are used extensively for steepened slopes, but most retaining wall applications use geogrid reinforcement.*

**Table 4-1. Raw Material in Geotextile and Geogrid Production.** (Elias, 2001)

<u>Raw Material</u>	<u>% of Market</u>
Polypropylene	80
Polyester	14
Polyethylene	6
Other	< 1

**Table 4-2. Geotextile and Geogrid Structure.** (Elias, 2001)

<u>Structure</u>	<u>% of Market</u>
Nonwoven Geotextile	77
Woven Geotextile	16
Geogrid	6
Other	1

**Table 4-3. Geosynthetic Applications.** (Elias, 2001)

<u>Application</u>	<u>% of Market</u>
Asphalt Overlay	22
Separation/Stabilization	28
Reinforcement for Subgrade, Wall & Slope	9
Drainage	12
Lining Systems	17
Erosion Control	6
Silt Fence	6

For polyolefins, polypropylenes (PP) are manufactured from general purpose homopolymers (group 1, class 1, ASTM D4101), which may differ in grade (Melt Flow) slightly, based on the manufacturing process used. Grade 4 appears to be universally used.

The major suppliers of PP reinforcement geosynthetics and their properties as furnished by the manufacturers and listed in their catalogs are listed in Table 4-4.

The manufacturing of polypropylene geosynthetics requires that process stabilizers (e.g. antioxidants) and ultra violet (UV) inhibitors be used to maintain the required end-properties of the polymeric materials. The type and quantity of additives used is considered proprietary by manufacturers and can be expected to vary slightly between production runs.

A variation in additive concentration may result from the amount of "regrind" material used. Regrind generally results from material which has been manufactured but does not meet the quality assurance standards set for that particular product. For example, the leading portion of a nonwoven geotextile in a production run may have inconsistent density, or a density too low for the product style being manufactured. This is not an unusual occurrence at the beginning of a nonwoven geotextile production run. Because there is nothing inherently wrong with the resin used in the material, it may be reground, melted, and added to the resin used for other production runs. Most manufacturers add resin obtained from polypropylene regrind to "virgin" batches of resin which do not yet contain stabilization additives. The entire batch is then treated as if it was comprised entirely of virgin resin and the corresponding amount of additives is then introduced. Therefore, the amount of stabilization additive contained in a batch of resin which includes previously used resin will have a higher concentration of additives than batches which are truly virgin. This procedure is employed to ensure that a minimum standard of geosynthetic stability is consistently adhered to.

Only one high density polyethylene (HDPE) product, a geogrid, is currently being manufactured and distributed in the U.S. and it is widely used in reinforcement applications. Its composition is shown in Table 4-5.

The major product groups and the properties of polyester products (PET) that are produced for geogrids and geotextiles are listed in Table 4-6. PET geogrids are coated with a polyvinyl chloride (PVC) or an acrylic polymer to provide some protection from construction induced damage and to ensure dimensional stability during manufacturing.

From Table 4-6, in which all products in this group are listed, high tenacity polyester fibers used in grids and high-strength woven geotextiles are characterized by molecular weights (Mn) in the 25,000-30,000 range and Carboxyl End Group (CEG) numbers of 12 to 29. The non-woven geotextiles are typically manufactured using PET with a somewhat lower molecular weight and substantially greater CEG number. No additives are used other than a surface finish on the high tenacity fibers. It should be noted that high tenacity fibers classified by the producers and/or manufacturers as "first quality" only, will have carefully controlled physical and chemical properties. Manufactured woven products may be produced from "industrial grade" PET fibers, which are composed of material not meeting the requirements for the end use for which they have been formulated and manufactured. Accordingly, physical and chemical properties may vary widely from production run to production run.

## **b. Materials Structure and Manufacture**

The engineering properties of a geosynthetic, i.e., its functionality in terms of tensile strength, water permeability, and filtration efficiency are significantly influenced by their composition and structure. The most commonly used geosynthetics for transportation applications are:

- Woven geotextiles:
  - monofilaments
  - multifilaments
  - slit tape
  - combinations
- Nonwoven geotextiles:
  - needlepunched
  - heat bonded
  - resin bonded
  - combinations
- Geogrids:
  - extruded uniaxial
  - extruded biaxial
  - woven coated fiber
  - welded
- Geocomposites
  - geogrid/nonwoven geotextiles
  - inline filament knitted to nonwoven geotextiles
  - geonet with nonwoven geotextile

**Table 4-4. Major PP Product Groups. (2000)**

Product Name	Structure	Manufacturing Process	Polymer Class & Grade ASTM D4101	Melt Flow g/10 min. ASTM D1238	Antioxidants Process/ End Use Additives	Weight Range g/m <sup>2</sup>	Ultimate Wide Width Strength kN/m
1. Tensar "BX"	Grid	Extruded Sheet, punched & stretched	Grade 2 Group-1, Class-1	0.3 -1.0	Irganox 1010 Carbon Black	195- 470	13 - 39
2. Tenax "MS"	Grid	Extruded & stretched Multi layer	Grade 2 Group-1 Class-1	0.3 - 1.0	Carbon Black	220- 350	18 - 31
3. a) TenCate Mirafi Construction Products	Woven	Fibrillated yarns woven	Grade 4 Group-1 Class-1	4	1% Carbon Black	284 - 866	35 - 158
3. b) TenCate Mirafi Construction Products	Woven	Monofilament yarns	Grade 4 Group-1 Class-1	3	2% Total of 36% Carbon Black 63% LLDPE	170 - 270	26 - 47
4. a) LINQ "Typar"	Non Woven	Thermally spun bonded	Grade 4 Group-1, Class-1	4.5	HALS Carbon Black	N/A	N/A
4. b) LINQ "GTF"	Woven	Extruded tape	Grade 4 Group 1, Class 1	8	N/A	N/A	15 - 38
5. a) Propex Fabrics Inc.	Woven	Monofilament	Group 1, Class 1	0.1-12	Carbon Black	N/A	30
5. b) Propex Fabrics Inc.	Woven	Slit film tape fibrillated tape	Grade 4 Group 1, Class 1	N/A	N/A	N/A	21 - 70
5. c) Propex Fabrics Inc.	Non Woven	Needle punched, staple fiber	Grade 4 Group 1, Class 1	N/A	N/A	N/A	N/A
7. Tensar TG	Non Woven	Needle punched continuous filament	Grade 6 Group 1, Class 1	< 22	HALS	N/A	5 - 25

N/A - Not Available

**Table 4-5. HDPE Product Group. (2000)**

Product Name	Structure	Manufacturing Process	Polymer Class & Grade ASTM D1248	Melt Flow ASTM D1238 (g/10 min)	Antioxidants Process/End Use Additives	Weight Range (g/m <sup>2</sup> )	Ultimate Wide Width Strength (kN/m)
Tensar "UX"	Grid	Extruded Sheet Punched & Stretched	Type III, Class A, Grade E-5, Category 5	< 0.4	Irganox 1010 Carbon Black	360-1360	35 - 180

**Table 4-6. Major PET Product Groups. (2000)**

Product Name	Structure	Geogrid Coating (Grid Only)	Fiber or Polymer Source, Type	Molecular Wt. g/mol or Viscosity	CEG meq/kg.	Additives	Weight Range g/m <sup>2</sup>	Ultimate Wide Width Strength kN/m
1. STRATA Systems, Inc. "Stratagrid"	Grid knitted	PVC dipped	N/A	27,000	24	none for PET PVC- fungicide, UV inhibitor	200 - 600	22 - 146
2. Reco "Matrex"	Grid	PVC dipped	T-811 Hoechst/Celanese	I.V. = 0.89	17	none	330 - 1290	48 - 370
3. TenCate Mirafi Construction Products "Miragrid"	Grid	PVC Coating	T-811 Hoechst/Celanese	30,000 (I.V. = 0.91)	26 - 29	none	200 - 1500	29 - 370
4. Huesker "FORTRAC"	Grid	PVC dipped & cured	AKZO	30,000 (I.V. = 0.91)	20	none	170 - 1600	15 - 600
5. TenCate Mirafi Construction Products HS Series	Woven filament yarn		Hoechst/Celanese T-800, 100, 236	I.V. = 0.88 I.V. = 0.905 I.V. = 0.910	17 28 12	none	290 - 2400	70 - 735
6. LINQ "GTF"	Woven filament yarn		T-800 Hoechst/Celanese	I.V. = 0.88	17	none	N/A	70 - 175
7. Huesker "COMTRAC"	Woven filament yarn		AKZO Allied Fibers	I.V. = 0.91	17	none	250 - 1560	50 - 800

N/A - Not available

In manufacturing geotextiles, elements such as fibers or yarns are combined into planar textile structures. The fibers can be continuous *filaments*, which are very long thin strands of a polymer, or staple fibers, which are short filaments, typically  $\frac{3}{4}$  to 2 in. (20 to 50 mm) long. The fibers may also be produced by slitting an extruded plastic sheet or film to form thin flat tapes. In both filaments and slit films, the extrusion or drawing process elongates the polymers in the direction of the draw and increases the filament strength. After the drawing process, filaments and tapes may also be fibrillated, a process in which the filaments are split into finer filaments by crimping, twisting, cutting or nipped with a pinned roller. This process provides pliable, multifilament yarns with a more open structure that are easier to weave.

Geotextile type is determined by the method used to combine the filaments or tapes into the planar structure. The vast majority of geotextiles are either *woven* or *nonwoven*. Woven geotextiles are made of *monofilament*, *multifilament* or *fibrillated* yarns, or of slit films and tapes. The weaving process derives from textile cloth-making. Nonwoven textile manufacture is a process by which synthetic polymer fibers or filaments are laid onto a moving belt. Then the mass of filaments or fibers are either *needlepunched*, in which the filaments are mechanically entangled by a series of small needles, or *heat bonded*, in which the fibers are *welded* together by heat and/or pressure at their points of contact in the nonwoven mass.

Geogrids with integral junctions are manufactured by extruding and drawing polyolefin (polyethylene or polypropylene) sheets. These types of geogrids are often called *extruded or integral* geogrids. Geogrids may also be manufactured of multifilament polyester yarns, joined at the crossover points by a knitting or weaving process, and then encased with a polymer-based coating. These types of geogrids are often called *woven or flexible* geogrids. A third type, a *welded* geogrid is manufactured by welding polymeric strips (e.g., strapping material) together at their cross over points. All these manufacturing techniques allow geogrids to be oriented such that the principal strength is in one direction, called *uniaxial* geogrids, or in both directions (but not necessarily the same strength), called *biaxial* geogrids.

## 4.2 FUNDAMENTALS OF POLYMER DEGRADATION

Polymers principally consist of long chains of carbon atoms with various branches and side groups. Under certain conditions, this structure can be attacked by oxidation promoted thermally, catalytically, or by ultraviolet light; by other forms of chemical attack including hydrolysis; by the combined effect of chemicals and mechanical load; or, by microorganisms.

Most polymers used in geosynthetics contain additives and stabilizers that improve the resistance of the basic polymer; however, these additives themselves can be susceptible to leaching or to biological attack, ultimately leaving the polymer unprotected. In addition, the structure can be damaged during compaction or by subsequent abrasion. The principal results of these degradative mechanisms are loss of mechanical strength and changes in elongation properties. This section outlines the potential degradation mechanisms and the available testing methods to quantify tensile strength losses and identifies the role of additives/antioxidants in enhancing long-term, in-ground durability.

#### **a. Oxidation of Polyolefins (PP and HDPE)**

The predominant degradation mechanism for most polymeric materials is chain scission, which is a polymeric reaction that breaks a bond on the backbone of a polymer chain, reducing the chain length and thereby reducing molecular weight (Koerner et al., 1986). This in turn significantly changes the polymeric structure and material properties, chiefly strength and elongation. The oxidation process is initiated by heat, light (UV radiation), mechanical stress, catalyst residue from manufacturing remaining in the geosynthetic, or reaction with impurities.

Antioxidants are additives that interrupt the degradation process in different ways, depending on their structure. The two major classifications are:

- chain terminating primary antioxidants and
- hydroperoxide decomposing secondary antioxidants.

