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16. Abstract

Despite over 25 years of experience with the problem of bridge reinforcement corrosion due to chlorides from deicers or sea water exposure, the solution to the problem is still unknown. Many different corrosion protection systems have shown promise in laboratory and preliminary field studies, but their performance on the basis of long-term field tests either has not been evaluated or has been questionable. In the portion of the work discussed in this report, an extensive literature search was conducted to identify corrosion protection systems and field test methods. In addition, a survey of TxDOT districts was conducted to identify corrosion protection systems used in Texas and to determine the districts' experiences concerning the different systems. Most importantly, a field testing program was developed to evaluate and characterize the corrosion performance of bridges and to increase the information learned from corrosion protection system test installations in Texas.

Preliminary questionnaire surveys distributed to all the TxDOT districts revealed that many different types of corrosion protection systems have been installed in Texas, but that little information on their performance has been collected from the structures since their completion. The field testing program developed in this study was used to investigate eight preexisting bridges representing different protection systems and various overall exposures and service conditions.

The test program, which included determination of half-cell potentials, concrete permeability, chloride content, cracking patterns, and delaminations, was found to accurately reflect the corrosion condition of bridges protected by a variety of corrosion protection measures. The tests selected for use in the field surveys were generally successful and accurate, with different tests confirming each other's results.

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FIELD EVALUATION OF BRIDGE CORROSION PROTECTION MEASURES

by

Matthew R. Sherman
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D. W. Fowler

Research Report 1300-1

Research Project 3-5-92/4-1300
Evaluations of Current Corrosion Protection Measures for Bridge Decks

conducted for the

Texas Department of Transportation

in cooperation with the

U.S. Department of Transportation
Federal Highway Administration

by the

CENTER FOR TRANSPORTATION RESEARCH
Bureau of Engineering Research
THE UNIVERSITY OF TEXAS AT AUSTIN

March 1993

IMPLEMENTATION STATEMENT

This report contains background information about the reinforcement corrosion problem, the corrosion process, methods for preventing corrosion, and monitoring procedures. In particular, the report contains a detailed description of a field testing program developed for evaluating and characterizing the corrosion performance of bridges, and the results of the examination of eight bridges in Texas.

The field testing program developed through this work was found to accurately indicate the corrosion condition of bridges and to provide performance information about the corrosion protection systems. In particular: the individual tests confirmed each other's results, corrosion activity could be detected before any visual manifestations of damage occurred, test span surface treatments could be directly compared, and the corrosion condition of a structure could be accurately determined using the test regime.

The work performed to date has provided an outline of the information that can be gained through an extensive and comprehensive corrosion protection testing program. A field testing program as described in this report should be implemented to allow TxDOT to better monitor both its test installations and its conventional structures. This will serve as an early indicator of the performance of the protection systems and as an indicator of corrosion damage to come. The information gained through a monitoring program would also assist TxDOT in financial planning and budgeting, as the information would allow more accurate predictions of ultimate structure life and necessary repair timetables.

Prepared in cooperation with the Texas Department of Transportation and the
U.S. Department of Transportation, Federal Highway Administration

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METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	2.54	centimeters	cm
ft	feet	0.3048	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	millimeters squared	mm^2
ft^2	square feet	0.0929	meters squared	m^2
yd^2	square yards	0.836	meters squared	m^2
mi^2	square miles	2.59	kilometers squared	km^2
ac	acres	0.395	hectares	ha
		MASS (weight)		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2,000 lb)	0.907	megagrams	Mg
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.0328	meters cubed	m^3
yd^3	cubic yards	0.0765	meters cubed	m^3

NOTE: Volumes greater than 1,000 L shall be shown in m^3.

Symbol	When You Know	Multiply by	To Find	Symbol
		TEMPERATURE (exact)		
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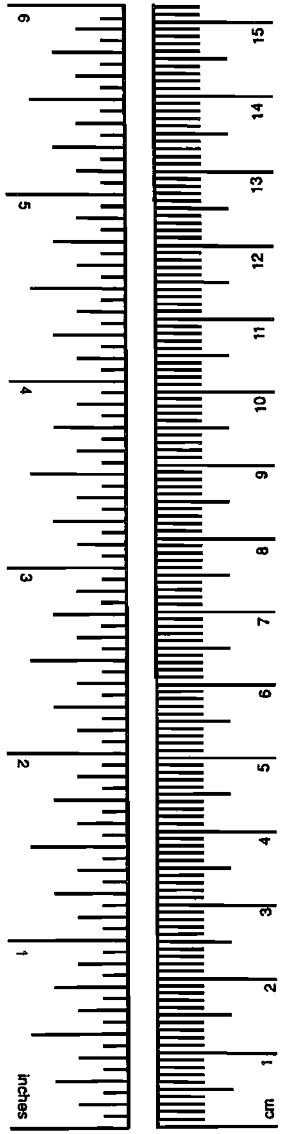

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	millimeters squared	0.0016	square inches	in^2
m^2	meters squared	10.764	square feet	ft^2
m^2	meters squared	1.20	square yards	yd^2
km^2	kilometers squared	0.39	square miles	mi^2
ha	hectares (10,000 m^2)	2.53	acres	ac
		MASS (weight)		
g	grams	0.0353	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams (1,000 kg)	1.103	short tons	T
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	meters cubed	35.315	cubic feet	ft^3
m^3	meters cubed	1.308	cubic yards	yd^3
		TEMPERATURE (exact)		
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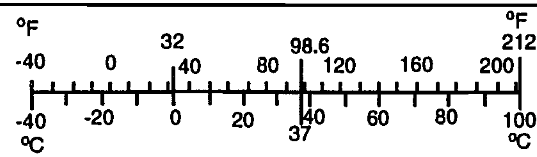

These factors conform to the requirement of FHWA Order 5190.1A.

* SI is the symbol for the International System of Measurements

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SUMMARY

Despite over 25 years of experience with the problem of bridge reinforcement corrosion due to chlorides from deicers or sea water exposure, the solution to the problem is still unknown. Many different corrosion protection systems have shown promise in laboratory and preliminary field studies, but their performance on the basis of long-term field tests has either not been evaluated or has been questionable. In the portion of the work discussed in this report, an extensive literature search was conducted to identify corrosion protection systems and field test methods. In addition, a survey of TxDOT districts was conducted to identify corrosion protection systems used in Texas and to determine the districts' experiences concerning the different systems. Most importantly, a field testing program was developed to evaluate and characterize the corrosion performance of bridges and to increase the information learned from corrosion protection system test installations in Texas.

Preliminary questionnaire surveys distributed to all the TxDOT districts revealed that many different types of corrosion protection systems have been installed in Texas, but that little information on their performance has been collected from the structures since their completion. The field testing program developed in this study was used to investigate eight preexisting bridges representing different protection systems and various overall exposures and service conditions.

The test program, which included determination of half-cell potentials, concrete permeability, chloride content, cracking patterns, and delaminations, was found to accurately reflect the corrosion condition of bridges protected by a variety of corrosion protection measures. The tests selected for use in the field surveys were generally successful and accurate, with different tests confirming each other's results.

