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CORROSION SUSCEPTIBILITY OF INTERNALLY REINFORCED SOIL RETAINING STRUCTURES

Research, Development,
and Technology

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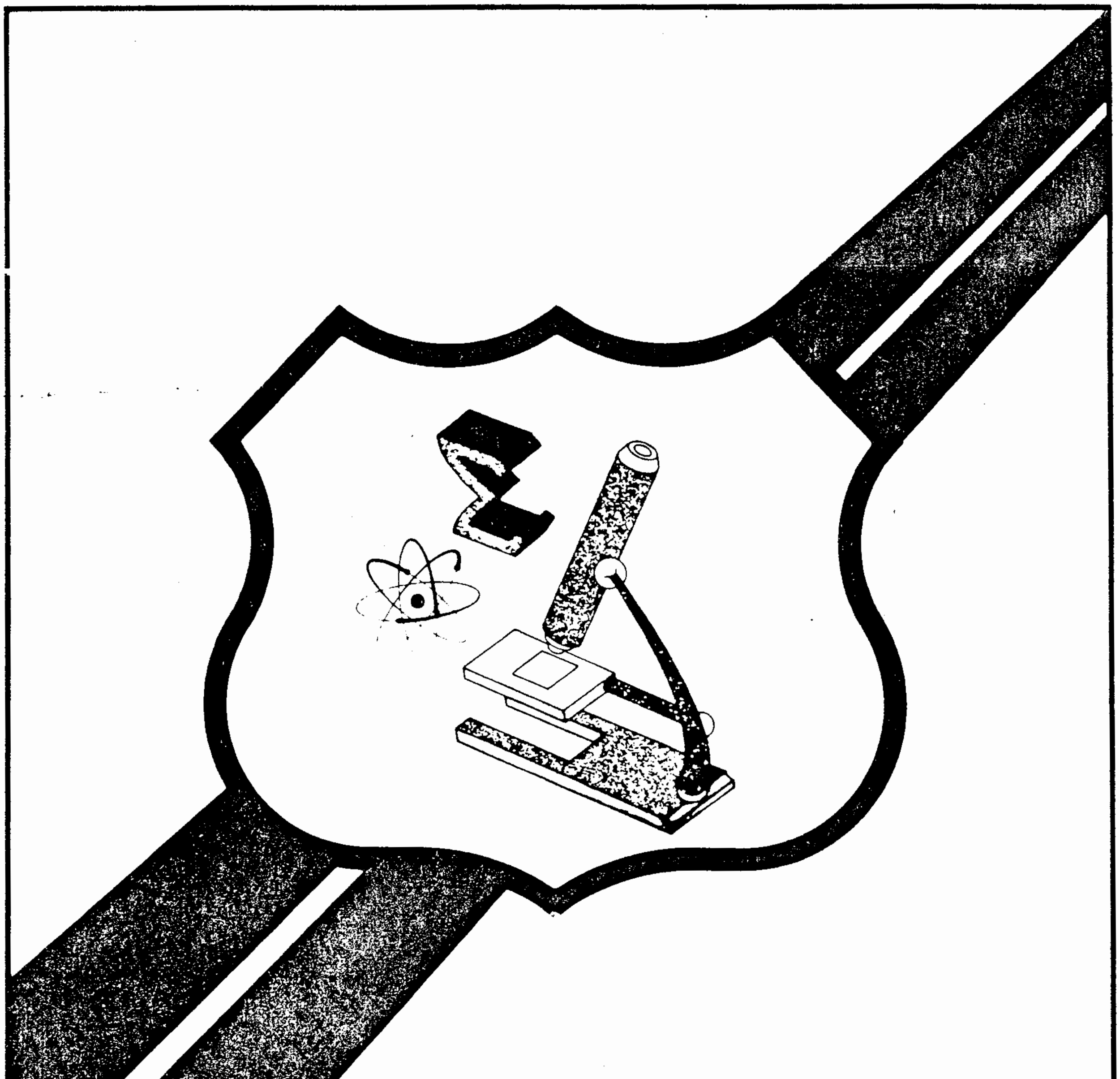

FOREWORD

This report will be of interest to persons concerned with the problem of corrosion of reinforcing metals in internally reinforced soil retaining structures.

The report describes the current state of knowledge in the above area as well as findings of a field study of four Reinforced Earth* walls. It was determined that corrosion problems exist that may reduce the design life of these structures. Further research is recommended in order to assess the magnitude of the above problems and determine the safe limits of the reinforced earth concept.

Richard E. Hay
Director, Office of Engineering
and Highway Operations
Research and Development

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* Reinforced Earth is a trademark of the Reinforced Earth Company and is one type of internally-reinforced soil system.

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16. Abstract

The current state of knowledge in the area of metal corrosion in reinforced soil retaining walls has been assessed. It was found that data is missing in several areas including the performance of the metal reinforcement in a highly alkaline environment as well as in the presence of high chloride concentrations. Moreover, no quantitative information was found on the effect of climatological conditions.

In order to determine whether there are any corrosion problems in completed Reinforced Earth walls, four such walls were selected for study. All four walls are in relatively severe environments. It was determined that two of the above walls may have corrosion problems which could reduce their design life.

It is recommended that further field studies be undertaken in order to assess the magnitude of the problems. It is further recommended that research be conducted to determine the safe limits of the reinforced earth concept.

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Retaining Walls, Reinforced Earth Structures, Corrosion, Soil Reinforced Systems

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METRIC CONVERSION FACTORS

APPROXIMATE CONVERSIONS FROM METRIC MEASURES

LENGTH

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
in	inches	2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km

AREA

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.6	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha

MASS (weight)

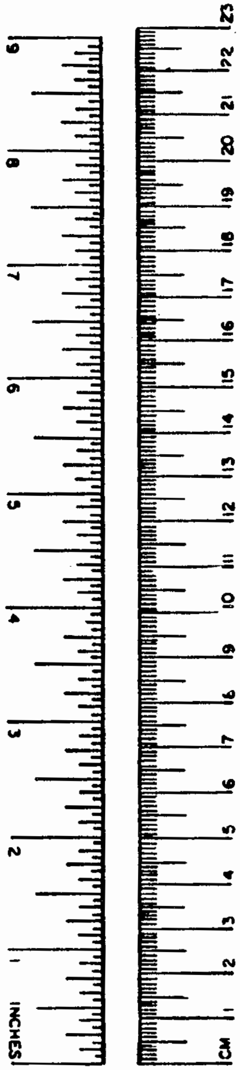
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons(2000 lb)	0.9	tonnes	t

VOLUME

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
tsp	teaspoons	5	milliliters	ml
tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3

TEMPERATURE (exact)

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C



APPROXIMATE CONVERSIONS FROM METRIC MEASURES

LENGTH

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi

AREA

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares(10,000 m^2)	2.5	acres	

MASS (weight)

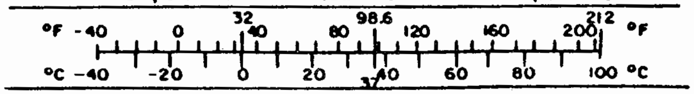
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	

VOLUME

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
ml	milliliters	8.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	36	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3

TEMPERATURE (exact)

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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SUMMARY

In 1966, Henri Vidal, in France, proposed the modern concept of Reinforced Earth using thin flat metal strips placed in controlled directions within earth masses. The above concept was first introduced in the United States in 1969 in an experimental way; it was removed from the Federal Highway Administration experimental status in 1974 and gained tremendous popularity since then. Indeed, by 1979 more than 130 Reinforced Earth retaining walls - the most popular application of the Reinforced Earth concept - had been completed in 26 States, and since then the applications of the Reinforced Earth concept have kept increasing in an exponential mode. At this point the taxpayer invests tens of millions of dollars in this concept every year.

Reinforced Earth retaining walls have a design life of 70 to 120 years. Presently, the oldest structures employing this concept in France are only about 17 years old; in the United States they are only about 12 years old. A few failures attributed to corrosion of the reinforcement have been reported in France and Spain; however, the overwhelming majority of structures show no evidence of impending failure. The industry is confident that the structures will perform throughout their design life.

The objective of this study is to assess the corrosion susceptibility of Reinforced Earth retaining walls.

The scope of the research is limited to Reinforced Earth retaining walls that have a concrete facing. Moreover, the scope of the work is limited to a critical survey of existing knowledge as well as a field survey of four walls. In the four walls, the corrosion of the reinforcing metals embedded in soil and concrete has been assessed.

It was found that gaps in knowledge exist in several areas including the effect of high chloride concentrations and highly alkaline environments. Moreover, data does not exist on the durability of some reinforcements in some environments. Additionally, no quantitative data exists on the effect of the climate.

The four structures that were selected for study were between 11 and 6 years of age. Given the young age of Reinforced Earth walls, care was taken to select structures in relatively severe environments that could produce measurable deterioration.

All four structures were visited and studied through visual inspection of the concrete facing units, delamination survey of concrete, cover surveys of the reinforcement, and electrical potential measurements. In addition, concrete chloride analysis, concrete chloride permeability, concrete resistivity measurements, and soil resistivity studies were also performed. Moreover, soil samples and samples from the reinforcing strips were analyzed for corrosiveness and corrosion, respectively.

Findings include that two of the four selected projects may have corrosion problems and further study of these structures is recommended. One of these projects is in a marine environment and employs a chloride-

resistant aluminum alloy. The other project has a backfill resistivity that is relatively low. Moreover, pH values are probably higher tan 12.5 and responsible for the high corrosion rates.

It is recommended that additional field studies be undertaken in order to assess the magnitude of the problems. It is further recommended that research be conducted that will establish the limits of safe use of the Reinforced Earth concept.

Chapter 1 INTRODUCTION

Soil reinforcement methods are becoming increasingly popular in this country. This popularity is primarily the result of changes that have occurred in our national highway program (1). The emphasis in transportation projects is changing from new construction to upgrading of existing facilities. Moreover, a large percentage of new roadways that are being constructed are in an urban environment and require some modification of an active roadway facility. The above conditions have severely limited the application of many tools traditionally employed as solutions to geotechnical problems. Treatments which require movement of large earth volumes, shifting of roadway alignment, right-of-way acquisition, or extensive construction delays are being seriously questioned.

During recent years soil reinforcement methods such as reinforced earth, stone columns, array of small diameter cast in place piles and deep chemical stabilization have often provided cost-effective solutions under the conditions mentioned above. By far the most popular of the soil reinforced methods has been the Reinforced Earth.

It has long been realized that the mechanical properties of soil can be improved by the inclusion of other materials. This principle has been used by man and animals through the ages. For example, the Chinese built dikes for thousands of years with earth and branches and animals intertwine earth and vegetation to construct habitats and other structures.

Henri Vidal, in 1966, proposed the modern concept of Reinforced Earth using thin flat metal strips placed in controlled directions within earth masses. Reinforced Earth can therefore be thought of as a composite material formed by the association of soil and reinforcing materials.

When Reinforced Earth was first introduced it was used to construct road embankments in France. Since that time it has been used world wide for a variety of purposes such as bridge abutments, marine structures, dikes, discharge hoppers, and foundation slabs.

The Reinforced Earth concept was first introduced in the United States in 1969. Convinced of its potential advantages, in 1970 the Federal Highway Administration initiated a number of demonstration projects. In 1974, as a result of experimental testing and experience acquired during the demonstration programs, Reinforced Earth was removed from experimental status. By 1979 over 200 transportation-related structures had been completed in 26 States.

Of the various applications of the Reinforced Earth principle in the United States, the Reinforced Earth retaining wall has enjoyed, by far, the greatest popularity (2) to the point that the taxpayer presently invests tens of millions of dollars every year in the above type of walls.

Reinforced Earth retaining walls have a design life that is anywhere from 70 to 120 years (3, 4). For this life to be realized the embedded metal should not fail because of any corrosion problems during the life expectancy of the system.

The oldest Reinforced Earth retaining walls are in France and are only 17 years old. The oldest such walls in this country are only 12 years old. A few failures of reinforced walls because of corrosion problems have been reported in France and Spain: a Reinforced Earth wall, built in 1974 in a marine environment, in Cap d'Agde, France, failed of a rapid corrosion that proceeded at an observed rate that was 17 times the design rate (5). This was attributed to use of an aluminum alloy to reinforce the wall that contained too much copper that crystalized. In another incident in Nice, France, corrosion of a Reinforced Earth wall was attributed to use of organic soil as a backfill material. In this case sulphate-inducing bacteria caused rapid corrosion of the metal reinforcement (6). Failure of a Reinforced Earth wall in Spain was due to a truck accident which caused corrosive chemicals to penetrate the wall (7). Yet, at this young age, the overwhelming majority of structures show no signs of corrosion problems. Based on the above rather limited and for the most part happy experience the industry is confident that Reinforced Earth systems will perform throughout their design lives, the above systems are used in more and more demanding applications and contractors are pushing for more relaxed standards for the backfill material as well as more relaxed metal corrosion requirements (less galvanization and less cross section).

The significance that the Reinforced Earth concept has acquired together with its relative "newness" calls for research that will assess the corrosion susceptibility of Reinforced Earth retaining systems.

Chapter 2 COMPONENTS OF REINFORCED EARTH RETAINING WALLS

Retaining structures constructed of Reinforced Earth consist of four elements (8). These are:

1. The reinforcing elements.
2. The connections between the facing units and reinforcing elements.
3. The facing units.
4. The soil fill.

The reinforcing elements provide the soil with a tensile strength which is derived from the frictional forces developed along the length of these elements.

The reinforcements used in the overwhelming majority of cases are made of metals. During the last 10 years, in order to solve corrosion problems, several nonmetallic reinforcing materials have been developed and marketed.

Among these new materials are fiber glass-reinforced plastics (FRP) and textiles. The constituents of FRP are glass fibers oriented in preferential directions within chemically resistant resins. Forerunners of these materials have been used satisfactorily during the last 20 years for underground pipes and tanks in aggressive soils and to contain corrosive chemicals. The glass fibers used in FRP are stiff and have a high strength to weight ratio. Additionally, they are chemically inert, and are claimed stable to creep.

Mallinder (9) has performed accelerated tests on FRP using water close to boiling point for periods of up to 10 years. His results show that even though FRP lose strength when immersed for long periods, their long-term strength after several years of immersion is still adequate. Moreover, biological activity was found to have no effect on FRP. Still, the industry feels that not enough is known about the long-term effects of burial of these materials and their use is presently limited (8, 10).

Demand for textiles made with synthetic fibers for use in Civil Engineering projects has risen rapidly over the past few years. Currently woven or nonwoven materials are available most of which are made of polyester, polyamides, or polyolefins.

Ideally, the materials should be manufactured to resist tension most effectively. The most efficient fabrics, therefore, use high-strength, specifically oriented fibers so that the load is relatively evenly distributed and the corresponding load deformation modulus is high. This can be achieved with woven materials. In nonwovens no particular fiber orientation exists resulting in individual fibers being stressed to different levels which leads to a lower modulus and lower overall strength.

Some researchers (11) have claimed satisfactory performance with synthetic textiles over a period of about 15 years; however, this time period is less than 20 percent of the desirable minimum service life of a structure.

Little is known about the effect of continuous immersion on the load carrying capacities of synthetic fibers. Moreover, the latter do creep at a rate that appears to be independent of the applied load (11). Additionally, these materials are subject to degradation upon exposure to ultraviolet rays (e.g., sunlight). (Polyester is affected to a slightly lesser extent than the other compounds). Therefore, any exposed material should be coated to provide protection. Carbon black pigments protect most fabrics from ultra-violet rays. Gunite and asphalt also have proved successful for this purpose.

Metallic reinforcements used in earthwork reinforcement systems with concrete facing units may be in the form of strips or welded steel bar mats. Of the five systems that currently exist, the Reinforced Earth System employs strips that are usually made of galvanized steel. Originally flat strips (often 3mm thick) were used, but more recently ribbed strips (usually 5mm thick) have generally been employed. The latter not only provide greater corrosion resistance due to the increased thickness but also have improved frictional qualities.

Work done at the California Department of Transportation has shown that welded mats when used as reinforcement can provide a better resistance to pull out (4 to 5 times the resistance of ribbed strips and 7 times that of plain strips) (12). Accordingly, the Department developed a system, the Mechanically Stabilized Embankment System, that employes a welded steel bar mat made of 1/4 to 1/2 in (6 to 13mm) bars in a 6 by 24 in (15 by 61 cm) size mesh (13). The bars are of plain steel. With this welded mat reinforcement highway officials in California feel that they may be able to use lower quality backfill material. On the other hand the cost of the metal reinforcement is higher with the welded steel bar mat.

The Georgia Department of Transportation followed the California lead and developed the Georgia Stabilized Earth System. This, too, employs a welded mat that is made of 3/8 in (10mm) steel bars. Longitudinal bars are 6 in. (15 cm) apart while transverse bars are 24 in. (61 cm) apart. The mat is made of galvanized ASTM A 82 steel.

The Reinforced Soil Embankment System, developed by the Hilfiker Company in California, is similar to the above two systems. It utilizes a wire mat at 6 by 24 in (15 by 61 cm) spaces.

The VSL Corporation Retained Earth System is very similar to the other mat systems. It too is based on welded mat made of ASTM A 82 steel.

Most of the older internally reinforced retaining structures completed in this country use reinforcement in the form of metal strips. The four structures that have been selected for field evalution in this study are Reinforced Earth structures because these are the oldest and therefore would be more likely to show signs of corrosion.

In Reinforced Earth structures, the connections between the facing units and the reinforcing elements are also made of metals. Often the design involves strips of A570 steel that have been galvanized and are embedded a few inches in the soil and a few inches in concrete. In between them and bolted to them through A325 galvanized bolts is an A36 galvanized reinforcing strip that is at least 3.0 m (10.0 ft) long and lies exclusively in the soil.

Different designs have also been used in severe environments to avoid corrosion problems. For instance, where stray currents are a problem, welding (instead of bolted connections) has been used to provide electrical continuity and protect the system. In this case the metal strips are also electrically connected to each other. As another example, in cases where there is a high chloride content, or acidic conditions, epoxy-coated reinforcement has been used. In still other cases, aluminum alloy has been used as reinforcement.

In most cases, the facing units are made of precast reinforced concrete (8). Facing units can also be made from other materials, such as metals. Such systems fall outside the scope of this study. Precast concrete facing units are reinforced with steel bars. The steel bars are needed to provide the panels with adequate strength during fabrication, transportation and erection and also for the purposes of shrinkage and differential temperature. Corrosion of these bars might cause cracking, spalling, and delamination of concrete and eventually the facing unit may need to be replaced.

The soil used in Reinforced Earth structures should be essentially granular to ensure adequate frictional properties.