Primary antioxidants are often sterically hindered phenols. They react rapidly to terminate chain scission and protect the polymer chain. Secondary antioxidants are most effective at elevated temperatures, as during manufacture processing, and effectively protect both the polymer and the primary antioxidant. They would include but not be limited to phosphite/phosphonite compounds. A new class of UV stabilizers, sterically hindered amines (HALS), is very effective in imparting stability at the lower temperatures consistent with in-ground use. Often, the protection obtained against oxidation by using a mixture of primary and secondary antioxidants in certain proportions is stronger than the sum of the protection effects obtained with individual compounds used separately. These synergistic mixtures are known as "master batch" and are proprietary to each producer. They can be varied to satisfy the intended usage and use regime.

***For long-term protection against oxidation induced strength losses, the geosynthetic should be produced with primary antioxidants that are not consumed during the manufacturing process.***

## b. Hydrolysis of Polyester (PET)

Hydrolysis is the reverse reaction of the mixing (synthesis) of terephthalic acid and ethylene glycol, which forms polyethylene terephthalate (PET) and water. Since this is an equilibrium reaction, it is reversible. Therefore, it is possible for the PET to react with water and to revert back to acid and glycol, which is a non-reversible process. In neutral environments (pH = 7), the reaction is initiated by the carboxyl end group (CEG) of the macro molecule of PET and is relatively slow. In alkaline environments, the reaction is more rapid due to the presence of the more reactive OH<sup>-</sup> ion compared to the water molecules as reagents in neutral (pH = 7) reactions. ***The effect of these reactions is a decrease in the molecular weight (Mn) with a corresponding decrease in strength.***

The rate of hydrolysis is primarily affected by:

- Carboxyl End Group (CEG) Concentration— These end groups are situated at the end of the molecular chains. The amount of carboxyl end groups in a particular PET product is dependent on the polymerization process used. Typically, the high tenacity fiber produced for geogrid and high strength woven products have lower CEG numbers compared to fiber produced for nonwoven geotextiles. Research has indicated that the hydrolysis rate of PET with higher CEG numbers proceeds faster under equivalent conditions. (Koerner et al., 1986)
- Molecular Weight – Molecular weight directly affects the CEG concentration under the same polymerization conditions. Therefore, PET polymers with a higher molecular weight contain less CEG than those with lower molecular weight and are less susceptible to hydrolysis under equivalent conditions.
- Temperature – As with oxidation, hydrolysis proceeds at a faster rate with increasing temperature.
- pH Level – High levels of environmental alkalinity will cause fiber dissolution in addition to hydrolytic reactions.
- Relative Humidity – The rate of hydrolysis increases as relative humidity increases.

***For long term usage, PET products of high molecular weight (Mn) and low CEG will be least susceptible to strength losses due to hydrolysis. PET should not be used in highly alkaline environments characterized by pH greater than 9 without significant test data to document suitability.***

### **c. Stress Cracking**

Semi-crystalline polymers such as high density polyethylene (HDPE) have a potential for stress cracking, which is a material failure caused by tensile stresses less than the short term mechanical strength. The failure is characteristically brittle, with no elongation adjacent to the failure. This phenomenon has two phases: crack initiation and crack growth. Environmental stress cracking (ESC) is the rupture of a polymer in a stressed state when exposed to a chemical environment. ESC tests are, therefore, differentiated from chemical resistance tests, in general, by the fact that the test specimens are exposed to a chemical environment while under stress.

Experience in the plastic pipe industry has shown that certain grades of polyethylene (PE) can experience stress cracking under certain conditions, and recent data has suggested a potentially similar behavior for some grades of PE used in geomembranes. It follows, therefore, that a possibility of stress cracking in geogrids fabricated of PE exists. Under "low" stresses at ambient temperatures, PE could fracture by slow crack growth given sufficient time. This mode of failure may limit the lifetime of and/or stress levels on PE used for critical load-bearing applications such as pressurized pipes, landfill linings, and reinforcement applications.

The stress-crack resistance of HDPE geogrids has been studied in FHWA RD-97-142 and a preliminary testing protocol was developed. For the presently available geogrid it was concluded that stress cracking is a less stringent than or equal consideration to creep for intact geogrids. For severely damaged geogrids, stress cracking may occur at load levels lower than inferred by the material creep limit. (Elias et al., 1997)

***The maximum stress level under either mode of failure determines a nominal stress basis.***

Therefore, if the backfill maximum size is limited to  $\frac{3}{4}$ -in. (20 mm), ensuring a low level of construction damage, no consideration for stress cracking appears warranted for the geogrid detailed in Table 4-5.

### **d. UV Degradation**

UV degradation occurs when geosynthetics are exposed to the influence of sunlight, rain, temperature, and oxygen. This type of degradation is caused primarily by the UV content of sunlight, which initiates the photo-oxidation process. The rate of degradation depends on the intensity of the relevant wave length and such additional factors as temperature, the presence of water and of certain atmospheric components such as ozone, nitrous oxides, hydrocarbons,

etc. Other factors are the material structure and the rate at which degraded layers are removed by rain and wind and new surfaces are exposed to UV radiation.

For polyolefin geosynthetics, significant resistance to UV radiation is obtained with the addition of antioxidants such as phenolics, hindered amine light stabilizers (HALS) and carbon black. These act as a screen to harmful portions of the light spectra. Once the geosynthetic is buried, the UV light-induced degradative process ceases because exposure to the UV source is terminated. Polyester is less affected by UV radiation because of the resistance of ester bonds to breakage.

Recent research has shown that the outdoor degradation process is a synergistic one in which both photo-oxidation caused by UV radiation and oxidation caused by elevated temperatures have an effect on the rate of degradation (Husan et al., 1994). *The data has indicated that the major effect is photo-oxidation, and, therefore, consideration of annual average energy incidence alone at a site may be sufficient to evaluate the effects of UV exposure.*

The resistance to UV degradation is measured in the laboratory by exposure to xenon arc light for a duration of up to 500 hours in accordance with ASTM D4355, or outdoors by direct exposure to sunlight and weather conditions under ASTM D5970.

#### **e. Biological Degradation**

Microorganisms causing deterioration are found in a wide range of environmental conditions. These microorganisms require a source of carbon for growth and obtain it from reactions degrading organic-based materials such as some of the polymers and additives potentially used in geosynthetics. Environmental factors controlling biodeterioration are temperature, humidity, pH, etc. Microorganisms of importance in biodeterioration are bacteria, fungi, actinomycetes, algae, and yeast. In general, elevated temperatures, high humidity, and the absence of UV light are required conditions. (Bright, 1993)

To grow, microorganisms excrete enzymes into the surrounding medium. The enzymes degrade the host material by breaking down its large molecular units into much smaller units that serve as food for the microorganism. The net effect is a reduction in molecular weight, with ensuing deterioration of physical properties such as weight, strength and elongation.

High-molecular-weight, high-density polymers used for geosynthetics, do not appear to be susceptible to direct enzymatic degradation by microorganisms such as fungi and bacteria. Several biodegradability studies have shown little loss in strength of any typical polymers used in geosynthetics when exposed to biologically active environments (e.g., mildew) for

periods of 1 year or more. There is some indication that very low molecular weight polymers can be consumed especially in the presence of nutrient fillers such as starch. (Potts, 1973)

***No completely relevant test to measure the resistance of geosynthetics to biological effects in unstressed states is presently available. ASTM D3083 has been used and can be adopted on an interim basis.*** Statistically, significant strength losses measured from this test should disqualify a candidate geosynthetic for long-term in-ground applications.

#### **f. General Chemical Dissolution**

Exposure of polymeric materials to extremely aggressive chemicals may accelerate the oxidation/hydrolysis processes in conjunction with a process of dissolution which is a separation into component molecules by solution. *Such regimes are not likely to be found naturally in soils*, but may be encountered in hazardous waste sites.

With regard to specific chemicals that may affect polymers, numerous chemical compatibility tables have been published by geosynthetic manufacturers such as the Plastic Pipe and Wire Insulation Institute and others in the polymer industry. There are several considerations if using such tables. Test conditions, including the exposure time (always short, less than 1 year), temperature, chemical concentration (usually very high), and strength evaluation methods, vary between the tables. For any specific polymer, the plastic formulations may vary considerably, especially between industries. Also, the form of the material evaluated (e.g., strap, fiber, block) and the material additives will have an effect.

***Therefore, these tables of compatibility are only useful in identifying specific regimes that are aggressive and therefore incompatible with specific geosynthetics.***

The resistance of geosynthetics to chemical effects in unstressed states can be measured in accordance with:

- ASTM D5322 "Practice for Immersion Procedures for Evaluating the Chemical Resistance of Geosynthetics to Liquids". This is a relatively short-term test (120 days) that should be modified for longer durations. A minimum of 9 months is recommended. The selection of immersion liquids is not specified.
- EPA 9090, "Compatibility Tests for Wastes and Membrane Liners" is a similar test at higher than ambient temperatures (122° F {50° C}), conducted with specific chemicals considered present at the investigated site.

Statistically significant strength losses measured from these short-term tests should disqualify a candidate geosynthetic for long-term, in-ground applications where the chemical condition is anticipated. Neither test, however, provides a sound basis for determining a reduction factor for strength.

#### **g. Summary**

The principal mechanisms of polymeric degradation have been outlined, all of which decrease tensile strength and change elongation characteristics. Geosynthetics are seldom degraded by a single environmental condition or mechanism, but often by a combination of synergistic actions or events. A list of commonly identifiable degradation mechanisms, their source, effect and test procedures to identify and quantify (by short-term laboratory tests) their consequence on long-term strength is presented in Table 4-7.

A general approach to quantifying geosynthetic durability and making lifetime predictions requires that the following objectives be achieved:

- Identify the nature of potential degradation mechanisms within a particular site and functional use, by examining the mechanisms and sources listed on Table 4-7.
- Identify the nature of the physical and chemical effects that these mechanisms have on candidate geosynthetics and their properties.
- Identify the type of test data necessary.
- Evaluate the degradation from available test data.

This process is illustrated in an example of a permanent geosynthetic-reinforced retaining wall. The wall is to be built in Arizona along a stream bank with a wrapped facing using local gravel as wall fill. The soil is determined to have a pH value of 8.5. Based on this information, the probable aging mechanisms can be identified as oxidation, hydrolysis, stress-cracking, UV degradation, and installation damage. Therefore, the design engineer should require the following for the geosynthetics being considered for use:

1. Polypropylene and polyethylene materials contain an antioxidant package to inhibit oxidation.
2. Polyester materials have suitable high molecular weight and low CEG numbers to inhibit hydrolysis.
3. Polyethylene materials manufactured from stress-crack resistant grade polymer.
4. The material is UV stabilized and is handled in a manner which minimizes exposure to sunlight on the project site. Further, a UV-resistant coating, such as bitumen or shotcrete, should be applied to the wall face or a wood or concrete panel facing can be constructed.
5. Installation damage testing be available for each candidate material consistent with the available gravel fill.

**Table 4-7. Commonly Identifiable Degradation Mechanisms. (Horrocks, 1990)**

<b>Mechanism</b>	<b>Source</b>	<b>Effect</b>	<b>Variables</b>	<b>Test Procedures</b>
<b>PHYSICAL</b>				
Stress/ pressure	Installation/in use	Rupture, creep, stress cracking	Stress level, Backfill grain size	ASTM D5262 & ASTM D6992 Stress Rupture Tests ASTM D5818 Installation Damage
Water	Installation/in use	Leaching of additives and plasticisers, hydrolysis	Temperature, pH	ASTM D5496 Immersion Testing
Solvents/ hydrocarbons	Installation: diesel mineral oils hot bitumen In use: bitumen	Leaching of additives, swelling and embrittlement, plasticization	Temperature, chemical concentration	ASTM D5322, EPA 9090 and Leaching tests.
Biological	Installation/in use: birds, animals, insects	Localized damage	Soil type and density	Not Available
<b>CHEMICAL</b>				
Heat (+ oxygen)	Installation: hot bitumen In use: ambient environment temperature	Chain scission and oxidation; Loss in tensile properties	Temperature Oxygen Content Transition metals	Oven Aging Tests at multiple temperatures
Light (+ oxygen)	Installation: UV exposure	Chain scission and oxidation; Loss in tensile properties	Radiation Intensity, Temperature, Humidity	ASTM D4335
Water (pH)	In-use: hydrolysis in acid, neutral and alkaline soils	Chain scission; Loss in tensile properties	Temperature, pH concentration, acid and alkali exposure	Hydrolysis testing at multiple temperatures
General chemicals	In use: exposure to natural soils and waste deposits	Degradation of polymer structure via oxidative/ hydrolytic chain scission	Temperature, Concentration	Immersion tests EPA 9090, ASTM D5322
Micro- organisms	In use: bacterial and fungal attack in soils	Polymer chain degradation; Loss in tensile properties	Temperature, pH Soil type, Organism type	ASTM D3083

### **4.3 IDENTIFICATION OF SOIL ENVIRONMENTS WHICH ACCELERATE DEGRADATION**

Soil environments that could accelerate degradation can be identified by their geological origins and composition. The physical regime (temperature and groundwater) can accelerate the degradation rate. Since not all polymers are subject to accelerated degradation in the same environments, it follows that an appropriate geosynthetic (polymer type) material can be chosen.

