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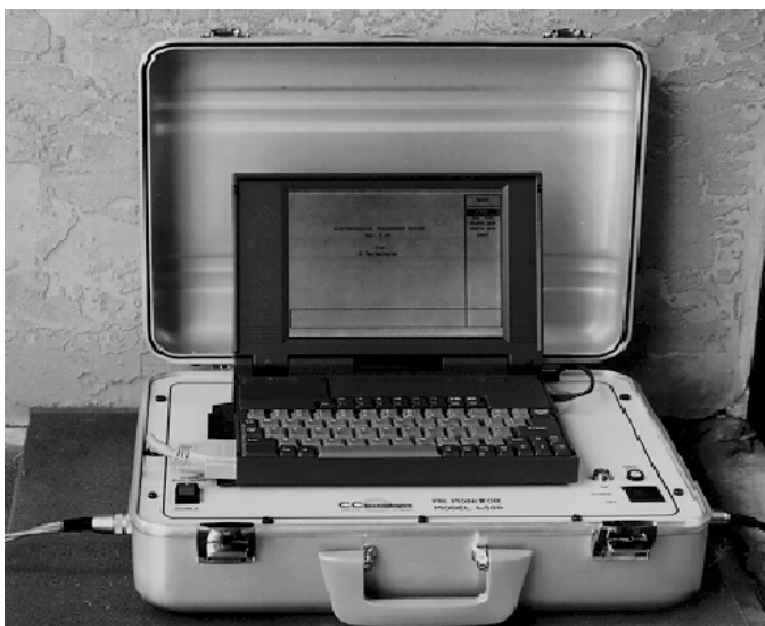
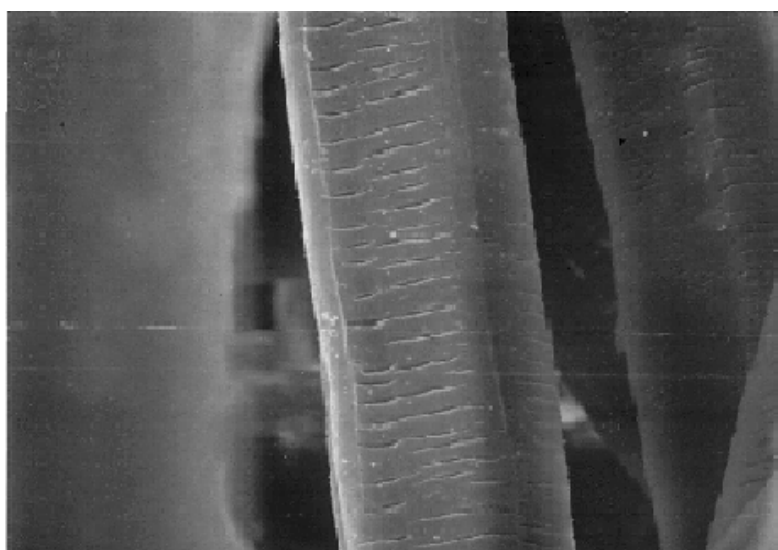


U.S. Department
of Transportation
**Federal Highway
Administration**

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MECHANICALLY STABILIZED EARTH WALLS AND REINFORCED SOIL SLOPES

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



National Highway Institute
Office of Bridge Technology

SEPTEMBER 2000

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16. ABSTRACT This manual is intended to provide 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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SI CONVERSION FACTORS				
APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
AREA				
mm ²	square millimeters	0.0016	square inches	in ²
m ²	square meters	10.764	square feet	ft ²
m ²	square meters	1.195	square yards	yd ²
ha	hectares	2.47	acres	ac
km ²	square kilometers	0.386	square miles	mi ²
VOLUME				
ml	milliliters	0.034	fluid ounces	fl oz
l	liters	0.264	gallons	gal
m ³	cubic meters	35.71	cubic feet	ft ³
m ³	cubic meters	1.307	cubic yards	yd ³
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
TEMPERATURE				
°C	Celsius	1.8 C + 32	Fahrenheit	°F
WEIGHT DENSITY				
kN/m ³	kilonewton / cubic meter	6.36	poundforce / cubic foot	pcf
FORCE and PRESSURE or STRESS				
N	newtons	0.225	poundforce	lbf
kN	kilonewtons	225	poundforce	lbf
kPa	kilopascals	0.145	poundforce / square inch	psi
kPa	kilopascals	20.9	poundforce / square foot	psf

PREFACE

Engineers and specialty material suppliers have been designing reinforced soil structures for the past 25 years. During the last decade significant improvements have been made to design methods and in the understanding of factors affecting the durability of reinforcements.

In order to take advantage of these new developments, the FHWA has developed this manual. The primary purpose of this manual is to serve as the FHWA standard reference for highway projects involving reinforced soil structures.

A second 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 on going research projects:

! Durability/Corrosion of Soil Reinforced Structures; V. Elias, FHWA RD-89-186

! Testing Protocols for Oxidation and Hydrolysis of Geosynthetics, FHWA RD-97-144.

The authors recognize the efforts of Mr. Jerry A. DiMaggio, P.E. who was the FHWA Technical Consultant for this work, and served in the same capacity 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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CHAPTER 1

INTRODUCTION

1.1 OBJECTIVES OF MANUAL

The use of mechanically stabilized earth systems for the construction of retaining structures and steepened slopes has gained widespread acceptance among owners, as evidenced by the many thousands of completed structures.

There is, 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.

The design of Mechanically Stabilized Earth (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 (95 percent of applications to date), connected to a precast concrete facing. Aluminum alloys and stainless steel have been used for reinforcements mainly in France, but their use has been discontinued due to extremely poor performance.

In the last decade, polymeric reinforcements have been introduced and used with increasing frequency in both Mechanically Stabilized Earth walls and Reinforced Soil Slopes.

The 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 has been developed in support of a FHWA Demonstration Project on the design, construction and monitoring of MSEW retaining structures and RSS and its principal function is to serve as a reference source to the materials presented in that project.

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 recommendations of design procedures.
- 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 backfill materials used in MSE structures.
- 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*, 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*.⁽²⁴⁾

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 backfill with controlled electrochemical properties is specified.

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. Failures of this type have been documented in a few instances in the United States, Europe and South Africa.

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 average fictitious 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 and copper exist in nature in their metallic state. 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.⁽¹⁾

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. The authoritative reference work to date on underground corrosion is National Bureau of Standards (NBS) Circular 579.⁽¹⁾

The corrosion mechanism of ferrous and other metals in soils is essentially electrochemical. For corrosion to occur, there must be a potential difference between two points that are electrically connected in the presence of an 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. Bacterial corrosion is associated with the presence of anaerobic sulfate-reducing bacteria that reduce any soluble sulfates present in the soil to sulfides. It has not been reported as a problem with galvanized steels. 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, which consists of depositing in a bath, a zinc layer. 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.

b. Fusion Bonded Epoxy Coatings

As an alternative to galvanized coatings, fusion-bonded epoxy coatings on steel reinforcements have been used on a number of projects and provide a possible alternative solution.

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.

2.2 IDENTIFICATION OF CORROSIVE ENVIRONMENTS

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 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 17 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 the anions Cl and SO₄.

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 and in the peat bog areas of Michigan 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.

Industrial fills such as slag, fly ash, and mine tailings may be either acidic or alkaline depending on their origin. Slags in particular are likely to be acidic and contain significant amounts of sulphates. Modified soils, cement, or lime treated can be characterized by a pH as high as 12.

b. 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 vary 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 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 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.

c. 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.⁽⁵⁾ 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.⁽³⁾ 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 backfill, 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, 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.⁽²⁾ Current research projects (2000), NCHRP 21-06 and 24-13, are focused on developing better links between laboratory measurements of index properties and in situ corrosion rates.

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 dS/m is the identical value to 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 increases as resistivity decreases. 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-288-91 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.

The relative level of corrosiveness, commonly accepted by the engineering community as indicated by resistivity levels, is shown on table 1.

Table 1. Effect of resistivity on corrosion.⁽⁴⁾

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

Based on the above, resistivity ranges in the moderately corrosive to mildly corrosive ranges are generally chosen as lower bound values. From the National Bureau of Standards (NBS) data shown on figures 1 and 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.

A quantitative limit has been established for MSE backfills 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.

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.⁽¹⁾

Therefore, 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.

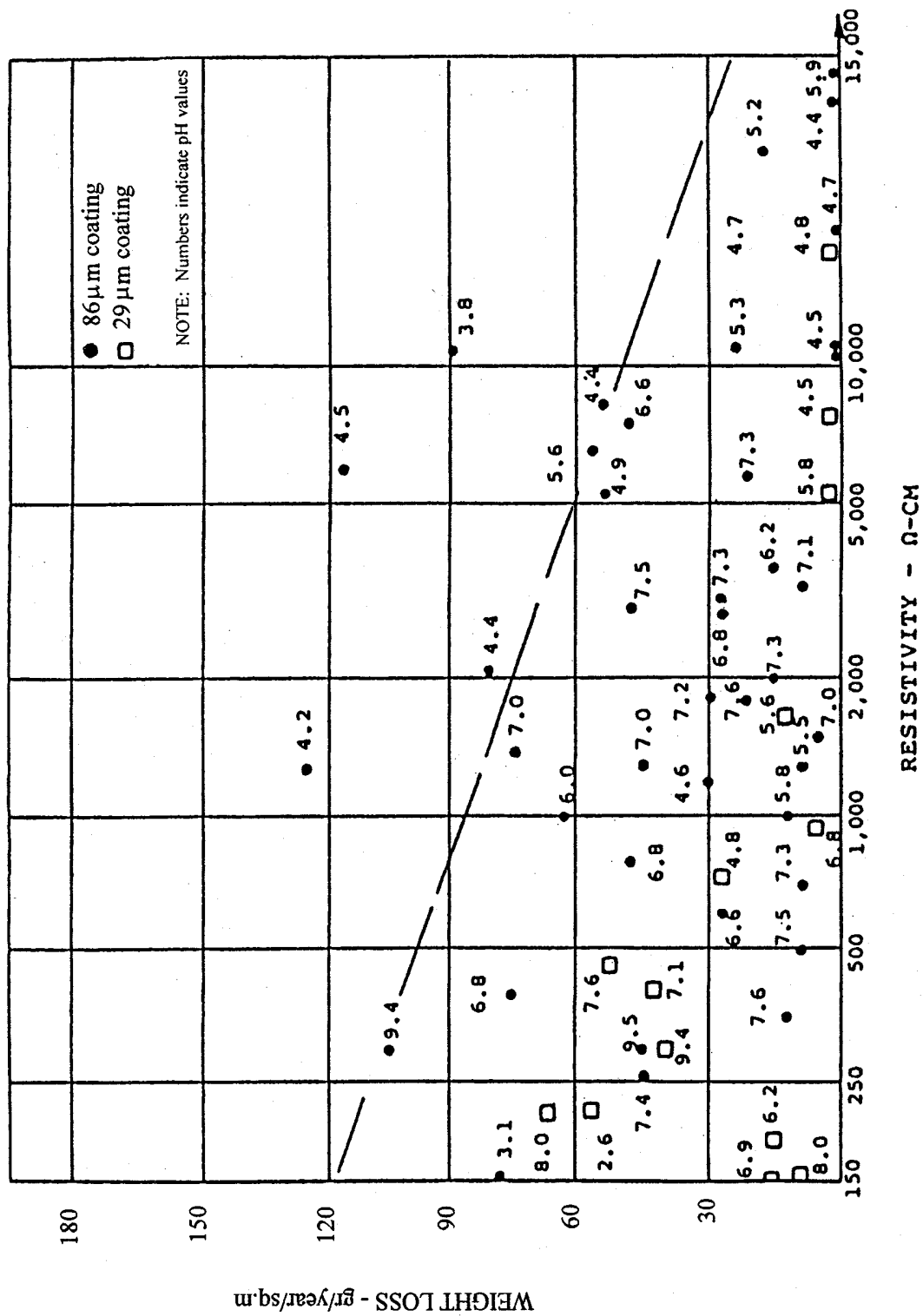
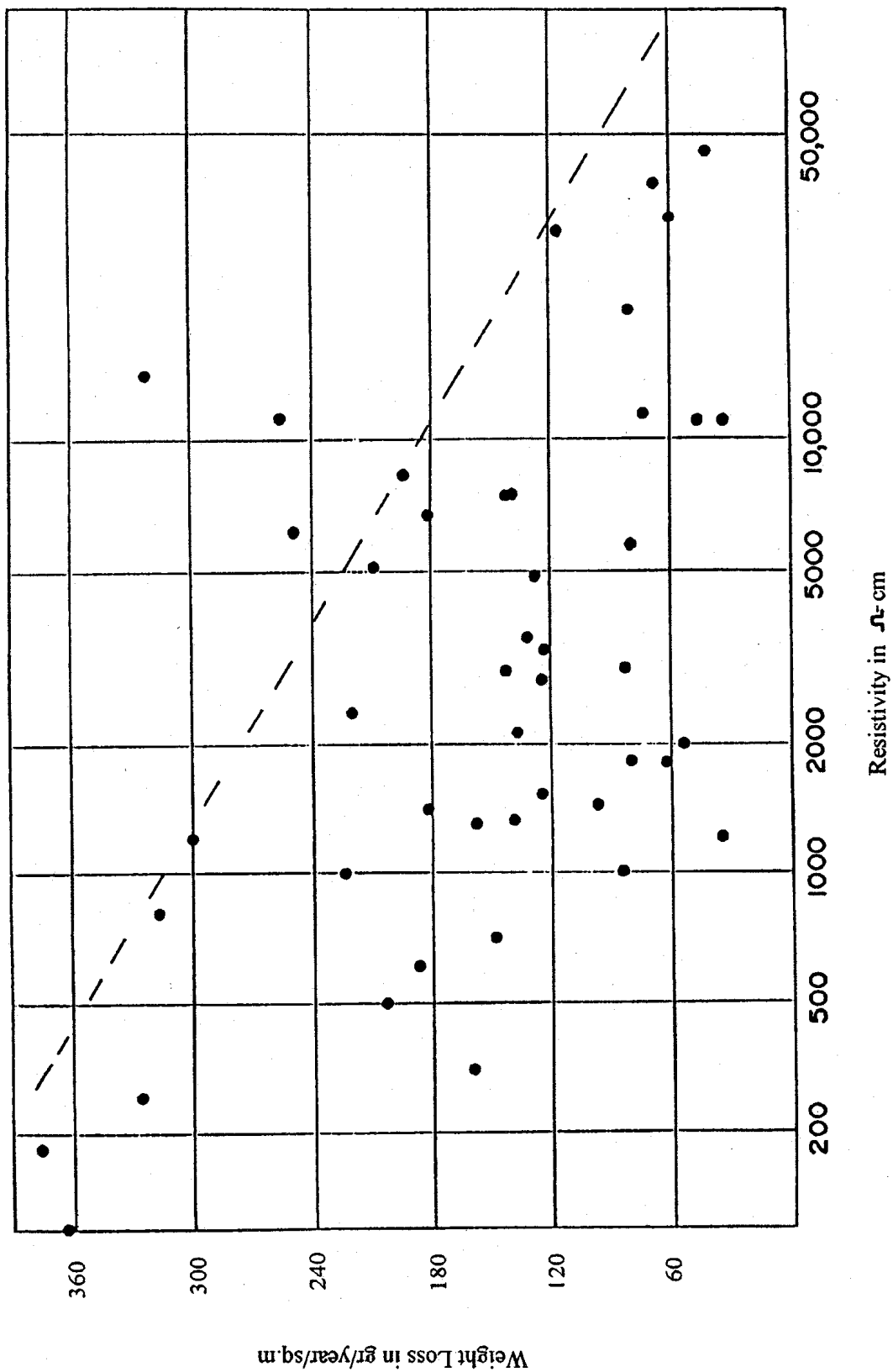