The three areas of corrosion risk in the backfill are shown in Figure 1 (14). Various methods to reduce the corrosion risk in area 1 - zone of greatest risk - have been proposed. These include plastic sheeting over the structure, a clay cap over the non-cohesive fill, and provision for extensive drainage systems. Failure of the drainage or protective system will result in very localized channeling of water into the structure with the risk of severe localized corrosion attack.

In addition to what has been discussed here, several properties of the backfill will determine its corrosivity. These are discussed in Chapter 3 of this report.

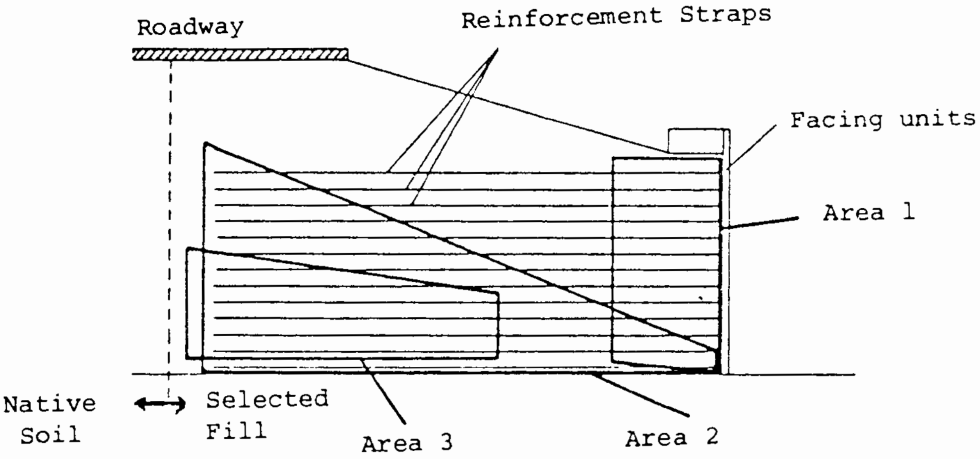


Figure 1. Areas of corrosion risk in structures of noncohesive fill. (14)

Area 1 — Area of less compacted soil, and zone of attachment of facings to straps. Road run-off, rain on facing units, water channelling from damaged drain systems will cause repeated wetting and drying of this zone.

Area 2 — Drainage area of natural soil water from the hinterland. Soil water movement will bring salts into the fill and lower resistivity. Corrosive biogenic sulphide may be brought through here if the native soil is affected by bacteria.

Area 3 — Area most probably saturated with water and stagnant and most likely to suffer microbiological corrosion.

Chapter 3 CORROSION OF REINFORCING ELEMENTS IN REINFORCED EARTH RETAINING WALLS: CURRENT STATE OF KNOWLEDGE; GAPS IN KNOWLEDGE

3.1 Nature and Mechanisms of Corrosion of Metals

Corrosion may be defined as the deterioration of a metal due to chemical interaction with its environment. The reaction may be brought about by direct chemical attack, but more commonly, it is electrochemical in nature. In the latter case, to initiate corrosion the fundamental requirement is the existence of a potential difference between two points, the anode and the cathode, that are electrically connected in the presence of an electrolyte. In reinforced soil situations, the electrolyte will typically be water containing certain concentrations of dissolved salts and metals. In the case of reinforced concrete the electrolyte will be the water and other ions in concrete. A complete circuit is commonly referred to as a galvanic cell.

A galvanic cell is set up when two dissimilar electrodes, separated by an electrolyte, are electrically connected by a conductor. A galvanic cell is also set up when two similar metals are separated by dissimilar electrolytes.

The actual mechanisms involved when reinforcement in concrete or soils is subject to corrosive attack is similar to these basic galvanic cells but much more complicated because of the variable characteristics and properties of the different soils and concretes which influence the corrosion mechanism (8,15,16).

The actual reactions that can take place when a metal is buried in soil are covered in References 8, 17, 18, and 19. Similar reactions that can take place when reinforcing steel bars are embedded in concrete are reviewed in Reference 15.

3.2 Corrosion of Metals Embedded in Soil

3.2.1 Soil Properties Related to Corrosive Aggressiveness

The major factors governing corrosivity in a given soil are electrical resistivity, degree of saturation, acidity or alkalinity, content of dissolved salts including depolarizers or inhibitors, porosity, and redox potential.

Resistivity is defined as the inverse of conductivity and is expressed in units of ohms times length (e.g., ohm-cm).

The resistivity of soil is measured by passing a current through the soil and measuring the voltage drop along the line of current flow. It can be measured most reliably (where reliability is regarded as a measure of the scatter of the result) in noncohesive soils (20).

The risk of metal corrosion increases as the soil resistivity decreases and could, in extreme cases, lead to current flow over distances of 30 m (98 ft) or more. However, situations as critical as this are unlikely to occur within a properly designed reinforced soil structure.

Some soil resistivities measured by Romanoff are reported by Elias in Figure 2 (16). It can be seen from the latter that a well-defined relationship between weight loss and resistivity does not exist. However, a trend of smaller metal losses with increased resistivity is clearly defined especially in resistivity ranges greater than 10,000 ohm-cm. In line with the above, it is suggested in Reference 22 that a soil with a resistivity of more than 5,000 ohm-cm is noncorrosive. The above figure is only 4,000 ohm-cm in Reference 8.

It is recommended by the French Highway Administration that backfill resistivities be greater than 1,000 ohm-cm for structures outside the water and more than 3,000 ohm-cm for structures in soft water (3,6). Indeed, it is reported in Reference 23 that typically the resistivity of soils used in French Reinforced Earth projects varies between 1,000 and 20,000 ohm-cm. Similarly, Elias reports (16) that about 98 percent of all the nonmarine structures built in this country were built in sites with a resistivity of more than 1,000 ohm-cm. Even so, there are 2 percent of the nonmarine structures that are built in sites that have resistivities that are less than 1,000 ohm-cm; moreover, resistivity varies from location to location within the same site and also varies as a function of water content, salt concentration, etc., as discussed below. Therefore, it is possible that low resistivities can be found at times even in structures that have been assumed to have resistivities of at least 1,000 ohm-cm.

Various factors affect the magnitude of measured resistivity including water content of the fill material (and thus porosity and saturation), water composition (pH and dissolved salts), and the degree of compaction.

The water-content affects resistivity (24,25). Resistivity decreases as the water content increases and reaches a minimum value at 100 percent saturation.

Maximum general corrosion occurs in the range of 30-60 percent saturation. Pitting predominates below 30 percent and above 60 percent saturation. Above 60 percent saturation aeration decreases because void paths are being filled making circulation increasingly difficult. The aggressiveness of material with less than 5 percent water content is low, dry soil being noncorrosive.

Water composition, particularly pH and dissolved salts, significantly affects soil corrosiveness.

The soil pH has a variable effect depending on the solubility of metals and their oxides in acidic and alkaline conditions. Metal oxides are normally insoluble and inhibit corrosion, until a pH is reached at which they are soluble; at this point the corrosion rate will increase.

In general, the most corrosive soils contain large concentrations of soluble salts, especially in the form of sulphates, chlorides, and bicarbonates and may have very acid or highly alkaline pH values.

It is suggested in Reference 8 that all reinforcing materials will perform satisfactorily in a neutral pH. On the other hand galvanized steel and aluminum alloys will suffer in a highly alkaline environment while galvanized, low-alloy and copper steel, as well as aluminum alloys and copper, will suffer in an acidic environment (8).

It is recommended by the French Highway Administration (3,6) that the pH of the backfill be between 5 and 10, that chloride ion concentration remains less than 200 ppm, and that sulphate ion concentration remains below the level of 1000 ppm. Histograms of measured pH, chloride, and sulphate values in French soils (26) suggest that the above recommendations have in general been followed.

Similarly, Elias suggests (16) that of all the nonmarine projects in this country, more than 96 percent have a pH in the range of 5 to 9.5, chloride concentrations of less than 200 ppm, and sulphate concentrations of less than 1000 ppm.

Romanoff, in a National Bureau of Standards (NBS) study, has measured values of soil pH, as well as their relationship to metal loss, in several United States sites. These values are reported by Elias in Figure 3 (16). It can be seen from this figure that a well-defined relationship between metal loss and pH does not exist. It can also be seen that no sites have been selected in the above study that have a pH of more than 9.5. Moreover, it can be seen in Reference 26 that there is a paucity of French data on sites with a pH that is greater than 9.

Similarly, Reference 26 indicates that very little French data exist on sites with high chloride concentrations. A reason for this is that Europeans are not using deicing salts at the extent that they are used in this country. It follows that data on the effect of high chloride concentation on the corrosion of buried metals needs to be produced in this country. Some work on diffusion and distribution of deicing salts through soils (27-35) will be useful to evaluate corrosion rates of reinforcing elements.

Differences in soil compaction along the reinforcing strips will promote corrosion. Only light methods of compaction can be applied to reinforced soil close to the facing units (1-2 m·(2.8-5.7 ft.)) while further from the face the use of heavy equipment is acceptable. This results in a zone of material close to the wall that is less compact than the rest of the fill.

Redox potential is a measure of the oxidation or reduction characteristics of a soil. Probes are inserted into the soil and the potential measured between them is compared to a reference value (36). This test is of most use in cohesive soils. (8)

Rate of corrosion. In 1910 the NBS initiated a series of tests of ferrous and non-ferrous metals buried in different soils. The results of these experiments, which lasted until 1955 and were published by Romanoff in 1957 (21), provide the most comprehensive data available on underground corrosion. As has already been discussed, some of these results are being reported by Elias in Figures 2 and 3.

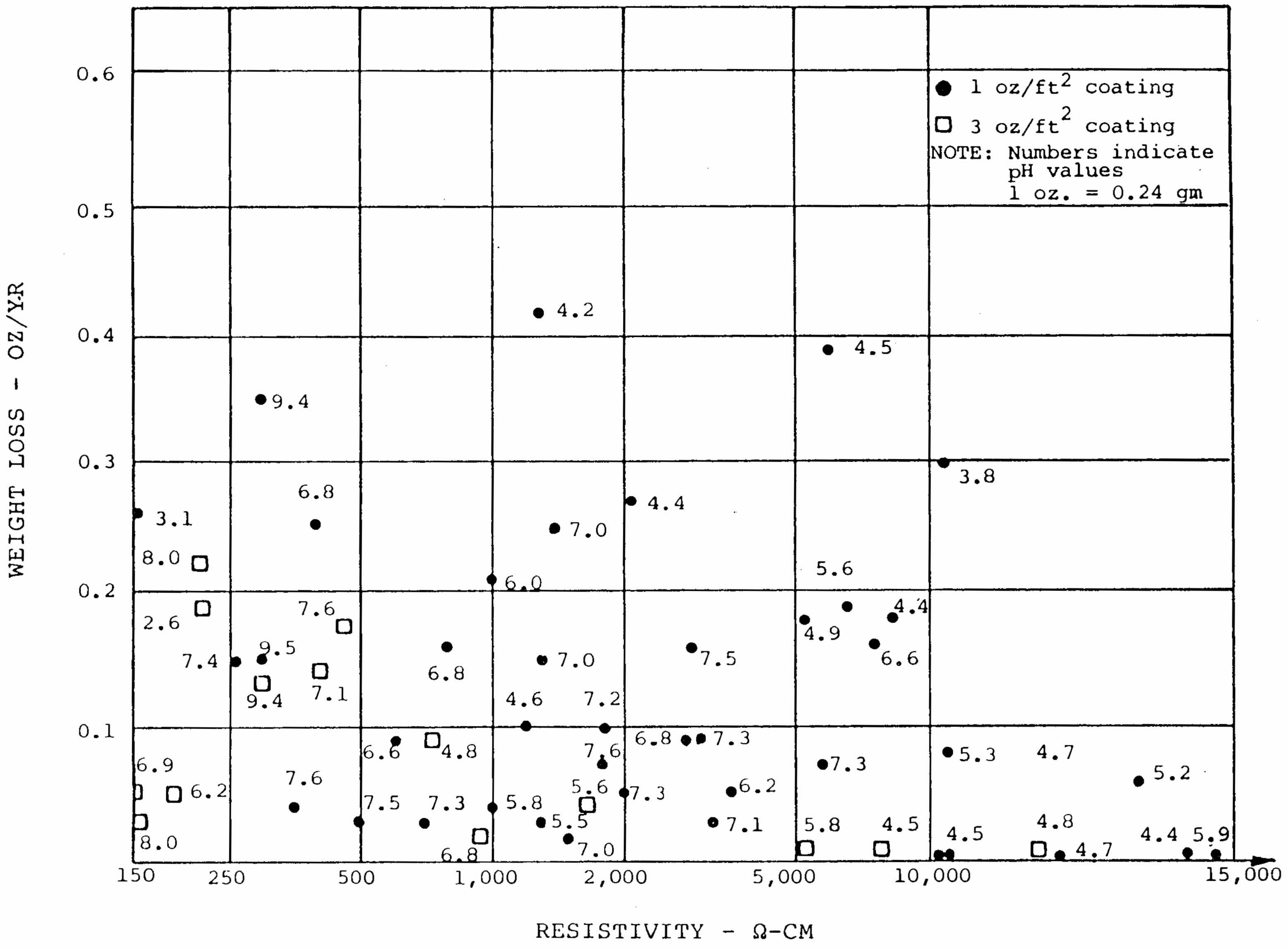
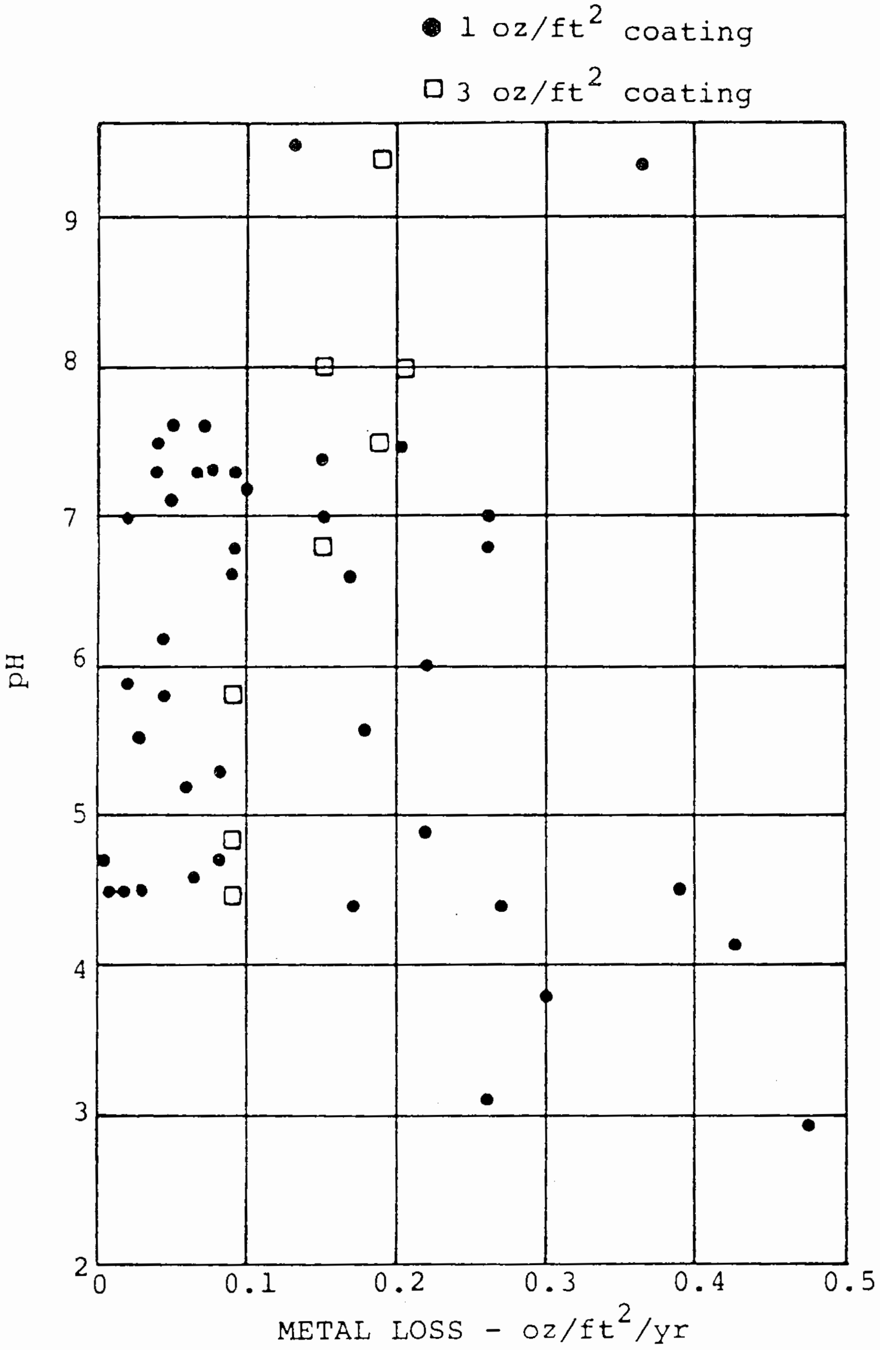


Figure 2. Metal loss as a function of resistivities as measured by Romanoff and reported by Elias (16)



NOTE: Data is plotted on the assumption that metal loss on galvanized samples can be extrapolated linearly from 10 year data.

Figure 3. Metal loss as a function of pH as measured by Romanoff and reported by Elias (16)

Additional available 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, is substantially in good agreement with the burial tests conducted by NBS (21).

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 the above, Elias (16) suggests the following scenario: Bare steel corrodes rather fast (at a rate V_1) during the first years of exposure. Galvanized steel also corrodes rather fast though at a lesser rate than bare steel, during the first years of exposure (rate V_1'). After that, corrosion rate is reduced to V_2 for bare steel and a lower V_2' for galvanized steel. Once zinc is consumed, the bare metal will corrode at a rate V_2.

The above scenario is clearly plausible and might even describe a typical case. Yet, there are several cases that are not covered by the above. For instance, in a highly alkaline environment the zinc will corrode at a rapid rate which will not decrease with time. On the other hand, in the alkaline environment the bare metal will corrode at a moderate rate. Additionally, the above model is not applicable to some reinforcing materials. As an example, aluminum alloys tend to show localized, as opposed to generalized corrosion, in which case the corrosion rate will not decrease after the first years of exposure (37). Moreover, there is not enough information on the long-term performance of materials, such as epoxy coated metals, that are now occasionally being used in Reinforced Earth structures.

Romanoff (25) suggested that the following exponential equation can be used to predict the amount of general corrosion in bare and galvanized steel at some time (t) after burial:

$$x = kt^n$$

where x is the average loss of thickness in μm at time (t) and k and n are constants that are soil and site dependent (n is always less than unity).

The above equation can also be used to calculate the depth of pit(x) at a specific time (t). The constants k and n will have different values.