#### **a. Background**

Soil contains both inorganic and organic chemicals, with the inorganic material derived largely from the weathering of rocks and minerals and the organic materials from plants, animals, and microorganisms. In a large majority of soils, inorganic substances constitute the bulk of the soil material. In addition, the inorganic fraction contains acids and alkalies. Organic matter normally varies from less than 1 to 10 percent in soils that may be considered as highway construction fills, although in separation or stabilization applications, the geosynthetic may be placed directly over highly organic soils that may contain as much as 95 percent organic matter.

The bulk of most soils is made up of inorganic matter that ranges from 60 to 99 percent of the total weight, averaging 95 percent. About 47 percent is oxygen, the most abundant element, with oxides being the most prevalent form.

The physical soil environment, which includes such factors as temperature and moisture, varies widely. In-ground temperatures at a depth of 5 ft (1.5 m) vary from 32° to 75° F (0° C to 24° C) in the continental U.S. Surface temperatures vary between below freezing to 110° F (43° C).

The annual rainfall in the continental United States varies from nearly zero in the western desert to 60 in. (1500 mm) of rain in the Southeast and Pacific Northwest. Highway fills where reinforcement may be used are generally compacted near optimum moisture, which for many of the soils used would mean saturation percentages in excess of 65 percent and often near 95 percent. Geosynthetics used in separation/stabilization functions are often found at or below the piezometric water levels and therefore under fully saturated conditions.

Inorganic chemicals that are believed to affect buried geosynthetics comprise mineral acids, alkalies, salts, certain bivalent metals, gases and water. The organic compounds in soils affecting durability of geosynthetics are understood to be organic acids and solvents.

Certain natural soil environments can contain significant amounts of chemical substances which are degradable. The following natural processes have been identified as sources of chemicals, with water, oxygen and water, or heat being the catalyst:

- Sulfur Transformation - producing sulfuric acid, sulphur dioxide, hydrogen sulphide and water.
- Ammonification - producing ammonia in gaseous and aqueous state, ammonia bearing salts.
- Nitrification and denitrification - producing nitrates, nitric acid, nitrogen dioxide and nitrous oxide.
- Ferralitisation - producing hydroxides/oxides and ionized forms of iron and aluminum.
- Phosphorous Transformation - producing phosphate and phosphoric acid.

These and other processes form aggressive soils such as acid sulphate soils, organic soils, saline-alkali soils and calcareous soils. Other chemically reactive soils are ferruginous soils, which are high in iron content, and soils containing metals of manganese, copper, cobalt, and chromium (transition metals), as well as modified soils that may contain cement, limes, or de-icing salts. Cinders or slags may contain significant amounts of iron or other metals and sulfur.

The composition of some of the major natural soil groups identified as being potentially aggressive are further discussed below:

#### **b. Salt-affected Soils**

Salt-affected soils are generally found in arid and semiarid regions where there are high evaporation and transpiration rates. In the U.S. they primarily occur in 17 western states. Sodic soils, a sub group of salt-affected soils, are characterized by a low permeability and thus restricted water flow. The pH of these soils is high, usually >9 or 9.5, and the clay and organic fractions are dispersed because of the high levels of monovalent sodium and OH<sup>-</sup> ions. *They are the most alkaline environments found in the U.S.*

#### **c. Acid-Sulphate Soils**

Acid-sulphate soils are extremely acidic with pHs of < 3.5 and even lower. Such low pH levels are indicative of the presence of strong acids in the soils and thus hydrogen is the main acidic culprit. The origin of these strong acids is often the oxidation of pyrite (iron sulfide), which is oxidized to sulfuric acid.

Generally, rock containing pyritic sulfur in excess of 0.5 percent and containing little or no alkaline minerals will produce pHs of less than 4.5, which has considerable potential to produce sulfuric acid. These soils or rock are identified by the presence of noticeable yellow mottles attributable to pyrite oxidation. Typically, acid sulphate soils contain soluble levels of iron, manganese, copper, zinc, aluminum, and chlorides, although levels vary greatly and are abundant in the Appalachian regions. *When excavated and in the presence of groundwater, these soils produce sulfuric acid in significant quantity.*

#### **d. Calcareous Soils**

Calcareous soils are those that contain large quantities of carbonate such as calcite (calcium carbonate), dolomite (calcium-magnesium carbonate), and sodium carbonates and sulfates such as gypsum. These soils are characterized by alkaline pHs but are not saline. Calcareous soils are widespread and occur in Florida, Texas, New Mexico, and many of the western states. *Under certain conditions, they are characterized by pH in the range of 9 to 10.*

#### **e. Organic Soils**

Organic soils are referred to as bogs, peats, mucks, moors, organic silts, and organic clays. Most of them are water-saturated for most of the year unless they are drained. They contain organic soil materials to a great depth. The major concentrations are found in the Everglades of Florida and in the bog regions of Michigan and Minnesota. They are, however, widespread throughout the U.S.

The major organic components are fulvic, humic, and humin materials. Organic acids are generally negligible. *Biological degradation of geosynthetics in these environments is possibly due to the presence of nutrients for bacteria and microorganisms.*

#### **f. Soils Containing Transition Metals**

The literature has indicated adverse effects on polyolefin oxidation rates when transition metals such as copper, iron, chromium, manganese, and cobalt are present (Koerner et al., 1986). These metals are generally not found in the free state but rather as sulfides and oxides. Iron, the most abundant metal in the earth's crust, is not generally found in a free state but rather as sulfides such as pyrite ( $\text{FeS}_2$ ) or ferrous silicates ( $(\text{Mg Fe})_2\text{SiO}_4$ ) or from weathering in the form of oxides such as ferric oxide, hydrous oxide, ferrous carbonate and ferrous-ferric oxide, which characterize the "red earth" ferruginous soils.

The rest of these metals are rarely found in nature other than in spoil areas developed from mining operations or fills constructed from these spoils. Their presence, therefore, would indicate the potential for accelerated degradation by oxidation of any polyolefin geosynthetic (PP, HDPE).

#### **g. Modified Soils**

Modified soils such as cement or lime-treated fills can be quite alkaline depending on the soil type and the quantity of additive. Sandy soils of low plasticity treated with cement are often characterized by a pH greater than 10. Lime modification (1-2 percent lime) of sodic soils is also likely to increase the pH to 10 or more. Lime stabilization (5-10 percent lime) will always raise any soil pH above 12.

#### **4.4 IDENTIFICATION OF POLYMER CHARACTERISTICS/ADDITIVES TO MITIGATE DEGRADATION AND TESTING METHODS**

Each of the degradation mechanisms and specific environments will have a varying degree of effect, as the soil environment is quite diverse and changeable. Certain aggressive environments have been identified, but the level of in-ground aggressiveness for each polymer type is a function of such variables as oxygen availability, relative humidity (saturation level), concentration of aggressive elements (pH, transition metals, etc.), and temperature. The resistance of each polymer type is a function of its molecular structure and/or the additives (antioxidants) used to enhance its resistance to a specific degradation mechanism and environment.

The relative resistance of polymers and, therefore, their potential uses on specific soil environments identified as aggressive is indicated in Table 4-8. Certain polymers should not be considered without site-specific testing for their long-term durability or specific knowledge of their additives or molecular structure as indicated in Table 4-8. It should be noted that polymers identified as "Exposure Tests Required", may perform satisfactorily if formulated with specific antioxidants or additives to prevent degradation in that specific environment. **The durability of geosynthetic products can be significantly increased by the addition of antioxidants for polyolefin products (PP, HDPE) and longer polymerization to achieve high molecular weight (Mn) and low Carboxyl End Groups (CEG) for PET.** An understanding of these issues provides the background for discussion with manufacturers in selecting an appropriate product and/or site-specific testing for products listed as "Exposure Tests Required" in the potentially aggressive environments identified in Table 4-8.

**Table 4-8. Anticipated Resistance of Polymers to Specific Soil Environments.**

<u>Soil Environment</u>	<u>Polymer</u>		
	<u>PET</u>	<u>PE</u>	<u>PP</u>
Acid Sulphate Soils	NE	ETR	ETR
Organic Soils	NE	NE	NE
Saline Soils pH < 9	NE	NE	NE
Ferruginous	NE	ETR	ETR
Calcareous Soils	ETR	NE	NE
Modified Soils/Lime, Cement	ETR	NE	NE
Sodic Soils, pH > 9	ETR	NE	NE
Soils with Transition Metals	NE	ETR	ETR
NE =No Effect ETR =Exposure Tests Required			

**a. Polyolefins (PP and HDPE)**

Resistance to Oxidation

The relatively poor thermal stability of unstabilized polyolefin requires the addition of stabilizers during the heat processing of the raw polymer and its conversion to filaments, tapes, or sheets. Antioxidants are always added during melt processing to minimize the degree of thermal degradation on extrudates, and they may be varied in both character and concentration where long-term thermal stability is required based on ultimate functional use.

The antioxidant package and its quantity (loading) are often proprietary to each manufacturer. Polyolefin geosynthetics are often stabilized through the use of a hindered phenol as a primary antioxidant in combination with a phosphite. Selection of the hindered phenol depends on performance requirements such as thermal stability and on extraction/chemical resistance. In this case, the phosphite can be used to sacrificially stabilize the polymer, thus preserving the primary antioxidant for later use as a long-term thermal stabilizer. It has been further reported that the use of hindered amine light stabilizers (HALS) provides additional thermal stability when used with certain primary hindered phenol stabilizers (Gray, 1990). Carbon black, usually added to provide UV stability, is also believed to have some slight antioxidant property.

One measure of antioxidant effectiveness is resistance to leaching and chemical attack. While polyolefins are resistant to hydrolytic and other chemical attack, additives within them may not be and may leach out during exposure to water or other liquids.

The quantity and composition of antioxidants used for stabilization are often proprietary. The usual quantity of phenolic antioxidants is on the order of less than 1.0 percent by weight, somewhat less for hindered amine light stabilizers (HALS). Carbon black varies from 0.5 to 2.0 percent. The level of metal deactivators, where used, is unknown and proprietary.

A measure of the effectiveness of the antioxidant package or relative effectiveness to other formulations can be obtained from oven aging tests. In such tests, samples are incubated at a given elevated temperature for a specified period of time and their residual strength is measured. The relevance of these tests to long-term oxidative resistance is discussed in FHWA RD-97-144 (Elias et al., 1997a).

ASTM D-4355 is usually employed to measure effectiveness against UV radiation. This test incubates specimens in a weatherometer chamber, with retained strength measured after specific exposure times. *Many current specifications require a minimum retained strength of 70 percent after 150 hours, although a higher level of resistance such as 70 percent at 500 hours would be more appropriate for critical applications of permanent soil reinforcement in MSE walls and RSS slopes, especially in Southern areas with higher solar intensity.* This higher level of resistance is obtained by increasing the quantity of antioxidant which can increase product costs 10 to 15 percent, as the cost of the antioxidant by weight is roughly 100 times that of the polymer.

#### Stress Crack Resistance

High-density polyethylene materials are susceptible to stress cracking; therefore the final product for soil-reinforcement use should be formulated with additives to increase its stress-cracking resistance. Stress crack resistance of polymers as defined by ASTM D1248 is a function of its grade with eleven grades out of twenty being designated as having environmental stress-crack resistance. These are, E-4, E-5, E-8, E-9, E-10, E-11, J-3, J-4, J-5, P-24 and P-34.

Stress-crack resistance is measured by constant stress methods which quantify resistance and generate data for design purposes. Because of the long testing times required, the notched constant stress method ASTM D5397 has been introduced. This test is capable of determining relative ESC resistance among products. Testing to obtain design nominal strength based on stress-cracking limits is detailed in FHWA RD-97-142 (Elias et al. 1997b).