Figure 1. Metal loss as a function of resistivity (galvanized steel).



Chlorides

Chloride minerals are very soluble and thus completely removed by an aqueous extract. Chloride determination methods can be categorized as electrometric or colormetric. 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 colormetric method has been devised for application for autoanalyzers.

AASHTO has adopted an electrometric Method T-291-91 as the method for measuring chlorides concentrations.

ASTM D-4327-88 is a recently adopted standard test to measure anions, including chloride, by ion chromatography. It is the most accurate and reproducible of all methods and is well suited for automated laboratories.

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 coprecipitation 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-290-91 as the method of measuring water soluble sulfate concentrations. This is a chemical titration method.

As with chloride measurements, ASTM D-4327-88 methods by ion chromatography are the most accurate and reproducible of all methods.

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 effect of chlorides and sulfates on resistivity is shown on figure 3 for both theoretical considerations and controlled laboratory tests. ⁽³⁾

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 backfills with metallic reinforcements has been established in the current AASHTO Standard Specifications for Highway Bridges Division II, as shown on table 2.

Table 2. Maximum permissible levels of soluble salts.

<u>Soluble Salt</u>	<u>Maximum Level</u>
Chlorides	100 PPM
Sulfates	200 PPM

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. Even though two soils may have identical pH values, their total exchangeable acidities (capacity factor), and thus the lime requirements to neutralize their acidities, may be quite different.

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-289-91 test method.

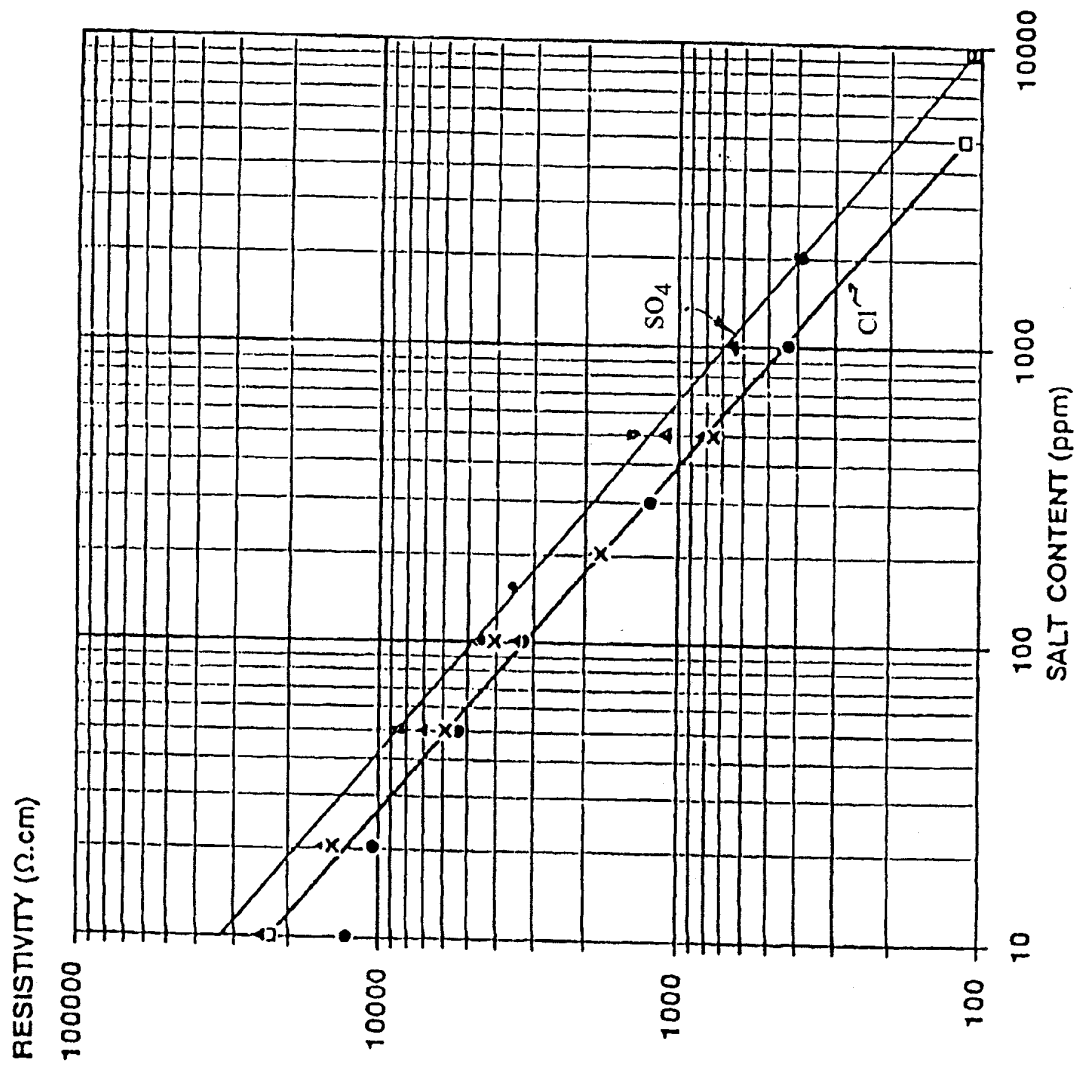


Figure 3 Resistivity vs. soluble salts

Figure 3. Resistivity vs. soluble salts.

The corrosion literature suggests that for bare steel on the pH range of 4-10, the corrosion rate is independent of pH and depends only on how rapidly oxygen diffuses on the metal surface.

Corrosion data suggest that 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. In addition, zinc is strongly attacked in strongly acidic and alkaline soils. Therefore, the galvanized coating in these regimes will have a significantly lower life expectancy.

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 backfill 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-267-86 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.

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.⁽¹⁾

Additional data include the results of many studies conducted in the United States on the performance of metal highway culverts and buried piling. This data, generally qualitative rather than quantitative, are substantially in good agreement with the extensive burial tests conducted by NBS.⁽⁴⁾

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:

$$x = Kt^n \quad (1)$$

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 μm at the end of the first year. For galvanized steels, "n" constants were not evaluated, but "K" constants varying from 5 to 70 μm 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 backfill required for MSE structures.

Various transportation departments have conducted corrosion studies with reference to metal culvert durability, summarized in NCHRP-50.⁽⁴⁾ 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 wide-spread 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.⁽⁵⁾ 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 these studies are shown on figure 4 and 5. 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.

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 suggest that pitting depths could be significantly deeper than depths suggested by uniform loss that have been substantiated by several studies.⁽⁶⁾ 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 which relate to the relative loss of tensile strength to relative **average** thickness loss. The data developed from buried samples of 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.⁽³⁾

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:

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

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

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

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

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

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 backfills.

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.^(2,4,5)

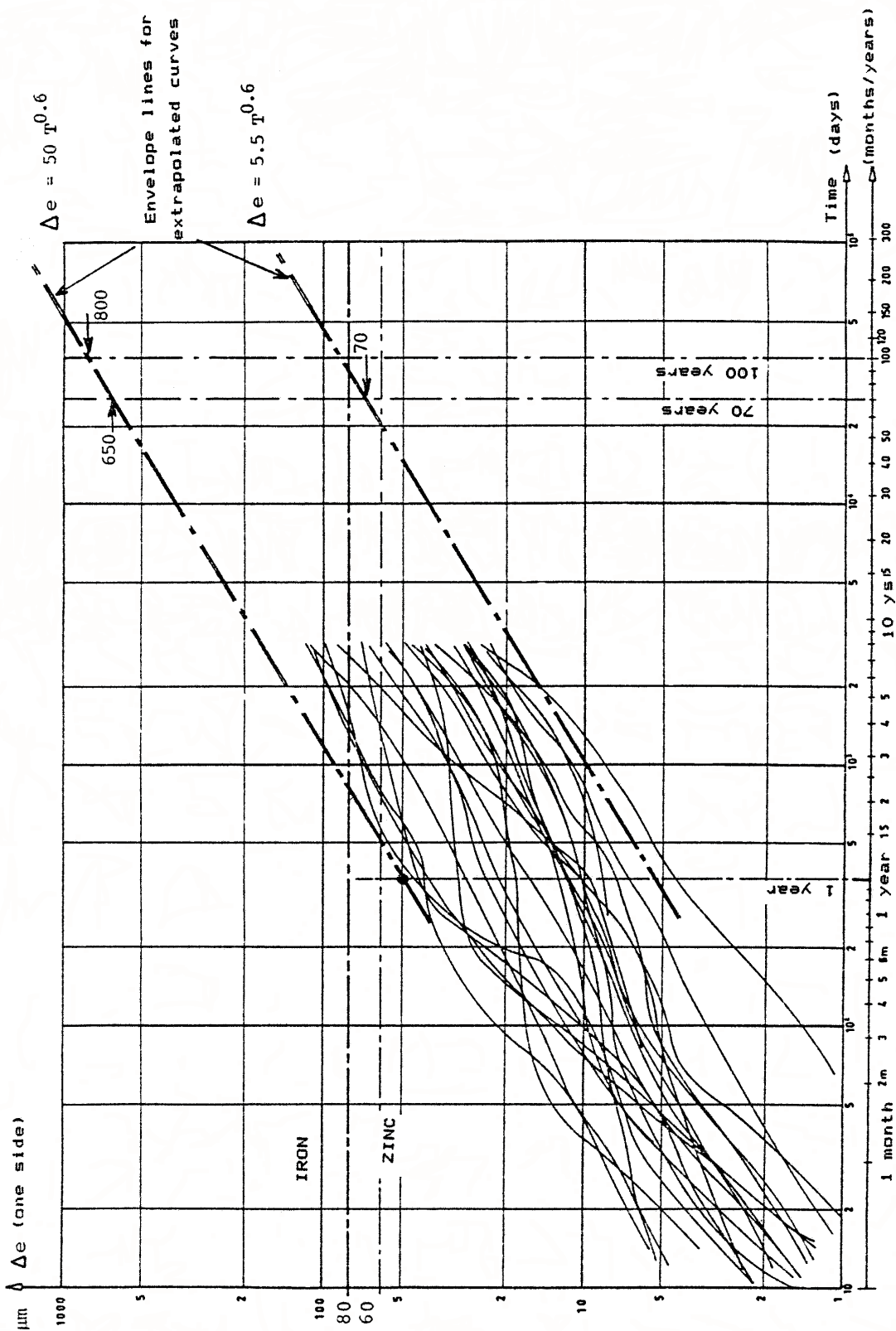


Figure 5. Summary of electrochemical cell test data at 50% and 100% saturation.