The selection of k and n is subjective; however, for well-drained soils and ferrous metals considering general corrosion a value of $n = 0.15$ is normally selected. In poorly drained soils and ones of high microbiological content, n varies and a value nearer to unity is more appropriate. Typical values for pitting calculations are n = 0.18, 0.35, 0.48, and 0.68 for soils with good, fair, poor, and very poor aeration characteristics, respectively.

Values of x for $n = 0.2$ and $n = 0.5$ have been plotted in Figure 4 (16). On the same figure the above values can be compared to actual loss of thickness of galvanized strips buried in non-marine soils with resistivities that are more than 1,000 ohm-cm, pH in the range of 5 to 9.5, chloride concentrations that are less than 200 ppm, and sulphate concentrations that are less than 1,000 ppm.

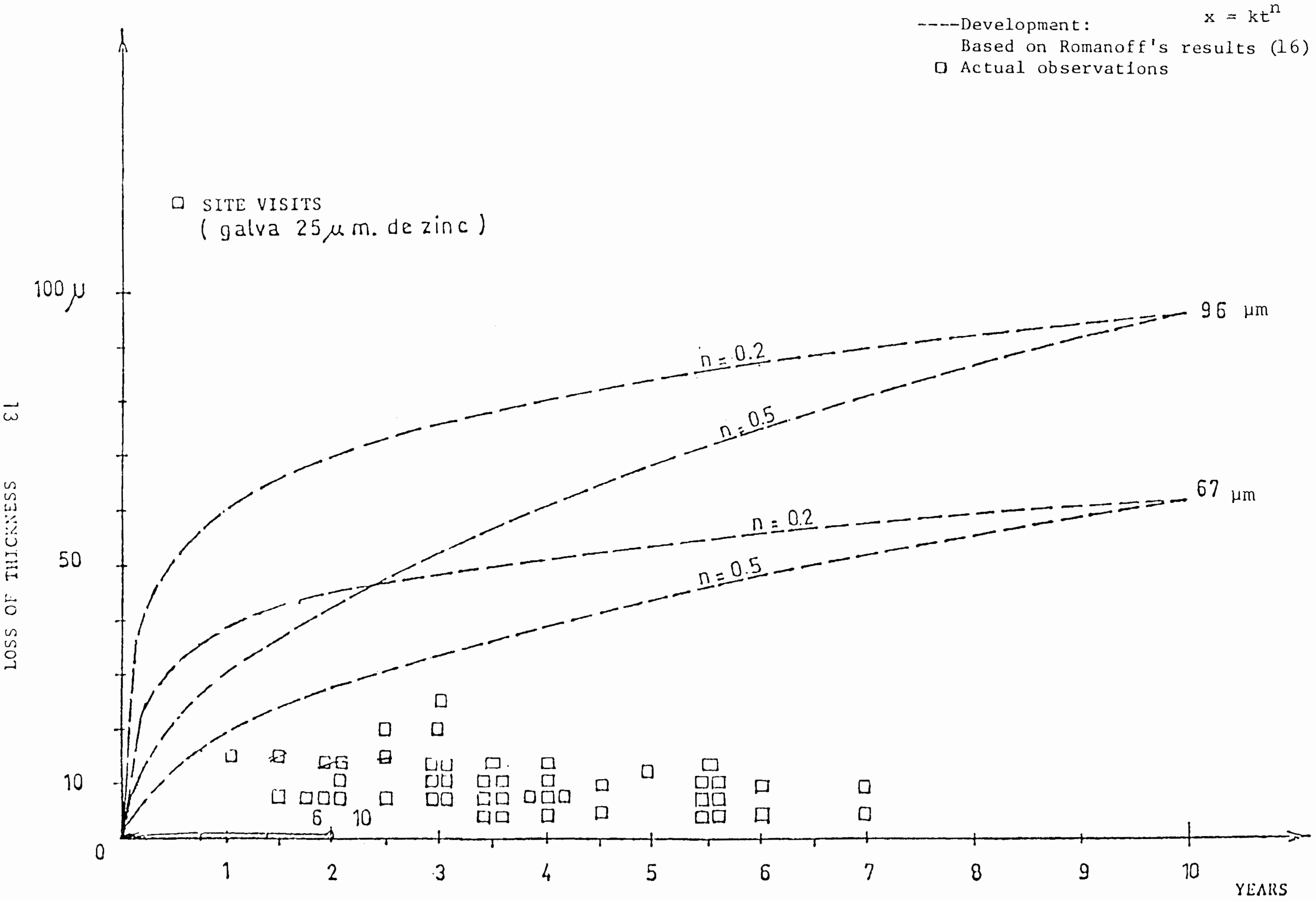


Figure 4. Galvanized steel-loss of thickness with time (11)

The studied galvanized strips are buried for no more than 7 years and are still covered with zinc coating; therefore, they can provide no data for actual loss of bare metal. Data in Figure 4 show Romanoff's equation to be conservative. According to Elias (16) one reason for this is that Romanoff worked primarily with fine-grained soils of poor aeration characteristics which are more aggressive than the coarse-grained or well-aerated soils that are typically used in reinforced soil structures. Indeed Romanoff's findings suggest an average metal loss of 0.39 $gm/m^2/year$ (0.15 $oz/ft^2/year$) with a maximum of 0.65 $gm/m^2/year$ (0.25 $oz/ft^2/year$). Elias suggests (11) that in well-aerated soils the average rate is 0.21 $gm/m^2/year$ (0.08 $oz/ft^2/year$) with a maximum rate of 0.39 $gm/m^2/year$ (0.15 $oz/ft^2/year$).

Elias (16) suggests that the following are conservative assumptions on corrosion rates that the industry can use in the design of a typical non-marine Reinforced Earth wall:

Anticipated Zinc Losses

Initial 2 years: $V_1 = 6\ \mu m/year$

After the first 2 years: $V_2 = 2\ \mu m/year$

Anticipated Steel Losses

Initial 3 years: $V_1 = 45\ \mu m/year$

After the first 3 years: $V_2 = 9\ \mu m/year$

Elias suggests (16) that 2,222 μm (2.2 mm) of thickness is necessary to carry the design load. If 5 mm strips have been used, and these are coated with 86 μm of zinc (ASTM A-123), then the design life of the structure will be as follows:

5,000 μm initial thickness - 2,222 μm required final thickness = 2,778 μm available for sacrificial losses

At a rate of 9 μm loss per year the above will yield a life of $\frac{2,778}{2 \times 9} = 154$ years.

To the above, 39 years of useful life should be added. This is the life of the zinc coating. Therefore, the total life of the structure is 154 + 39 = 193 years.

However, until recently, Reinforced Earth walls were built with 3 (as opposed to 5) mm thick metal strips. Therefore, they only have 3,000 μm - 2,222 m = 778 μm available for sacrificial losses.

At a rate of 9 μm loss per year the life of the bare metal will be only $\frac{778}{2x9} = 43$ years (as opposed to 154 years estimated above).

Additionally, the above calculations are only useful in the case of general corrosion. In the case of perforations, stress concentrations will be formed that will disproportionately decrease the load carrying ability of the metal strip.

The above losses are suggested for a typical nonmarine structure. In a saline environment the anticipated maximum loss rates are as follows:

For zinc:

V_1 = 17 μm/year (first 3 years)

V_2 = 2 μm/year (after the first 3 years)

For carbon steel:

V_1 = 80 μm/year (first 2 years)

V_2 = 12 μm/year (after the first 2 years)

Data in Reference 38 suggests that sometimes losses in marine environments might exceed the above values.

Additional research on corrosion in marine environments is summarized in Reference 39 which concludes that corrosion in marine environments is mainly influenced by the type of exposure, temperature, bacteria and actual chemical composition of the metal, and that corrosion in the splash zone occurs up to ten times faster than in the submerged condition. The above work concludes that on the average, corrosion of metals in sub soil areas is approximately one-fifth of corrosion in the splash zone.

At the present time the Reinforced Earth Company in the United States and the Laboratoire Central des Ponts et Chaussees in France are conducting laboratory testing and field studies of completed Reinforced Earth structures to check the design principles used (23). Much of this ongoing research was initially focused on determining the effect of chlorides and sulphates on the corrosion rate of buried galvanized strips. Preliminary results indicate that chlorides in concentrations of up to 200 parts per million and sulphates up to 1000 parts per million have no significant effect on corrosion rates (16).

3.2.2 Additional Factors Influencing the Rate of Corrosion

Additional factors that will affect the corrosion of the buried metals include climatological conditions.

In general the higher the temperatures the higher the rate of corrosion (40, 41). Therefore, corrosion rates will proceed faster in a warm climate. Moreover, corrosion rates will proceed faster in a humid environment.

At this point more actual data is needed on the effect of the above in a quantitative way (19).

3.3 Corrosion of Steel Reinforcement Embedded in Concrete

Corrosion of reinforced concrete occurs in a variety of different types of structures notably in marine structures, building facades, parking garages, underground pipelines, and bridge decks. In the United States and Canada, corrosion in bridge decks is the most publicized of the above problems. Its cause is use of deicing salts (chlorides) as part of highway winter maintenance programs.

The literature in the area of reinforced concrete corrosion is voluminous. Studies in the published literature include corrosion variables associated with concrete (42, 43), factors associated with steel (44, 45), protection methods (46, 47), corrosion measurements (48), special problems (49), and corrosion under special concrete applications (50, 51). In spite of the above, to the author's knowledge, there is no published work in the area of corrosion of reinforced concrete walls in Reinforced Earth retaining systems.

CHAPTER 4 FIELD SURVEY

4.1 Introduction

In order to assess whether any corrosion problems exist in Reinforced Earth retaining structures, four walls were selected for study. Given the recent age of Reinforced Earth walls, effort was made to identify structures that are in relatively severe environments so that measurable deterioration might exist.

The number of studied structures is not enough to provide a statistical proof of the existence or absence of corrosion problems. Rather, the purpose of this study is to assess whether there is any indication of significant corrosion and whether further research on this is warranted.

To identify structures that were good candidates for this study, the industry as well as highway departments were consulted. Lists were received from the industry on the oldest structures in the most severe environments. Transportation agencies in all States that have such projects were contacted, and reports were received on the age and condition. of the structures as well as the environmental severity of the site.

The following four structures were selected for field evaluation:

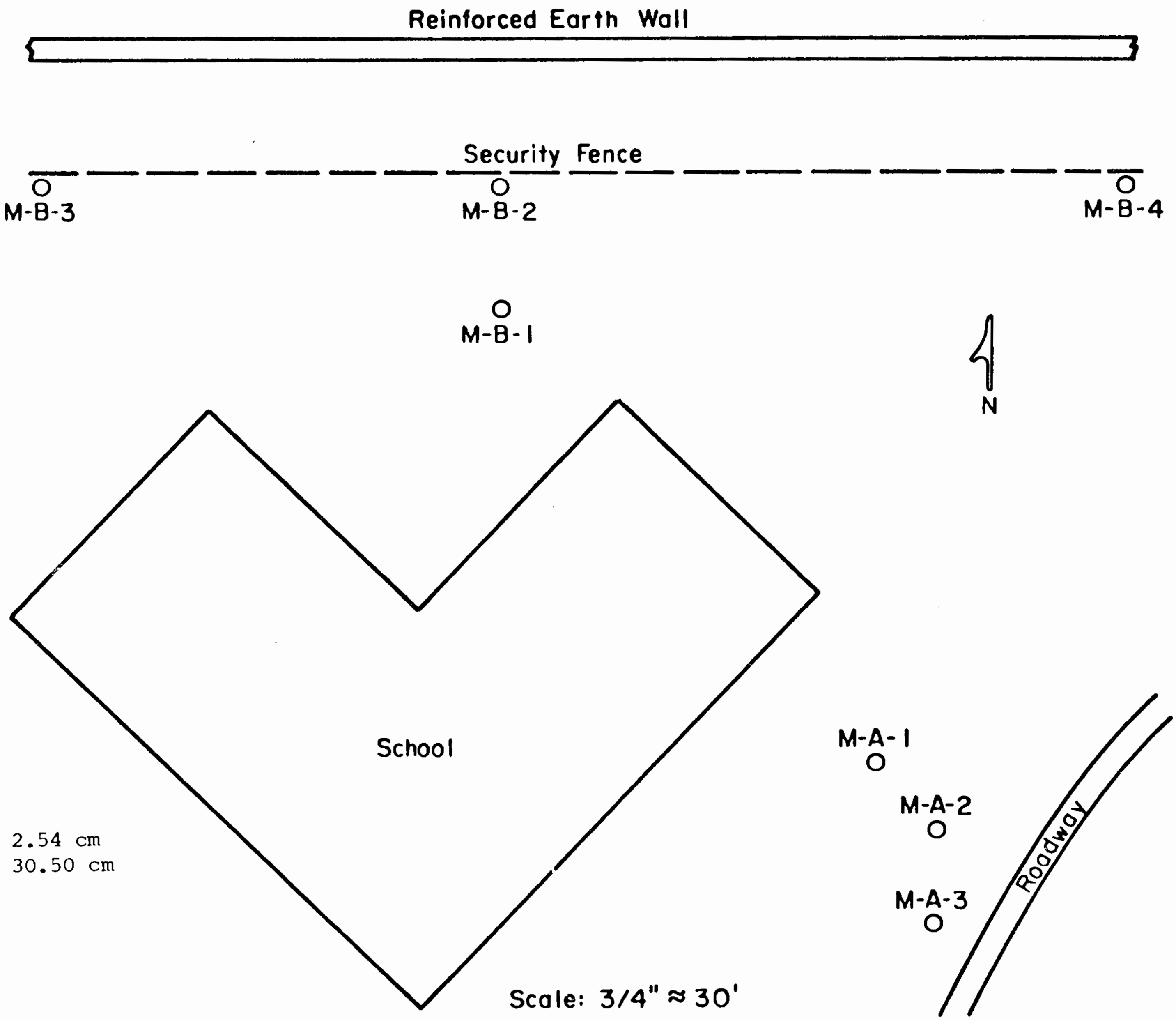
Structure 1 Metropolitan Atlanta Rapid Transit Authority - (MARTA) West Line Retaining Wall C, Atlanta, Georgia. This was selected for study because of the stray currents in the area.

The structure is approximately 180 m (600 ft) long and 9 m (30 ft) high at its crest. It is built with 3mm (0.121 in) thick galvanized steel metal strips that were electrically connected during construction with No. 4 collector bars. Provisions were made for drainage of possible stray currents through the rails via junction boxes located on each end of the structure. However, examination of these boxes indicated such connections had not yet been made.

Nine panels were chosen for testing. These were the first nine topmost panels at the west end of the wall starting from the first full unit (panel 1).

Additionally, soil samples were taken, and soil properties were measured for both the native soil and the backfill material. Soil sampling locations for the native soil are shown in Figure 5. Depths for each of the samples taken are shown in Table 1. To assess the properties of the backfill material, holes were dug at panels 3, 4, and 5 (M-C and M-D samples in Table 1).

Structure 2 Georgia Department of Transportation - New Castle Street Retaining Wall, Academy Creek, Brunswick, Georgia. This was selected for study because it is in a marine environment.



Note: 1 in = 2.54 cm
1 ft = 30.50 cm

Figure 5. Structure 1: Sketch of area south of MARTA Reinforced Earth wall.

Table 1. Structure 1: Soil Sample Identification.

Code No.	Location[a]	Depth (inches)
M-A-1-1	M-A-1	2 to 4
M-A-1-2		4 to 9
M-A-2-1	M-A-2	2 to 5
M-A-2-2		5 to 9
M-A-2-3		13 to 16
M-A-2-4		20 to 23
M-A-3-1	M-A-3	2 to 5
M-A-3-2		5 to 9
M-A-3-3		13 to 17
M-A-3-4		20 to 22
M-B-1-1	M-B-1	2 to 5
M-B-1-2		5 to 9
M-B-1-3		13 to 16
M-B-1-4		20 to 24
M-B-2-1	M-B-2	2 to 4
M-B-2-2		5 to 9
M-B-2-3		13 to 17
M-B-2-4		20 to 25
M-B-3-1	M-B-3	2 to 4
M-B-3-2		11 to 14
M-B-3-3		22 to 26
M-B-3-4		34 to 38
M-B-4-1	M-B-4	2 to 4
M-B-4-2		11 to 14
M-B-4-3		22 to 26
M-B-4-4		34 to 38
Panel 3[b]	M-C-1[c] M-C-2[c] M-C-3[c]	First layer of strips (approx. 30 in deep)
Panel 4[b]	M-D-1 M-D-2	"
Panel 5[b]	M-D-3 M-D-4	"

NOTE: 1 in = 2.54 cm

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[a] See Figure 5.

[b] There is a concrete gutter at the inner top of the wall. The holes were dug on the opposite side of the concrete gutter from the respective panel.

[c] All samples were at the same depth. However, they were at different horizontal locations.

The New Castle Street retaining wall was constructed in 1974. The structure is approximately 366m (1200 ft) long and 6m (20 ft) high. It is topped with a cast-in-place concrete cap which stabilizes the units and supports a guardrail. The structure is subject to tides which submerge the lower level of the panels twice daily.

One unique feature of this structure is that all metallic elements, including strips in the soil and tie-ins in the panels, were specified to be composed of an aluminum alloy (ASTM B209-5086-H34) chosen for its resistance to marine environments. The only steel in the structure is encased in the concrete cap and in tie bars extending from the upper course of panels into the cast-in-place cap. All panels and strips were found to be electrically isolated from each other, except for those strips sharing a common tie-in. Thus, to make ground connections for potential surveys, each panel had to be drilled separately. In addition, the particular alloy used had only a slight effect on the magnetic field of the "R-meter". Thus, location of tie-ins was tedious and time consuming. Finally, work in the tidal zone was limited due to tidal schedules.

For the above reasons, only two areas (consisting of three panels each) were chosen for detailed study. These are shown in Figures 6 and 7. To denote panel locations, the southernmost panel was marked as No. 1. Thus, the first southernmost panel at the top of the wall is 1-1. The lowest level of accessible panels was the third level. Thus, the lowermost panel at the center of the structure would be denoted 121-3. The two areas chosen were selected to represent an area that always remained above tide (Panels 27-1, 28-1, 29-1) and an area subject to daily tidal inundation (121-3, 122-2, 123-3).

Soil samples were obtained from two locations at the top of the structure by digging holes into the earth in the vicinity of the topmost layer of reinforcing strips. Samples were also obtained through core holes drilled through the face of panels 123-3 (center of the wall) and 192-3 (70 panel locations north of 123). In addition, a sample of sand was taken from an area at the base of the wall subjected to daily inundation. Code number and location data are presented in Table 2.

<u>Structure 3</u> Nevada Department of Transportation IR80-PE-15.6 E-W Reinforced Earth wall, Lovelock, Nevada. The poor quality of the backfill suggested that some corrosion problems might exist.