Some limited information about the performance of different corrosion protection systems used in Texas was also learned. Dense concrete overlays were found to work well in both remedial and original installation applications, exhibiting very low permeabilities and low chloride contents even after ten winters with deicer exposure. The dense concrete overlays did, however, show a high incidence of cracking and associated chloride penetration at the cracks. Sealers were found to decrease concrete permeability, with water-carried silanes and linseed oil performing the best in terms of reducing permeability and chloride contents.

CHAPTER 1. INTRODUCTION

In the late 1960's, many states adopted "Clear Roads" policies to increase winter roadway safety. Soon after the extensive use of deicing salt was started under those policies, widespread corrosion of the reinforcing steel in bridge structures was noted nationwide. Many different corrosion protection methods were formulated to combat the new deicing salt threat and were implemented to reduce the damage caused by the corrosion and subsequent expansion of the reinforcing steel in the bridges.

1.1 RESEARCH SIGNIFICANCE

Although Texas is often thought of as a warm-weather state, northern portions of the state experience multiple snow and ice storms each year. There are also numerous freeze-thaw cycles, as the structures freeze nightly and thaw during the warmer daytime hours. As a result, many of the state's highways are heavily salted. In Texas, corrosion problems became apparent soon after the implementation of the clear roads policy. Distress was noted on several portions of the structure, such as the deck, piers, bents, and rails, and was evidenced by scaling, spalling, and delamination of the concrete caused by the corrosion and subsequent expansion of the reinforcement contained within the structure. The early onset of corrosion following the deicing salt applications was probably exacerbated by the frequent nighttime freezes and daytime thaws experienced in the affected portion of the state.

1.2 PROBLEM HISTORY

Texas responded to the new corrosion problem by revising its concrete specifications to include lower water-cement ratios, increased air entrainment, higher cement contents, and cleaner aggregates. Also, construction practices were improved. The Texas Bridge Deck Protection System was developed for use in areas where deicing salts were applied. The Texas Bridge Deck Protection System was used to augment or replace the standard boiled linseed oil application used statewide for the purpose of keeping the chloride-laden water from penetrating the concrete deck. This system consists of two or three layers of asphalt in addition to one or two asphaltic seal coats on top of the concrete bridge deck.

In time, however, it became apparent in Texas and elsewhere that asphalt systems such as the Texas Protection System were less than ideal. Apparently, when saturated, asphalt overlays hold chloride-laden water in contact with the concrete surface, while retarding evaporation. One bridge with an asphalt overlay in Virginia was found to contain in excess of 11 pounds of chloride per cubic yard of concrete in the top 1/2 inch of concrete (Ref 32), a large quantity considering that the corrosion threshold is commonly accepted to be 1.1-1.5 lb/yd^3 (Ref 39). More importantly, from an owner's point of view, the asphalt overlays were found to conceal distress in the underlying deck and to allow the deterioration to spread undetected until complete rehabilitation was required. In Texas, slightly distressed overlays were frequently removed, revealing severely damaged decks. This resulted in higher repair costs, increased lane closure times, and motorist inconvenience, and it also required that modifications be made to the rehabilitation contract scope. Because of these experiences, Texas has focused its search for bridge corrosion protection strategies on methods which permit visual inspection of the concrete surface.

1.3 PREVENTION PRINCIPLES

Corrosion of steel in concrete is a complicated electrochemical process and involves many factors such as chloride content, permeability, concrete electrical resistance, water and moisture content, electrical continuity, steel passivity, temperature, pH, and electrochemical potentials. All factors must be favorable to allow extensive corrosion damage to occur. It is this necessary large combination of factors required for corrosion that allows reinforced concrete to be used corrosion-free in most applications.

As the corrosion process involves many "links," the numerous corrosion protection schemes used in Texas and nationwide are similar in objective but different in approach and strategy. Some methods, such as latex-modified or dense concretes, are based on the principle of limiting the permeability of the upper portion of the deck, thus reducing the intrusion of chloride into the deck. Epoxy-coated steel is used in an effort to keep the chlorides in the concrete from contacting and "depassivating" the steel and accelerating the corrosion process. Sealers such as silanes, linseed oil, and siloxanes employ a two-pronged approach: reducing the amount of chloride penetration into the concrete, and reducing the moisture content of the concrete. Other approaches such as calcium nitrite admixtures try to stabilize the "passivating layer" of the steel in high chloride environments, thus preventing corrosion. Lower water-cement ratio concretes, fly ash concretes, and silica fume concretes are used in an effort to reduce the chloride permeability of the concrete, limit the chloride intrusion, increase the electrical resistivity of the concrete, and slow ionic diffusion. Cathodic protection systems use an externally applied voltage to resist the corrosion cell's electrical potential. Last, an increase in cover depth over the reinforcement can also be used to prolong the structure's life by increasing the time required for chlorides to reach the level of the reinforcement.

1.4 RESEARCH OBJECTIVES

Texas has implemented many of the strategies described above on an experimental basis and has sponsored much laboratory research into the underlying protection strategies and their field implementation. Unfortunately, very little full-scale performance evaluations have been undertaken, and follow-up studies have been limited. Typically, the Federal BRINSAP bridge inspections have been relied on as indicators of condition, and no long-term analyses have been conducted to determine why a particular installation was or was not successful in a particular location. A detailed comparison of the protection strategies has not been performed in terms of effectiveness, overall costs, and applicability.

The study's main objective is to identify and evaluate current corrosion protection methods in use in Texas and nationwide, and to determine their effectiveness and applicability in Texas. (Cathodic protection is the topic of study of another research project and thus is not considered in this report.) To this end, the study will establish an investigation and monitoring program to evaluate existing structrues in Texas. Field and laboratory tests to determine the effectiveness of the corrosion protection measures will be performed. The ultimate work product of this project will be the development of implementation guidelines to assist TxDOT engineers in selecting the best corrosion protection to be used for a particular structure.

As an interim product, this report concerns the first portion of the work performed in the project, including the literature search of currently utilized protective strategies and current field testing methods, the corrosion mechanism, the Texas Department of Transportation's experiences with various corrosion protection systems, the results of a survey of current practice, and the findings of the first portion of our field testing program.

The report starts in Chapter 2 by describing the corrosion mechanism and the protection mechanisms of the various protection strategies. In Chapter 3, the various corrosion protection mechanisms are described in greater detail. Chapter 4 gives the background and theory of tests used to characterize corrosion-related performance. The history of corrosion protection in Texas is covered in Chapter 5. Chapter 6 is a "how-to" description of the field surveys performed during the course of the project. The results of the field evaluations are given in Chapter 7, with general conclusions and guidelines given in Chapter 8. A number of appendices are included, each containing the complete results of the investigations conducted at each bridge during the field survey portion of the project.

CHAPTER 2. THE CORROSION MECHANISM

2.1 INTRODUCTION

To determine the most effective way to prevent the corrosion of reinforcing steel in concrete, the mechanism through which the corrosion takes place must be understood. Once the electron and chemical transfer mechanisms and the role of the chloride ion are defined, numerous protection strategies can be adopted to interrupt any one or more of the concurrent activities required for the corrosion to take place.