Careful study of the NBS corrosion results in soils meeting the stringent requirements for MSE structures suggests that an alternate simplified model more accurately represents the corrosion losses. ***This latter model has been adopted by AASHTO to predict uniform maximum loss rates for determination of sacrificial thickness as follows:***

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

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	ohm-cm >3000	AASHTO T-288-91
pH	>5 <10	AASHTO T-289-91
Organic Content	1% Max.	AASHTO T-267-86
Chlorides	< 100 PPM	AASHTO T-291-91
Sulfates	< 200 PPM	AASHTO T-290-91

Because of significant variability of backfill sources, multiple samples must be tested to assess mean conditions.

- *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 15 $\mu\text{m/yr.}$
- Zinc corrosion to depletion 4 $\mu\text{m/yr.}$
- Carbon steel rate 12 $\mu\text{m/yr.}$

The resulting sacrificial thickness for a 75-year life based on initial galvanization of 86 μm is approximately **1.5 mm**. 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.

- *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-884-88

The coating thickness should be on the order of 450 μm , and the design life should be considered as equal to that of a galvanized reinforcement with a coating thickness of 86 μm (16 years).

Therefore, for a 75-year design life, a sacrificial thickness of 1.5 mm of the structural steel section is presently recommended.

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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.

Two techniques have been used to monitor corrosion rates of buried metallic elements for research and structure assessment purposes:

- **Retrieval of buried coupons** with measurement of weight loss and section thickness at each retrieval interval. This is a destructive test method which requires excavation 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 backfill soils which are never totally homogeneous.
- **Remote electrochemical methods** using potential and polarization resistance measurements to obtain an instantaneous average in-situ corrosion rate of the whole reinforcement element in its stressed state. This method is non destructive and measurements can be taken at any time to more closely monitor performance.

Given the advantages, the implementation of remote electrochemical methods is highly recommended. These techniques can be utilized on both existing structures and new construction. Devices to perform these measurements have been developed and are available for general use. 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 from a typical installation.

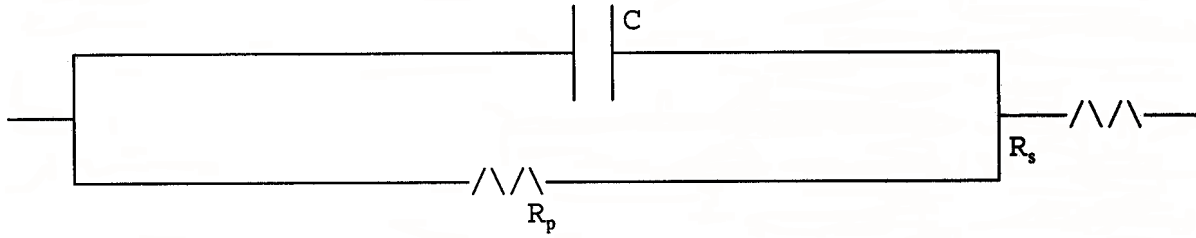
3.1 CORROSION MONITORING FUNDAMENTALS

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 a current (flow of electrons) associated with it. Numerous methods exist to measure the current or the rate of corrosion. 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 specimen surface involved in the corrosion process can be represented by the following equivalent circuit:



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^2 .

The classic description of the relation between R_p and corrosion rate was derived by Stern and Geary and is a modification of the fundamental equation for electrochemical kinetics, the Butler-Volmer Equation.(7) The classic equation of Stern and Geary is:

$$i_{\text{CORR}} = \frac{B_A B_C}{[2.3 (B_A + B_C) R_p]} \quad (7)$$

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 on a log current density basis as the voltage of the specimen is shifted away from its rest or corrosion potential. 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 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 Wisert 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.⁽⁸⁾

Electrochemical Impedance Spectroscopy (EIS) or AC Impedance is a fairly new electrochemical technique to determine the values of R_p , R_s , and C in the equivalent circuit.

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.
- That the polarization resistance be determined independently of the ohmic resistance (soil resistance). This can be accomplished by use of the EIS approach to polarization resistance determination 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.
- That the conversion value 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 for this constant and show it to be independent of the soil type. Pipeline companies routinely use a constant of 0.035 for steel pipe. Montuelle published data for galvanized steel in numerous soils suggest a conversion constant on the order of 0.050.^(3,9) Therefore, it can be assumed that conversion constants range between 0.020 and 0.050.
- That 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 to ultimately down to the carbon steel base.

The use of polarization resistance measurements in general compliance with ASTM-G59-78, 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 the 0.035 for steel and 0.050 for galvanized steel to obtain an average corrosion rate should be considered where exact data for a particular soil are not known.*** The basis of selection for the constant used should be potential measurements that will distinguish existing surface composition.

For implementation of this technique, potential measurements of representative reinforcements are made first to establish the average composition of the surface of the reinforcement (fully galvanized, partially galvanized, no galvanization).

The basis for potential measurements are outlined below.

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. Once bare steel is exposed to the soil environment, the corrosion rate of the sacrificial zinc

(galvanization) may be accelerated due to the galvanic couple with the steel. Comparisons between the potential of the reinforcements and the potentials of buried zinc and carbon steel coupons will provide the information necessary for determining when significant surface area of bare steel is being exposed to the soil.

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 ($\text{Cu}/\text{Cu}_2\text{SO}_4$). 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

The need for a field corrosion monitoring program should be established based on satisfying at least one of the following criteria:

- The desire to assess the validity of the design corrosion rates.
- The use of reinforced backfills that do not meet the minimum electrochemical criteria specified in current AASHTO.
- A suspicion that the groundwater regime established in the structure is acidic or highly alkaline.
- Some visible distress in the MSE structure or a structure of critical importance where early warning of impending failure is of utmost importance.
- Structures in or adjacent to warm marine environments.

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 should be considered for MSE structures 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 3 to 4 m because differences in oxygen content, moisture content, and salt concentration can produce different corrosion behavior. 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.

Ideally, three types of coupons should be placed at each location and depth; zinc, steel and galvanized (new structures only). At existing structures, sections of the actual reinforcement wall replace the galvanized coupons and will be periodically removed.

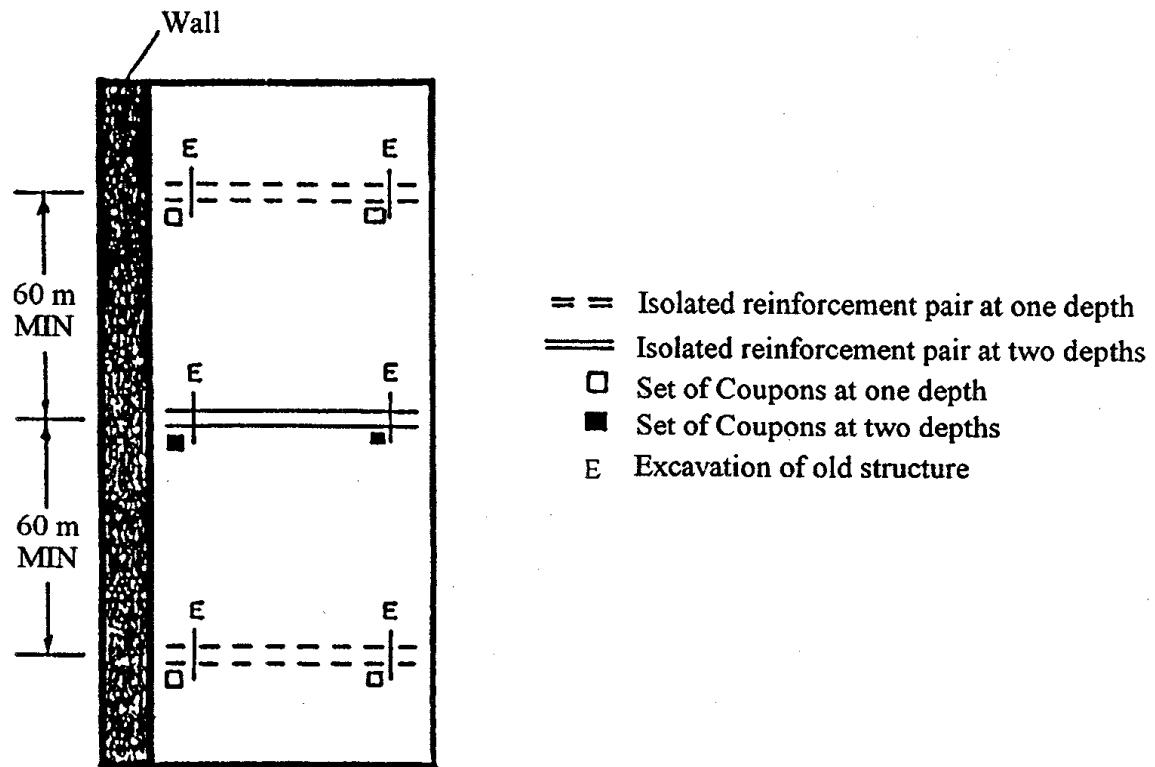
For monitoring, it is desirable to have one-zinc, one-steel and up to four galvanized coupons at each depth. The multiple galvanized coupons are for periodic removal. Coupons each have two leads to provide back-up in case one connection fails.

At each location, the test station should be 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.

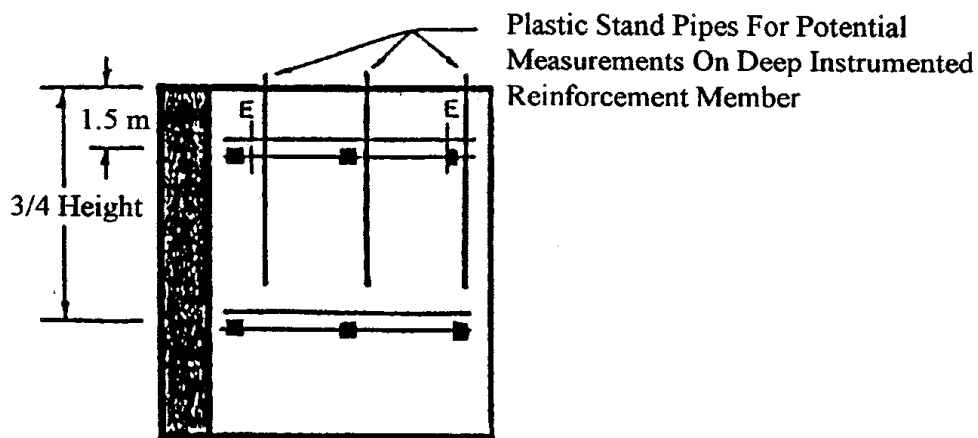
Figure 6 shows the location of the coupons and instrumented reinforcement members. For complete monitoring, three test locations (top view) are desirable for structures over 250 m in length. For structures under this length, two test positions will suffice. One critical location (center of structure) should be selected for establishing test locations at both shallow and deep positions. For structures over 250 m in length, two or more locations should be considered for monitoring at two or more depths. The shallow depth stations should be approximately 1.5 m in depth, and the deep position should be approximately at one-fourth of the structure height from base level. Intermediate positions, where desired, should be spaced at intervals of 2 to 3 meters.

Similar location selection criteria apply for existing and new structures. However, it is realized that for existing structures, only the shallow depth may be practical. This location may not yield the maximum corrosion rates.

The rationale for measurements can be explained by viewing the corrosion process as having 3 Stages as shown on figure 7.



(a) Top View



(b) Side View

Figure 6. Schematic diagram showing locations of coupons and instrumented reinforcement members.