The Big Meadow Interchange retaining wall (IR80-PE-15.63 E&W) was constructed in 1974. The structure consists of two walls, one on the north and one on the south bridge abutments. Each wall is approximately 60 m (200 ft) long, and 7m (23 ft) high at a point between the east and westbound roadways. A latex paint was used to coat the exposed face of the panels. This was done shortly after construction. Standard 3-mm (0.12 in) thick galvanized strips were used, as well as galvanized tie-ins within the face units. Black steel reinforcing was also included as hoop reinforcement in the panels.

The southeast portion of the wall was chosen for detailed survey of the facing units and strips. This portion was selected because of accessibility of a majority of the units, plus the tendency of traffic to throw road salt toward this location. Deicing salt mixed with sand is applied 5

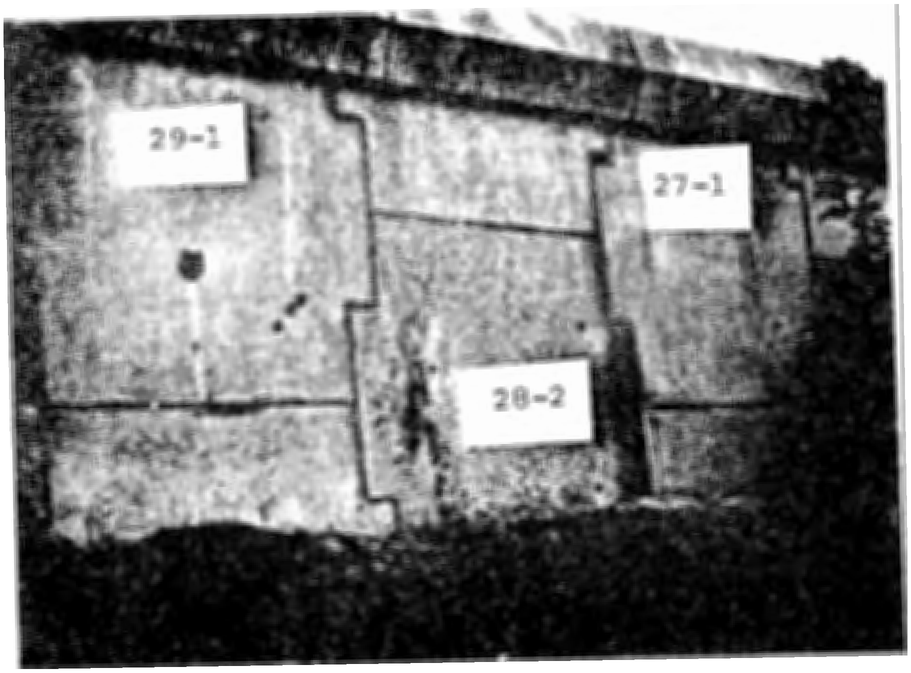

Figure 6. Structure 2: Work location above tidal zone. Panels 27-1, 28-2 and 29-1

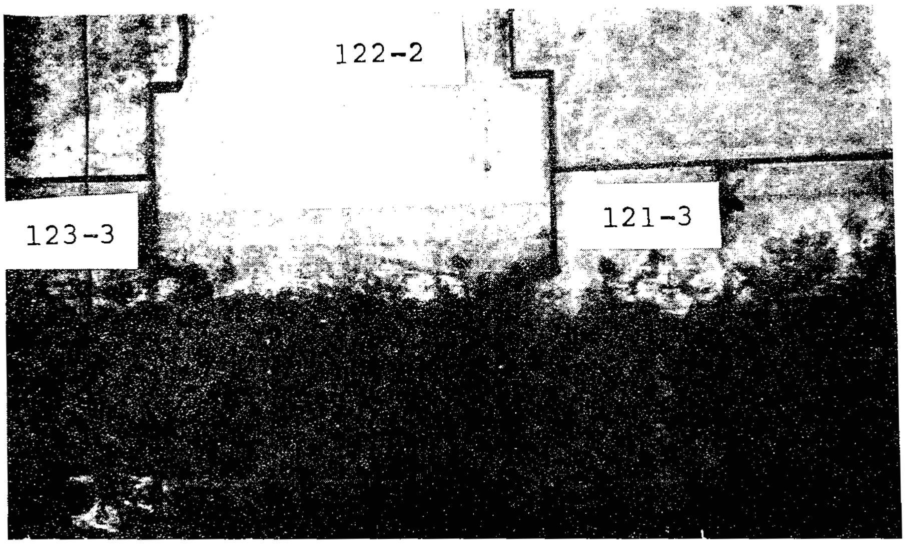

Figure 7. Structure 2: Work location in tidal zone. Panels 121-3, 122-2 and 123-3

Table 2. Structure 2: Soil Sample Identification.

Code No.	Location
B-28-2-T	Panel 28 - at concrete cap - 8 in deep
B-28-2-8[a]	Panel 28 - at concrete cap
B-39 Top[b]	Panel 39 - 25 ft from wall - 18 in deep
B-39 Bot[c]	Panel 39 - 25 ft from wall - 18 in deep
B-Tidal	Sand near Panel 39
Tidal-1	Panel 123-3: At rear face of panel
Tidal-2	Panel 123-3: 2 ft past rear face
192-3-0	Panel 192-3: At rear face of panel
192-3-2	Panel 192-3: 2 ft past rear face
192-3-4	Panel 192-3: 4 ft past rear face

NOTE: 1 in = 2.54 cm

1 ft = 30.50 cm

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[a] Minus No 8 sieve material

[b] Taken from top of exposed reinforcing strip

[c] Taken from bottom of exposed reinforcing strip

to 6 times during an average winter. During the severe winter of 1982-83, deicing salt was applied about 15 times.

The 12 panels chosen for examination are shown in Figures 8, 9, and 10. Panels 1 through 11 were used for electrical potential and other tests. Panel 12 was used only for chloride sampling.

Soil samples were obtained at the seven locations shown in Figure 11. Locations 1, 3, 7, and 11 are immediately adjacent to the panels with corresponding numbers. Location R is located about 0.6m (2 ft) in from the guardrail on eastbound I-80 at the bridge wingwall. Location A is located 4.5 m (15 ft) from the bridge abutment on westbound I-80, 1.2 m (4 ft) away from the wall panels. Location B is the approximate center of the median area, 6.7 m (22 ft) from the wall panels. At locations A and B soil samples were taken around the reinforcing strips, portions of which were also removed. Sample identifications are given in Table 3.

Structure 4 Colorado Department of Transportation I70-2, Reinforced Earth wall, Vail Pass, Colorado. This wall was selected because deicing salts are used in the area.

The wall at Vail, Colorado, was constructed in 1974. It is approximately 244 m (800 ft) long and 9 m (30 ft) above ground level at its highest point. A considerable number of panels lie below finish grade level on the facing side of the wall. Standard 3-mm (0.12 in) thick galvanized steel reinforcing strips were used. Galvanizing was in accordance with ASTM A525 G210 which requires a minimum of 5.5 g/m^2 (2.1 oz/ft^2) total for both sides. This implies a coating thickness of about 42 μm. The metal strips were bolted to galvanized tie-ins within the facing units. Black steel reinforcing was also used in the face units.

The westernmost portion of the wall plus two units at the lowest part of the wall were selected for detailed examination. Considerable amounts of deicing salt are used in this area during the winter, a sand/salt mixture being applied at least daily, and often 2 or 3 times a day. Some of the concrete barriers adjoining the shoulder of the highway have deteriorated badly, presumably by the action of this deicing agent. Some scaling of the topmost level of facing panels was also noted.

Panels chosen for examination are shown in Figures 12 and 13. A total of 9 panels were surveyed. Holes in the panels are the result of drilling and coring operations carried out by the survey team.

Soil samples were obtained at seven locations. Three of the locations correspond to holes dug into the earth immediately adjacent to the wall at panels 1, 2, and 3. Three additional locations were situated 1.5, 3, and 4.5 m (5, 10, and 15 ft) directly up the slope from panel 1. The remaining soil samples were taken at the base of panel 9. Sample identifications are given in Table 4.

Each of the above structures were visited, and the progress of corrosion was assessed for both the metal bars embedded in concrete and the metal strips embedded in the soil. Additionally, the aggressiveness of the soil was also assessed.

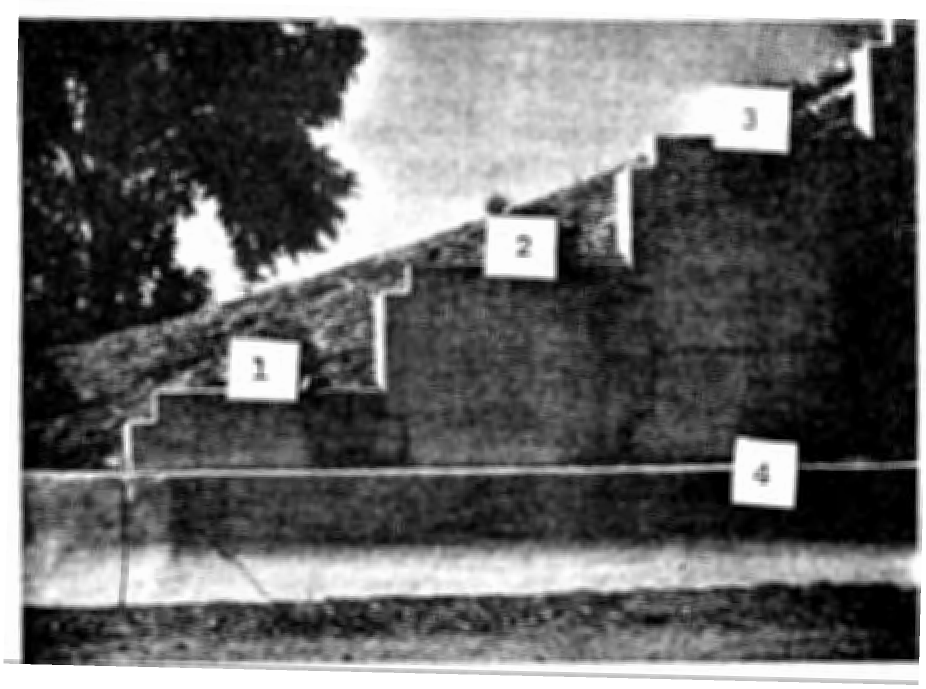


Figure 8. Structure 3: Panels 1, 2, 3,and 4 chosen for study.

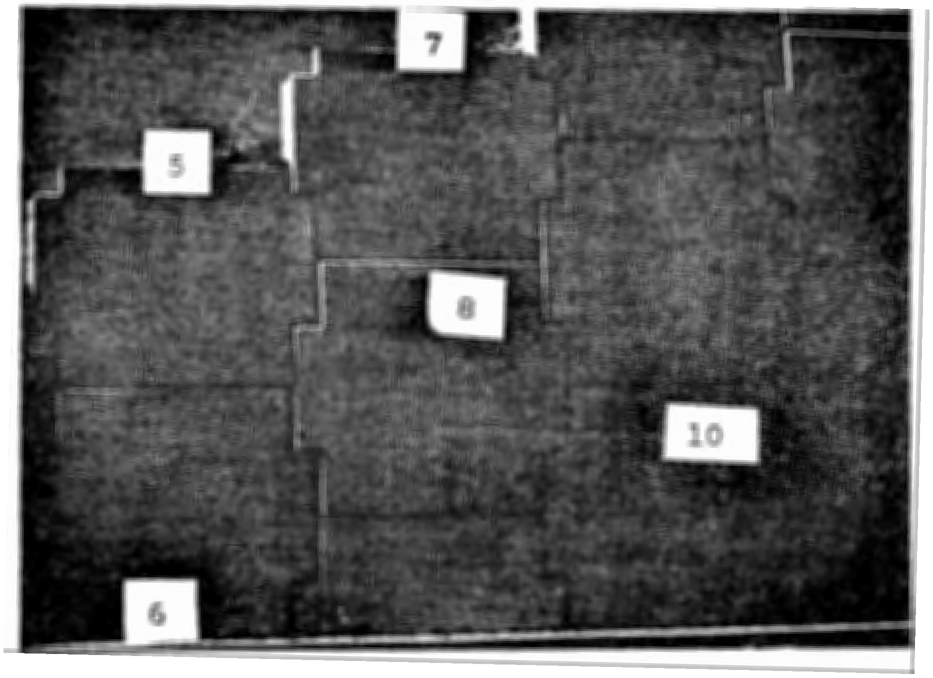


Figure 9. Structure 3: Panels 5, 6, 7, 8, and 10 chosen for study.

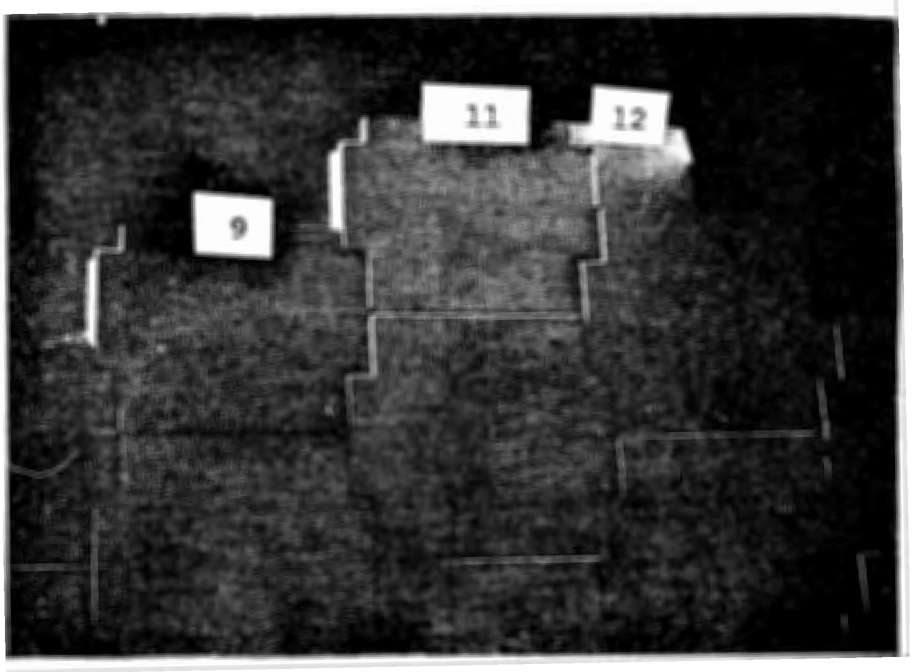
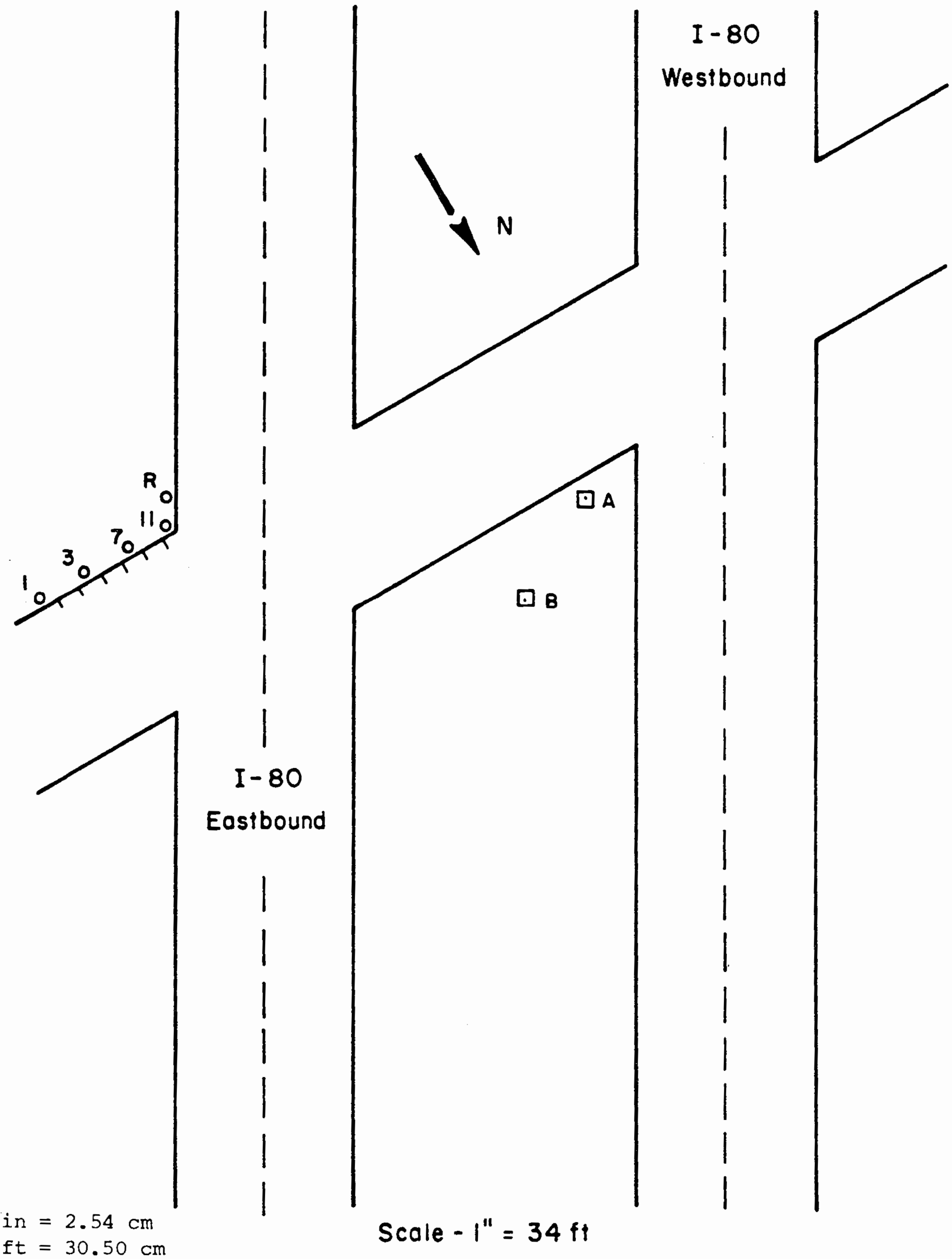


Figure 10. Structure 3: Panels 9, 11, and 12 chosen for study.



NOTE: 1 in = 2.54 cm
1 ft = 30.50 cm

Figure 11. Structure 3: Soil sampling locations.

Table 3. Structure 3: Soil Sample Identification.