2.2 CORROSION MECHANISM

In an alkaline environment such as concrete, reinforcing steel is thought to form a "passivating layer" approximately 30 Angstroms thick (Ref 39). This layer stabilizes the basic iron to iron ion reaction

$$Fe_{(s)} \rightarrow Fe^{2+}_{(aq)} + 2e^- \qquad \text{(Eq 2.1)}$$

which takes place naturally in the presence of water and results in a non-corroding equilibrium. The passivating layer offers protection to the reinforcing steel in a manner similar to the way in which the green patina that develops on exposed copper prevents further deterioration. When the passivating layer comes into contact with a sufficient amount of approximately 1.0 to 1.5 lb/yd^3 (Ref 39) chloride ion in a moist environment, the protection breaks down. The exact mechanism for this loss of protection is not known (Ref 39), but is thought to take one of the following two forms.

The first theory is that the chloride ions disrupt the passivating layer allowing the chloride ions to react directly with the iron ions via the following reactions:

TRANSPORT

$$Fe^{++}_{(aq)} + 6Cl^-_{(aq)} \rightarrow FeCl_6^{-4}{}_{(aq)} \qquad \text{(Eq 2.2)}$$

$$Fe^{+3}_{(aq)} + 6Cl^-_{(aq)} \rightarrow FeCl_6^{-3}{}_{(aq)} \qquad \text{(Eq 2.3)}$$

PRECIPITATION

$$FeCl_6^{-3}{}_{(aq)} + 2OH^-_{(aq)} \rightarrow Fe(OH)_{2\,(s)} + 6Cl^-_{(aq)} \qquad \text{(Eq 2.4)}$$

Note that the ions formed by the first two reactions are in solution and migrate away from the reinforcement through the concrete by ionic diffusion before reacting with the hydroxides to precipitate out as $Fe(OH)_2$ solids. This reduces the Fe^{++} concentration in the region of the reinforcing steel, requiring the drawing out of more steel ions in order to maintain ionic equilibrium. In essence, under this theory, the transport and precipitation reactions continue indefinitely.

The second theory of depassivation is based on the ability of the chloride ions to pass directly through the passivating layer once a sufficient concentration gradient has been established. The theory follows the following reactions:

TRANSPORT

$$2Fe_{(s)} + 6Cl^-_{(aq)} \rightarrow 2FeCl^-_{(aq)} + 4e^- \qquad \text{(Eq 2.5)}$$

PRECIPITATION

$$FeCl^-_{(aq)} + 2OH^-_{(aq)} \rightarrow Fe(OH)_{2\,(s)} + 3Cl^-_{(aq)} \qquad \text{(Eq 2.6)}$$

In both of the above reactions, it should noted that the precipitated corrosion product is not "red rust" as commonly thought, but is instead a pasty grey-black $Fe(OH)_2$ solid.

For either of these theoretical reactions to take place, a simultaneous oxygen-reduction must take place to "absorb" the electrons produced in the "transport" reaction and to supply the hydroxides used in the "precipitation" reaction. This cathodic reaction takes the form:

$$\frac{1}{2}O_{2\,(g)} + H_2O + 2e^- \rightarrow 2OH^-_{(aq)} \qquad \text{(Eq 2.7)}$$

As a group, all of the above reactions form the corrosion cell graphically represented by Figure 2.1.

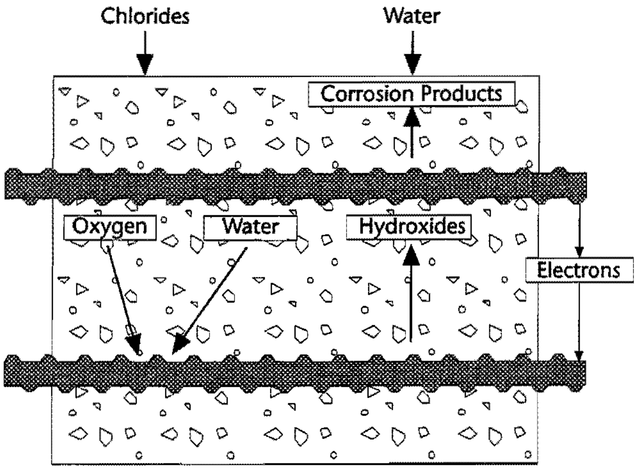


Figure 2.1 Schematic of the corrosion cell

2.3 IMPLICATIONS FOR CORROSION PROTECTION

The most important information to be gained from the knowledge of these chemical reactions is the role of oxygen, chloride, water, and electrons in the reactions. In order to use the knowledge of the corrosion process in combating the problems experienced in bridge decks, the following properties of the process must be recognized and understood:

1. Although the chloride ion is necessary for the initiation of the corrosion process in the steel, it is not consumed or bound in the reactions. Once it has reached the steel, the chloride is there to stay. It acts only as a catalyst or accelerator to the reaction.
2. The oxygen, although a critical part of the corrosion process, is not a part of the primary reactions taking place at the corroding anode. The only time that oxygen is active at the corrosion location is in the secondary formation of Fe_2O_3 "red rust" at the anode as described by the following reaction:

$$Fe(OH)_2 \rightarrow FeO + H_2O \qquad \text{(Eq 2.8)}$$

$$4FeO + O_2 \rightarrow 2Fe_2O_3 \qquad \text{(Eq 2.9)}$$

Note that these reactions do not have to occur for the corrosion process to take place, and that they frequently occur only after cracking has allowed oxygen to reach the surface of the steel. An example of this is the observed transformation of black corrosion products on a piece of corroded reinforcing steel retrieved from an underwater piling into red rust only upon exposure to air at the water surface (Ref 39). The oxygen's critical role is at the non-corroding cathode. In bridge deck applications, the cathodic area is typically the bottom mat of steel. In substructures, this would typically be the steel in the above-ground surface areas.

3. Water plays a critical role in two portions of the corrosion of steel in concrete. First, water is needed at the cathode to participate in the production of the hydroxides needed to consume the electrons produced by corrosion at the anode. Second, the presence of water in the concrete allows all of the ions generated during the corrosion process to migrate through the concrete.
4. Last, the electron flow is important. The entire corrosion process relies on the flow of electrons through an electrically conductive material (such as reinforcing bars, chairs, wire ties, etc.) to perpetuate the reaction. If electrons cannot be consumed at the cathode, they will not continue to flow from the anode. This will stop the corrosion process.

2.4 CORROSION PROTECTION STRATEGIES

In terms of corrosion protection, the consequences of the above mentioned necessary roles of the chloride ions, oxygen, water, and electron flow are that the interruption of any one of their activities will stop the entire corrosion process. This is what allows corrosion protection strategies to work. Corrosion protection systems typically use one of five theoretical approaches to take advantage of the properties of the corrosion cell to reduce or eliminate corrosion damage to the bridge reinforcing steel.