## **b. Polyesters (PET)**

The chief degradation mechanism for PET is hydrolysis in any aqueous solution, especially in inorganic acids, halogenated organic acids, inorganic and organic bases, benzyl alcohol, and halogenated bases. In acid media, degradation in laboratory tests has been observed at a pH of 1 or less, *a condition never found in nature* (Elias et al., 1997). In highly alkaline media, (pH >9) a dual degradative process of dissolution and hydrolysis has been reported in laboratory tests (Davis, 1988; Elias et al., 1997).

In an acid sulphate soil environment, sulfuric acid is generated to pH levels of as low as 1.5, but usually in the pH range of 2 to 3. In an alkaline environment, soils can exhibit pH greater than 10 with monovalent OH<sup>-</sup> ions present. Therefore, sodium hydroxide immersions in the pH range of 10-12 could model extreme in-soil regimes in some sodic alkaline areas as found in the West and Southwest.

*For hydrolysis to occur as modeled in immersion tests, the soil would have to be saturated, or sufficient moisture present for the PET to absorb some of the moisture and maintain high levels of humidity during its functional period of use.*

Calcareous soils are also alkaline and generally contain calcium (Ca) salts, although their pH is generally less than 9. These soils, including dolomitic soils, are widespread in the United States. Therefore, calcium hydroxide immersion tests in the pH range below 10 could model these extreme in-soil conditions.

The resistance to hydrolysis of PET geosynthetics is impacted during the polymerization process and is primarily a function of the molecular weight (Mn) and carboxyl end group (CEG) obtained from it. It has been reported that hydrolysis reactions can be slowed by the addition of certain stabilizers such as carbox di (ionides) or ethylene oxides. The function of these stabilizers is to convert the carboxyl end group to a non-acidic component. (Koerner et al., 1986)

Based on a survey of available products at present, it appears that high-strength PET geosynthetics (primarily woven and grid products) are produced with a molecular weight (Mn) generally, in excess of 25,000 and a CEG of less than 30. In contrast, nonwoven products are produced with molecular weights (Mn) of about or less than 20,000 and CEGs upward to 50.

Therefore, the (PET) geosynthetics produced with higher molecular weight are anticipated to be more resistant to hydrolysis, potentially by a factor of 1.5, which is roughly the ratio of the square root of their CEG concentrations (Koerner et al., 1986).

The presence of coatings (PVC or acrylic) over the load-carrying fibers of PET grids should not affect the potential rate of hydrolysis, as these coatings primarily function to protect the fibers from construction damage and provide dimensional stability during manufacture. During installation these coatings are pierced and provide an entry for the aqueous environment necessary to support the hydrolytic reaction.

***In the selection process for reinforcement applications, the use of high molecular weight ( $M_n > 25,000$ ) and low Carboxyl End Group (CEG  $< 30$ ) PET geosynthetics should be considered as most applicable. In alkaline soils ( $pH > 9$ ) where the geosynthetic may become saturated either because of its position below the water table or from rainfall infiltration, PET geosynthetics should be considered only if long-term immersion testing has been conducted in an aqueous media with the salts present in the proposed backfill soils.***

#### **4.5 EVALUATION OF INSTALLATION DAMAGE**

Significant loss of tensile strength has been attributed to geosynthetic damage during construction, in soil reinforcement applications. This damage is not time dependent since it occurs during the backfill placement and compaction operations. Installation damage could be a significant reduction factor in the determination of an allowable design strength for permanent applications. Assessment of the damage can be made by any of the following:

- Conducting field installation damage testing for each candidate geosynthetic and the proposed backfill in accordance with the procedures outlined in Chapter 5, Installation Damage Testing of this manual which are modifications to ASTM D5818 "*Construction Damage Practice for Obtaining Samples of Geosynthetics from a Test Section for Assessment of Installation Damage*".
- Use of estimates of the damage based on past testing, summarized below.
- Use of default values.

## a. Summary of Available Installation Damage Results

The level of damage for each geosynthetic is a variable and a function of:

- Weight and type of construction equipment used for fill spreading.
- Weight and type of compaction equipment.
- Weight and type of geosynthetic.
- Lift thickness of backfill material.
- Gradation and angularity of backfill.

Significant data have been generated to assess this important performance characteristic and provide quantitative data useful for design (Elias, 1989; Bush and Swan, 1986; Rainey and Barksdale, 1993). The range of strength loss reported for a wide range of geosynthetics varies between 10 and 77 percent. Current data strongly suggests that extreme damage is associated with coarse angular backfills spread in relatively thin lifts, compacted with heavy compaction equipment, and heavy construction traffic (e.g., loaded dump trucks) traversing the thin lift. The most important variables affecting the level of damage appear to be angularity, average backfill size used ( $D_{50}$ ), weight and type of construction equipment used for fill spreading and weight or thickness of the geosynthetic.

The results further suggest that:

- Slit film geotextiles are most subject to damage.
- Damage decreases substantially with increasing geotextile weight.
- Minimum geotextile weights of 8 oz/yd<sup>2</sup> (270 g/m<sup>2</sup>) should be considered based on construction damage survivability. This recommendation is consistent for gravelly sandy fills often used for MSE construction.

Extensive construction damage testing has been reported on HDPE geogrids (Bush and Swan, 1986). The variables examined were:

- Geogrid thickness.
- Compactive effort and lift thickness.
- Grain size distribution of backfill.

The results indicated the following:

- Damage and resulting loss of initial strength increased with decreasing geogrid thickness and weight.
- Damage and resulting loss of initial strength increased logarithmically with increasing maximum backfill size as denoted by the  $D_{50}$  size. Backfills with  $D_{50}$  sizes greater than 1 in. (25 mm) significantly increased the level of damage with correspondingly greater losses of strength.

- Varying compacted lift thicknesses between 6 and 9 in. (150 and 230 mm) had very little effect on the loss of strength.
- Varying compactive effort from four to more than eight passes with a heavy vibratory compactor had only a minor effect on the resulting damage and loss of strength.

The effects of installation damage on geosynthetic reinforcement strength should be determined for each product from results of full scale installation damage testing. Test results from damaged specimens should be compared to ASTM D4595 test results obtained from undamaged (i.e., not exposed to installation conditions) specimens taken from the same lot, and preferably the same roll, of material as the damaged specimens. The installation damage reduction factor for ultimate limit state design is then determined as follows:

$$RF_{ID} = \frac{T_{undam}}{T_{dam}} \quad (4-2)$$

where,  $T_{undam}$  is the lot specific tensile strength before installation, and  $T_{dam}$  is the lot specific tensile strength after subjection to installation. ***In no case should  $RF_{ID}$  be less than 1.10.***

To select an appropriate reduction factor for design, the project site installation conditions must be related to the installation test conditions. To relate the installation damage test conditions to the actual site conditions, one should primarily consider the backfill characteristics (mean particle size, potential for oversize material, particle angularity, and gradation), and to a lesser degree type of compaction equipment and initial backfill lift thickness over the geosynthetic, provided that the initial lift thickness is 6 in. (150 mm) or more.

Note that engineering judgment is required to characterize the site conditions, as there are many combinations of conditions which can occur. This creates some uncertainty when relating the test conditions to the project site conditions, if the test was not conducted at the project site. If the anticipated installation conditions are poorly defined or unknown and the installation damage data is not site specific, a conservative interpretation of the available installation damage data may be warranted. It is best to obtain installation damage test data at the actual project site so that relating test conditions to site conditions is unnecessary. Even in the case of testing at the project site, however, the measured average geosynthetic strength loss for a sample due to installation damage may vary within a given geosynthetic structure even though the installation conditions appear to be the same at each point, which may justify exhuming and testing at more than one sample location.

Based on published results of construction damage for a wide range of geosynthetic reinforcements, a preliminary estimate of the *range* of reduction factors (partial factors of safety) for construction damage has been prepared and is shown on Figure 4-12 (based on Elias, 1989; Bush and Swan, 1986; Rainey and Barksdale, 1993; Sandri et al., 1993). The partial factor of safety is defined as the inverse of the retained strength which is defined by wide-width tensile strength of the material before and after the construction damage trials as discussed above. The lower-bound damage level is generally associated with the products having the highest mass per unit area.

The wide range indicated in Figure 4-12 suggests that geosynthetic and backfill specific testing is necessary to evaluate the minimum installation damage strength loss. ***Figure 4-12 should not be used directly to obtain this reduction factor, but rather as a check of manufacturers supplied data.***

**b. Summary**

To account for installation damage losses of strength where full-scale product-specific testing is not available, Table 4-9 can be used with consideration of the project specified backfill characteristics.

**Table 4-9. Installation Damage Reduction Factors.**

<b>Reduction Factor, RF<sub>ID</sub></b>			
<b>No.</b>	<b>Geosynthetic</b>	<b>Type 1 Backfill Max. Size 4 in. (100 mm) D<sub>50</sub> about 1¼ in.(30 mm)</b>	<b>Type 2 Backfill Max. Size ¾ in. (20 mm) D<sub>50</sub> about #30 (0.7 mm)</b>
1	HDPE uniaxial geogrid	1.20 - 1.45	1.10 - 1.20
2	PP biaxial geogrid	1.20 - 1.45	1.10 - 1.20
3	PVC coated PET geogrid	1.30 - 1.85	1.10 - 1.30
4	Acrylic coated PET geogrid	1.30 - 2.05	1.20 - 1.40
5	Woven geotextiles (PP&PET)(1)	1.40 - 2.20	1.10 - 1.40
6	Non woven geotextiles (PP&PET)(1)	1.40 - 2.50	1.10 - 1.40
7	Slit film woven PP geotextile (1)	1.60 - 3.00	1.10 - 2.00
<sup>(1)</sup> Minimum weight 8 oz/yd <sup>2</sup> (270 g/m <sup>2</sup> )			

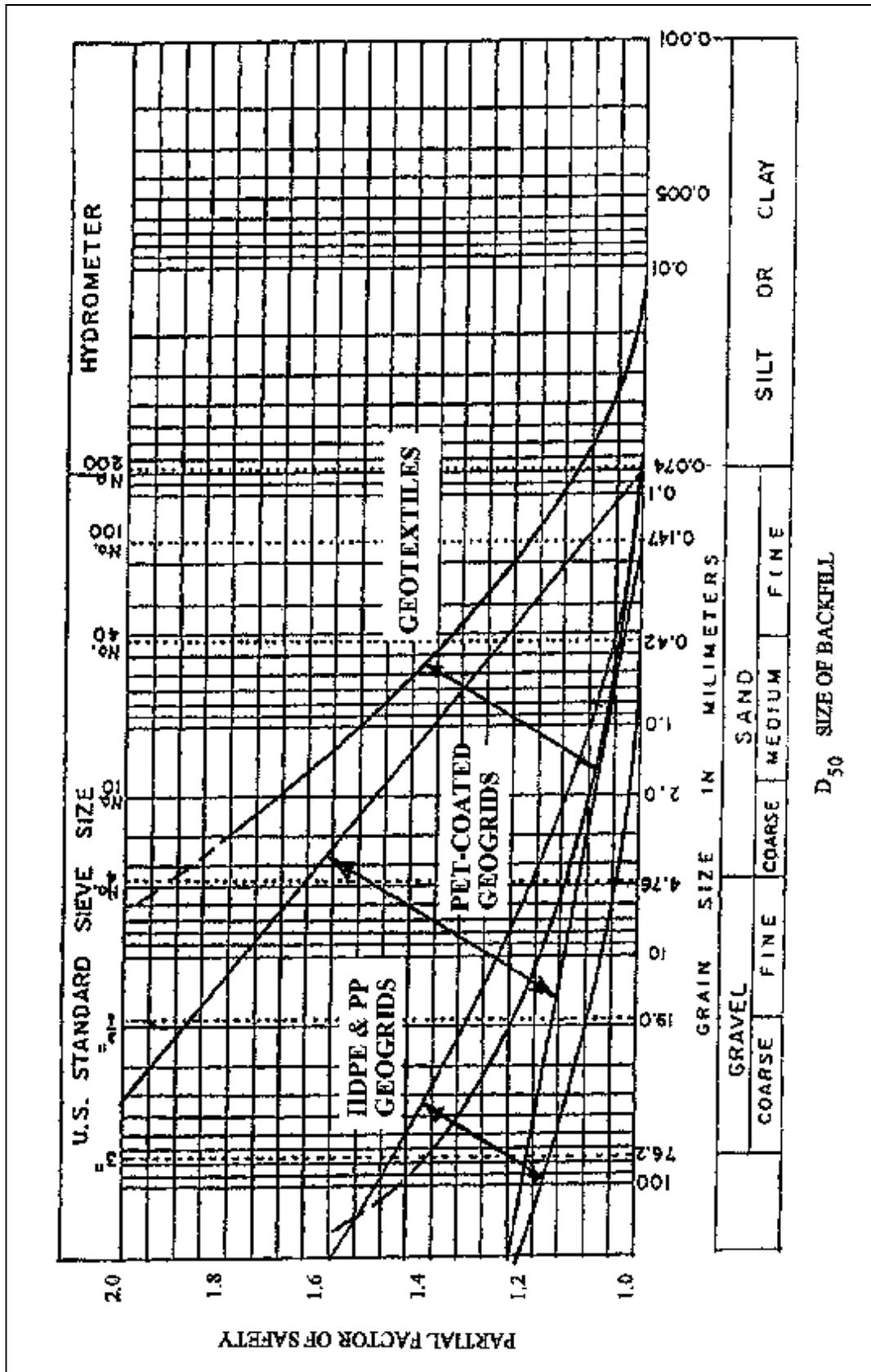


Figure 4-1. Installation damage vs. backfill size.