Code No.	Location[1]	(inches)
L-1-0	At Panel No. 1	2 to 5
L-1-1	At Panel No. 1	12
L-1-2	At Panel No. 1	24
L-3-0	At Panel No. 3	2 to 5
L-3-1	At Panel No. 3	12
L-3-2	At Panel No. 3	24
L-7-0	At Panel No. 7	2 to 5
L-7-1	At Panel No. 7	12
L-7-2	At Panel No. 7	24
L-11-0	At Panel No. 11	2 to 5
L-11-1	At Panel No. 11	12
L-11-2	At Panel No. 11	24
R	At Bridge Wall	2 to 5 12 24
L-A-1	A	6 to 8
L-B-1	B	6 to 8
L-B-2	B	6 to 8
L-B-3	B	50
L-B-4	B	30
L-B-5	B	Surface

NOTE: 1 in = 2.54 cm

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[1] See Figure 11

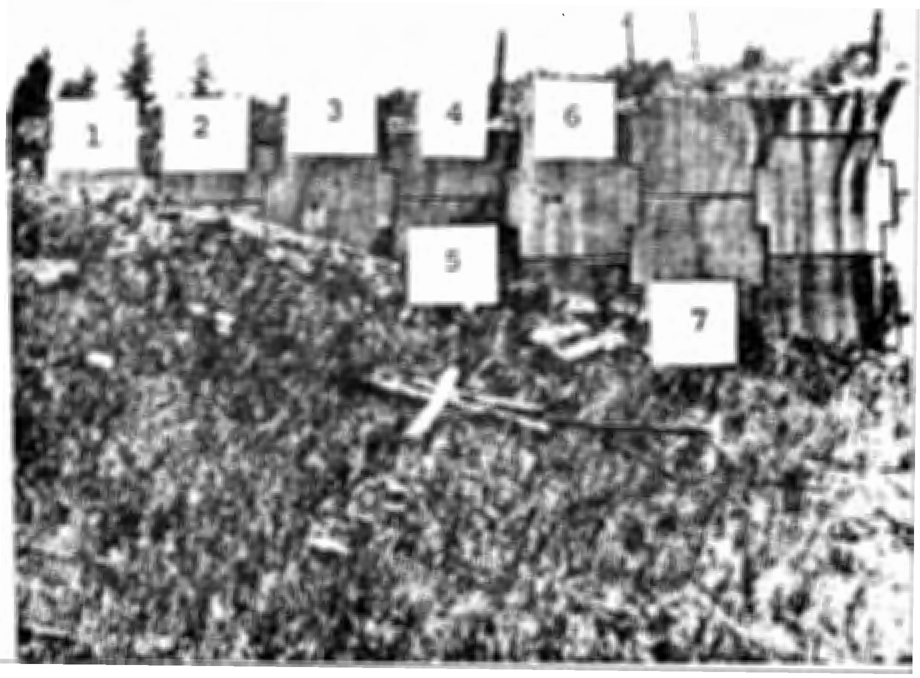

Figure 12. Structure 4: Panels chosen for study on west end of wall.

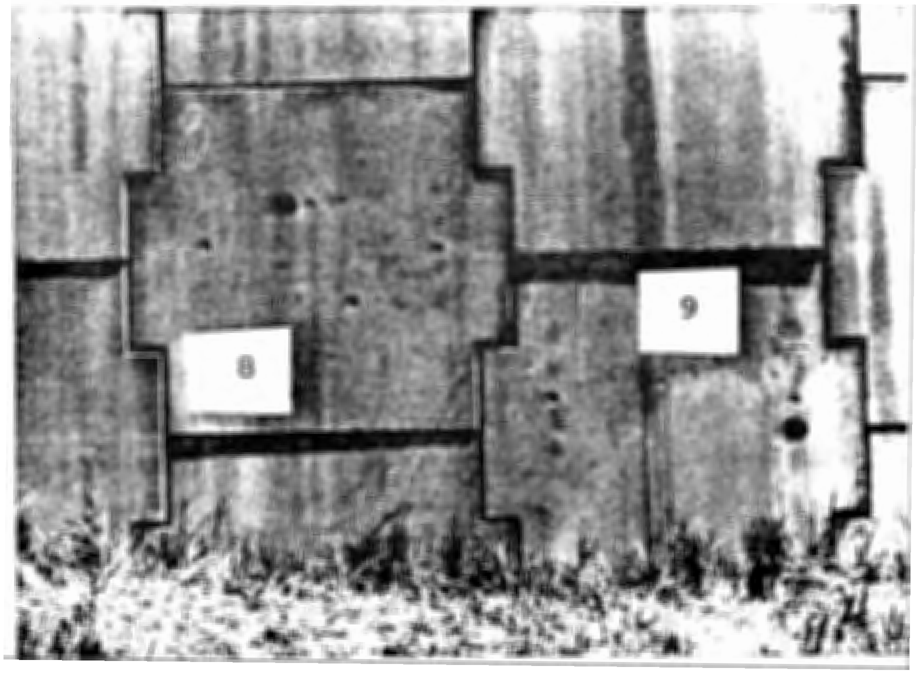

Figure 13. Structure 4: Panels chosen for study at low point on wall.

Table 4. Structure 4: Soil Sample Identification.

Code No.	Location	Depth (inches)
V-1-1	At Panel No. 1	0 to 2
V-1-2	At Panel No. 1	4 to 6
V-1-3	At Panel No. 1	12
V-1-4	At Panel No. 1	24
V-1-5	5 ft up slope from Panel 1	4 to 6
V-1-10	10 ft up slope from Panel 1	4 to 6
V-1-15	15 ft up slope from Panel 1	4 to 6
V-2-1	At Panel No. 2	0 to 2
V-2-2	At Panel No. 2	4 to 6
V-2-3	At Panel No. 2	12
V-2-4	At Panel No. 2	24
V-3	At Panel No. 3	24
V-9-1	At base of Panel No. 9	2 to 5
V-9-2	At base of Panel No. 9	5 to 10

NOTE: 1 in = 2.54 cm
1 ft = 30.50 cm

4.2 Surveys and Test Measurements and Results for Each of the Selected Structures

Following is a description of tests performed and results obtained for each one of the selected structures.

4.2.1 Visual Examination of the Concrete Surface

The concrete facing units were inspected visually for rust staining, scaling, cracking, spalling and exudation of corrosion product from the concrete (52).

Structure 1 All concrete units appeared to be in excellent condition. The only evident damage was some spalling of concrete at points between units. This apparently occurred during positioning and placing of the panels.

Structure 2 Most panels appeared to be in excellent condition. A few examples of surface imperfections and incomplete consolidation were found. In one area, poor consolidation had left part of a strip tie-in exposed to the environment. At this area concrete was broken away, exposing the tie-strap.

Structure 3 All panels appeared to be in excellent condition. Some minor construction damage and misalignment of units were noted. A few stains appeared to be surficial.

Structure 4 The vast majority of panels appeared to be in excellent condition. A few of the panels along the top level of the wall exhibited some surface scaling. Some construction damage, most likely due to impact of panels, was also seen on a very few panels.

4.2.2 Delamination Surveys

These were conducted on selected concrete panels in each of the structures. In these surveys a 0.9 kg (2 lb) hammer was used to manually sound panels for hollow areas (53).

Structure 1 Nine panels were chosen for testing, starting from the first full unit on the west end of the wall. Only the topmost panels were surveyed. No hollow areas typical of those encountered in delaminated concrete were detected. However, a dull tone was noted around the periphery of each panel. This was believed to be due to an edge effect, possibly reflecting reduced rigidity at the panel edges.

Structure 2 A delamination survey was carried out on those panels accessible from the base of the structure along its length. No hollow areas were detected.

Structure 3 A delamination survey was carried out by sounding those panels chosen for examination on the southeast portion of the wall. No obvious hollow areas were detected.

Structure 4 A delamination survey was performed by sounding all accessible panels between the west end of the wall and the location of panel 9. No obvious hollow areas were detected.

4.2.3 Depth of Concrete Cover

This is a nondestructive test to determine the depth of concrete cover on reinforcing bars.

Structure 1 No cover measurements were made because drawings on the location of reinforcing steel were not available.

Structure 2 The cover over tie-straps has been found to range from 5.0 to 2.9 cm (2 to 1-3/4 in). This is not enough to keep sea water from the metal.

Structures 3 and 4 The measured values of the cover over tie-straps and reinforcing bars in each panel are reported in Table 5.

4.2.4 Half-Cell Potential Measurements

This is a standard ASTM C876 test method that measures the electrode potential of steel reinforcing bars in the concrete environment by comparison with the known electrode potential of a reference electrode (half/cell), which, by definition, must maintain a constant value (54, 55). The technique is non-destructive. It is good at detecting corrosion but gives no information about the rate of corrosion. It has been mostly used for bridge decks; retaining walls is a new application.

Structure 1 The fact that all reinforcing strips had been electrically connected to each other and to steel contained within the panels made it possible to conduct an electrical potential survey from the top of the wall. Ground connection fcr the survey was made at the MARTA junction box. A spray bar was then used to wet each panel at locations spaced approximately 30 cm (1 ft) apart across the face of each panel. A copper-copper sulphate electrode (CSE) was placed at each location in order to obtain readings. The porous plug of the electrode was covered with a saturated sponge to improve electrical contact.

Results for all panels surveyed are shown in Table 6. With the exception of panel 1, average potentials for the remaining panels are below the level commonly accepted as being indicative of a corrosive state for steel in concrete (more negative than about 350 mV vs CSE). When MARTA trains passed by the structure, large swings in potential were noted. The swings were as much as 100 mV or more. As panel 1 is closest to the west end of the wall, and this is at a point of maximum current discharge, it is possible that the higher potentials on this panel represent the influence of residual track currents.

Table 5. Cover Over Tie-Straps and Reinforcing Bars.

Structure	Panel No.	Cover (Inches)	
		Tie-Straps	Reinforcing Bars
3	1	1-1/4, 1	2-1/2
	2	1-1/4, 1-1/4	2-3/8, 3
	3	7/8, 7/8	3, 1-3/4
	4	1-1/8, 1-1/8	1-7/8
	5	1, 1	1-5/8
	6	1-1/8, 1-1/8	2-1/4, 2-3/8
	7	3/8, 1/2	1-3/4, 2
	8	1-1/4, 7/8	2, 1-5/8
	9	3/4, 1-1/8	2-1/8
	10	1-5/8, 2-1/4	1, 1-3/8
	11	1, 5/8	2, 2-1/2
4	1	1-7/8	1-3/4
	2	1-7/8, 1-1/2	1-3/4
	3	1-1/2, 1-3/8	2
	4	1-5/8, 1-1/2	1-7/8
	5	2-1/8, 1-3/4	2-5/8
	6	1-3/8, 1-1/4	2-1/4, 1-3/4, 2-3/4
	7	1-1/2, 1-1/4	1-1/2, 1-3/4
	8	2-1/8, 2-5/8	2-5/8, 2-1/2
	9	1-1/4, 1-1/2	1-1/2

NOTE: 1 in = 2.54 cm

Table 6. CSE Potentials.

Structure	Panel No.	Average CSE Potentials (in negative millivolts)		
		Along Tie-Straps	Along Reinforcing Bars	Along a grit[a]
1	1			300
	2			240
	3			230
	4			220
	5			170
	6			150
	7			190
	8			160
	9			110
2	Above Tidal Zone 27-1	770		
	28-2	830		
	29-1	800		
	Tidal Zone 121-3	920		
	122-2	860		
	123-3	870		
3	1	570	60	
	2	580	30	
	3	620	45	
	4	585	50	
	5	590	110	
	6	530	90	
	7	600	30	
	8	545	140	
	9	420	70	
	10	470	40	
	11	440	50	
4	1	810	360	
	2	790	165	
	3	820	320	
	4	760	75	
	5	780	280	
	6	650	170	
	7	740	80	
	8	460	Bar 1: 570 - Bar 2: 35	
	9	470	230	

---

[a] This is an arbitrary grit at 30 cm (1 ft) spacing along the face of each panel

Structure 2 Electrical potentials (vs CSE) were measured along each of the two tie straps in each panel chosen for examination. Results are shown in Table 6. As previous data on potentials of this particular alloy in concrete apparently do not exist, an interpretation of the absolute values of potentials is not possible. It does appear, however, that potentials are lower at the base of each unit and that the highest average potential is recorded for panel 28-2 which is the closest to the tidal level. The average potentials of panels in the tidal zone are measurably higher than those in the above tidal zone area.

Structure 3 Electrical potentials were measured along two vertical tie straps made of galvanized steel and two horizontal carbon steel reinforcing bars in each panel chosen for examination. Results are shown in Table 6.

Average tie strap (galvanized steel) potentials range from close to 600 mV for panels 1 through 8 to 450 mV for panels 9 through 11. These values that, as expected, are higher than the steel values reflect the highly electronegative contribution of the zinc coating on the strips. The lower values (i.e., 400 to 500 mV range) are similar to those recorded by Stark and Perenchio (56) on a number of concrete bridge decks constructed with galvanized steel reinforcing bars. The higher values, though not typically encountered in field installations of galvanized rebars, have been measured by FHWA (41) in laboratory specimens.

Average reinforcing bars potentials are very low, ranging from 30 to 140 mV negative to CSE. This is to be expected, considering the extreme dryness and low chloride content of the panels (see following sections).

Structure 4 Electrical potential measurements were made along vertical tie-straps made of galvanized steel and horizontal reinforcing bars made of carbon steel in each panel. In a number of panels it was difficult to locate reinforcing bars. For this reason, only one bar was measured in panels 1, 2, 3, 4, 5, and 9.

Results are shown in Table 6. Average tie-strap potentials range from 470 mV on panel 8 to 820 mV on panel 3. The two lowest panels surveyed (8 and 9) exhibit the lowest (less negative) potentials. On the whole, values of strap potentials are higher (more negative) than those measured at Structure 3. This may indicate that less of the zinc coating is consumed at Structure 4; however, environmental conditions may also account for the difference.

Reinforcing bar potentials cover a wide range. On panels 2, 4, 6, 7, 9, and on bar 2 in panel 8, potentials are generally below 250 mV, with many less than 100 mV. On panels 1, 3, and 5, potentials approach and exceed (panel 1) the value of 350 mV generally accepted as indicative of a corrosive state. However, as mentioned in the section dealing with the visual survey, no manifestations of such corrosion were visible on the surface of the units.

Bar 1 in panel 8 exhibited unusually high (negative) potentials, averaging 570 mV vs CSE. A core was taken through this reinforcement, but when the bar was broken free of concrete there was no evidence of corrosion. The cause of these anomalously high potentials remains unexplained.

The measured potentials in the case of aluminum reinforcement (Structure 1) cannot provide an indication of corrosion because, as argued above, there are no reference points to relate these figures to corrosion activity. In the case of carbon steel reinforcement, most of the measured potentials are not indicative of corrosion. In the case of galvanized steel, values were higher as expected. This, by itself, is not conclusive evidence of corrosion.

4.2.5 Concrete Chloride Analysis

Concrete samples were taken using a 2.54 cm (1 in) carbide drill bit and were analyzed for chloride ions using a potentiometric titration (57). Results from all structures are shown in Table 7.

Structure 1 All chloride values are within the range commonly considered as "baseline" values. They are representative of background contributions from aggregates, cement, water, and other mix ingredients in the concrete.

Structure 2 Panel 28-2, located above the tidal zone, has chloride contents that are only slightly over what may be considered baseline values. Those panels within the tidal zone, in comparison, exhibit very high chloride contents. For panel 121-3 and 123-3, significant amounts of chloride have penetrated to the level of the tie-straps. Penetration is less for panel 122-2, as this panel is higher up the wall and subject to less tidal inundation.

Structure 3 Most values lie within what is normally considered a baseline level of chloride ion concentrations. The first increments on panels 9 and 11 are somewhat higher than baseline levels, but are still below levels of 0.025 percent (0.58 kg/m^3 (1 lb/yd^3)), generally accepted as a lower threshold limit for inducement of corrosion on metals embedded in concrete.

Structure 4 Relatively high chloride contents are evident in the first increment of sample obtained from the face of panels 1, 3, and 8, and from the rear of panel 9 (core sample). The first increment of sample taken from the rear of panel 1, however, shows baseline chloride content.

It is probable that the higher chloride contents at the face of the panels are due to deicing salts, which run down the slope and onto the face of the panels. Likewise, the higher chloride content at the rear of the panels could be due to intrusion of chlorides into the soil, although the lack of chloride at the rear of panel 1, which is closest to the top surface, conflicts with this hypothesis.

4.2.6 Concrete Chloride Permeability

Concrete cores of about 9.5 cm (3.75 in) in diameter were obtained from concrete panels. A 5 cm (2 in) thick slice was taken from each core and subjected to the rapid test for chloride permeability of concrete (48).

All results are shown in Table 8.

Table 7. Chloride Analyses.

Structure	Panel No.	Location	Sampling Depth (inches)	Chloride Content (wt. percent of concrete)
1	1	A	0 to 1/2	0.009
			1.2 to 1	0.012
			1 to 1-1/2	0.010
			1-1/2 to 2	0.007
	2	A	0 to 1/2	0.011
			1.2 to 1	0.009
			1 to 1-1/2	0.007
			1-1/2 to 2	0.012
	4	A	0 to 1/2	0.019
			1.2 to 1	0.012
			1 to 1-1/2	0.005
			1-1/2 to 2	0.013
	7	A	0 to 1/2	0.011
			1.2 to 1	0.009
			1 to 1-1/2	0.005
			1-1/2 to 2	0.009
2	28-2	36 in[B]	0 to 1/2	0.020
			1.2 to 1	0.009
			1 to 1-1/2	0.015
			1-1/2 to 2	0.015
	121-3	12 in[B]	0 to 1/2	0.759
			1.2 to 1	0.244
			1 to 1-1/2	0.136
			1-1/2 to 2	0.040
	122-2	8 in[B]	0 to 1/2	0.277
			1.2 to 1	0.058
			1 to 1-1/2	0.020
			1-1/2 to 2	0.012
	123-3	13 in[B]	0 to 1/2	0.753
			1.2 to 1	0.661
			1 to 1-1/2	0.257
			1-1/2 to 2	0.083
3	5	C	1/2 to 1	0.012
		C	1 to 1-1/2	0.010
		C	1-1/2 to 2	0.013
	9	D	0 to 1/2	0.014
		D	1/2 to 1	0.013

Table 7. Chloride Analyses (continued).

Structure	Panel No.	Location	Sampling Depth (inches)	Chloride Content (wt. percent of concrete)
3	9	C	1/2 to 1	0.019
		C	1 to 1-1/2	0.014
		C	1-1/2 to 2	0.009
	11	E	0 to 1	0.020
		C	1 to 1-1/2	0.013
		C	1-1/2 to 2	0.009
	12	C	0 to 1/2	0.008
		C	1/2 to 1	0.007
		C	1 to 1-1/2	0.005
		C	1-1/2 to 2	0.006
4	1	C	1/2 to 1	0.010
			1 to 1-1/2	0.004
			1-1/2 to 2	0.004
		D	0 to 1/2	0.026
			1/2 to 1	0.009
			1 to 1-3/4	0.005
		F	0 to 1/2	0.008
			1/2 to 1	0.013
			1 to 1-1/2	0.015
			1-1/2 to 2	0.013
	3	C	1/2 to 1	0.007
			1 to 1-1/2	0.005
			1-1/2 to 2	0.003
		D	0 to 1/2	0.033
			1/2 to 1	0.017
			1 to 1-3/4	0.003
	8	D	0 to 1/2	0.040
			1/2 to 1	0.004
			1 to 1-1/2	0.005
			1-1/2 to 2	0.005
			2 to 2-5/8	0.002
	9	F	0 to 1/2	0.032
			1/2 to 1	0.023
			1 to 1-1/2	0.010
			1-1/2 to 2	0.011

NOTE: 1 in = 2.54 cm.