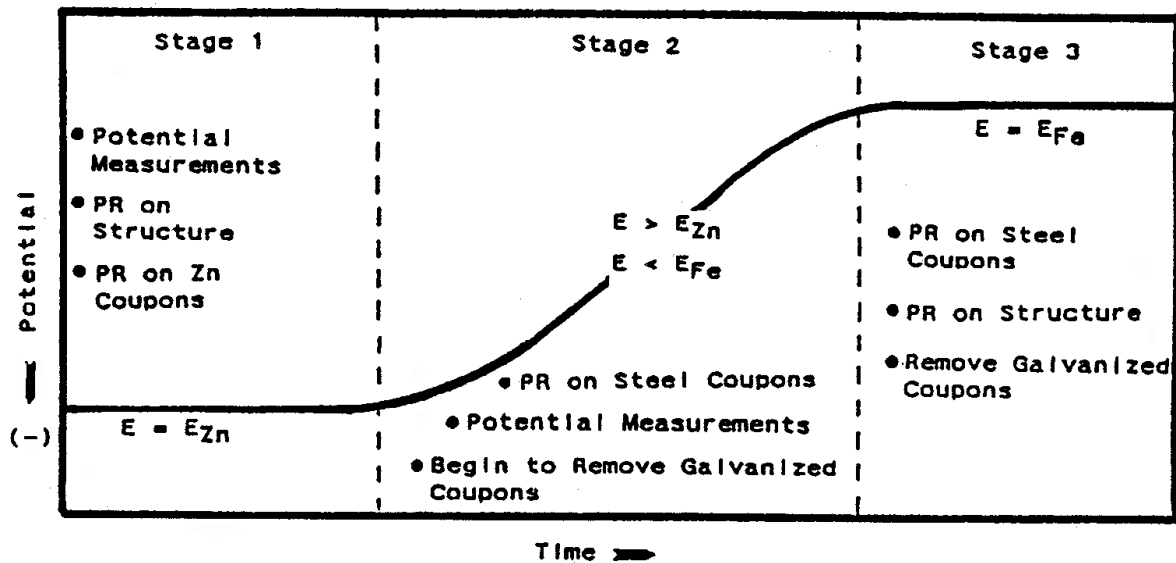


Figure 7. Stages of galvanized structure performance.

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 7.

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. 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. 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 measurements. Otherwise the area polarized is unknown and the resulting calculated corrosion rate is in significant error.

Instrumentation of the reinforcement strips should be performed in pairs, such that two parallel strips are instrumented. On new structures, "dummy" pairs of reinforcement members are instrumented and placed in the structure at the desired location such 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 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 8. Two connections will be made on each instrumented strip for redundancy.

For MSE structures that have grid-type reinforcing members, the procedures for instrumenting 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)

For existing structures, excavation must be performed to reach the desired instrumentation levels. If excavation is performed from the top of the structure isolation should be accomplished by removing a 0.3 to 0.6 m section of the reinforcement strip or grid at a point 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 0.3-m sections cut and removed for evaluation. The 0.3-m sections removed from existing structures serve the same purpose as the retrievable galvanized coupons used for new structures.

Instrumentation lead connections on existing structures are made in a similar manner as described for the reinforcement strips and are shown on figure 8.

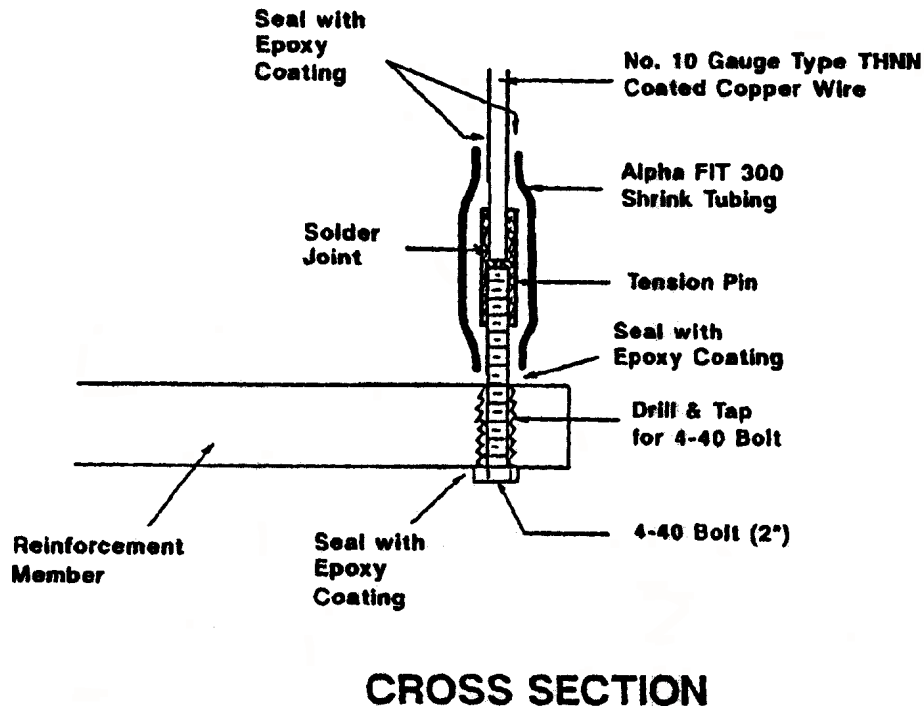


Figure 8. Schematic diagram for connection to reinforcing members.

Alternately to minimize excavation, more limited but generally sufficient data can be obtained by accessing the reinforcements by drilling multiple 60 mm holes thru the facing adjacent to reinforcement locations. Electrical isolation of the reinforcements must be assured by testing prior to any potential or polarization measurements. If electrical continuity exists, the working reinforcing element must be detached or isolated from the facing system. One of the access holes can then be used for the placement of the reference electrode and subsequent potential and polarization measurements.

Procedures using this alternate method have been developed and demonstrated.⁽²⁵⁾ Figures 9 and 10 illustrate the necessary field setup.

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. 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 are shown in figures 11 and 12:

- 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 (red) 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 9. 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.

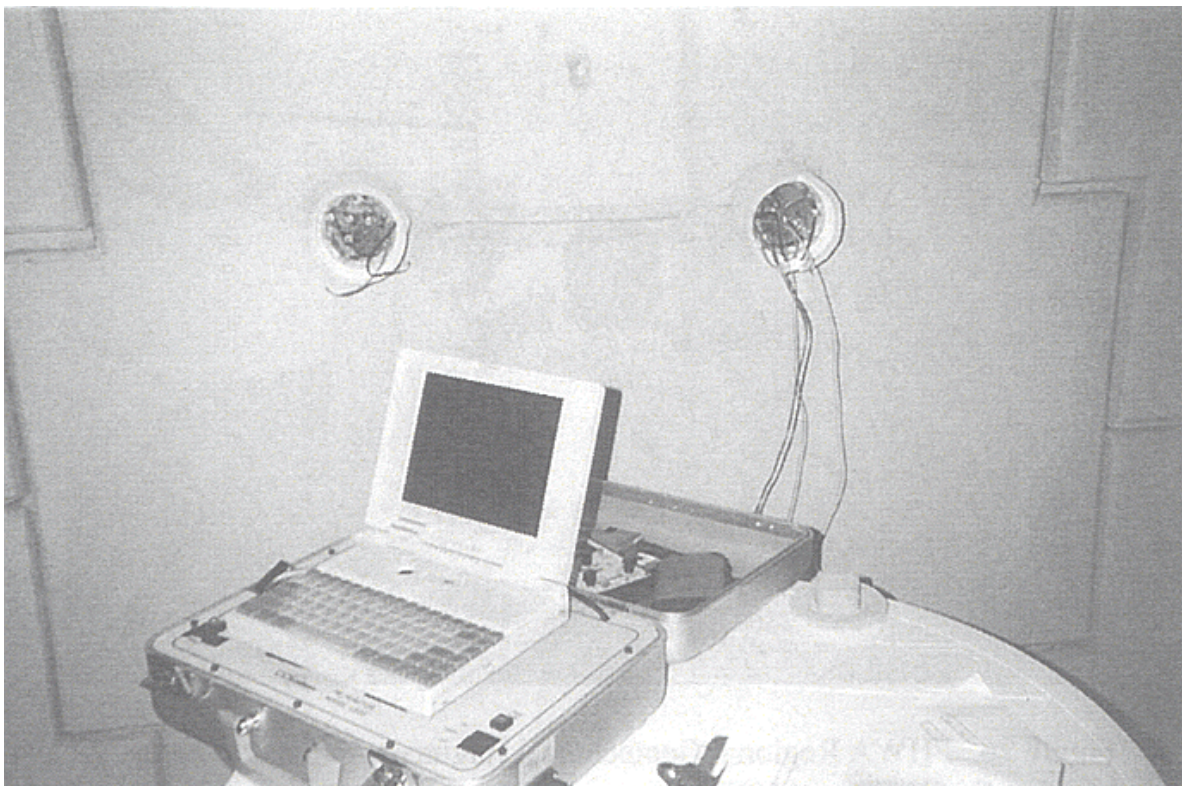


Figure 10. PR Monitor evaluation of a test location. Note horizontal adjacent access holes for cross testing.

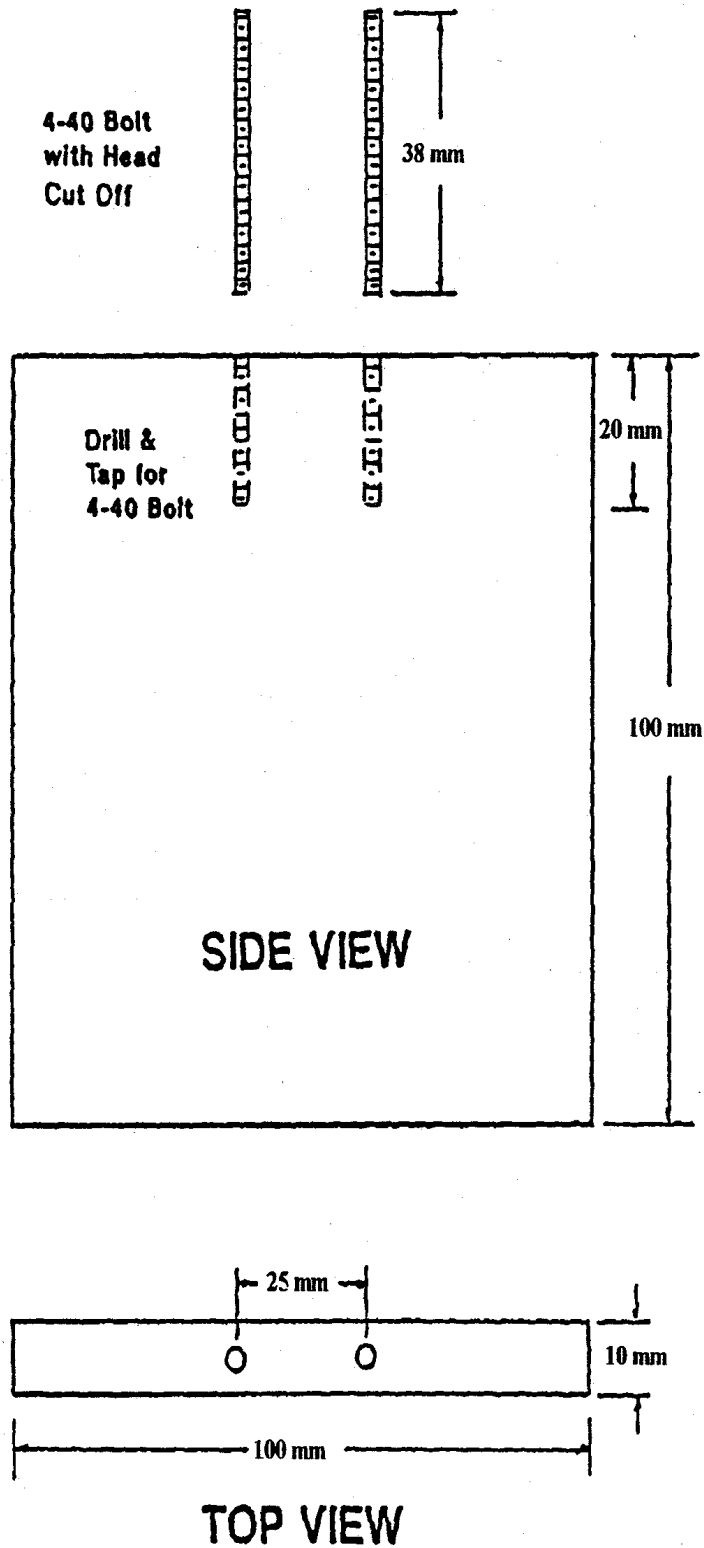
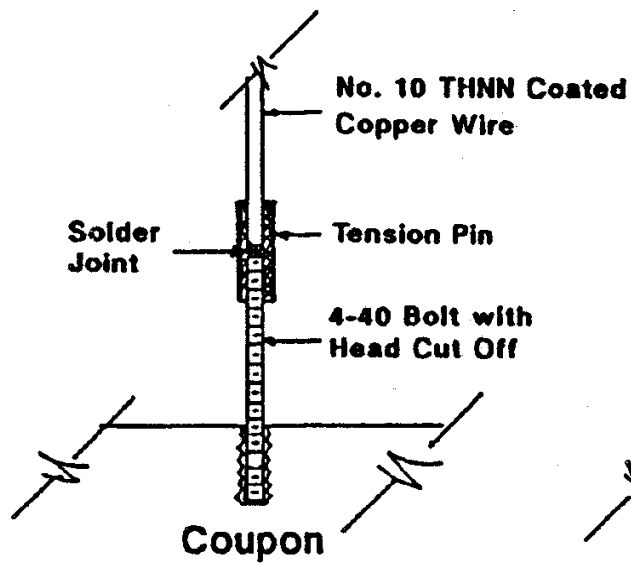
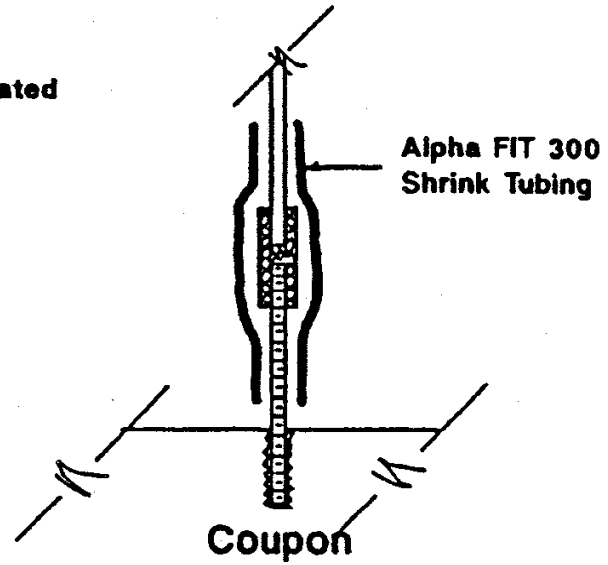