## 4.6 AGING REDUCTION FACTORS

Laboratory methods to predict aging strength loss for geosynthetics for in-ground use (reinforcement, stabilization) have been developed and are outlined in FHWA RD-97-144 (Elias et al. 1997b). Conceptually, there are two approaches to provide data usable in determining reduction factors against aging (degradation), namely:

- Excavation and retrieval of geosynthetics from construction projects with subsequent relevant testing to isolate the changes in mechanical properties attributable to a degradation mechanism; and
- Use of accelerated laboratory testing in selected environments to model the degradation phenomena and to measure the changes in mechanical properties with time.

The relevance and validity of both methods and recommendations for implementation are further discussed below:

### a. Field Retrievals

This apparently direct method is fraught with practical and technical difficulties, namely:

- Geosynthetics have been used extensively for MSEW and RSS applications for about 25 years. Therefore, the potentially available sample age is considerably less than the usually required 50 to 100 year life span for the product.
- The composition (polymer grade, additives) of products manufactured 15 to 25 years ago is quite different from those available today, and further it is likely to be quite different again, 10 to 25 years from now.
- The major change in mechanical properties occurring during construction is due to installation damage. Unless this was measured separately during the initial construction, it is almost impossible to separate initial damage from the long-term time-dependent damage attributable to aging degradation mechanisms.
- Archive samples or complete mechanical strength data and polymer composition index data must be available for comparison. This is seldom, if ever, the case.
- Sufficient samples must be recovered to obtain a statistically significant analysis.

Given the above, very few sites qualify for this type of investigation, and the reported data in the literature is indeed sparse on this subject. Alternately temperature accelerated laboratory testing provides a potential method of obtaining an order of magnitude for degradation losses, in a relatively short time (1 to 3 years), for some products.

## b. Accelerated Laboratory Testing

The predictive technique most widely used by industry for polymer degradation is based on a time-temperature superposition principle referred to as "Arrhenius modeling". It uses high-temperature incubation of the polymeric material, followed by laboratory testing to define physical or chemical properties in order to extrapolate the experimental behavior to a site-specific and lower temperature. (Koerner et al., 1992)

Aging losses for polyolefin products (PP and HDPE) are initiated by oxidation (availability of oxygen) and can be thermally accelerated. Therefore, aging tests are conducted in ovens at various temperatures and controlled oxygen content to model in-soil behavior, where oxygen concentration can vary from atmospheric (21 percent) to a fraction thereof. Note that these testing protocols are only applicable to products which do not exhibit crazing or cracking in their as manufactured state.

Aging losses for polyester (PET) products are principally initiated by hydrolysis (availability of water or moisture) and can be thermally accelerated. Therefore, aging tests are conducted in aqueous solutions at various temperatures, where the solution pH is controlled to model a given in-soil regime.

All rates of chemical degradation are temperature-sensitive according to the exponential term of the Arrhenius rate equation:

$$K = Ae^{-E/RT} \quad (4-3)$$

where:

K = the rate constant at temperature T (Kelvin),

A = the pre-exponential factor (often termed the collision factor),

E = the activation energy (J mol<sup>-1</sup>) and

R = the gas constant (= 8.3136 J mol<sup>-1</sup>K<sup>-1</sup>).

This equation, strictly speaking, may only be applied to homogeneous chemical reactions and degradation mechanisms of polymers below their melting point. Degradation reactions of most polymers may not be simple and are usually heterogenous, which would involve both simultaneous and competing reactions that themselves may give rise to secondary reactions. Therefore, standard methods of determining the pre-exponential factor, A, and more importantly, the activation energy, E, only yield apparent values which must be constant for the same material within the temperature range studied, to be useful, using this model. The validity of this general approach has been demonstrated in FHWA RD-97-144 (Elias et al., 1997a).

Testing temperatures must be below the melting or phase-transition temperatures for each polymer and as close as possible to the temperature under which the product will be ultimately used. In practical terms to achieve significant and measurable property changes, incubation temperatures of 105° to 160° F (41° to 71° C) are used for PET incubations, 85° to 175° F (29° to 79° C) for PP, and 160° to 195° F (71° to 91° C) for HDPE. At these temperatures, testing times are on the order of 2 to 4 years. Typical laboratory equipment is shown on Figure 4-13.

The usual method of analyzing laboratory incubation data is to plot the log of a Reaction Rate or the inverse of a Reaction Time for a preselected property against the inverse of temperature as shown on Figure 4-14. For geosynthetic durability studies a typical reaction rate would be the degradation rate in tensile strength with time at a given laboratory elevated temperature. From this type of data plot, reactions at lower temperatures can be predicted from higher-temperature experimental data.

A wide range of physical and chemical properties may be extrapolated using this technique as extensively discussed by Koerner et al. (1992), provided that:

- The logarithm of the reaction rate vs. the inverse of temperature is linear.
- The investigative temperatures are spaced reasonably apart and are below any transition-phase temperatures.

In oxidation studies where the logarithm of the reaction rate vs. the inverse of temperature is not linear or where no strength loss is measured initially (induction period) at a given temperature, similar analysis techniques consistent with the well developed basic autoxidation scheme (BAS), as outlined in FHWA RD-97-144 (Elias et al., 1997a), must be applied (Kelen, 1982).

Of primary interest to design engineers are mechanical properties such as tensile strength and elongation. For example, if the time to reduce tensile strength by 50 percent of its unaged value is required for a site ambient temperature (e.g., 68° F {20° C}), multiple samples of the product are aged at three elevated temperatures (e.g., 122°, 149°, 176° F {50°, 65°, 80° C}) as shown on Figure 4-14, High Temperature.

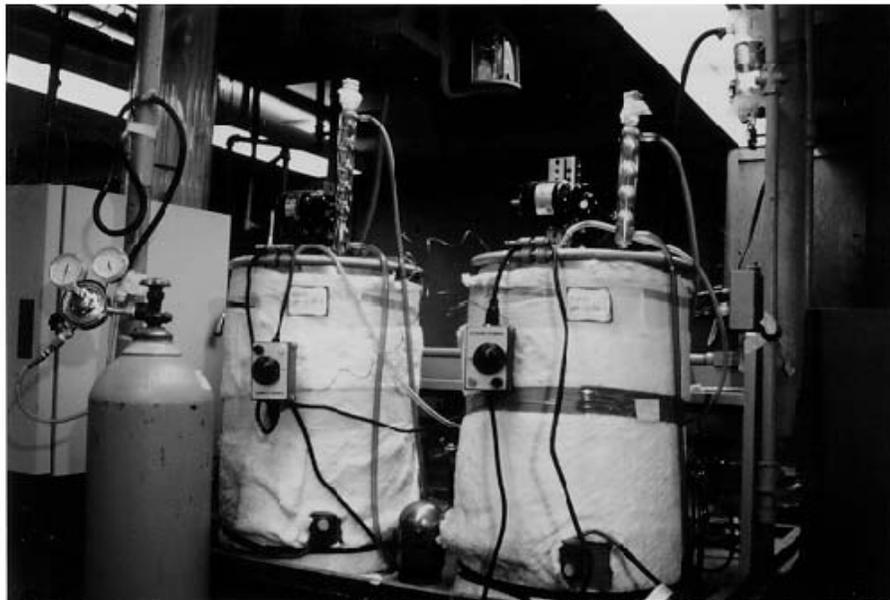


Figure 4-2. Laboratory aging equipment setup.

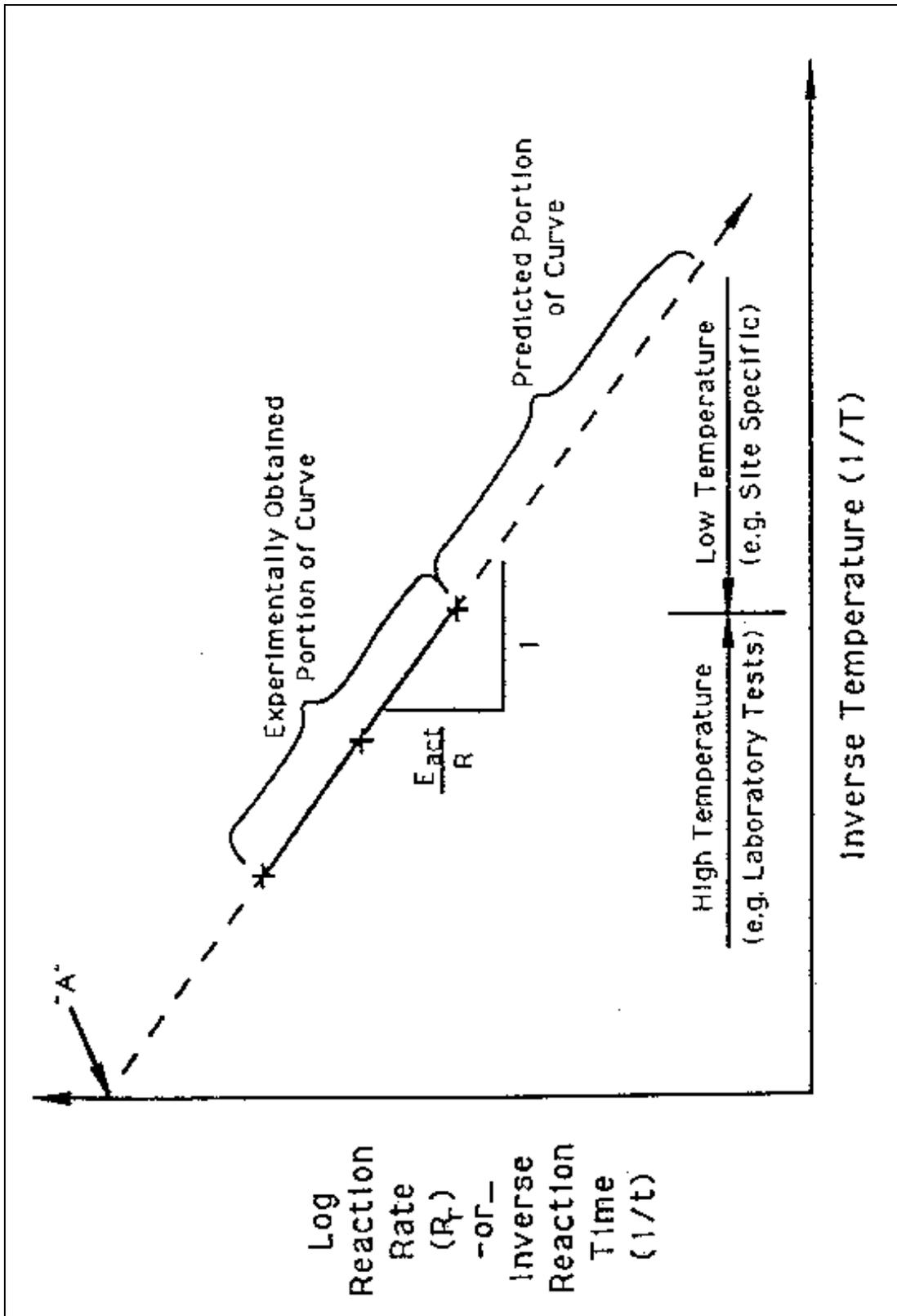


Figure 4-3. Generalized Arrhenius plot used for low-temperature predictions form high-temperature experimental data.