Table 7. Chloride Analyses (continued)

FOOTNOTES:

A: Sample from an area at the rear of the panel, between the top of the panel and the concrete gutter.

B: Distance from base of panel.

C: Depths are from back (soil) side of panels.

D: Depths are from front side of panel.

E: Drill "punched through" from 15 cm (6 in) deep to rear face of panel. Specimen represents last 2.5 cm (1 in) of panel thickness.

F: Core sample through panel. Depth referenced to rear face.

Table 8. Rapid Chloride Permeability.

Structure	Panel No.	Charge Passed (coulombs)	Chloride Permeability[1]
1	1	3950	Moderate
	3	1720	Low
	5	2320	Moderate
2	29-1	1350	Low
	123-3	930	Very Low
	192-3	1500	Low
	196-2	2010	Moderate
	202-2	2440	Moderate
3	2	2890	Moderate
	4	1250	Low
	6	2280	Moderate
	10	2250	Moderate
	Unmarked[2]	1500	Low
4	3	1040	Low
	6	1783	Low
	8	1339	Low

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[1] Reference 43, p. 127

[2] Panel lying just below panel 8.

Structure 1 All samples exhibit low to moderate chloride permeability.

Structure 2 All samples, with the exception of the core taken from panel 123-3, exhibit low to moderate chloride permeability. The core taken from the tidal zone (panel 123-3) exhibits very low permeability. This may appear to be in conflict with the chloride analyses, which show that larger amounts of chloride have penetrated this panel. However, the discrepancy can be resolved by consideration of the effect of long-time immersion on reduction of concrete permeability. The chloride penetrated the panel over a period of 9 years, during which time the permeability was slowly decreasing. As the rapid test was conducted only at a single point in time (i.e., after 9 years), it reflects the current permeability, not the permeability when the panel was first put into service.

Structure 3 All samples exhibit low to moderate chloride permeability.

Structure 4 All specimens exhibit low permeability of chloride ions.

4.2.7 Concrete Resistivity

Concrete resistivity was measured using a miniature 4-pin electrode system similar to that developed by Stratfull (58). This technique is used only to gain a rough indication of concrete resistivity.

Results obtained from all structures are shown in Table 9.

Structure 1 The high values reflect the extreme dryness of the concrete at the top of the wall. It is surmised that the thin, dry surface layer exerts an inordinate influence on the readings, even after a short period of soaking prior to obtaining the reading. Similar problems were noted by Stratfull (58) when using this technique under hot, dry ambient conditions.

Structure 2 Resistivities of the panels above the tidal zone range from 23,000 to 29,000 ohm-cm. For the two locations toward the bottom of the panels, resistivities in the tidal zone are much lower. This is to be expected in view of the daily inundation with highly conductive tidal waters.

Structure 3 Difficulties were encountered in measurement of concrete resistivity, due to both the extreme dryness of the concrete and the presence of the painted surface. Measurement on the face of panel 2 yielded a value of 510,000 ohm-cm after about 20 minutes of wetting. An area on top of panel 3 where the paint had debonded was selected as an alternative site. Resistivity at this location was measured at 280,000 ohm-cm after 20 minutes of wetting. In view of the problems encountered, no further resistivity tests were carried out on this structure.

Structure 4 Resistivity values are indicative of very dry concrete, at least at the face side of the panels.

Concrete resistivity measurements in most cases yielded high values which indicate that concrete panels at that point in time were dry.

Table 9. Concrete Resistivity

Structure	Panel No.	Resistivity (ohm - cm)
1	1*	84,300
	2*	95,800
	3*	99,200
	4*	106,000
	5*	106,600
2	27-1**	24,470
	28-2**	29,000
	29-1**	22,880
	121-3**	10,430
	122-2*	27,930
	122-2**	6,120
	123-3*	21,550
	123-3**	3,500
4	1	52,700
	2	68,400
	3	278,000
	4	119,700
	5	99,200
	6	67,300
	7	256,700
	8	204,800
	9	157,470

*top of panel
**center of panel
***bottom of panel

Low values of potential, low values of chloride, high resistivity, and dry conditions indicate that corrosion is not present inside the panels.

4.2.8 Soil Resistivity

Soil resistivity was determined by both 4-pin and soil box techniques (59) at a number of locations.

Results are reported in Table 10.

Structure 1 Soil resistivity was determined by both 4-pin and soil box techniques at a number of locations and depths that varied from 6 to 2 m (20 to 6.6 ft) and was consistently found to be very high.

Structure 2 Soil resistivities range from 16,000 ohm-cm at the top of the structure to 80 ohm-cm for backfill taken from behind the wall in the tidal zone. This reflects the gradient of moisture and salt content existing from bottom to top of the structure.

Intermediate values were obtained from the location 46 cm (18 in) from top ground level and through the core hole drilled through panel 192-3 (Table 10). Based on these data, the backfill on the tidal zone and also at the level of panel 192-3 would be considered highly corrosive.

Structure 3 Soil resistivities were measured both on as-received soils (excluding material retained on the No. 8 sieve), and on soils saturated with tapwater. As-received resistivities exhibit a wide range of values, most likely reflecting varying moisture conditions. Saturated values range from 3750 to 6800 ohm-cm, indicating that if moisture levels ever approach saturation, corrosive resistivity ranges may be encountered, depending on other soil constituents.

Structure 4 Soil resistivities were measured on three samples. It was very difficult to saturate this dense soil in the soil box after measuring as-received resistivity. In lieu of this, sample was removed from the soil box, combined with additional sample from the same location, and saturated on a polyethylene surface. This resulted in a very fluid mixture, probably beyond the maximum saturation that could actually be achieved in-place.

4.2.9 Chemical Analysis of Soil Samples

All of the soil samples from the selected sites were analyzed for the strong acid anions, Cl^-, NO_3^-, and $SO_4^=$. Ion Chromatography (IC) was used to identify and quantify ions in solution. The American Society of Testing and Materials older method was not used because this newer method (IC) is widely recognized as being faster, more economical, and far more precise for the analysis of multianion mixtures in a complex matrix. A description of the IC method can be found in the 1983 Annual Book of ASTM Standards, Volume 11.01, pp. 696-703, as a proposed test method of anions in water by anion chromatography. Moreover, it is currently being considered for inclusion in the EPA Standard Methods Book.

Table 10. Soil Resistivities.

Structure	Code No.	Resistivity (ohm-cm)	
		As Received	Saturated
1	M-A-2	400,000	
	M-A-3	410,000	
	M-B-1	70,000	
	M-B-2	50,000	
	Panel 3[a]	210,000	
	Panel 4[a]	110,000	
	Panel 5[a]	130,000	
2	B-28-2-8[b]	16,000	
	B-39 Bot[c]	7,200	
	B-Tidal	320	
	Tidal 2	80	
	192-3-0	430	
	192-3-2	2,800	
	192-3-4	880	
3	L-1-0	7,100	-
	L-7-2	8,000	-
	L-A-1	15,000	-
	L-B-1	200,000	6,000
	L-B-2	41,000	6,000
	L-B-3	9,900	4,500
	L-B-4	8,300	3,750
	L-B-5	310,000	6,800
4	V-1-4	9,100	4,000
	V-2-4	10,000	4,500
	V-3	10,500	-

NOTE: 1 in = 2.54 cm

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[a] Samples taken from holes dug on opposite side of concrete gutter from respective panel.

[b] Minus No. 8 sieve material

[c] Taken from bottom of exposed reinforcing strip

Following is a brief description of IC.

Determination of inorganic and organic species is possible, often in concentration of 10 parts per billion or less. Because analysis time is frequently less than 20 minutes and minimal sample quantities are needed, ion chromatography is a fast and economical technique.

The various steps in IC measurements of anions are as follows:

1. The sample is injected into a stream of $NaHCO_3$ eluent delivered by a reciprocating liquid pump which delivers constant volume over time.

2. The sample then goes through a strong base anion exchange separator resin (separator column) which separates sample anions in a background of $NaHCO_3$ eluent.

3. The suppressor column -- a strong acid suppressor resin -- removes the $NaHCO_3$ eluent and converts sample anions to their acids (which pass unretarded through the suppressor column).

4. Finally, the anion acids in a background of dilute carbonic acid are quantified by conductivity.

Further information on ion chromatography; its procedures, theory, and practices, are given in References 60, 61, and 62.

In this work, the samples were run on the IC and concentrations were determined by using a standard calibration curve with a $r^2 \geq .999$. Fresh standards bracketing the concentrations were made daily. Internal standard additions to the soil itself were deemed unnecessary as leachable salts were the primary concern. (Soil constituents which are not leachable under environmental conditions are not active in the corrosion process. The extraction method chosen provides a reasonable upper limit on the concentrations of leachable acid anions).

Structure 1 Results of the chemical analysis of the soil appear in Table 11 and in the histograms in Figures 14, 15, and 16. The native soil at this site is quite acidic (pH 4 to 6), with decreasing pH gradient with depth at the sites near the road. The low chloride, sulphate, and nitrate concentrations also show these decreasing values with depth near the road. In areas adjacent to the track, the concentrations are in the same range; but the vertical gradients are eliminated or even partly reversed. The fill material exhibits a neutral pH, low chloride and sulphate concentrations, and no systematic gradients.

The above results are compared in Figures 14, 15, and 16 to those tabulated in Reference 26 for 235 samples of fill material utilized in France for Reinforced Earth structures. The fill material at the site of Structure 1 is rather typical of such material with perhaps lower than average chloride and sulphate concentrations.

Table 11. Structure 1: Chemical Analysis of the Soil.

Sample	pH	µg/g soil (ppm)		
		Cl^-	NO_3^-	$SO_4^=$
MA 1-1	5.28	2.90	1.96	4.82
MA 1-2	5.27	1.89	.50	9.38
MA 2-1	5.46	2.21	1.67	7.46
MA 2-2	5.24	1.34	.35	5.05
MA 2-3	4.37	1.18	.19	8.13
MA 2-4	4.43	.78	.25	4.99
MA 3-1	4.98	3.91	.42	10.56
MA 3-2	5.16	1.42	.18	12.64
MA 3-3	4.49	.86	.15	7.73
MA 3-4	4.52	.78	.23	5.83
MB 1-1	4.60	1.80	0	8.10
MB 1-2	4.54	1.89	0	6.88
MB 1-3	5.27	2.91	0	4.96
MB 1-4	4.67	5.20	.10	5.74
MB 2-1	5.11	1.26	.14	4.16
MB 2-2	4.72	1.43	0	4.97
MB 2-3	5.90	1.66	0	4.29
MB 2-4	Empty bag			
MB 3-1	5.08	1.12	1.43	4.00
MB 3-2	4.71	1.53	0	3.97
MB 3-3	4.86	.96	0	4.08
MB 3-4	5.59	.94	0	1.56

Table 11. Structure 1: Chemical Analysis of the Soil (continued).

Sample	pH	µg/g soil (ppm)		
		Cl^-	NO_3^-	$SO_4^=$
MB 4-1	4.90	2.72	1.71	5.02
MB 4-2	5.20	1.61	0	4.35
MB 4-3	5.39	2.58	0	4.36
MB 4-4	5.48	.96	0	5.57
MB 5-1	4.54	1.87	.099	9.70
MC 1	6.36	1.97	.099	4.59
MC 2	6.97	.40	0	9.21
MC 3	7.11	1.55	.11	6.85
MD 1	7.60	.39	0	5.86
MD 2	7.73	.39	0	4.42
MD 3	7.15	.96	0	5.00
MD 4	6.97	.52	0	6.33

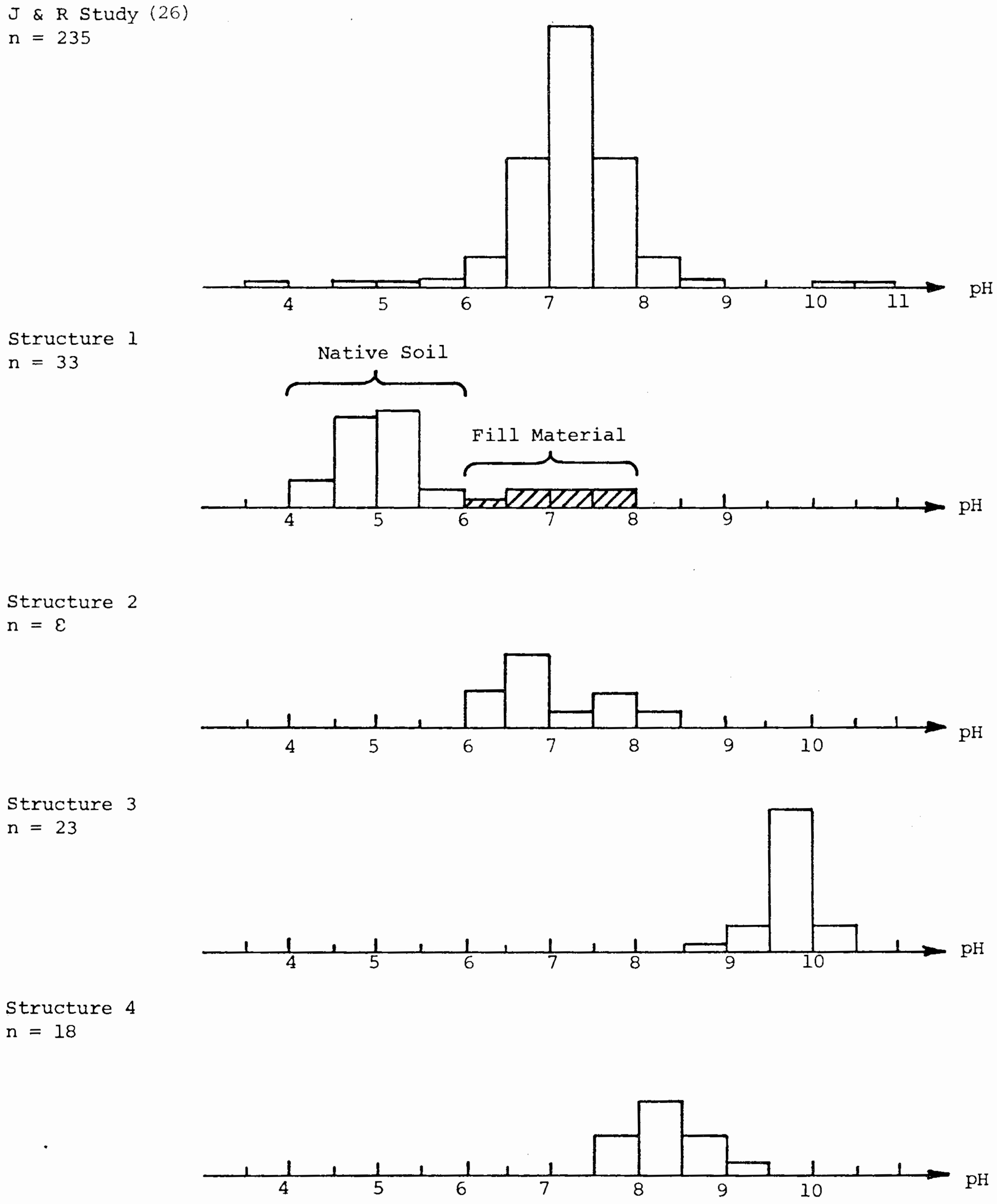


Figure 14. Histograms of pH values.

NOTE: The vertical axis represent the number of samples in each range of values. The total number of samples (n) is shown on each figure.

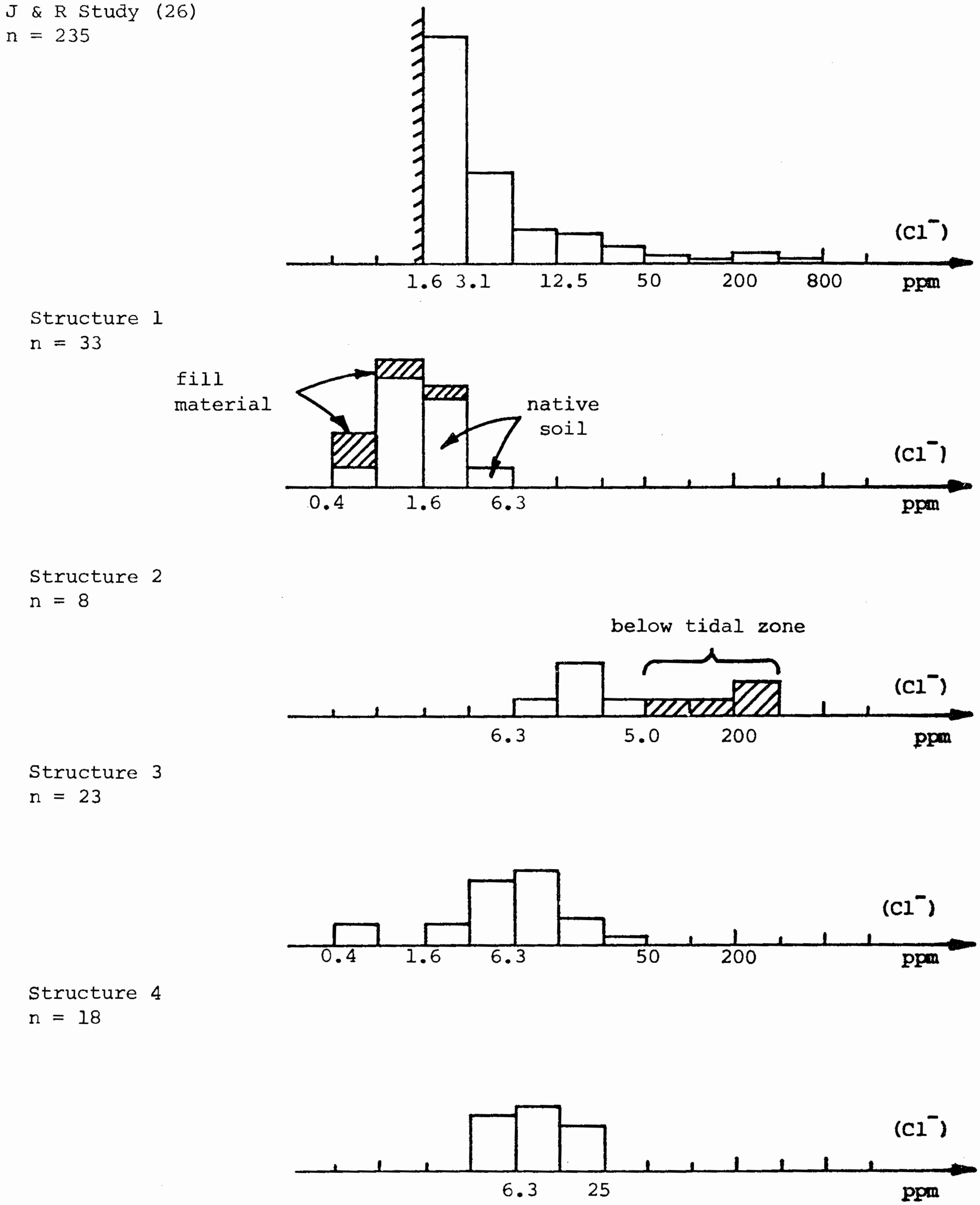


Figure 15. Histograms of chloride concentrations.