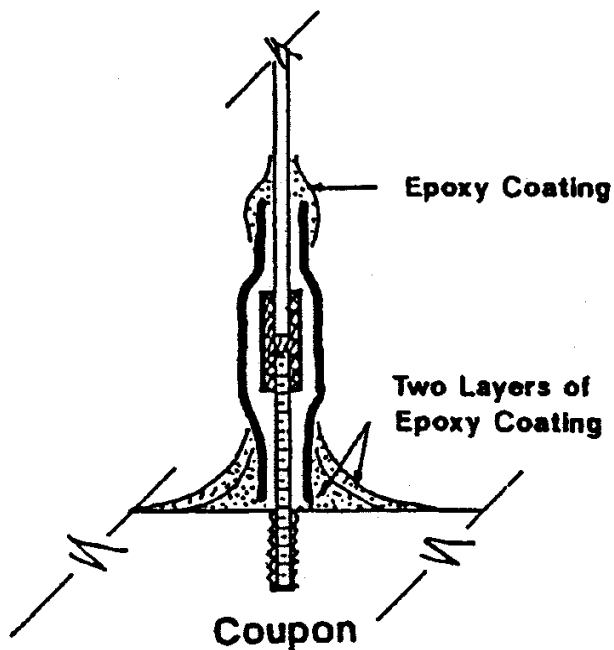
Figure 11. Schematic diagram for coupons.



(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 12. 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 structure (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 70 to 120 mm in diameter.
- The test leads are black.
- 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. The cut edges of the coupons should be redipped in a zinc bath to provide a galvanized coating at the cut edges. 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 1.2 m long.

The instrumented coupons should have the following configurations:

- Coupon size is 0.3 m long (thickness or diameter will be the same as the reinforcement element).
- A single 4-40, 50 mm 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).

f. Measurement Procedures and Equipment

After the installation of leads to the coupons, reinforcing element, and backfilling of at least one lift, initial measurements of potential and polarization resistance can be made.

Figure 13 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 (100 Mohm or greater) and a copper/copper sulfate ($\text{Cu}/\text{Cu}_2\text{SO}_4$) electrode. Multiple measurements (every $2 \pm \text{m}$) 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.

Polarization resistance measurements require the application of a potential (current) and resulting current to the specimen being recorded, 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 performing polarization resistance are outlined in ASTM G-59-78 (reapproved 1984). Soil resistance measurement equipment is typically needed for correcting polarization resistance measurements for R_s .

Fully automatic equipment is now available that can apply current, make polarization and soil resistance measurements, and integrate all data. (See FHWA RD 89-186). 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 14.

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.

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



Figure 13. Typical installation.



Figure 14. Automated polarization resistance measurement equipment.

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, the isolation of the reinforcement member should be accomplished by removing a 0.3-m section at a point 1.5 to 3 m from either end of the reinforcement member. 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 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"*.

The data obtained at Site 4, lower level of a newly constructed wall, for 26 months is shown on table 3. At this site the reinforced backfill 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.

Table 3. 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 ^(b) , Steel, in μm	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 μm	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 μm	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 μm	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 μm	4.5	0.8	0.6	1.3	0.6

(a) Free-corrosion potential, $V(\text{Cu}/\text{CuSO}_4)$.

(b) Corrosion rate estimated from polarization resistance corrected for soil resistance, $\mu\text{m}/\text{yr}$ unless otherwise noted.

The free corrosion potential measurements, E_{cor} , for the steel coupons, using a copper sulfate reference electrode, $\text{Cu}/\text{Cu}_2\text{SO}_4$, 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.

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 backfills.

This typical data confirms the suitability and practicality of the monitoring techniques using electrochemical principles.

Additional confirmation of the suitability and practicality of techniques using potential monitoring was developed by Caltrans in their monitoring of fourteen MSE wall sites.⁽²⁶⁾

Complete confirmation of the suitability of the design corrosion rates previously recommended can only be made after at least a few years of measurements.

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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 is increasing. 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 allowable strength, T_{al} is then obtained from:

$$T_{al} = \frac{T_{ult}}{RF_{CR} \times RF_{ID} \times RF_D} = \frac{T_{ult}}{RF} \quad (8)$$

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, (kN/m)

This chapter is intended to provide a background on available geosynthetic materials for reinforcement applications. It also 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 provides the basis for selecting aging (RF_D) and installation damage (RF_{ID}) reduction factors, consistent with the in-ground regime and the corresponding allowable 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 polypropylene, polyester and polyethylene. 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 a market survey conducted by Industrial Fabric Association International (IFAI)⁽²⁸⁾ the total 1995 geosynthetic product market in North America was estimated at 441 million square yards, which can be divided on a raw polymer material basis as shown on table 4.

Table 4. Raw material in Geotextile and Geogrid production.

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

The market as further divided on a geosynthetic structure basis is shown on table 5.

Table 5. Geotextile and Geogrid structure.

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

The geosynthetic market based on an application basis, is shown on table 6.

Geogrids, wovens, and non wovens geotextiles, are used extensively for steepened slopes, but most retaining wall applications use geogrid reinforcement.

For polyolefins, polypropylenes (PP) are manufactured from general purpose homopolymers (group 1, class 1, ASTM D-4101), which may differ in grade (Melt Flow) slightly, based on the manufacturing process used. Grade 4 appears to be universally used.

Table 6. Geosynthetic applications.

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

Table 7 lists the major suppliers of PP reinforcement geosynthetics and their properties as furnished by the manufacturers and listed in their catalogs.

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 regrinded, melted, and added to the resin used for other production runs. Most manufacturers add resin obtained from regrinded polypropylene to "virgin" batches of resin which do not yet contain stabilization additives. The entire batch is then treated as if it were comprised entirely of virgin resin and the corresponding amount of additives are then introduced. Therefore, the amount of stabilization additive contained in a batch of resin which included previously used resin will have higher concentrations of additives than batches which are truly virgin. This procedure is employed to insure that a minimum standard of geosynthetic stability is consistently adhered to.

Table 7. Major PP product groups. (2000)

Product Name	Structure	Manufacturing Process	Polymer Class & Grade ASTM 4101	Melt Flow g/10 min. ASTM 1248	Antioxidants Process/ End Use Additives	Weight Range g/m ²	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) Nicolon	Woven	Fibrillated yarns woven	Grade 4 Group-1 Class-1	4	1% Carbon Black	284 - 866	35 - 158
3. b) Nicolon	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. Synthetic Industries	Woven	Slit film/fibrillated yarns	Grade 4 Group 1, Class 1	0.1-12	Carbon Black	N/A	21 - 70
	Woven	Monofilament			Carbon Black	N/A	30
6. Amoco	Woven	Slit film tape fibrillated tape	Grade 4 Group 1, Class 1	N/A	N/A	N/A	21 - 70
	Non Woven	Needle punched, staple fiber		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

Only one high density polyethylene (HDPE) product, a geogrid, is currently being manufactured and widely used in reinforcement applications. Its composition is as follows:

Table 8. HDPE product group. (2000)

Product Name	Structure	Manufacturing Process	Polymer Class & Grade ASTM 1248	Melt Flow ASTM D 1248 (g/10 min)	Antioxidants Process/End Use Additives	Weight Range (g/m ²)	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 9 lists the major product groups and the properties of polyester products (PET) that are produced for geogrids and geotextiles. 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 9, in which all products in this group are listed, high tenacity 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 by 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.

Table 9. 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 ²	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. Mirafi "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. Hoechst/ Celanese "TREVIRA"	Continuous filament, spun bonded needle-punched non-woven		Hoechst/ Celanese	20,000-21,000	45-50	0.02% carbon black	N/A	9 - 36
6. Mirafi 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
7. LINQ "GTF"	Woven filament yarn		T-800 Hoechst/Celanese	I.V. = 0.88	17	none	N/A	70 - 175
8. Huesker "COMTRAC"	Woven filament yarn		AKZO Allied Fibers	I.V. = 0.91	17	none	250 - 1560	50 - 800

N/A - Not available

The most commonly used geosynthetics for transportation applications are:

- Woven geotextiles : monofilaments
multifilaments
slit tape and combinations
- Nonwoven geotextiles : needlepunched
heat bonded
resin bonded and combinations
- Geogrids : extruded uniaxial
biaxial
coated fiber

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 20 to 250 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.

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 sheets. These types of geogrids are usually called *stiff* geogrids. Geogrids are also manufactured of polyester yarns joined at the crossover points by a knitting or weaving process, and encased with a polymer-based, plasticized coating. These types of geogrids are generally called *flexible* geogrids.

4.2 FUNDAMENTALS OF POLYMER DEGRADATION

Polymers consist of long chains of, principally, 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 micro-organisms. 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.⁽¹¹⁾ 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), are 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 a 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 more reactive OH⁻ ion present compared to the water molecules as reagents in neutral (pH = 7) reactions. ***The effect of these reactions is to decrease the molecular weight (Mn) with a corresponding decrease in strength.***

The rate of hydrolysis is primarily effected 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.⁽¹¹⁾
- 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 have been studied in FHWA RD-97-142 and a preliminary testing protocol developed. For the presently available geogrid it was concluded that stress cracking is a less stringent or equal consideration than creep for intact geogrids. For severely damaged geogrids, stress cracking may occur at load levels lower than inferred by the material creep limit.⁽²⁷⁾

The maximum stress level under either mode of failure determines an allowable stress basis.

Therefore, if the backfill maximum size is limited to 20 mm, ensuring a low level of construction damage, no consideration for stress cracking appears warranted for the geogrid detailed in table 8.

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. Also 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.⁽²²⁾ *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 ASTM D-4355 for duration of up to 500 hours, or outdoors under ASTM D-1435.

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.⁽¹²⁾

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 micro organisms 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.⁽¹³⁾

No completely relevant test to measure the resistance of geosynthetics to biological effects in unstressed states is presently available. ASTM-3083-89 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 D-5322 "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 (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. Table 10 presents 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.

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 10.
- Identify the nature of the physical and chemical effects that these mechanisms have on candidate geosynthetics and their properties.