2.4.1 Concrete Barrier

The most direct approach to corrosion protection is to prevent the chlorides in the concrete surface from reaching the reinforcing bars, thus maintaining the passivating layer and preventing corrosion. This approach is represented, among others, by the use of dense concrete, asphalt-based membranes, latex-modified, and polymer concrete overlays. All of these measures decrease the permeability of the top portion of the concrete. This reduces the amount of chlorides that can reach the level of the reinforcing steel. Other methods such as increased cover, lower water-cement ratio, silica fume admixtures, and fly ash admixtures result in lower permeability concrete and in a decrease in the ability of the chloride ion to reach the level of the reinforcement. As a result, the service life is increased by increasing the time required for the threshold concentration of chlorides to build up at the reinforcing steel.

2.4.2 Steel Coating

Epoxy-coated reinforcement assumes a strategy similar to that for the concrete barrier discussed in Section 2.4.1, but instead of preventing the necessary chloride build-up in the concrete at the reinforcing steel, the coating prevents the chlorides from reaching the surface of the reinforcing steel itself. The coating also serves to increase the electrical resistance of the bar-to-bar connections, breaking the electrical circuit present in macrocell, or multiple bar, corrosion. By electrically isolating the reinforcement, the corrosion cells are forced to establish themselves on individual bars, rather than over large bar areas. Since ion transport is a large factor in corrosion rates, the necessity for having both anodic and cathodic regions present in the same bar, or even at different regions within the same coating break, can greatly limit the corrosion activity. The coating also serves to reduce the oxygen and water available for the cathodic reactions.

2.4.3 Chemical Stabilization

Calcium nitrite and other similar proprietary concrete admixtures work by stabilizing the passivating layer and competing with the chloride ions for the available Fe^{++} ions. By stabilizing the passivating layer, the chloride concentration threshold level for corrosion can be increased beyond the conventionally accepted 1.1-1.5 lb/yd^3 to much higher values. This prolongs the structure's life by increasing the time required for the chlorides to build up to the necessary concentrations for corrosion to occur.

2.4.4 Surface Sealers

Surface sealers such as silanes, siloxanes, methacrylates, and linseed oil aim to prevent chloride intrusion into the concrete. By providing a surface barrier, the underlying concrete is kept relatively chloride-free, and thus corrosion-free. Some sealers also work as breathable moisture barriers, preventing water intrusion while allowing water vapor to exit the concrete. This allows the underlying concrete to dry over time as the excess water exits during warm and dry periods as vapor. By reducing the moisture content, the mobility of chloride ions, complex iron ions, and hydroxides is reduced, reducing the corrosion rate. The water reduction effect further reduces corrosion by reducing the concrete's ion transmissibility and increasing its electrical resistance.

2.4.5 Electrochemical

Cathodic protection is the only corrosion protection system recognized by the Federal Highway Administration as stopping the corrosion of reinforcing steel in bridges (Ref 202). External impressed cathodic protection systems work by applying an external voltage, or potential, to directly counter the corrosion potential. The externally applied potential forces the reinforcing steel to act as a cathode, while an external anode of various form attracts the chlorides while undergoing no corrosion due to its material properties. This form of protection is not being considered in this study, as it is being investigated in another ongoing TxDOT project.

2.5 CORROSION DETECTION AND MONITORING

In order to minimize the impact of corrosion-related damage, corrosion activity must be detected and monitored before widespread damage has taken place. Unfortunately, the first visible signs of widespread corrosion are delaminations and surface spalling. The absence of early visible warning is often compounded by the presence of a damage-hiding asphalt overlay. This has necessitated the development of a non-destructive test for detecting the presence of active corrosion. The most commonly utilized test for this purpose is the half-cell potential test, detailed in ASTM specification C876. Other tests, such as the linear polarization or corrosimeter, have been used, but have not gained widespread use.

Only the half-cell test was used for corrosion detection and monitoring in this project. Among the advantages of this test are low cost, simplicity, and ease of conduct. Although the use of the test is described in detail in Section 4.2.1 of this report, the mechanism of the test will be discussed in this section.

The presence of corroding steel inside concrete produces an electrical potential, much as the lead plates and acid of an automobile battery do. As active corrosion begins and the Fe^{++} ions are transported away from the reinforcing steel as described in Section 2.1, some of the reinforcing steel must decompose into its ion form to retain ion equilibrium. This "need" to change form is measured in terms of the steel's potential. This potential is an indicator of how much the solid substances need to change into their ionic states to preserve equilibrium, and thus is an indicator of the corrosion potential of the material.

In order to determine the corrosion potential of the reinforcing steel in the concrete, a reference cell must be used. The use of a reference cell offers a known stable value to measure against. For the ASTM test, this reference cell consists of a copper rod immersed in a saturated copper sulphate solution. By measuring the voltage between the reinforcing steel and the reference cell placed on the surface, the relative potential of the steel can be determined. This allows the determination of the corrosion state of the reinforcing steel.

It must be noted, however, that while the corrosion potential can be an indicator of corrosion activity by showing the relative concentrations of Fe^{++} and complex iron ions at the steel-concrete interface, it cannot show the *amount* of corrosion taking place. The amount of steel lost to corrosion is purely a function of the corrosion *current* flowing in the corrosion cell, and is described by the following equation:

$$W = kIt \qquad \text{(Eq 2.10)}$$

where W is the weight loss, k is a constant, I is the corrosion current, and t is the elapsed time. In corrosion of steel in concrete, the corrosion current is a function of the driving potential, the resistivity of the concrete, and the hydroxide mobility. The hydroxide mobility governs the speed with which the ions can move through the concrete to carry the negative charges, and is a function of temperature, chloride content, and permeability.

CHAPTER 3. LITERATURE SEARCH: STATE OF THE ART—CORROSION PROTECTION SYSTEMS

3.1 INTRODUCTION

A thorough literature search for corrosion protection system information was performed utilizing the library facilities at the Center for Transportation Research at The University of Texas at Austin. This was done to identify the most promising corrosion protection systems for further investigation during the field survey portion of the project. A number of protection systems were identified as potential alternatives for use in Texas highway bridges. A detailed listing of the references cited in the literature search is contained in the reference section of this report.

As discussed in Chapter 2, corrosion protection strategies fall into five main categories: Steel Coating, Electrochemical, Concrete Barrier, Chemical, and Sealers. Specific strategies will be discussed in the following sections, including the primary corrosion protection mechanism and the strengths and weaknesses of each system. Emphasis is placed on the mechanisms through which the protection measures work, as this will aid in understanding the merits and limitations of the protective measures.

3.2 STEEL COATING METHODS

3.2.1 Epoxy-Coated Reinforcing Bars

The purpose of epoxy-coating steel is to prevent chloride and moisture from reaching the surface of the reinforcing steel. If the chlorides cannot contact the surface of the steel, they cannot break down the stable passivating layer of the steel and the corrosion process can never begin.