NOTE: The vertical axis represent the number of samples in each range of values. The total number of samples (n) is shown on each figure. The hatched area indicates that the precision of the measurements did not allow distinction of the low chloride values.

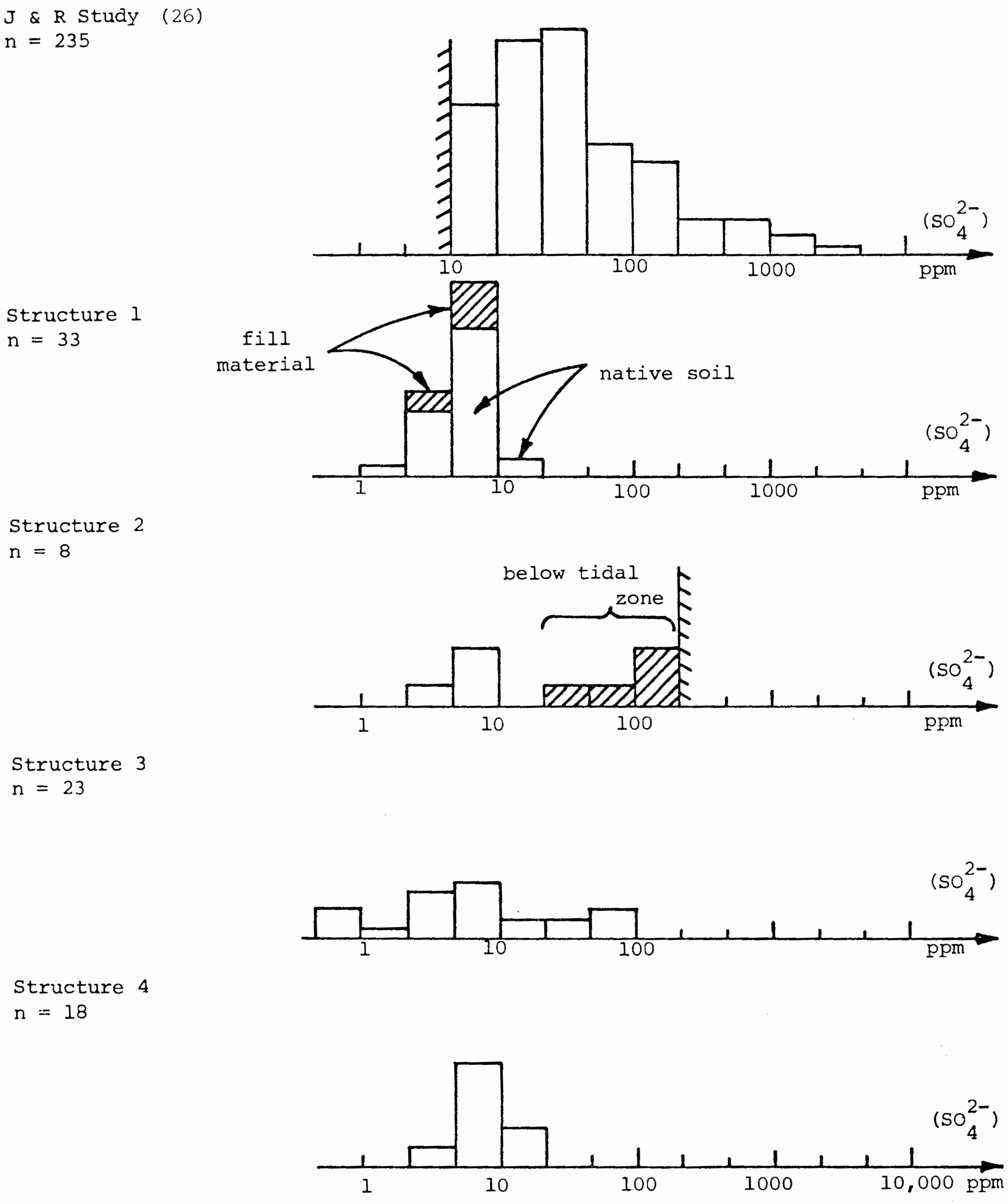


Figure 16. Histograms of sulphate concentrations.

NOTE: The vertical axis represent the number of samples in each range of values. The total number of samples (n) is shown on each figure. The hatched area indicates that the measuring device only gave a lower limit in sulphate.

To complete the comparison with other fill material, histograms for soil resistivity are shown in Figure 17; and, of course, resistivity at the site of Structure 1 is very high.

Structure 2 Results of the chemical analysis of the soil appear in Table 12 and in histograms in Figures 14, 15, and 16. The pH in the fill material is neutral to slightly alkaline, as expected for a marine site. For the chloride and sulphate concentrations, there is a clear difference between the regions above and below the tidal zone; much higher concentrations are encountered in the low region. Chloride concentrations are relatively high everywhere and exceed the recommended limit of 200 ppm below the tidal zone. In one case, at least, the concentrations exhibit a systematic decrease from the facing to the interior of the structure. Compared to fill material utilized in France for Reinforced Earth structures, and due to the infiltration of sea water, the chloride and, to a lesser degree, the sulphate concentrations at the site of Structure 2 are on the high side of the distribution (Figures 15 and 16).

To complete the comparison with other fill materials, it can be seen from Figure 17 that, as expected, resistivity is very low at the Structure 2 site, particularly below the tidal level.

Structure 3 Results of the chemical analysis of the soil appear in Table 13 and in histograms in Figures 14, 15, and 16. The site of Structure 3 is characterized by very alkaline pH's, even exceeding in places the recommended limit of 10. (At this point, it must be emphasized that the techniques used for measuring soil pH tend to underestimate the pH of the soil because both the collection and the equilibration procedures are apt to introduce CO_2 in the measured sample and thus partly filtrate the strong bases with carbonic acid. In situ, pH's at the site of Structure 3 could well be in excess of 10 and reached values higher than 12.

To complete the comparison with other fill materials one can see from Figure 17 that the resistivity at Structure 3, though in the normal range, is certainly on the low side.

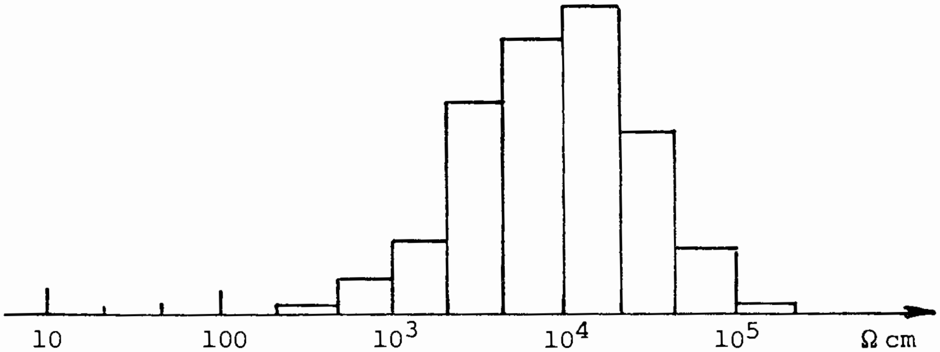
Structure 4 Results of the chemical analysis of the soil appear in Table 14 and in histograms in Figures 14, 15, and 16. Of all the projects studied the Structure 4 site demonstrates the tightest distribution of chemical parameters: slightly alkaline pH, moderate chloride, and low sulphate. There is a a systematic increase in pH with depth.

To complete the comparison with other fill material, the resistivities at this site are shown in Figure 17 to be in the normal range.

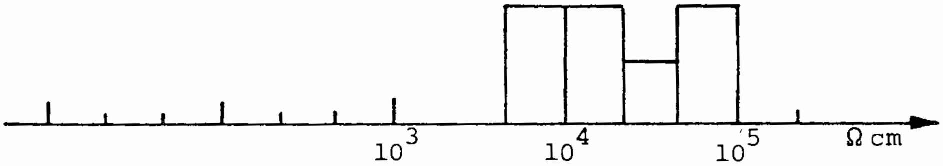
4.2.10 Examination of Metal Strips Embedded in Soil

This was performed through microscopy and Energy Dispersive X-ray Analysis (EDAX).

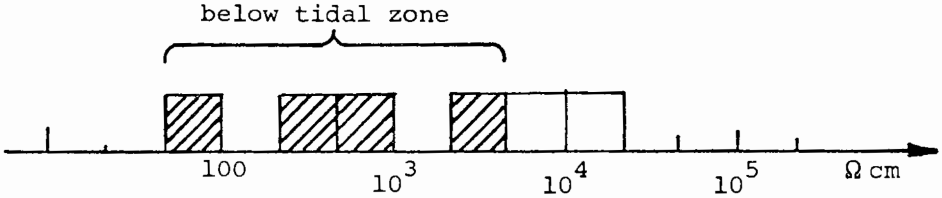
J & R Study (26)
n = 235



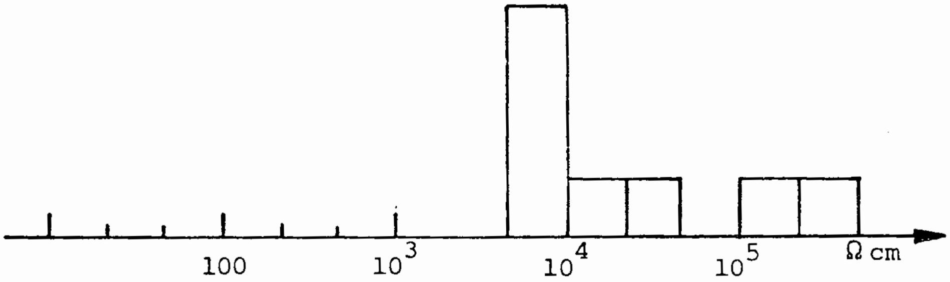
Structure 1
n = 7



Structure 2
n = 6



Structure 3
n = 8



Structure 4
n = 3

Figure 17. Histograms of soil resistivities.

NOTES: The vertical axis represent the number of samples in each range of values. The total number of samples (n) is shown on each figure.

Table 12. Structure 2: Chemical Analysis of the Soil.

Sample	pH	µg/g soil (ppm)		
		Cl^-	NO_3^-	$SO_4^=$
B-28-2-T	7.08	20.14	11.12	4.13
B-28-2-B	6.74	22.72	23.69	5.88
B 39 top	8.14	15.95	13.62	8.08
B 39 bot	7.66	11.39	9.62	5.14
192 3-0	6.47	111.15	obscured	56.44
192 3-1	4.18	86.89	obscured	205.73
192 3-2	7.90	34.45	2.31	30.12
123-3-tidal-1	6.90	> 206	obscured	182.69
123-3-tidal-2	6.81	> 206	obscured	obscured
Brunswick Tidal	6.49	Cl^- too		
B-tidal	7.79	high to run		

Table 13. Structure 3: Chemical Analysis of the Soil.

		µg/g soil (ppm)		
Sample	pH	Cl^-	NO_3^-	$SO_4^=$
L 1-0	9.17	5.26	33.29	7.60
L 1-1	9.51	6.77	17.17	4.63
L 1-2	9.59	13.65	15.02	4.75
L 2-0	9.53	7.46	9.23	3.36
L 2-1	9.77	8.49	6.91	2.92
L 2-2	9.71	8.28	2.50	6.33
L 3-0	9.92	10.66	12.21	4.37
L 3-1	9.97	6.13	7.33	5.81
L 3-2	9.75	15.34	2.22	2.76
L 5-0	10.11	8.19	2.11	14.07
L 5-1	9.74	22.58	.55	96.25
L 5-2	10.01	28.98	1.04	62.42
L 7-0	9.86	8.90	.0124	10.95
L 7-1	9.62	6.83	.78	10.33
L A-1	10.25	3.56	5.28	24.62
L B-1	9.93	.76	2.24	.36
L B-2	9.33	3.85	4.56	5.45
L B-3	8.63	5.08	2.06	.81
L B-4	9.61	3.55	3.32	54.58
L B-5	9.40	3.21	3.28	33.35
L R-0	9.66	.16	5.84	.34
L R-1	9.90	2.44	.30	1.80
L R-2	9.91	2.14	.51	2.35

Table 14. Structure 4: Chemical Analysis of the Soil.

Sample	pH	µg/g soil (ppm)		
		Cl^-	NO_3^-	$SO_4^=$
V 1-1	7.95	6.68	15.02	8.42
V 1-2	8.20	8.56	4.81	7.08
V 1-3	8.75	4.96	.83	3.66
V 1-4	8.07	5.74	11.92	12.23
V 2-1	7.72	9.42	3.92	8.97
V 2-2	7.99	5.91	5.89	6.89
V 2-3	8.09	20.03	4.30	7.78
V 2-4	9.07	12.66	3.16	9.11
V 3-4	8.35	10.44	4.94	9.58
V 9-1	8.01	7.30	1.94	3.92
V 9-2	8.00	4.15	6.71	6.03
V 1-10-2	7.90	11.28	3.75	15.10
V 1-10-1	8.42	18.85	3.36	7.32
V 1-15-1	8.09	4.04	1.71	5.67
V 1-15-2	8.90	15.63	4.22	10.91
V 1-5-1	8.44	11.24	1.80	7.75
Rocky Mt water	7.06	810.8 µM	8.39 µM	58.5µM
V 8 Rocks	8.85	4.19	0	1.66

Structure 1 Two cross sections from each one of three specimens of metal strips reinforcing the structure were examined. Figure 18 shows a metallographic section of a specimen. It is in very good condition. The zinc coating is still conspicuous. Indeed, the average coating thickness measured was 54 m.

Structure 2 Three cross sections of two specimens of metal strips reinforcing the structure were examined. Figure 19 shows a metallographic section of a specimen. In this as well as in all other specimens that were examined, metal loss is due to localized corrosion. At regions this localized attack has occurred at a high rate. Indeed the cross section in the photograph in Figure 19 has been locally reduced to 1.4 mm (0.05 in). In the tidal zone, one of the examined cross sections has been locally reduced to only 1.2 mm (0.045 in). The initial cross section was 2mm (0.08 in).

The EDAX analysis indicates that the corrosion product, in addition to aluminum oxide, contains elements such as Na, Cl, Ca, Si, and Fe. Since the metal strip is made of an aluminum alloy containing no iron, it is possible that the soil is iron rich.

Structure 3 One cross section of each of the six specimens of metal strips reinforcing the structure was examined. Representative metallographic cross sections of the specimens from Structure 3 are shown in Figure 20. The specimen shown in the top of Figure 20 demonstrates considerable corrosion. It is encrusted with sand and thick layers of iron oxide and has had almost a 7 percent reduction in thickness due to uniform corrosion. Contrary to the above, the specimen shown in the bottom of Figure 20 is in excellent condition and shows no signs of corrosion.

Coating thickness measurements showed that zinc was completely gone on the specimen shown in the top of Figure 20. On the other hand an average thickness of 40 m was measured on the specimen in the bottom of Figure 20. This is close to initial thickness.

Structure 4 One cross section of each of four specimens of metal strips reinforcing the structure was examined. A metallographic cross section of the strips is shown in Figure 21. The zinc coating thickness measured on all specimens obtained in this structure showed that the galvanized coating is almost intact, apparently unaffected by 9 years in the soil.

4.2.11 Potential of Earth Reinforcing Strips

The potential of the earth-reinforcing strips was measured using equipment and techniques similar to ASTM C876 (50) with the half cell being placed on the soil above the strip. The soil was wetted prior to making the measurement.

Structure 1 The potential of the strips was measured in 5 locations and was found to vary from 723 to 825 mV vs CSE.

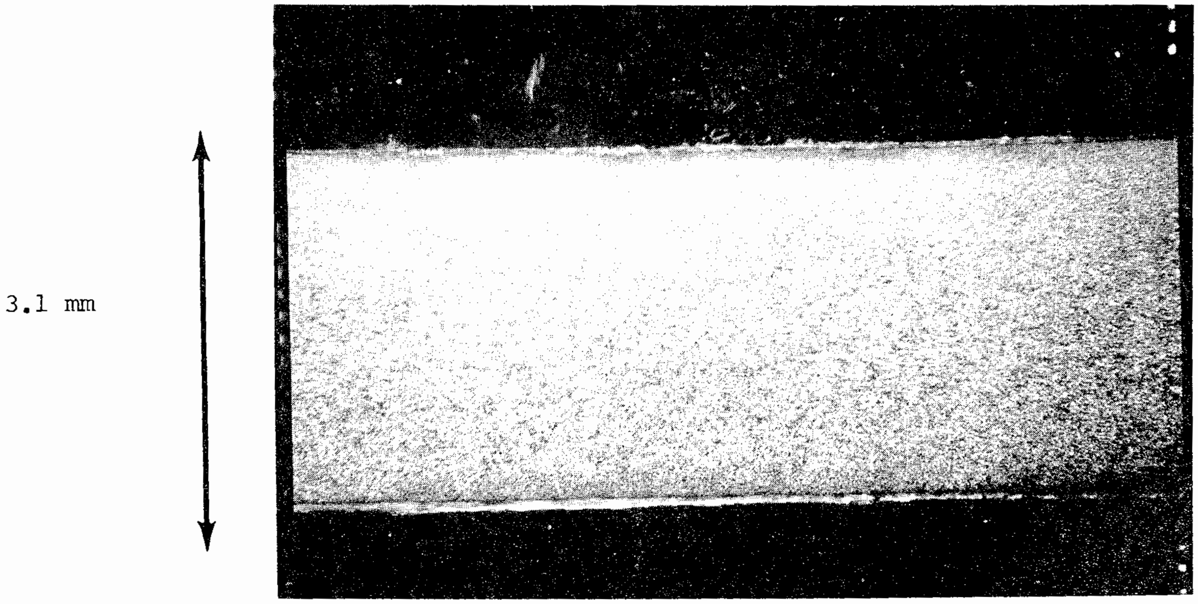


Figure 18. Structure 1: Metallographic cross section of a specimen from the strips in the structure.