Table 10. Commonly identifiable degradation mechanisms.⁽¹⁵⁾

Mechanism	Source	Effect	Variables	Test Procedures
PHYSICAL				
Stress/pressure	Installation/in use	Rupture, creep, stress cracking	Stress level, Backfill grain size	ASTM D-5262 Stress Rupture Tests ASTM D-5818
Water	Installation/in use	Leaching of additives and plasticisers, hydrolysis	Temperature, pH	ASTM-5496 Immersion Testing
Solvents/hydrocarbons	Installation: diesel mineral oils hot bitumen In use: bitumen	Leaching of additives, swelling and embrittlement, plasticisation	Temperature, chemical concentration	EPA 9090 and Leaching tests.
Biological	Installation/in use: birds, animals, insects	Localized damage	Soil type and density	Not Available
CHEMICAL				
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 D-4335
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-5322
Micro-organisms	In use: bacterial and fungal attack in soils	Polymer chain degradation; loss in tensile properties	Temperature, pH soil type, organism type	ASTM-3083

- 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 backfill. 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, of geosynthetics considered for use:

1. Polypropylene and polyethylene materials contain an antioxidant package to inhibit oxidation.
2. Polyester materials have suitably 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. Alternatively, 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.

4.3 IDENTIFICATION OF SOIL ENVIRONMENTS WHICH ACCELERATE DEGRADATION

The 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 1.5 m vary from 0° C to 25° C in the continental U.S. Surface temperatures vary between below freezing to 45° C.

The annual rainfall in the continental United States varies from nearly zero in the western desert to 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 believed 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 precipitation is low and there are high evaporation and transpiration rates. In the United States 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-ions. *They are the most alkaline environments found in the United States.*

c. Acid-sulphate Soils

Acid-sulphate soils are extremely acidic with pH's 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 pH's 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 pH's 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 United States.

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.⁽¹¹⁾ 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 (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 ferrousferric 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 11. It should be noted that polymers identified as "Questionable Use, Exposure Tests Required", may perform satisfactorily if formulated with specific antioxidants or additives to prevent degradation in that specific environment.

Table 11. Anticipated resistance of polymers to specific soil environments.

<u>Soil Environment</u>	<u>Polymers</u>		
	<u>PET</u>	<u>HDPE</u>	<u>PP</u>
Acid Sulphate Soils	NE	?	?
Organic Soils	NE	NE	NE
Saline Soils pH<9	NE	NE	NE
Calcareous Soils	?	NE	NE
Modified Soils (Lime, cement treated)	?	NE	NE
Sodic Soils, pH>9	?	NE	NE
Soils with Transition Metals	NE	?	?

NE = No Effect

? = Questionable Use, Exposure Tests Required

Table 11 indicates that certain polymers should not be considered without site-specific testing for their long-term durability or specific knowledge of their additives or molecular structure.

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 Questionable Use in the potentially aggressive environments identified in table 11.

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. Under 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) provide additional thermal stability when used with certain primary hindered phenol stabilizers.⁽¹⁴⁾ 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.⁽²⁴⁾

ASTM D-4355 is usually employed to measure effectiveness against UV radiation. This test incubates specimens in a weathermeter 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 D-1248 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, notched constant stress methods such as ASTM D-5397 have been introduced. These tests are capable of determining relative ESC resistance among products, but their relevance for design purposes is currently being investigated.

Testing to obtain design-allowable strength based on stress-cracking limits is detailed in FHWA RD-97-142.⁽²⁷⁾

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.*⁽²⁴⁾

In highly alkaline media, (pH >9) a dual degradative process of dissolution and hydrolysis has been reported in laboratory tests.^(19,24)

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⁻ 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 it 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.⁽¹¹⁾

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.⁽¹¹⁾

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 D-5818 "*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.^(3,16,18)

The range of strength loss reported for a wide range of geosynthetics varies between 10 and 77 percent. Insufficient data presently exists to correlate strength loss to type or thickness of reinforcement or backfill characteristics, but a general trend emerges that strongly suggests that extreme damage is associated with coarse angular backfills spread in relatively thin lifts and compacted with heavy compaction equipment. The most important variables affecting the level of damage appear to be angularity, average backfill size used (D_{50}) 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 270 g/m^2 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.⁽¹⁶⁾ 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 25 mm significantly increased the level of damage with correspondingly greater losses of strength.
- Varying compacted lift thicknesses between 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 D-4595 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:

$$RF_{ID} = \frac{T_{undam}}{T_{dam}} \quad (9)$$

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 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 more than at one sample location.

Based on recent 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 by the author and is shown on figure 15.^(3,16,18,20) 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 15 suggests that geosynthetic and backfill specific testing is necessary to evaluate the minimum installation damage strength loss.

Figure 15 should not be used directly to obtain this reduction factor, but rather as a check of manufacturers supplied data.

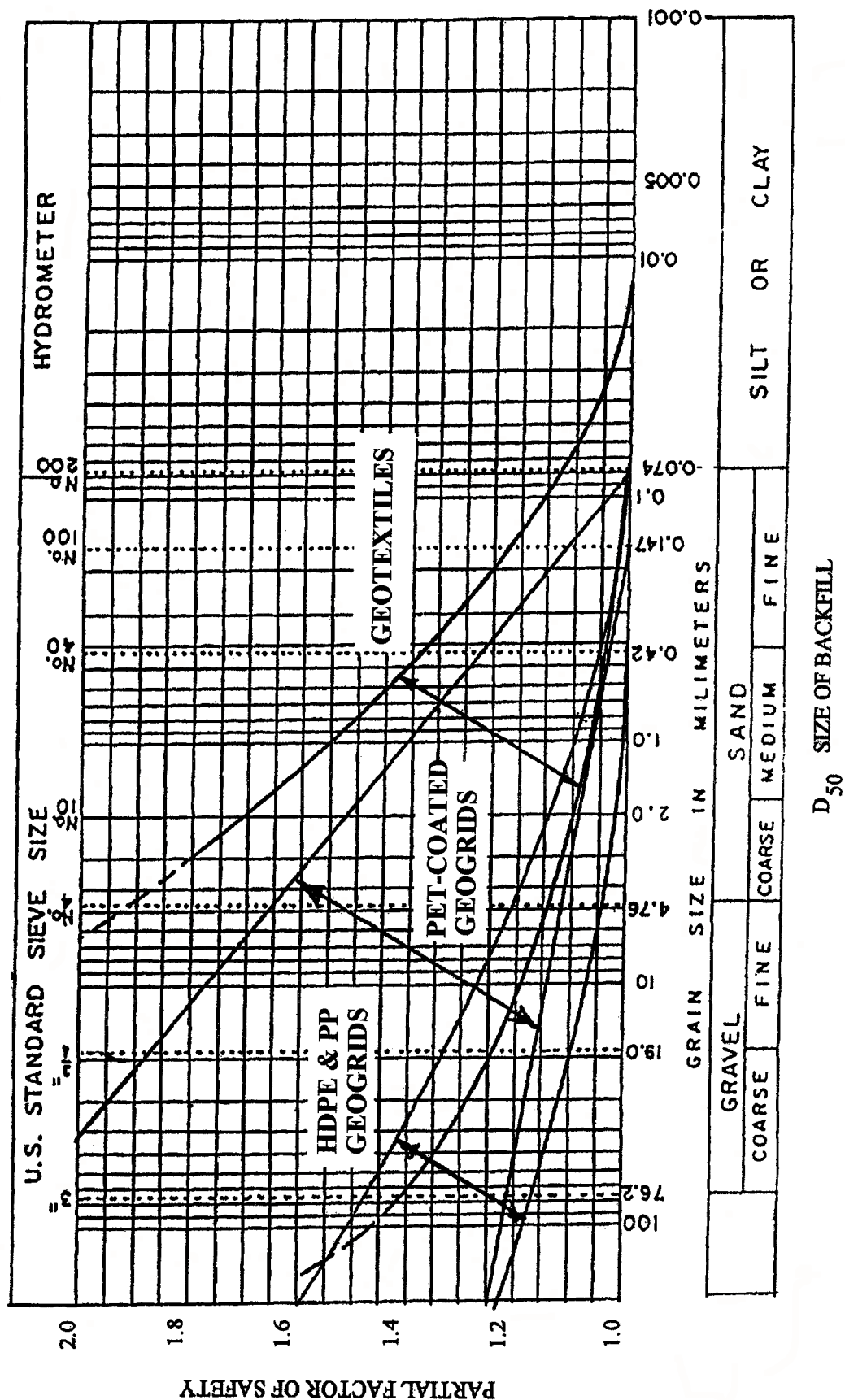


Figure 15. Installation damage vs. backfill size.

b. Summary

To account for installation damage losses of strength where full-scale product-specific testing is not available, Table 12 can be used with consideration of the project specified backfill characteristics.

Table 12. Installation damage reduction factors.

Reduction Factor, RF_{ID}			
No.	Geosynthetic	Type 1 Backfill Max. Size 102mm D_{50} about 30mm	Type 2 Backfill Max. Size 20mm D_{50} about 0.7mm
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

⁽¹⁾ Minimum weight 270 g/m²

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.⁽²⁴⁾

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 15 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 5 to 15 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.⁽¹⁷⁾

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 of atmospheric. 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:

$$K = Ae^{-E/RT} \quad (10)$$

where K is the rate constant at temperature T (Kelvin), A is the pre-exponential factor (often termed the collision factor), E is the activation energy (J mol^{-1}) and R is the gas constant ($= 8.3136 \text{ J mol}^{-1}\text{K}^{-1}$). 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 heterogeneous, 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.⁽²⁴⁾

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 40° to 70° C are used for PET incubations, 30° to 80° C for PP, and 70° to 90° C for HDPE. At these temperatures, testing times are on the order of 2 to 4 years.

Typical laboratory equipment is shown on figure 16.

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 17. 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.

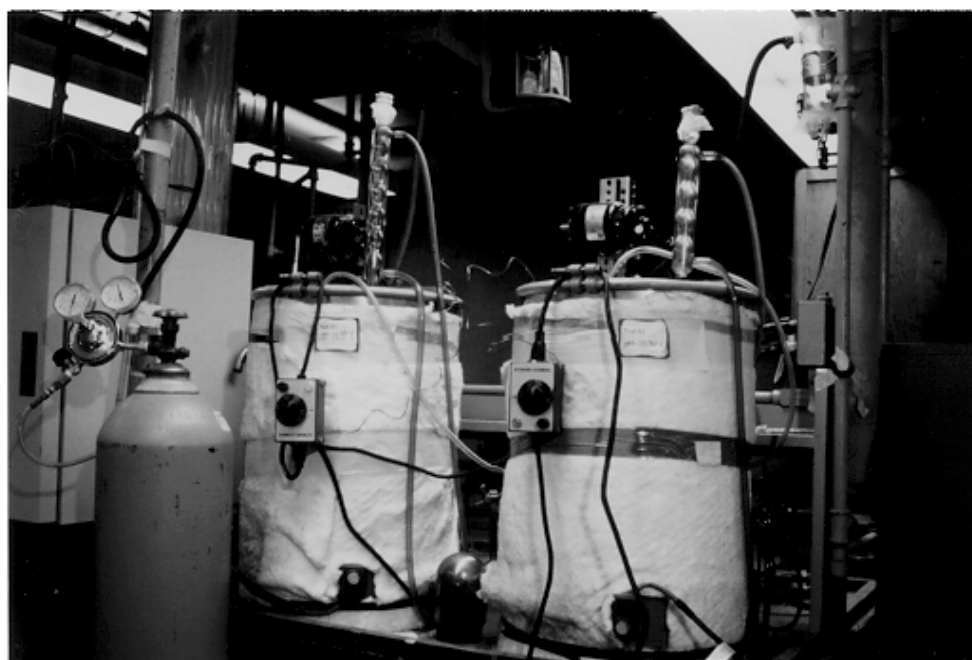


Figure 16. Laboratory aging equipment setup.