Epoxy-coated bars were first used in the early to mid-70's. The coating itself is a fusion-bonded bisphenol-amine epoxy coating applied to blast-cleaned reinforcing bars at high temperatures. It effectively prevents chloride ions, oxygen, and moisture from reaching the bar surface. To date, only four problems in structures containing epoxy-coated reinforcing outside the Florida keys have been detailed in the literature (Ref 165). Epoxy-coated bars tested in top mat and both mat configurations have been protected from corrosion even though the concrete chloride content was 20 times greater than the accepted corrosion threshold value (Ref 121). Epoxy-coated reinforcement's greatest advantage lies in its applicability to existing boilerplate designs and standard installations with no changes in load capacity or section size, requiring only modifications of required development length. Its apparent simplicity aids in general acceptance, as no new skills are required to design, build, and maintain concrete structures built with epoxy-coated reinforcement. Another strength of epoxy-coated reinforcement is its ability to protect all portions of a bridge structure, including the deck, substructure, and superstructure. Lastly, epoxy-coated reinforcement is a "one-time" protection method. It is installed as an integral part of the construction process and requires no further monitoring or maintenance.

To date, many tests and studies have investigated epoxy-coated reinforcing steel (Refs 22, 56, 61, 122, 162 to 166, 184, 186, 187) and have found the overall performance to be good. Tests have found that even if the bars develop corrosion at times comparable to uncoated bars, the corrosion rates are typically an order of magnitude less (Ref 165). This results in better overall performance with less steel loss and less concrete damage. It is important to note that the majority of installations are relatively new and that no life expectancy has been determined.

The major weakness of the epoxy-coated bar is its susceptibility to surface coating damage during fabrication and construction. Damaged areas in the coating can lead to local corrosion microcells and subsequent disbonding of the coating. These problems are best avoided through care in handling and fabrication, repair of surface defects, and education of the bar handlers and installers. The only major performance problems with epoxy-coated bars have been encountered in marine substructure use of epoxy-coated reinforcing bars (Refs 163, 186). In these cases, the bars have been installed vertically, crossing from the submerged to the air-exposed zone. Recent research has also revealed a possible disbonding problem in bars continuously exposed to moisture (Ref 186).

It is important to note that much of the basic background research regarding the corrosion of steel in concrete may not be directly applicable to the corrosion of epoxy-coated reinforcement in concrete. Conventional reinforcement has virtually unlimited surface area available for corrosion, whereas epoxy-coated steel has only a small area of exposed steel at nicks, cuts, and spalls in the coating. Because of this, the corrosion of epoxy-coated steel may be more dependent on the physical restraints of ion transport within the concrete (Ref 39), resulting in different performance characteristics.

3.2.2 Inorganic Coatings

Zinc-silicate inorganic coatings have been used to protect above ground structural steel since the late 60's (Ref 214). They consist of zinc chemically bonded to a silica-based carrier. The carrier facilitates film formation while sealing the surface of the steel (Ref 212). The material works by coating the reinforcement to prevent chloride and moisture access, as well as anodically sacrificing itself to protect the underlying steel should conditions favorable for corrosion form. Recently, the technology has been applied to the protection of reinforcing steel. No test results have been published, but research is being conducted.

The water-based materials are self-curing, compliant with all pending volatile organic compound (VOC) environmental regulations, and require no heat for fusion (Ref 213). They are painted on like normal paint and are cleaned up with water only. The paint can be field or shop applied without any loss in quality.

3.3 ELECTROCHEMICAL METHODS

3.3.1 Cathodic Protection

As discussed in Chapter 2, a battery-like electrolytic cell is established when reinforcing bars corrode in a concrete environment. Cathodic protection works by providing an external electric potential that opposes this "natural" corrosion cell. The electric potential established by the cathodic protection system prevents the flow of electrons associated with the reinforcement corrosion, and the corrosion process is stopped. The electrical potential is established between an electrically continuous current distribution system, working as an anode, placed on top of the existing deck and the existing reinforcing mat in the structure that is used as the cathode of the system.

The anode is usually covered with some type of overlay. In substructure applications, the anode is usually placed on the exterior surface of the member to be protected and covered with some form of shotcrete. The anode can consist of an embedded wire grid or a specially coated expanded metal mesh. During operation, an electric potential is applied to the system, causing negative ions such as chlorides to migrate towards the positively charged anode and away from the reinforcing bars. The applied potential also supplies electrons to the reinforcing steel, satisfying the cathodic demand for electrons due to any oxygen-reduction occurring at the steel surface that would normally drive the corrosion cell.

The primary drawbacks of cathodic protection systems are their technical sophistication and high initial expense. In their current evolution, cathodic protection systems cannot be monitored and maintained by non-technical personnel, making expensive engineer site visits necessary (Ref 9). Presently, maintaining the potential at a level above that necessary to prevent corrosion while keeping it below the level associated with cathodic disbondment and hydrogen evolution at the reinforcing steel is a major problem. The high installation and maintenance costs are also a barrier to widespread use. In order to be economically feasible, cathodic protection systems should be expected to be in use for approximately thirty years after installation, depending upon the actual bridge characteristics such as location, importance, and future use (Ref 119). Also, the dead loads on the structure can be substantially increased, reducing structural capacity. Lastly, the system requires an external power supply, restricting its use in rural areas.

The system's main merit lies in the fact that it is the only method proven to stop the corrosion process (Ref 202). For this reason, cathodic protection should be considered for the protection of high replacement cost or highly utilized structures such as long-span river crossings on highly traveled routes.

Although the systems can be retrofitted to existing structures, it is simpler to build with possible cathodic protection in mind. This entails ensuring that the entire reinforcing mat is electrically connected. Note that this effectively precludes the use of epoxy-coated steel (Ref 186). As cathodic protection halts the corrosion process at any time, it can be used as a remedial measure as well as an original protection method. In other words, a structure could be built with a cathodic protection system which would be activated only when the first signs of corrosion are seen. This would save monitoring and maintenance costs during the early part of the structure's life.

3.4 CONCRETE BARRIER METHODS

3.4.1 Introduction

In general, concrete barrier methods work by making all or part of the concrete less permeable to water and the associated chloride ions. Low permeability overlays create a protective layer over the conventional concrete base layers. By decreasing the permeability, the time required for the necessary threshold chloride concentrations to accumulate at the level of the reinforcing steel is increased. This increases the service life of the structure. Also, lower permeability concretes decrease the penetration of water, allowing for drying of the concrete which results in a reduction of corrosion ion mobility. Non-overlay methods work by decreasing the permeability of the entire concrete mass, slowing the ionic transport necessary for corrosion to occur in addition to slowing chloride ingress and reducing water penetration. Overlays work as armor, protecting the underlying concrete from the harmful environment. These methods slow the corrosion rate once it starts, as well as increase the overall time to the start of corrosion.

3.4.2 Latex-Modified Concrete Overlays

Latex-modified concrete consists of a conventional portland cement concrete supplemented by a 40 to 50 percent emulsion of styrene-butadiene latex. The concrete is usually mixed with a latex solids portion approximately 15 percent by weight of cement and with a water-cement ratio of less than 0.39. Due to cost, latex-modified concretes are usually used for overlays only, although they could be incorporated into mass concrete work. As in other overlay methods, the latex-modified concretes help to prevent chloride penetration into the deck. The latex emulsion lowers the overlay concrete's permeability to water and chloride by sealing some of the internal capillaries and voids in the concrete paste (Ref 120).