Based on the tensile strength of samples retrieved at various time intervals from the ovens, a linear regression analysis of retained strength versus exposure time is used to compute the rate constant of tensile strength degradation for each temperature tested. The natural logarithm of degradation rate at each temperature is plotted versus the reciprocal of temperature (Kelvin) to obtain a relationship which then can be extrapolated to any other lower temperature such as ambient. This process, known as Arrhenius modeling, is illustrated in Figure 4-14.

It should be noted that for stabilized polyolefin geosynthetics, the initial stages of oven aging at most elevated temperatures will yield no strength loss. This *induction* period defined as aging time with no strength loss is a direct measure of antioxidant effectiveness. After all of the antioxidant is consumed, strength loss of the now unstabilized polyolefin will begin.

The strength retained,  $T_D$ , at the end of the desired design life at the site temperature can then be calculated by considering no strength loss during the induction period, followed by a loss calculated directly from the linear Arrhenius equation. Once the tensile strength at a given design life has been estimated from the test data,  $RF_D$  is determined as follows:

$$RF_D = \frac{T_{ultlot}}{T_D} \quad (4-4)$$

where:

$T_{ultlot}$  = the average lot specific ultimate tensile strength for the unaged lot of material used for the durability testing, and

$T_D$  = the extrapolated lot specific tensile strength after degradation based on the laboratory aging tests.

***In no case should  $RF_D$  be less than 1.1.***

For better understanding of the chemical degradation process, chemical properties may also be extrapolated. Research is presently under way using these techniques to completely define testing methods and protocols, and develop design data to yield lifetime predictions for certain preselected, possibly typical geosynthetics.

### c. Summary

The outlined framework of procedures summarizes the present state of the art in developing credible long-term degradation strength losses for polymeric reinforcement. Interpretation of the test data is still somewhat subjective, and subject to revision based on future research efforts. Table 4-10 summarizes specific studies needed.

**Table 4-10. Summary of Product-Specific Studies Needed to Evaluate the Durability of Geosynthetic Reinforcement.**

<b>Environmental Factor</b>	<b>Polymer Resin Type for Which Studies are Needed</b>	<b>Geosynthetic Reinforcement Studies Needed</b>
Mechanical Damage	All geosynthetic reinforcements	Full Scale Installation Damage Tests
Stress Levels	All geosynthetic reinforcements	Creep tests
Stress Levels	All geosynthetic reinforcements	Stress crack evaluation: long-term stress rupture tests at ambient and elevated temperatures
Chemical Exposure (oxygen)	Polyolefins (e.g., polypropylenes and polyethylenes)	Long-term oxidation studies
Chemical Exposure (water, pH)	Polyesters and polyamides (and any polymer coatings present such as PVC or acrylic)	Long-term hydrolysis studies and short-term effects due to plasticization
Other potentially reactive chemicals present in the in-situ environment	Resin types which are potentially susceptible depend on the specific chemical present	Other specialized chemical tests
Microbiological Attack	Most resins used for geosynthetic reinforcements are generally not susceptible to this but should at least be checked	ASTM D3083

**Table 4-11. Aging Reduction Factors, PET.**

<b>No.</b>	<b>Product*</b>	<b>Reduction factor, RF<sub>D</sub></b>	
		<b>5 ≤ pH ≤ 8</b>	<b>3 ≤ pH ≤ 5 8 ≤ pH ≤ 9</b>
1	Geotextiles M <sub>n</sub> < 20,000, 40 < CEG < 50	1.6	2.0
2	Coated geogrids M <sub>n</sub> > 25,000, CEG < 30	1.15	1.3
* Use of materials outside the indicated pH or molecular property range requires specific product testing.			

With respect to aging degradation, current research results suggest the following:

#### Polyester geosynthetics

PET geosynthetics are recommended for use in environments characterized by  $3 < \text{pH} < 9$ , only. The following reduction factors for PET aging ( $\text{RF}_D$ ) are presently indicated for a 100-year design life in the absence of product specific testing:

#### Polyolefin geosynthetics

To mitigate thermal and oxidative degradative processes, polyolefin products are stabilized by the addition of antioxidants for both processing stability and long term functional stability. These antioxidant packages are proprietary to each manufacturer and their type, quantity and effectiveness varies. Without residual antioxidant protection (after processing), PPs are vulnerable to oxidation and significant strength loss within a projected 75 to 100 year design life at 68° F (20° C). Current data suggests that unstabilized PP has a half-life of less than 50 years. Therefore the anticipated functional life of a PP geosynthetic is to a great extent a function of the type and remaining antioxidant levels, and the rate of subsequent antioxidant consumption. Antioxidant consumption is related to the oxygen content in the ground, which is only slightly less than atmospheric.

Heat aging protocols for PP products, at full or reduced atmospheric oxygen, with subsequent numerical analysis are available for PP products which exhibit no initial cracks or crazes in their as manufactured state, typically monofilaments (Elias et al., 1997a). For PP products with initial crazes or cracks, typically tape products, or HDPE, heat aging testing protocols may change the nature of the product and therefore may lead to erroneous results. Alternate testing protocols using oxygen pressure as a time accelerator have also been developed (ASTM D5885).

Since each product has a unique and proprietary blend of antioxidants, product specific testing is required to determine the effective life span of protection at the in-ground oxygen content. Limited data suggests that certain antioxidants are effective for up to 100 years in maintaining strength for in-ground use.

A rough measure of antioxidant effectiveness for PP products formulated without significant carbon black is resistance to UV degradation measured in accordance with ASTM D4355. A retained strength of 90% at 500 hours or more generally indicates an effective antioxidant blend and potentially a reduction factor as low as 1.1 at 68° F (20° C) and 100 years. For HDPE geogrids presently available (Tensar UX Series), current research data indicates a Reduction Factor of 1.1 for use at 68° F (20° C) and 100 years.

Note that for products meeting the minimum requirements of Table 4-5 in *"Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines"* a total default reduction factor RF of 7 has been recommended. This total reduction factor includes a reduction factor for creep  $RF_{CR}$  not discussed in Chapter 4.

#### **4.7 USE OF DURABILITY DATA FROM "SIMILAR" PRODUCTS**

Long-term chemical/biological durability data obtained from tests performed on older product lines, or other products within the same product line, may be applied to new product lines, or a similar product within the same product line, if one or both of the following conditions are met:

- The chemical and physical characteristics of tested products and proposed products are shown to be similar. Research data, though not necessarily developed by the product manufacturer, should be provided which shows that the minor differences between the tested and the untested products will result in equal or greater chemical/biological degradation resistance for the untested products.
- A limited testing program is conducted on the new or similar product in question and compared with the results of the previously conducted full testing program.

For polyolefins, similarity could be judged based on molecular weight and structure of the main polymer (i.e., is the polymer branched or crosslinked, is it a homopolymer or a blend, percent crystallinity, etc.), percentage of material reprocessed, tenacity of the fibers and processing history, and polymer additives used (i.e., type and quantity of antioxidants or other additives used). For polyesters and polyamides, similarity could be judged based on molecular weight or intrinsic viscosity of the main polymer, carboxyl end group content, percent crystallinity, or other molecular structure variables, tenacity of the fibers and processing history, percentage of material reprocessed or recycled, and polymer additives used (e.g., pigments, etc.). The untested products should also have a similar macrostructure (i.e., woven, nonwoven, extruded grid, yarn structure, etc.), relative to the tested products.

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## **CHAPTER 5**

### **MONITORING METHODS, GEOSYNTHETIC REINFORCEMENTS**

The principal degradation mechanisms for geosynthetics discussed in Chapter 4 fall in the following two categories:

- Construction damage degradation that is a short-term phenomenon.
- Polymer degradation due to oxidation, hydrolysis, or UV radiation that is a long-term phenomena.

For MSE walls or slopes construction, the results of these degradative mechanisms are a loss of tensile strength and potential change of elongation properties. Monitoring schemes designed to assess these mechanisms are, therefore, quite different in nature and duration, although in all cases they would principally assess the change in tensile and elongation properties.

Short-term construction damage determinations lend themselves to model field testing programs with retrievals tailored either to evaluate a single geosynthetic with one or several backfills or multiple geosynthetics with a single or with multiple fills.

Polymer degradation monitoring is a long-term endeavor characterized by multiple retrievals spaced over a decade or two. The protocols for each are summarized below.

#### **5.1 INSTALLATION DAMAGE TESTING**

Where previous knowledge does not exist to assess the strength loss caused by fill construction and compaction on a specific geosynthetic and reinforced fill, field testing is required. The use of the ASTM D5818 *Standard Practice for Exposure and Retrieval of Sample to Evaluate Installation Damage of Geosynthetics* methodology as modified/expanded by the following procedure, is recommended:

1. Preliminary laboratory characterization testing of geosynthetics focused on geosynthetic strength properties, using the wide-width tensile strength test (ASTM D4595) or multi-rib geogrid tensile strength test (ASMT D6637, Method B), in both directions. Single rib strength, junction strength, grab strength, puncture strength, tear strength and burst strength may also be performed to relate performance to typically available index tests. The minimum number of samples should be in accordance with the requirements of

ASTM D4595 and ASTM D6637 to ensure that the coefficient of variation is less than 5 percent from true average values. A minimum of five samples is required.

2. Field placement of geosynthetics as follows:

- Place and compact 12 in. (300 mm) of soil (same as soil used to cover the geosynthetic) on a flat, level, relatively incompressible subgrade.
- Place the geosynthetic with the machine direction perpendicular to the face of a wall or embankment. Geosynthetics should be pulled taut with no wrinkles or folds. Pinning at the corners should be considered to maintain the position. Each adjacent sheet of geosynthetic should be overlapped a minimum of 6 in. (150 mm) with the upper sheet placed in the direction of soil placement. A total sample size of 16 by 10 ft (5 by 3 m) should be used as a minimum.
- Place 8- to 10-inch (200-to 250-mm) compacted thickness of backfill using a front-end loader or a D-4 to D-7 dozer.
- Compact the backfill using a 10,000 to 30,000-lb (4500 to 13,600 kg) vibratory smooth-drum roller with a set number of passes. The minimum number of passes should insure compaction equal to at least 95 percent of (Modified Proctor) AASHTO T 180 density. As a maximum, 10 passes are recommended.
- Traverse the complete area with construction equipment used to place subsequent lifts (e.g., loaded dump truck). The minimum number of passes should follow anticipated construction conditions (e.g., two passes for end dump placement and five passes for pass through traffic).
- Carefully remove the backfill by hand and document any observable geosynthetic damage, including photographs of entire area and visual survey of puncture holes per square yard (m).
- Select specimens of the geosynthetic for testing. Specimen selection should be guided by the placement of a primary template 24 by 32 in. (600 by 800 mm) shown in Figure 5-1 on the center of the sample.

Cut and number twelve adjacent specimens 8 in. (200 mm) by nominally 8 in. (200 mm) as shown in Figure 5-1. Specimens having areas marked with damage from the retrieval process, if any, should not be tested. A minimum of nine specimens with consecutive numbers should be initially tested. For high-strength polyester geotextiles and geogrids, the length of the specimens should be increased to accommodate the roller grips, which require up to 5 ft (1.5 m) long test specimens. For geogrids, the specimen size should be such to contain at least three ribs in the machine direction and five ribs in the cross direction. Additional cross direction ribs may be necessary to accommodate roller clamps.