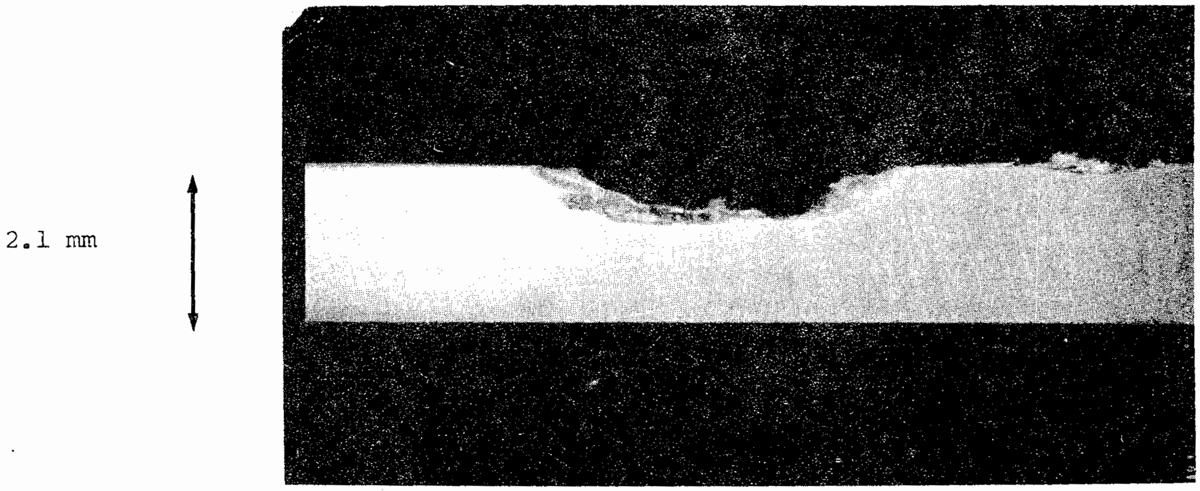


Figure 19. Structure 2: Metallographic cross section of a specimen from the strips in the structure.

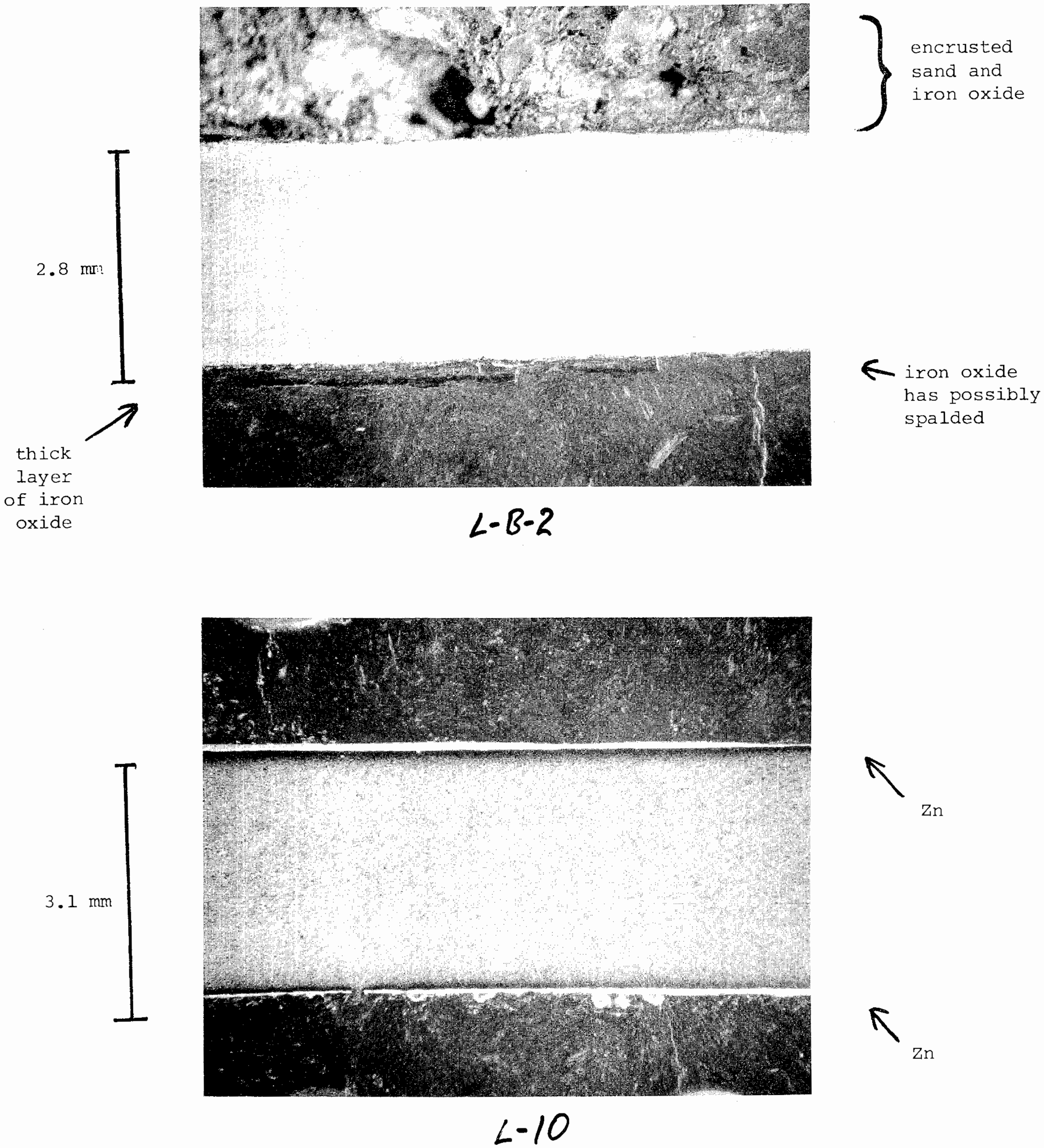
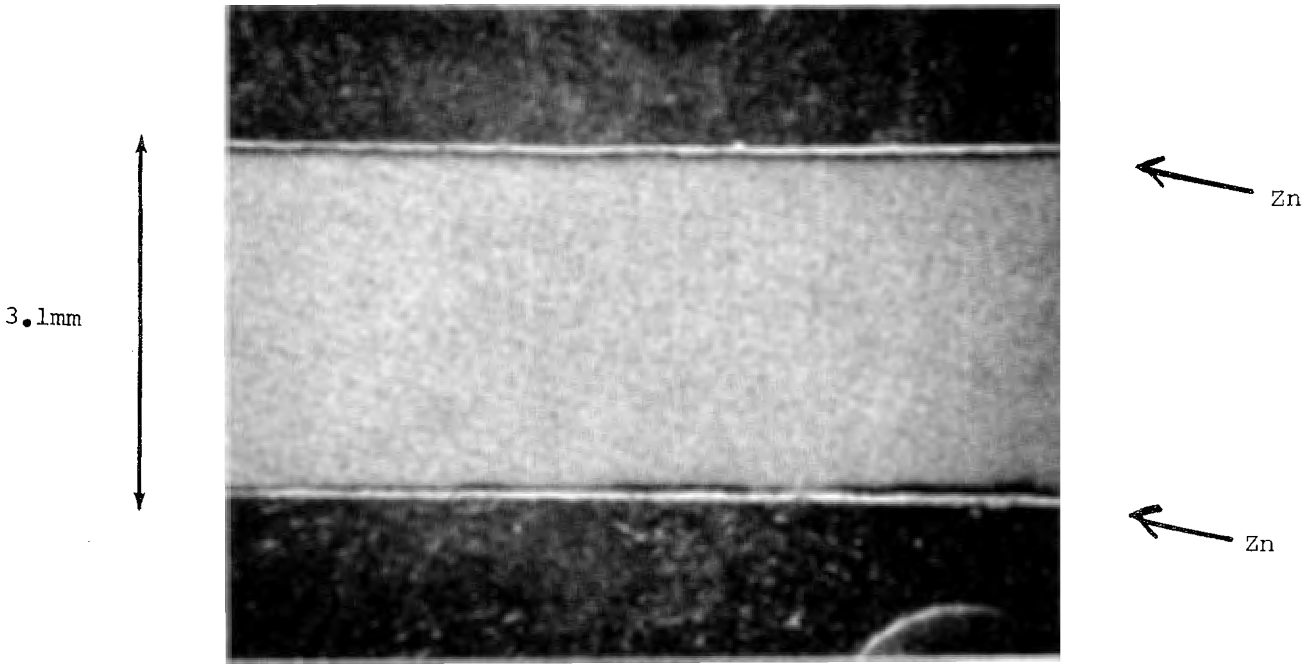


Figure 20. Structure 3: Metallographic cross sections of specimens from the strips in the structure.



Y-2

Figure 21. Structure 4: Metallographic cross sections of a specimen from the strips in the structure.

The above measurements are close to those expected from zinc exposed to underground soil conditions. Therefore, zinc is still there.

Structure 2 A core hole was drilled through the concrete at a location just below a strip in panel 123-3. A CSE was placed into this hole; the hole was backfilled, and the average potential was found to be 700 mV vs CSE.

Potentials (vs CSE) were also measured on a strip attached to panel 28-2. Measurements were taken on the soil immediately adjacent to the concrete cap, and 1.5 m (5 ft) out from the wall. Values were 845 mV and 830 mV, respectively.

Since not enough data exist on the aluminum alloy that was used in this structure, the above measurements cannot be used to assess corrosion progress.

Structure 3 Potentials of two reinforcing strips attached to panels 1, 2, 5, 7, 9, and 11, were measured by placing a CSE on the surface of the ground above the strip to be tested. Measurements were made at the wall and at 1.5 m (5 ft) increments along the surface away from the wall. Results are presented in Table 15. Potentials tend to be somewhat lower (less negative) near the wall, and then increase in magnitude at greater distance from the wall. The majority of potentials away from the wall are in the same range as those measured on Structure 1, reflecting the presence of zinc coating over steel strips.

The above figures are not indicative of any corrosion problems.

Structure 4 Potentials of reinforcing strips attached to panels 1 through 9 were measured by placing a CSE on the surface of the ground above the strip to be tested. Measurements were made at the wall, and at 1.5 m (5 ft) increments along the surface away from the wall. Results are given in Table 15. The same trend is noticed as on the previous structures; that is, potentials shift to more negative values as the cell is moved away from the wall. Here, too, this may result from the influence of steel in the panels on the measurements made on the portions of reinforcing strips that are closest to the panels.

Panels 1 through 7 exhibit potentials close to 1100 mV at locations away from the wall. This is close to theoretical solution potential for zinc (-1100 mV vs CSE), and indicates that much of the zinc coating remains intact. Potentials at the wall are lower (less negative) by about 200 mV, possibly reflecting the contribution of the tie-strap in the panel to the potential. Finally, potentials away from the wall at locations 8 and 9 are lower by about 200 mV than at the other locations. This may indicate that the zinc coating is being consumed at these locations, or that there is insufficient electrolyte (i.e., soil) to enable contact to be made to the straps from the top of the ground. In the latter case, potentials would be those of the tie-straps in the panels at these locations, the electrolytic path being completed by the dowel bars between the panels.

Table 15. Earth Reinforcing Strip Potentials.[1]

Structure	Distance (feet)	Panel 1	Panel 2	Panel 5
3	0	760;640	760;610	690;706
	5	790;780	810;810	700;710
	10	790;760	770;780	790;770
	15	770;770	770;770	760;760
	20	770;750	770;770	760;760
		Panel 7	Panel 9	Panel 11
	0	640;660	560;600	570;560
	5	630;670	580;570	570;560
	10	700;770	630;650	600;650
	15	680;750	690;680	630;690
	20	690;730	720;710	710;710
		Panel 1	Panel 2	Panel 3
4	0	980	860; 980	920;1000
	5	1090	1130;1120	1090;1170
	10	1120	1170;1140	1120;1130
	15	1100	1160;1140	1120;1130
	0	910; 900	890; 880	890; 930
	5	1130;1130	1130;1100	1090;1080
	10	1150;1140	1140;1100	1110;1110
	15	1170;1150	1140;1100	1120;1110
		Panel 7	Panel 8	Panel 9
	0	930; 910	860; 880	870; 880
	5	1140;1120	940; 970	920; 940
	10	1140;1140	960; 980	970; 980
	15	1130;1130	980; 980	960; 960

NOTE: 1 ft = 30.50 cm

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[1] All potentials shown are millivolts negative to CSE.

4.2.12 Stray Current Analyses for Structure One

The first task carried out as part of the stray current testing was determination of the shift in open-circuit potential of the structure due to passage of rapid transit traffic. It was found that the potential shifts are virtually negligible, changing at most by 2 mV during passage of traffic.

To gain an estimate of current flow to the structure, shunts were installed within the MARTA function boxes on the east and west ends of the structure. Currents measured across the shunts during passage of traffic were found to be relatively low, ranging from 0 to 3.5 amperes. Potentials due to the passage of this current through the structure were measured. The ratio of potential shift to current provides an indication of the amount by which potential would shift due to passage of current into the structure if the bond were made permanent. It also can be used as an estimate of the amount of current flowing onto the structure in its currently unbonded condition. The latter amount turned out to be about 66 milliamperes, which is considered a negligible amount.

Because of the above, and in view of the high resistivity exhibited by the backfill serious stray current corrosion problem is not expected. If operating or environmental conditions should change, installation of a reverse-current drainage bond* within the existing junction boxes would afford cathodic protection to the structure during periods of cathodic shift and completely halt current flow during anodic cycles.

4.3 Summary and Predictions

Structure 1 The native soil at the project site is an aggressive one. It was a good choice to replace it with a fill material that has properties within recommended values in terms of resistivity, pH, chloride, and sulphate concentrations. Moreover, the chosen design has solved successfully the stray current problems.

Based on the collected data it can be concluded that corrosion is not a particular problem at this project.

Structure 2 The environment at the project site is quite aggressive, exhibiting low resistivity and high chloride concentrations. Moreover, some heavy metals might exist in the soil.

Because of the above an aluminum alloy has been used that is known to have high resistance in marine atmospheric exposures. In this particular application, where the above metal was buried in the soil, it has exhibited a high rate of localized corrosion. Responsible for this might be the high

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*Such diode-type reverse current switches are commercially available at modest cost.

chloride concentration or alternatively it might be the presence of heavy metals, like iron, in the soil. (Iron being less active than aluminum, will chemically replace aluminum.)

At the exhibited rates of penetration the strips could be penetrated in 11 years.

The above might even be optimistic if one considers the following two factors: Firstly, we have not examined metal strips in the most severe locations - at the tidal level next to the facing units where maximum chloride concentrations exist. Secondly, perforations will act as stress concentrators that will disproportionately decrease the load carrying capacity of the section.

The present evidence does not encourage the use of aluminum alloy 5086 when buried in a marine soil even though it is considered to have acceptable resistance to marine atmospheric corrosion.

Since we have investigated only a limited number of specimens further investigation on this project is recommended.

Structure 3 The backfill material used in this project is nonuniform and has a high pH value. The latter is probably higher than 12.5 and is responsible for the exhibited high corrosion rates.

In this highly alkaline environment zinc has corroded at a high rate that did not decrease with time. On the other hand, in this environment bare steel will corrode at a moderate rate. The rate of deterioration found in some of the specimens suggest that the structure may have a life expectancy less than the design life. Given the limited amount of specimens further research is recommended on this structure.

Structure 4 In this project soil characteristics fall within recommended limits.

In spite of moderate use of deicing salts, the project is in excellent condition. Corrosion does not seem to be a particular problem.

CHAPTER 5 CONCLUSIONS AND RECOMMENDATIONS

There is a paucity of data in some areas of corrosion of reinforced soil retaining structures. Specifically, it was found that sufficient data is lacking on the effect of high chloride concentrations. One reason for this is that a good part of the research is conducted in Europe and Europeans are not using deicing salts to the extent they are used in this country. Additionally, there is a paucity of data on the effect of a highly alkaline environment on buried metals. Little data also exists on the long-term performance of materials such as epoxy-coated metals, when buried in the various soils. Moreover, no quantitative information was found to exist on the effect of climatological conditions on the rate of corrosion.

It was found that two out of the four Reinforced Earth walls selected for our field study have corrosion problems that may limit their life to significantly below the design life. Given the fact that not enough samples were taken to offer a statistical proof of the above statement, it is strongly recommended that additional field studies be undertaken in the above structures.

The findings of this work suggest that the safe limits of the reinforced soil method are not yet known. It is recommended that additional field studies, on structures not investigated in this project, be also undertaken. These structures should be located in severe environments.

It is recommended that studies be conducted on the properties of soil reinforcement methods other than Reinforced Earth.

Additionally, it is suggested that metal strips be inserted in future projects that can easily be removed to assess the progress of corrosion.

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FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH, DEVELOPMENT, AND TECHNOLOGY

The Offices of Research, Development, and Technology (RD&T) of the Federal Highway Administration (FHWA) are responsible for a broad research, development, and technology transfer program. This program is accomplished using numerous methods of funding and management. The efforts include work done in-house by RD&T staff, contracts using administrative funds, and a Federal-aid program conducted by or through State highway or transportation agencies, which include the Highway Planning and Research (HP&R) program, the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board, and the one-half of one percent training program conducted by the National Highway Institute.

The FCP is a carefully selected group of projects, separated into broad categories, formulated to use research, development, and technology transfer resources to obtain solutions to urgent national highway problems.

The diagonal double stripe on the cover of this report represents a highway. It is color-coded to identify the FCP category to which the report's subject pertains. A red stripe indicates category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, and green for category 9.

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Safety RD&T addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act. It includes investigation of appropriate design standards, roadside hardware, traffic control devices, and collection or analysis of physical and scientific data for the formulation of improved safety regulations to better protect all motorists, bicycles, and pedestrians.

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Traffic RD&T is concerned with increasing the operational efficiency of existing highways by advancing technology and balancing the demand-capacity relationship through traffic management techniques such as bus and carpool preferential treatment, coordinated signal timing, motorist information, and rerouting of traffic.

3. Highway Operations

This category addresses preserving the Nation's highways, natural resources, and community attributes. It includes activities in physical maintenance, traffic services for maintenance zoning, management of human resources and equipment, and identification of highway elements that affect the quality of the human environment. The goals of projects within this category are to maximize operational efficiency and safety to the traveling public while conserving resources and reducing adverse highway and traffic impacts through protections and enhancement of environmental features.

4. Pavement Design, Construction, and Management

Pavement RD&T is concerned with pavement design and rehabilititation methods and procedures, construction technology, recycled highway materials, improved pavement binders, and improved pavement management. The goals will emphasize improvements to highway performance over the network's life cycle, thus extending maintenance-free operation and maximizing benefits. Specific areas of effort will include material characterizations, pavement damage predictions, methods to minimize local pavement defects, quality control specifications, long-term pavement monitoring, and life cycle cost analyses.

5. Structural Design and Hydraulics

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