The latex-modified concrete is not waterproof, but has been found to have a permeability approximately 12 percent of conventional portland cement concrete (Ref 125). In 16-year tests performed in Virginia, it consistently outperformed conventional concretes and low-slump dense concrete. Latex-modified concrete overlays can be expected to last over 20 years (Ref 125) before needing replacement at 25 years due to rutting and general wear (10). Latex-modified concrete overlays can extend the life of a bridge even though some corrosion can continue in the conventional concrete under the overlay. Latex-modified concrete has also been shown to perform better in freeze-thaw resistance than conventional concretes (Ref 125). The overlays can be incorporated into the original construction, used as a remedial measure after some deterioration has started, or placed after ride quality has diminished. Remedial use, however, allows for less protection as a decrease in moisture penetration is the only protection offered to decks already contaminated with chloride.

Latex-modified overlays do not perform flawlessly. Problems with plastic shrinkage cracks that deepen with age have been encountered (Ref 125), as have scaling problems in continuously saturated areas such as gutters (Ref 10). The influence of cracking on corrosion is not well understood, but it is believed that extensive cracking will decrease the overlay's ability to resist chloride intrusion. Also, the permeability of the overlays has been found to increase with time, decreasing the overlay's effectiveness.

3.4.3 Dense Concrete Overlays

Dense concrete overlays are also known as "Low-Slump Dense Concrete Overlays" due to the stiffness of the material if a high range water reducing admixture or "superplasticizer" is not used. The concept of Low-Slump Dense Concrete (LSDC) is an old one. By increasing the cement content to approximately 820 lb/yd^3 and decreasing the water-cement ratio to approximately 0.30, the concrete's natural capillary void system can be reduced, decreasing the permeability and increasing the strength. Like latex-modified overlays, they are typically used as overlays due to higher material cost. LSDC overlays are used in original or retrofit construction and have the same performance limitations as latex-modified overlays in retrofit performance. Like latex-modified overlays, they work by preventing chloride and moisture intrusion into the underlying deck material.

LSDC overlays have performed relatively successfully, giving problem-free lives of 5 to 13 years (Ref 61). The Alberta DOT in Canada has installed over 100 LSDC overlays (Ref 127) and has found them to "adequately slow rebar corrosion rates ... where less salt and rain are present" (Ref 119). Because of their low-slump characteristics, LSDC overlays are well suited for use in high grade areas where there are placement problems with other systems. The permeability of LSDC overlays is somewhat higher than that of latex-modified overlays at first, but their permeability decreases with time to a "low" classification comparable to that of latex-modified overlays (Ref 120).

The main problems associated with LSDC overlays are difficult placement, surface cracking, and high material cost. The problems in placement can be overcome somewhat with the use of high range water reducers (superplasticizers), but the stiffness of the material can lead to overworking or the addition of water in the field, leading to reduced performance. Adding to the placement problems is the need for "quality" or extra curing to allow the maximum possible cement hydration. The lack of effective curing can exacerbate the cracking problem because shrinkage cracking increases with a decrease in curing time. As in latex-modified overlays, the effect of cracking on chloride resistance is not clearly understood, so cracking should be minimized. Scaling can also be a problem, especially in curb and gutter areas. Lastly, the skid resistance of LSDC overlays is slightly less than that of latex-modified overlays (Ref 10).

3.4.4 Polymer Concrete Overlays

Polymer concrete is simply concrete in which a polymer replaces portland cement. Typically, they are applied to the deck in layers of 0.5 inches or less in thickness, and have fine aggregate or sand "seeded" into them, resulting in higher skid resistance than conventional concretes (Ref 126). Polymer concretes are not typically used to protect curbs, rails, substructures or any other bulk applications due to the material costs. Polymer concretes with medium to large size aggregates added are used in repair applications because of the portability, short set time, and high early strength of the material.

Uncracked polymer overlays are nearly or totally waterproof and protect the deck by preventing the intrusion of both chlorides and water. Due to cost and application difficulty, they are typically used in bridge overlays as a remedial means in order to slow down the corrosion and to restore the structural aspects of the deck, or to restore its riding surface. In Alberta, thin polymer concrete overlays are used as a preventative measure on sound decks in which damage is anticipated due to active corrosion potential readings, or on decks with known air-void system deficiencies (Ref 119). Polymer concretes have also been used to repair heavily cracked and deteriorating low-slump dense concrete overlays. A major advantage of polymer concretes is their fast set and cure time. Also, their low thickness eliminates the need for transition zones or for rebuilding approaches. Repairs can be made in a matter of hours, with no major disruption of traffic.

Field application techniques are critical for polymer concretes. Overlay performance is highly dependent on the strength of the bond between the overlay and the underlying concrete. This bond is in turn highly dependent on deck preparation and cleanliness. The underlying concrete must be substantially dry at the time of application in order to avoid trapping moisture. With polymer concrete, extra surface preparation will pay off with extended life (Ref 127). Expert installation is required, as most failures are due to workmanship or improper handling of the materials (Ref 127). Reflective or "telegraph" cracking can also be a problem. In Alberta, where 66 polymer overlays have been installed, up to 70 percent of the underlying cracks reflect through to the surface, and Alberta Transportation and Utilities (ATU) no longer even attempts to repair cracks, stating that crack repair is "a waste of money" (Ref 127). Although they do not try to repair cracks, ATU reports that routine maintenance of polymer overlays is necessary to restore skid resistance to polished fine aggregate in the wheel paths and to repair minor spalls.

3.4.5 Asphalt Overlays

Asphalt overlays have been used for bridge deck protection since their introduction as a paving material. Many states traditionally used asphalt overlays in an effort to protect their decks against chloride and water intrusion. The use of asphalt-based membranes, both as initial protection and as a corrective measure, is currently used extensively in the northeast, especially in the New England states. Texas has included the "Texas Protection System" in its standard specifications for protection of bridges throughout the state. The Texas Protection System is a multi-layered asphalt overlay 1-1/2 to 2 inches in thickness, with various layers of latex asphalt, asphalt surface treatments, and asphaltic concrete overlays. By their nature, these asphaltic overlays are suitable for deck use only. They are installed any time after the underlying concrete deck has been completed, as original design, or later, to restore rideability and skid resistance.

After considerable field experience, it was determined that there were serious problems associated with the overlays. When overlays were removed to perform small repairs or resurfacing, large areas of deteriorated and delaminated concrete were found. This added great repair expense to the projects. Apparently, the overlays eventually leaked and allowed the water to become trapped in the sponge-like asphalt. With time and continued exposure, the chloride-laden water caused widespread deterioration in the underlying deck before any problems became apparent during surface inspection. Asphalt overlays are still used where the primary deterioration has been in the form of loss of skid resistance and rideability, but are falling into disfavor as corrosion protection strategies.