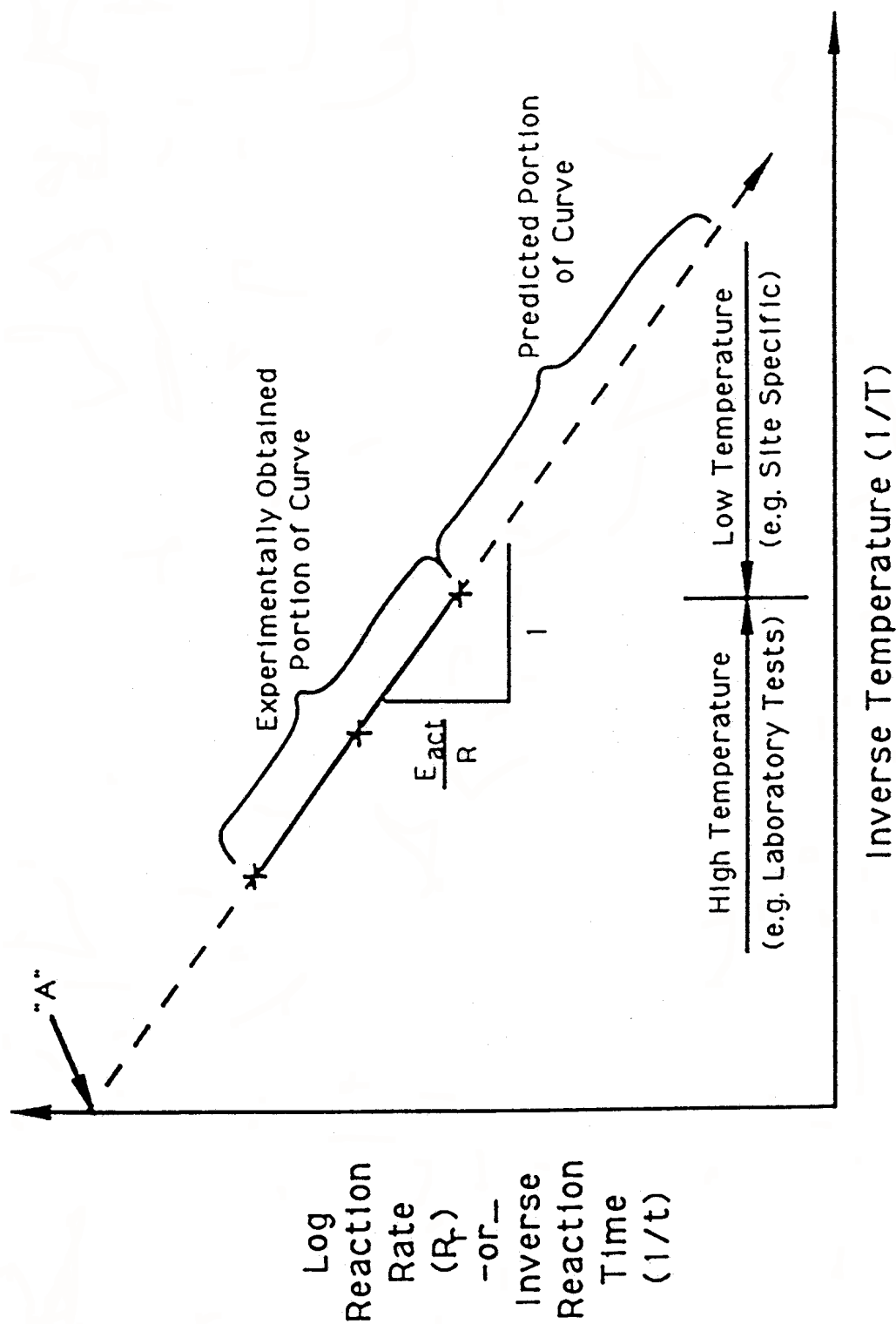


Figure 17. Generalized Arrhenius plot used for low-temperature predictions from high-temperature experimental data.

A wide range of physical and chemical properties may be extrapolated using this technique as extensively discussed by Koerner, provided that:⁽¹⁷⁾

- 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 analyses techniques consistent with the well developed basic autoxidation scheme (BAS), as outlined in FHWA RD-97-144, must be applied.^(23,24)

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 (eg., 20° C), multiple samples of the product are aged at three elevated temperatures (eg., 50°, 65°, 80°) as shown on figure 17, High Temperature.

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 17.

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:

$$RF_D = T_{ultlot} / T_D \quad (11)$$

where, T_{ultlot} is the average lot specific ultimate tensile strength for the unaged lot of material used for the durability testing, and T_D is 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 13 summarizes specific studies needed.

Table 13. Summary of product-specific studies needed to evaluate the durability of geosynthetic reinforcement.

Environmental Factor	Polymer Resin Type for Which Studies are Needed	Geosynthetic Reinforcement Studies Needed
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 3083-89

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 (RF_D) are presently indicated for a 100 year design life in the absence of product specific testing:

Table 14. Aging reduction factors, PET.

No.	Product*	Reduction factor, RF_D	
		$5 \leq \text{pH} \leq 8$	$3 \leq \text{pH} \leq 5$ $8 \leq \text{pH} \leq 9$
1	Geotextiles $M_n < 20,000$, $40 < \text{CEG} < 50$	1.6	2.0
2	Coated geogrids $M_n > 25,000$, $\text{CEG} < 30$	1.15	1.3

* Use of materials outside the indicated pH or molecular property range requires specific product 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), PP's are vulnerable to oxidation and significant strength loss within a projected 75 to 100 year design life at 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.

At present, 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.⁽²⁴⁾ 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 are under study and development.

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 D-4355. 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 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 20° C and 100 years.

Note that for products meeting the minimum requirements of table 8 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 either oxidation, hydrolysis, or UV radiation that is a long-term phenomena.

For MSE walls or RSS construction, the result 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 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 backfill, field testing is required. The use of ASTM D-5818 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 D-4595), in both directions. 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 D-4595 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 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 150 mm with the upper sheet placed in the direction of soil placement. A total sample size of 5-by 3-m should be used as a minimum.
- Place 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 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 AASHTO density. As a maximum, 10 passes are recommended.
- Carefully remove the backfill by hand and document any observable geosynthetic damage, including a visual survey of puncture holes per square meter.
- Select specimens of the geosynthetic for testing. Specimen selection should be guided by the placement of a primary template 600 by 800 mm shown in figure 18 on the center of the sample.

Cut and number twelve adjacent specimens 200 mm by nominally 200 mm as shown in figure 18. 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 geotextiles, the length of specimen should be increased to accommodate the roller grip requirement. For stiff geogrids, the specimen size should be such to contain three ribs in the machine direction and five ribs in the cross direction. For flexible geogrids up to seven cross direction ribs may be necessary to accommodate the required roller clamps.

Evaluation

Wide-width tensile strength obtained from these nine primary tests should be analyzed to determine the coefficient of variation as outlined in ASTM D-4595 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%, the required number of samples should be recomputed using the one sided student t variation (Table 1, ASTM D-4959). If greater than nine are required, then additional specimens as available should be secured from a secondary template location also shown on figure 18. No more than a total of 18 specimens per sample should be tested for wide-width strength.

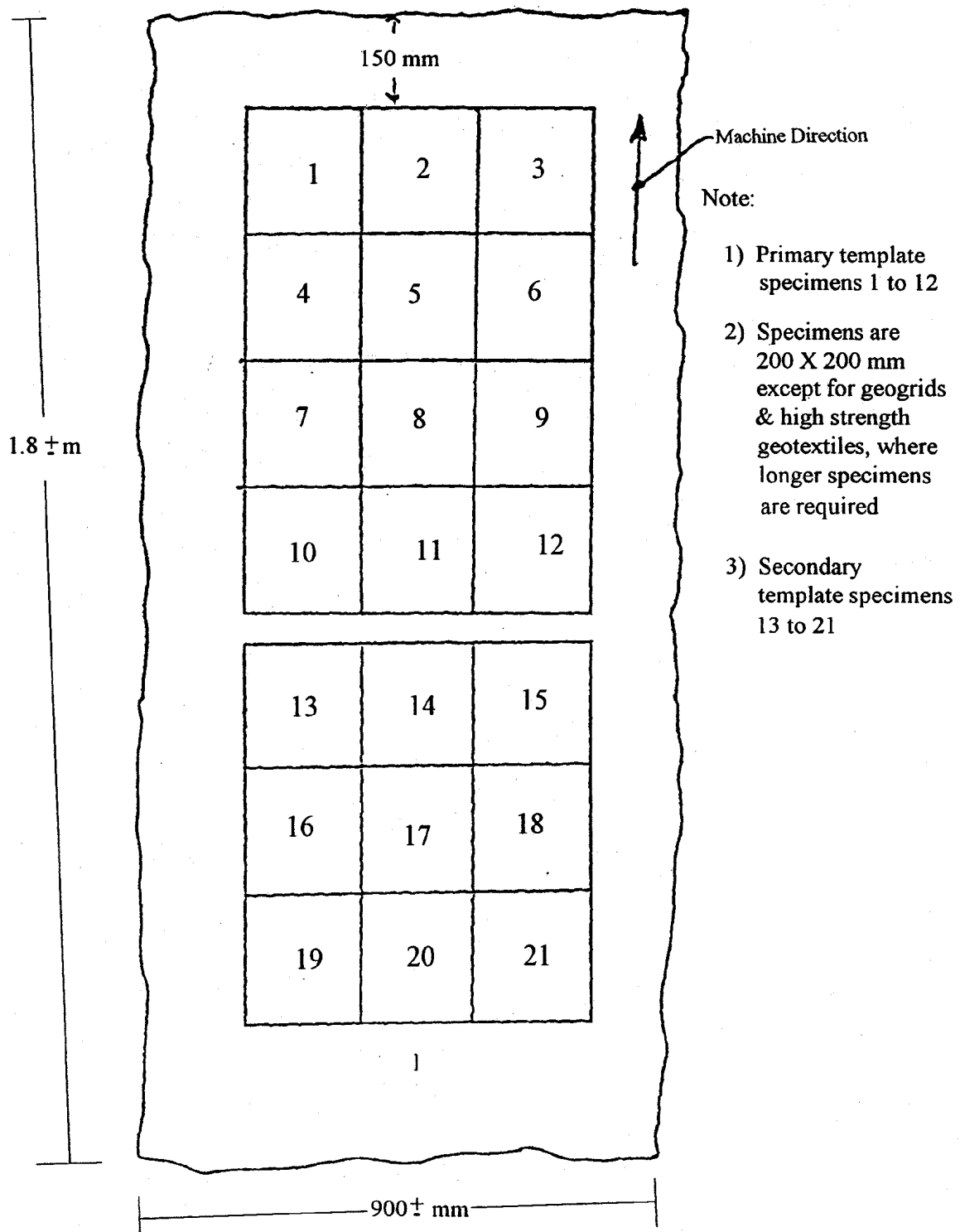


Figure 18. Scheme for sampling test specimens.

Prior to wide-width tensile testing, mass-per-unit area should be determined from specimens marked 9 through 12 in accordance with ASTM D-5261.

- 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:

Grab Tensile Strength	-	ASTM D-4632
Puncture Strength	-	ASTM D-3787
Trapezoidal Tear Strength	-	ASTM D-4533
Mullen Burst Strength	-	ASTM D-3786

- Evaluate the retained strength on the basis of the average results obtained from the wide-width 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 of 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 where available should be included. 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 have been developed under FHWA *Durability of Geosynthetics for Highway Applications*⁽²⁹⁾. 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 D-4595 in the machine direction. Where strength in previous retrievals has been characterized by the grab tensile test performed in accordance with ASTM D-4632, 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 D-3776, Option C, using at least one specimen retrieved. Thickness may be measured on polyolefin grid products in accordance with ASTM D-5199.
 - Density/Specific Gravity should be measured in accordance with ASTM D-792.
 - 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 to illustrate special conditions.

- 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) and determination of carboxyl end groups (CEG number). A minimum of three tests should be performed. No ASTM standards for these tests are currently available, and industry practice may be used.
- 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 D-4603.
- Thermogravimetric Analysis (TGA) in general accordance with ASTM E-1131, 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-1238 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 E-1131.
- 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 D-5262 for a 1000 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 AASHTO, section 5.8.; 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 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 1 by 2 m surface area. It is recommended that the 2-m length be parallel to the machine direction. High strength geotextile products requiring roller grips for testing will require a longer total sample. 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 600 by 800 mm should be laid out in the machine direction on the recovered sample by locating it 15 cm from the top edge and centering it along the 1 m recovered width. The primary template length should be increased for high strength geotextiles tested with roller grips.

Twelve adjacent specimens 200 by 200 mm should be cut and numbered in accordance with figure 18. 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 geotextiles, the length should be increased to accommodate the roller-grip requirement. For stiff geogrids, the specimen size should contain three ribs in the machine direction and five ribs in the cross direction. For flexible geogrids up to seven cross direction ribs may be necessary to accommodate the required roller clamps.

Wide-width tensile force data obtained from these nine primary tests should be analyzed to determine the coefficient of variation as outlined in ASTM D-4595, 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 (9), additional specimens as available should be secured from a secondary template location also shown on figure 18. No more than a total of 18 specimens per recovered sample should be tested for wide width strength.

Prior to wide-width tensile testing, mass-per-unit area should be determined from specimens marked 9 through 12 in accordance with ASTM D-5261. 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.

- 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 (5 kg) obtained above or below the retrieved geosynthetic specimen:

- Grain size distribution (ASTM D-854)
- Atterberg Limits for fine grained soils (ASTM D-4318)
- pH (AASHTO T-289-91)
- Electrical conductivity or resistivity (AASHTO T-288-91)
- Organic Content (AASHTO T-267)
- Transition metals (EPA SW 6010)
- Chloride, sulfate, carbonate (ASTM-4327)
- 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 19 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.