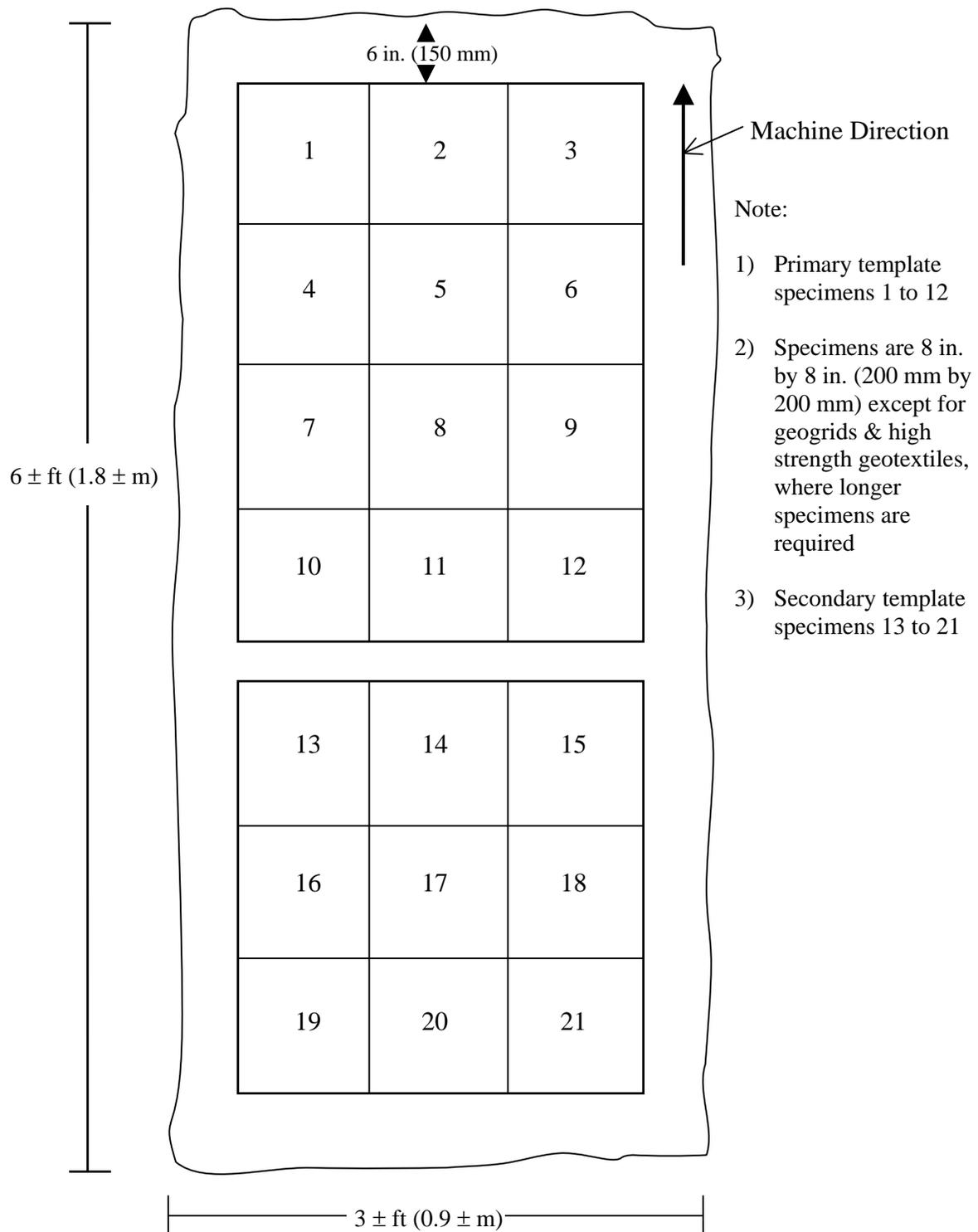


Figure 5-1. Scheme for sampling test specimens.

## Evaluation

Wide-width or multi-rib tensile strength obtained from these 9 primary tests should be analyzed to determine the coefficient of variation as outlined in ASTM D4595 or ASTM D6637 with an allowable 5 percent variation from true average values at a 95 percent confidence level.

If the coefficient of variation for the 9 primary samples is greater than 5 percent, the required number of samples should be recomputed using the one sided student t variation (Table 1, ASTM D4959). If more than 9 specimens are required, then additional specimens as available should be secured from a secondary template location also shown on Figure 5-1. No more than a total of 18 specimens per sample should be tested for wide-width or multi-rib strength.

Prior to wide-width or multi-rib tensile testing, mass-per-unit area should be determined from specimens marked 9 through 12 in accordance with ASTM D5261.

- If other index tests are desired or warranted for correlations to performance, then additional templates should be placed centered on the machine direction and samples obtained. These additional tests should be performed in accordance with the following standards:
  - For Geotextiles
    - Grab Tensile Strength – ASTM D4632
    - Puncture Strength – ASTM D6241
    - Trapezoidal Tear Strength – ASTM D4533
  - For Geogrids
    - Single Rib Strength – ASTM D6637
    - Junction Strength – GRI:GG2
- Evaluate the retained strength on the basis of the average results obtained from the wide-width or multi-rib tensile test.
- Determine the reduction factor (partial factor of safety) for construction damage by dividing the initial tensile strength by the retained tensile strength.

## 5.2 POLYMER DEGRADATION MONITORING

A protocol for exhuming and testing a geosynthetic for resistance to long-term degradation is characterized by multiple retrievals over a long period of time and identification of the soil/water environment in which the product is placed.

The essential steps are as follows:

- Identify site conditions.
- Select location and depths for future retrievals.
- Test control samples.
- Perform construction damage testing immediately after completion of construction.
- Develop subsequent retrieval schedule.
- Successive retrieval of samples.
- Test retrieved samples for both mechanical and chemical properties.
- Evaluate and extrapolate data.

It should be recognized that each retrieval requires a significant physical sample size and therefore the design of a test section must consider accessibility after the facility has entered its intended use. This task may not be easy to accomplish, especially where retrieved samples are significantly below the finished grade, which may require braced excavations to reach intended locations.

The developed protocol recommended for use is as follows:

### a. Identification of Site Conditions and Structure Description

Each site should be described as to the functional use of the geosynthetic mapped for retrieval. Such description shall include a location plan indicating the location, plan, and elevation of the geosynthetic in the structure; and an estimate of the loading conditions, including piezometric surfaces; design service life; and any other pertinent observations as to the functionality of the structure after construction. Available performance data such as deflection measurements and/or strain gauge data should be included where available. The information should be summarized on a Project Information Sheet.

Retrieval intervals at the approximate same location should be planned at about 5-to-7 year intervals for a minimum of four retrievals or 1/3 the expected life of the facility.

## b. Testing of Control Samples and Retrieved Samples

Testing methods for all required physical, endurance and chemical tests *Applications* (Bright, 1993) have been developed under FHWA *Durability of Geosynthetics for Highway* project. They are described as follows:

- The principal test method to characterize residual strength should be the wide width tensile test performed in accordance with ASTM D4595 or ASTM D6637 in the machine direction. Where strength in previous retrievals has been characterized by the grab tensile test performed in accordance with ASTM D4632, it should be considered the principal test for that specific site.
- Physical property tests on retrieved and control product specimens
  - Mass-per-unit area should be measured for geotextiles in accordance with ASTM D3776, Option C, using at least one specimen retrieved. Thickness may be measured on polyolefin grid products in accordance with ASTM D5199.
  - Density/Specific Gravity should be measured in accordance with ASTM D792.
  - An assessment of the number of holes per unit area of retrieved specimen for geotextiles should be made by placing the specimen on a light table and counting the number of holes visible. The level of abrasion, if visible, should be noted. For geogrids rib cuts, abrasions and percent of ribs severed across the specimen width should be noted. This assessment is subjective and no standard exists. Photographs should be taken and special conditions shown in photographs noted.
- Chemical tests on product specimens

For PET (polyester products)

- The principal tests performed on geotextiles and fibers of coated geogrid products are molecular weight determinations (Mn) using gel permeation chromatography and determination of carboxyl end groups (CEG number) using ASTM D7409. A minimum of three tests should be performed.
- Where product control data for molecular weight has been expressed in terms of intrinsic viscosity or where reagents and test temperatures are known, they may be substituted from molecular weight (Mn) determinations. Intrinsic viscosity may be determined in accordance with ASTM D4603.

- Thermogravimetric Analysis (TGA) in general accordance with ASTM E1131, to determine transition temperatures and any changes from archive samples.
- Scanning Electron Microscopy (SEM) at a magnification of at least 1000x, to determine any fiber diameter changes from archive samples.

For polyolefin products (PP and HDPE), the following tests are recommended:

- Melt Flow index, ASTM D1238 to establish changes from archive samples.
- Differential Scanning Calorimetry (DSC) to obtain oxidation induction time (OIT). Three specimens should be tested per retrieval sample. Methods outlined in FHWA *"Durability of Geosynthetics for Highway Applications"* should be used.
- Thermogravimetric Analysis (TGA) to obtain transition temperatures. Three specimens should be tested per retrieval sample ASTM E1131.
- High Performance Liquid Chromatography (HPLC) to determine the level of antioxidants present. Three specimens should be tested using industry standards. *This test should only be performed if the type of antioxidant in the product is known.*
- Scanning Electron Microscopy (SEM) at a magnification of at least 1000x to determine the presence of longitudinal or circumferential cracking.

- Performance Tests

In isolation creep tests in general accordance with ASTM D5262 for a 1,000 hour maximum duration may be conducted on selected retrieval specimens only if control data for this particular product is available for comparison or if the site will be available for future retrievals and the functional use of the geosynthetic on the project is for reinforcement/stabilization.

Where conducted, two specimens shall be tested at each of three load levels. The first load level shall be approximately equal to the specimen creep load limit as defined by the virgin creep limit (as a fraction of ultimate strength) multiplied by the ultimate tensile strength retained after damage; the second load level at the design load used for the project but no higher than the estimated limit state reinforcement tensile load as defined in Article 11.10.6.4 AASHTO (2007); and the third at a high load level approximately equal to 1.5 times the virgin creep limit of the virgin specimen.

### c. Retrieval Methods

Sampling must be performed carefully in order to avoid damage to the product. The excavation operation may begin with power equipment, but such excavation methods must terminate within 6 to 8 in. (0.15 to 0.2 m) of the geosynthetic. Excavation must then continue manually with a hand trowel, hand rake, and broom to remove the remaining soil gently over a minimum 3 by 6 ft (1 by 2 m) surface area. It is recommended that the 6-ft (2-m) length be parallel to the machine direction. High strength polyester geotextile and woven geogrid products requiring roller grips for testing will necessitate a longer total sample (e.g., 10 to 15 ft). Damage during this excavation phase should be noted and marked on the product. The site should be photographed and the visual appearance of the product be noted with emphasis on existing holes, tears, folds, root penetration, presence of water, and uniformity of backfill.

The sample is then cut along the sides, lifted carefully, excess soil shaken off and placed in black polyethylene bags, sealed and marked with appropriate identification. The sample must be identified with respect to machine and cross direction which must be noted on the location sketch.

Concurrently, a soil sample must be secured adjacent to the product sample retrieval. Separate samples are required if the soils above and below differ visually in composition.

#### Sample preparation for testing

- Total sample

Prior to specimen selection, the retrieved sample should be prepared by removing any soil by gently shaking the sample.

The full sample should then be hand washed gently under tap water, removing only any adhering surface soil cake that had formed. No attempt should be made to remove any soil that does not easily wash away.

The washed sample should then be laid out horizontally in a darkened room and allowed to dry under ambient temperature.

- Specimens for mechanical testing

To avoid bias, selection of specimens from the recovered product sample should be made in accordance with the following structured random process.

A primary template 24 by 32 in. (600 by 800 mm) should be laid out in the machine direction on the recovered sample by locating it 6 in. (150 mm) from the top edge and centering it along the 3 ft (0.9 m) recovered width. The primary template length should be increased for high strength polyester geotextiles and geogrids tested with roller grips, which require up to 5 ft (1.5 m) long test specimens (in the machine direction).

Twelve adjacent specimens 8 by 8 in. (200 by 200 mm) should be cut and numbered in accordance with Figure 5-1. Specimens having areas marked with damage from the retrieval process should not be tested. A minimum of nine specimens with consecutive marked numbers should be initially tested. For high-strength polyester geogrids and geotextiles, the length should be increased to accommodate the roller-grip requirement. For geogrids, the specimen size should contain a minimum of three ribs in the machine direction and five ribs in the cross direction. Additional cross direction ribs may be necessary to accommodate roller clamps.

Prior to wide-width tensile testing, mass-per-unit area should be determined from specimens marked 9 through 12 in accordance with ASTM D5261. Specimens tested for strength should be reserved for chemical testing, utilizing principally those portions of the product held in the jaws of the testing apparatus.

Wide-width tensile strength test (ASTM D4595 or ASTM D6637, Method B) data obtained from these nine primary tests should be analyzed to determine the coefficient of variation as outlined in ASTM D4595 or ASTM D6637, with an allowable 5 percent variation from true average values. Based on the obtained actual coefficient of variation, the required number of specimens should be recomputed. If greater than nine, additional specimens as available should be secured from a secondary template location also shown on Figure 5-1. No more than a total of 18 specimens per recovered sample should be tested for wide width strength.