3.4.6 Increased Concrete Cover

In the mid-70's, soon after the corrosion associated with shallow cover over the reinforcing steel was recognized, a number of agencies modified their protection schemes to include additional cover to the reinforcing bars. Although an increase in concrete cover does nothing to slow or prevent the corrosion process and as such is not a true protection scheme, it is included here as an alternative "protective" strategy. Instead of preventing or retarding the corrosion, the increase in cover simply delays the onset of corrosion. The increase in cover forces the chloride ions to migrate further, and thus take longer, before the critical corrosion concentrations can be reached at the reinforcing bar locations. In many cases this relatively low-cost alternative is sufficient to allow the bridge to reach a service life dictated by other factors such as capacity, clearance, and ride quality. In New York State applications, installations with 3-inch cover over the reinforcing are expected to provide damage-free performances of 16 years, and a mean service life of 37 years (Ref 113). This was partially confirmed by recent reports of 10-year problem-free performances (Ref 113).

Although an increase in cover could theoretically be used to protect all portions of bridge structures, only an increase in cover over deck reinforcing has been used extensively in practice. The cover increase is part of the construction process, and except for possible crack sealing, it requires no further maintenance.

The biggest question regarding the performance of three-inch cover applications is the role of cracking. Crack frequencies two or three times greater than those of conventional two-inch cover installations have been experienced (Ref 113), and extensive cracking has been linked to increased corrosion potentials. Even though a recent New York State report concludes that there is "no evidence that the increased [cracking] frequency has been associated with diminished performance" (Ref 113), the long-term effects are unknown at this time.

3.4.7 Polymer Impregnation

Polymer impregnation of concrete protects against corrosion by limiting the access of oxygen, chloride, and water to the reinforcing steel by sealing the concrete's surface pore system with an externally applied polymer material. Deep impregnation also works by replacing the chloride-water electrolyte with a corrosion-stopping dielectric polymer in the region around the reinforcement (Ref 12). Polymer impregnation has been found to reduce the rate of corrosion by approximately 50 percent in field applications, and performance in field trials has been comparable to that in laboratory trials (Ref 114). Also, polymer impregnation has been found to reduce wheel path wear in test installations.

Polymer impregnation is applicable to use in protecting all areas of bridge structures, but has been used mainly in protecting decks. This is due to the high material, labor, and equipment costs associated with a polymer impregnation treatment. As an externally applied system, polymer impregnation can be used during construction, at first distress, or whenever necessary to restore structure performance.

The primary difficulty associated with polymer impregnation has been achieving successful deep impregnation. To ensure sufficient depth of penetration, grooves may need to be sawn into the deck. Also, a low moisture content of the deck is critical to achieving deep impregnation depths. However, the drying of the deck can be difficult, and can lead to excessive thermal stresses during the heating and cooling process. Polymer impregnation has fallen into disuse because of application difficulties.

3.4.8 High-Performance Low Water/Cement Ratio Concretes

Another protection alternative is to simply improve the current protection method of encasing the reinforcing in a low permeability alkaline concrete environment. This is accomplished through improved finishing practices, by decreasing the water-cement ratio, and by increasing the tricalcium aluminate content of the cement. By producing a sound deck with no cracks and low permeability, the amount of chloride that penetrates to the reinforcing steel is reduced, minimizing the damage. Cracking has been linked to settlement cracking and surface finish problems (Ref 168). By waiting until no bleed water is left on the surface before finishing, the concrete quality can be increased.

Also, a minimum moist cure time of 72 hours can reduce surface cracking (Ref 168). Research has shown that the permeability is highly related to the water-cement ratio. A decrease in the water-cement ratio from 0.40 to 0.30 results in a 400 to 500 percent decrease in permeability. A decrease from 0.60 to 0.28 results in a decrease of 1,500 percent (Ref 60). Obviously, a decrease in water-cement ratio is beneficial in performance.

Lastly, corrosion performance has been linked to the cement's tricalcium aluminate (C_3A) content (Refs 168, 104, 105, 106). An increase in C_3A content of the cement from 0 to 12.6 percent results in a fivefold decrease in soluble chloride in the concrete matrix. The C_3A binds the chloride ions, preventing them from depassivating the reinforcement. Unfortunately, high C_3A contents are detrimental to sulfate attack resistance, and combine with the sulfates preferentially to the chloride. In other words, chlorides bound by the C_3A are released upon exposure to sulfates.

3.4.9 Silica Fume and Fly Ash Modified Concretes

Concretes are produced with the addition of silica fume and fly ash for a number of reasons, including strength, cost, and permeability. In these concretes, the mineral admixture combines with the by-products of the portland cement hydration to form more of the calcium silicate hydrate binder, resulting in a stronger and less permeable material. Concretes with added silica fume or fly ash can be used to protect all areas of bridge construction.

Although the two materials work similarly, their performance history is slightly different. Silica fumes are acknowledged to be beneficial, significantly delaying or preventing corrosion in the best case, and only slowing the corrosion rate in the worst (Ref 107). The addition of silica fume has been found to decrease the permeability of concrete by a factor of 10 or more (Ref 21). The corrosion performance of silica fume modified concretes is due to its lower permeability that makes the ionic transport of corrosion participants more difficult, and to its higher electrical resistance that lowers the overall corrosion rate. This increase in electrical resistance as compared to that of similar non-silica fume concretes has been found to be a factor of 6 to 16 (Ref 107).

The performance evaluation of fly ash modified concretes is less conclusive. While the addition of fly ash is generally accepted to lower the permeability of concrete, some studies using specimens with intentionally admixed chlorides have found an increase in corrosion over non-fly ash companion specimens (Ref 108). On the other hand, in studies using externally applied chloride solutions, the fly ash concretes have performed better than the control specimens (Ref 109). This difference in performance has been attributed to the fly ash's consumption of the free OH^-ions in the concrete. This reduces the Cl^-/OH^-ratio in the concrete pore solution, which some link to corrosion performance (Ref 109).

3.5 CHEMICAL CORROSION PROTECTION STRATEGIES

3.5.1 Introduction

Chemical protection strategies rely on changing the environment within the concrete to prevent or reduce corrosion. They typically work by preventing or slowing the "depassivating" effect of the chlorides on the reinforcing steel. By changing this aspect of the corrosion process, the effect of chloride and water intrusion of the deck is negated. Expensive waterproofing and protection strategies are not necessary when implementing a chemical corrosion protection treatment.

3.5.2 Calcium Nitrite

Calcium nitrite was used as a set accelerator long before its properties as a corrosion inhibitor were known. As an accelerator, calcium nitrite improves concrete strength (Ref 74). Calcium nitrite is usually prepared as a 40 percent stable solution and is added in the mix water, with a 2 gal/yd^3 dosage protecting against approximately 6 lb/yd^3 of chloride in the concrete. Calcium nitrite acts to stabilize the "passivating layer" that surrounds uncorroded steel in concrete via the following reaction:

$$2Fe^{++} + 2OH^- + 2NO_2^- \rightarrow 2NO\uparrow + Fe_2O_3 + H_2O \qquad \text{(Eq 3.1)}$$

The Fe_2O_3 formed is a stable material and serves to further protect the steel, while the $2NO_2^-$ in the left half of the equation directly "competes" with the Cl^- ions, further protecting the steel. A key feature of calcium nitrite is that it does not affect the concrete permeability. As a result, it does not prevent chloride intrusion, but rather competes with the chloride to react with the steel and subsequently reduces the corrosion rate by "more than an order of magnitude" (Ref 21). In other words, it does not delay the onset of corrosion, but instead lessens the severity to one-tenth of that of normal bars in untreated concrete.