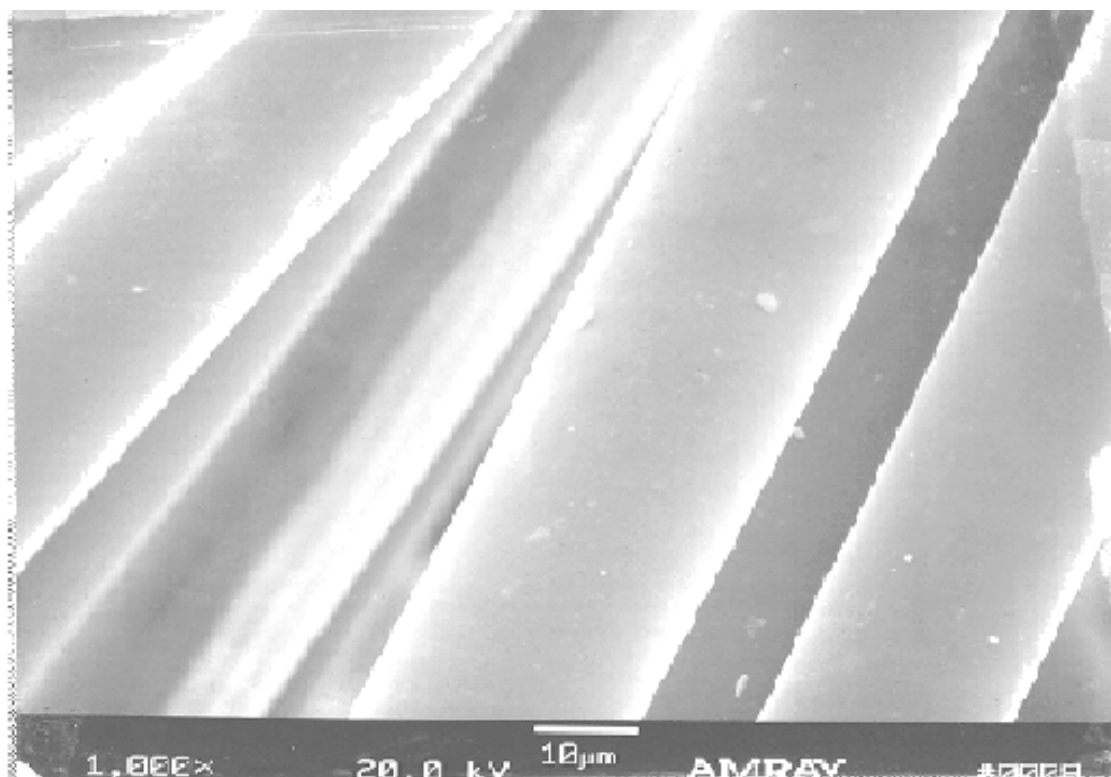
It would therefore appear that confirmation of strength losses measured (due to aging) *can only be obtained from successive* retrievals and may be confirmed by measurable loss of antioxidants, measurable loss of their effectiveness, or both.

If the type of antioxidant is known, the level present at anytime can be measured by High Performance Liquid Chromatography (HPLC) with a ± 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 a poor indicator.

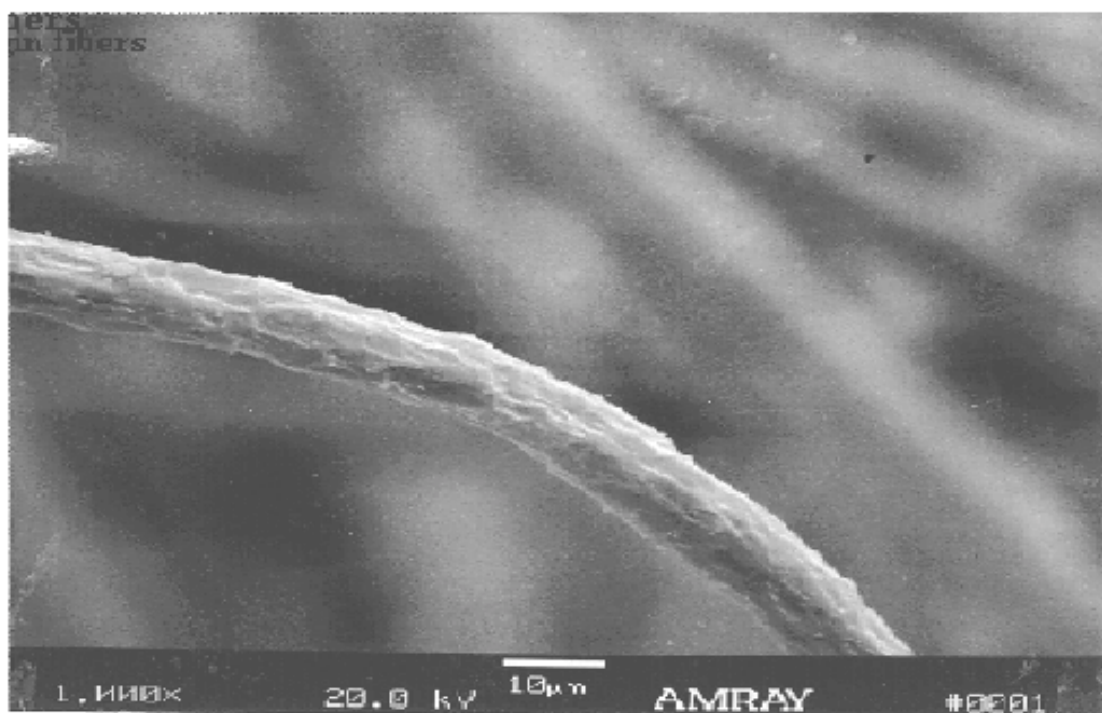
Based on our present knowledge, the decrease of the antioxidant level or OIT time is not evidence of strength loss, but can serve as a confirmation of either leaching or consumption of antioxidants. The complete loss of antioxidants as evidenced by HPLC, or decreases of OIT time greater than 90 percent, 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 20.

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.

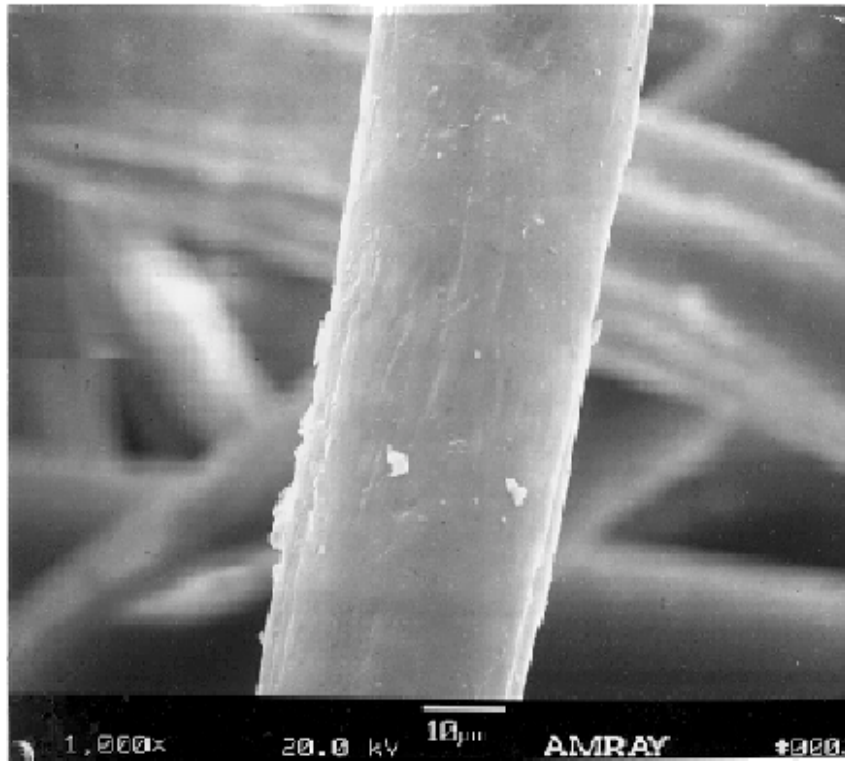


a) Virgin fibers

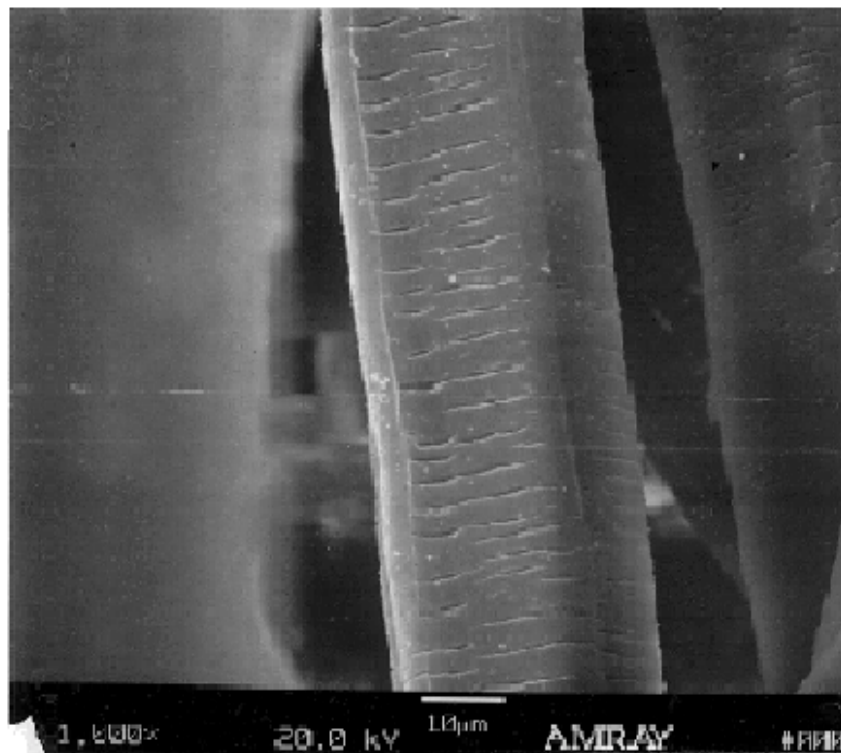


b) Fibers after alkaline hydrolysis

Figure 19. Scanning Electron Microscopy (SEM), polyester fibers.



a) Virgin fibers



b) Oven aged fiber with cracking

Figure 20. Scanning Electron Microscopy (SEM), polypropylene fibers.

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APPENDIX A

RELEVANT CORROSION TEST STANDARDS

American Association of State Highway Officials (AASHTO)

Electrochemical Properties of Soils

Specifications for:

- T-288 Resistivity of Soils
- T-289 pH of Soils
- T-267 Organic Content by Loss on Ignition
- T-291 Determination of Chlorides
- T-290 Determination of Sulfates

American Society for Testing and Materials (ASTM)

Specifications for:

- G-78 Polarization Resistance Measurements

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APPENDIX B

RELEVANT GEOSYNTHETIC TEST STANDARDS

American Society for Testing and Materials

Endurance Properties

Specification for:

- | | |
|--------|--|
| D 4355 | Deterioration of Geotextiles from Exposure to Ultraviolet Light and Water (Xenon-Arc Type Apparatus) |
| D 4594 | Effects of Temperature on Stability of Geotextiles |
| D 5322 | Immersion Procedures for Evaluating the Chemical Resistance of Geosynthetics to Liquids |

Practice for:

- | | |
|--------|--|
| D 5496 | In Situ Immersion Testing of Geosynthetics |
| D 3045 | Heat Aging of Plastics Without Load |

Guide for:

- | | |
|--------|--|
| D 4873 | Identification, Storage, and Handling of Geotextiles |
|--------|--|

Polymer Properties

Specification for:

- | | |
|--------|---|
| D 3418 | Transition Temperatures of Polymers by Thermal Analysis |
| D 1238 | Flow Rates of Thermoplastics by Extrusion Plastometer |
| D 4603 | Determining Inherent Viscosity of PET |

APPENDIX B (continued)

Mechanical Properties

Specification for:

D 4632	Breaking Load and Elongation of Geotextiles (Grab Method)
D 4833	Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products
D 4595	Tensile Properties of Geotextiles by the Wide Strip Method
D 4533	Trapezoid Tearing Strength of Geotextiles

Practices for:

D 4354	Sampling of Geosynthetics for Testing
D 4759	Specification Conformance of Geosynthetics, Determining

Test Method for:

D 5261	Measuring Mass Per Unit Area of Geotextiles
D 5199	Measuring Nominal Thickness of Geotextiles and Geomembranes