- Specimens for chemical testing

Chemical testing of specimens not sufficiently cleaned of colloidal soil particles may be ultrasonically cleaned using water and/or a mild wetting agent at room temperature. A 2 percent solution of Micro Cleaner has been successfully used in 2-to 5-minute cycles in a stainless steel basket.

#### **d. Soil Tests**

The ambient soil regime shall be characterized by performing the following tests on one sample of the representative soils (10 lb {15 kg}) obtained above or below the retrieved geosynthetic specimen:

- Grain size distribution (ASTM D854)
- Atterberg Limits for fine grained soils (ASTM D4318)
- pH (AASHTO T-289)
- Electrical conductivity or resistivity (AASHTO T-288)
- Organic content (AASHTO T-267)
- Transition metals (EPA SW 6010)
- Chloride, sulfate, carbonate (ASTM D4327)
- Calcium, sodium (EPA, SW 6010)

### **5.3 EVALUATION OF GEOSYNTHETIC DEGRADATION MONITORING DATA**

The principal method of evaluating aging degradation is to determine loss of tensile strength.

The short-term loss is totally due to construction damage. Therefore, potential aging polymer degradation is measured from the reduced average strength obtained after a new baseline initial strength is calculated by subtracting the construction damage from the initial control sample strength data.

Subsequent retrievals would therefore indicate the rate of aging degradation, which can be extrapolated to the design life of the product in a straight-line fashion. The measured chemical-property data at each retrieval may in some cases provide a confirmation of the measured rates and an indication that the process is linear or autocatalytic. The significance of changes in chemical properties varies with each base polymer additive combination and is under current research. Some preliminary conclusions can be drawn at this time as follows:

#### a. Polyester (PET)

The principal long-term, in-ground degradation mechanism of PET in acidic, neutral and slightly alkaline conditions characterized by pHs greater than three and less than nine, is hydrolysis. As a result of hydrolysis, molecular weight (Mn) decreases apparently almost linearly with loss of strength, to at least a point equal to a 50 percent strength loss. Correspondingly, CEG number increases although its relationship to molecular weight loss has not been established to date, but could be linear as well, as the two properties are directly related. Loss of strength as evidenced by tensile tests in successive retrievals can therefore be confirmed by an equal percentage decrease in molecular weight (Mn) up to a 50 percent strength decrease.

For degradation in more strongly alkaline media, an additional mechanism may cause losses of strength termed "outer hydrolysis" which is a physical dissolution of polyester material at the outer surface of each fiber. The effect of such loss of fiber mass on its circumference can only be measured by comparing fiber diameter under an SEM or more indirectly inferred by weight-per-unit-area measurements of the retrieved sample. This latter procedure, although simpler, is fraught with experimental problems, as it may be extremely difficult to completely clean the retrieved samples of embedded colloidal soil particles, especially for non-woven geotextile reinforcements.

Figure 5-2 shows the diameter change of a PET geotextile fiber subjected to laboratory immersion on a strongly alkaline solution. The geosynthetic is characterized by an initial number molecular weight (Mn) of 19,000 and a Carboxyl End Group of 47.

#### b. Polyolefins (PP and HDPE )

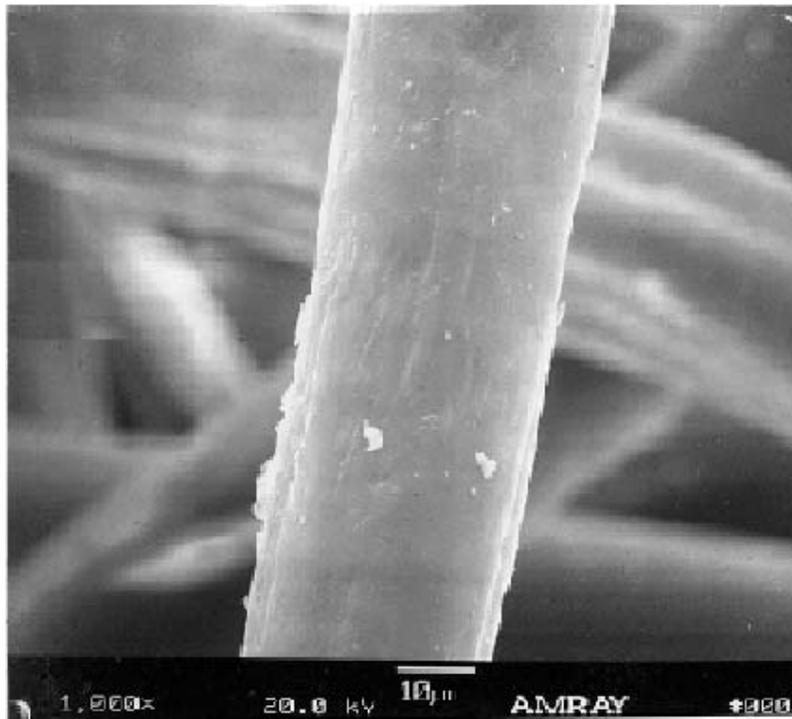
The principal long-term, in-ground degradation mechanism of polyolefin polymers is oxidation. The major resistance to oxidation is impacted by the antioxidant package included during the manufacturing process. It would therefore appear that confirmation of strength losses measured (due to aging) *can only be obtained from successive* retrievals and may be confirmed by measurable loss of antioxidants, measurable loss of their effectiveness, or both.

If the type of antioxidant is known, the level present at anytime can be measured by High Performance Liquid Chromatography (HPLC) with a  $\pm 20$  percent accuracy. The Oxidation Induction Time (OIT) is only a measure of an antioxidant's relative effectiveness within each product as a thermal or oxidative stabilizer and therefore is a poor indicator.

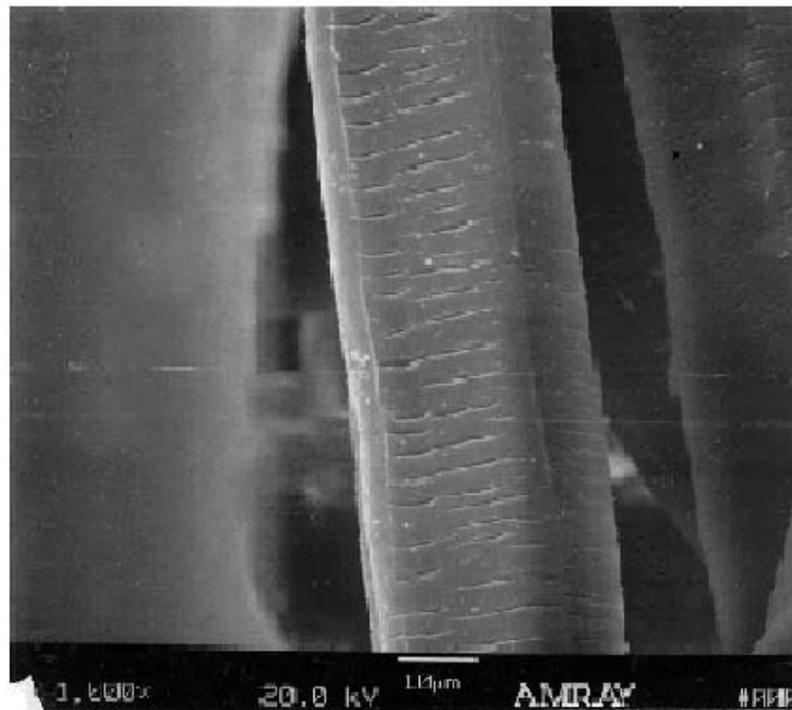
Based on our present knowledge, the decrease of the antioxidant level or OIT time is not evidence of strength loss, but can serve as a confirmation of either leaching or consumption of antioxidants. The complete loss of antioxidants as evidenced by HPLC, or decreases of OIT time greater than 90 percent, indicates active oxidation with strength loss accelerating.

A simpler qualitative determination of significant past oxidation activity is to examine archive and aged samples under an SEM. Oxidation activity may be reflected in circumferential cracking of the fibers as shown on Figure 5-3.

Note that some PP geotextiles, chiefly tape or staple products may exhibit initial longitudinal or transverse cracks or crazing in their virgin as manufactured state. For these products an increase in crack length or frequency is an indicator of oxidation activity.



**a) Virgin fibers**



**b) Oven aged fiber with cracking**

Figure 5-3. Scanning Electron Microscopy (SEM), polypropylene fibers.

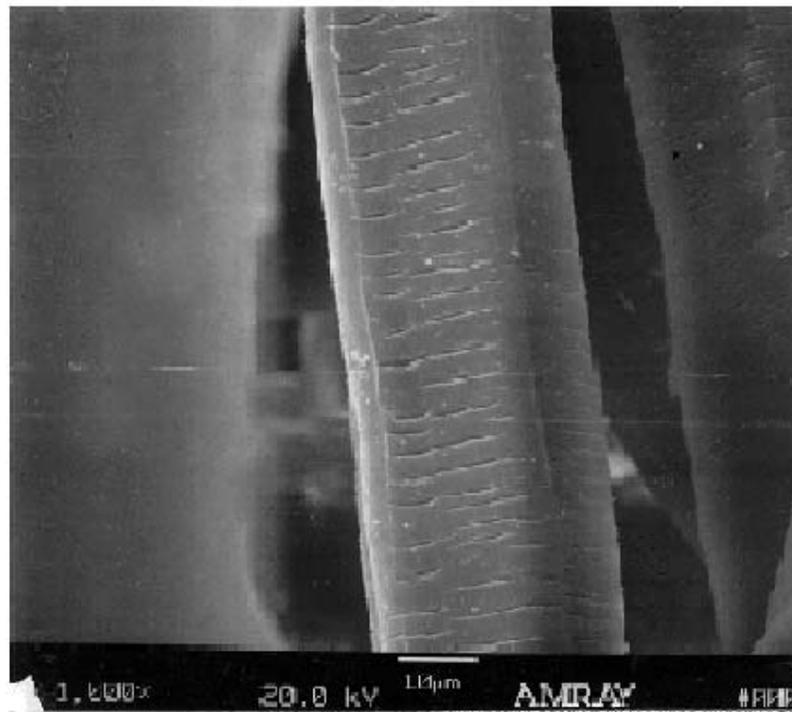
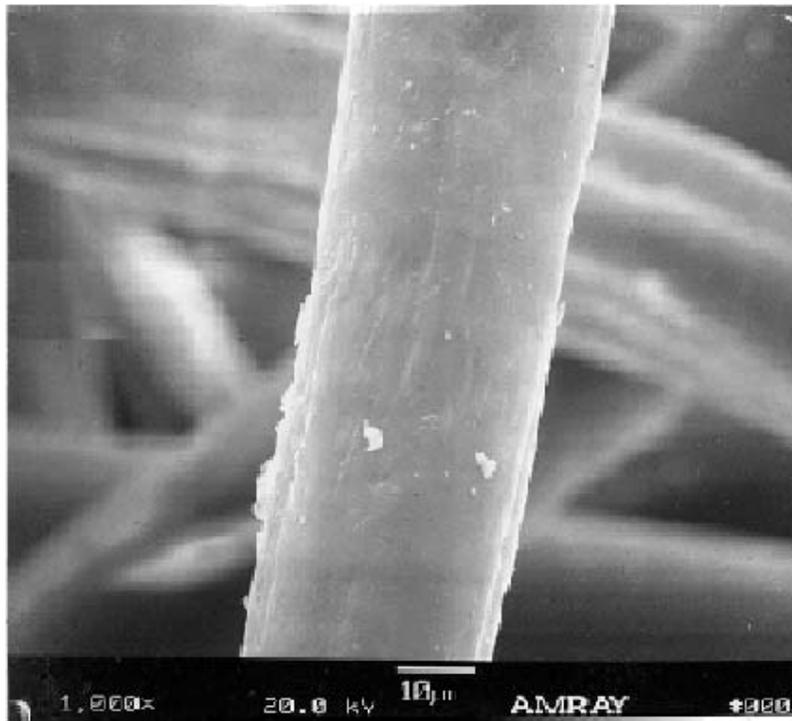


Figure 5-3. Scanning Electron Microscopy (SEM), polypropylene fibers.

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