As an admixture, the calcium nitrite must be added to the fresh concrete during construction. No additional maintenance is required once it is in place. Calcium nitrite protection is applicable to all areas of the bridge structure. It also has the added benefit of requiring no modifications to existing "boilerplate" designs or conventional design aids.

One of the drawbacks of calcium nitrite use is its accelerating property. Usually a retarder must be added to the concrete to offset this. Also, as a chemical binder system, the amount of chloride exposure must be estimated to properly dose the admixture. This can be difficult, as different areas of the bridge structure receive different exposures.

3.6 SEALER PROTECTION METHODS

3.6.1 Introduction

A multitude of concrete sealers are presently available for the protection of concrete structures. They all fight corrosion by preventing water and chloride from entering the concrete. Some sealers form an impermeable microscopically thin layer on the concrete surface, while others penetrate more deeply and typically form a "breathable" barrier. Because of skid resistance concerns, some impermeable surface sealers are not suited for bridge deck use. Of prime importance is the vapor transmission characteristics of the sealer. If moisture cannot pass through a sealer to escape from the concrete, high vapor pressures will build up inside the concrete during drying periods, and result in blistering and peeling of the sealer material. When a sealer allows water vapor to exit the concrete while keeping liquid water from entering, a one-way valve is created. This allows for drying of the concrete during dry periods.

3.6.2 Silane and Siloxane Sealers

Silanes and siloxanes are silica-based materials that react with water applied after application to form a silicone resin. This resin is a hydrophyllic compound which chemically bonds to the surface of the concrete capillary pores. Silanes and siloxanes do not seal the concrete per se, but instead they penetrate 0.2 to 0.3 inches into the concrete (Ref 133) and repel liquid water and the chloride contained therein while allowing water vapor to pass through. The permeability to water vapor prevents moisture trapping problems, allowing the concrete to dry internally while resisting the intrusion of outside water. Silanes have been found to decrease the permeability of concrete to chloride ion by as much as an order of magnitude, offering good protection "provided deterioration has not progressed too far" (Ref 135). The primary difference between silane and siloxane is the molecule size. Siloxanes are silanes that have been allowed to polymerize slightly, giving them a larger overall size.

Since the silanes and siloxanes must penetrate and line the concrete pores to provide protection, matching the treatment to the concrete is important. Different concretes will need different coverage rates and sealer molecule sizes depending upon total porosity and the capillary void sizes (Ref 135). Surface preparation is also critical. A dry surface will allow deeper penetration of the silane or siloxane, and proper cleaning is necessary to ensure that the treatment can reach the concrete surface to chemically react and bond to it. Consequently, all oil, laitance, curing compounds, and general road grime must be removed prior to application. Existing silanes or siloxanes need not be removed prior to the recommended 5-year reapplication, as subsequent applications typically improve the performance over that of the original applications (Ref 135).

As a surface sealer, silanes and siloxanes can be applied at any time during or after construction. Their chief strength is their ease of application. They are suitable for use on all portions of the structure.

The main difficulties associated with these sealers are the surface preparation requirements and the difficulty in testing and screening materials submitted for use in projects. Also, high material costs for small purchase amounts can increase project costs. Some owners have combated these problems by testing and purchasing these materials on a district-wide basis and providing the selected materials to the contractors on an "at-cost" basis. This eliminates testing and purchasing on a job-by-job basis.

3.6.3 High Molecular Weight Methacrylates

High Molecular Weight Methacrylates (HMWM's) have been used extensively in the United States in California and Virginia, as well as in Alberta, Canada. They are generally used as crack sealers or thin overlays utilizing sand and fine aggregate to aid in skid resistance. HMWM's are typically three-component systems: a monomer, a promoter, and an activator, all of which must be properly mixed prior to application. Many different formulations are available, ranging from high modulus overlay materials to flexible crack sealers (Ref 148). They are typically low-viscosity materials, and can be applied by spraying, brooming, or squeegee methods.

The effectiveness of HMWM's as waterproofers has been found to decline over time due to the reopening of cracks through the monomer. The treatment should not be expected to restore the strength of cracked concrete, nor should the filling of entire cracks be expected. The primary problems encountered with their use have been the unfamiliarity of field crews with their characteristics and with the odor that accompanies their application. Low odor materials are available, but at extra cost. Also, the set and hardening of the materials is highly environmentally sensitive, leading to possible field application problems.

3.6.4 Linseed Oil

Linseed oils are the oldest of the bridge concrete sealers. They were first used to reduce the surface scaling associated with deicer applications (Ref 141), and their use was carried over to corrosion protection when the role of deicing salts in bridge corrosion was recognized. Linseed oils have been applied to decks, parapets, rails, and wingwalls (Ref 141). Typically, boiled linseed oil is used to speed drying time, and the material is mixed with a solvent to reduce its viscosity to increase penetration depth (Ref 136). Linseed oil works as a breathing sealer, preventing chloride and water intrusion while allowing water vapor to escape the concrete.

In comparison tests, linseed oil has been found to perform well in general, although it has shown significant variability in some tests (Ref 131). It has been shown to be effective at resisting scaling and chloride penetration (Refs 131, 132, 133). It is a well-known product, and contractors are familiar with the application procedures. Also, it remains as one of the least expensive bridge protection strategies (Ref 132).

Linseed oil has a number of drawbacks, however. Its performance in standardized testing is known to be variable (Ref 159), leading to difficulty in drawing conclusions about field performance. This is partially due to the fact that the material must be exposed to ultra-violet light to polymerize (Refs 159, 133), and many of the experimental studies have not exposed the material to UV. Its penetration depth is less than that of some other sealants (Ref 132), and it requires periodic reapplications after 2 to 5 years to maintain its performance. TxDOT's Division of Bridge and Structures recommends reapplication of linseed oil every 3 to 5 years.

CHAPTER 4. FIELD TEST METHODS—STATE OF THE ART

4.1 INTRODUCTION

In order to determine the effectiveness of various corrosion protection strategies, some means of quantitative comparison are needed. A number of tests for possible use in long-term monitoring of bridges to determine the performance of the various corrosion protection systems were investigated through a literature search and field trials. A number of criteria were developed to determine the most appropriate tests to be used for field condition surveys. Among the most desirable characteristics of a field test method are:

- *Nondestructive.* This would allow multiple visits with minimal damage to the bridges, allowing long-term monitoring of the structures.
- *Reproducible.* An appropriate test should return consistent results if repeated with no change in conditions.
- *Simple.* To produce the greatest database of information regarding corrosion protection performance, a large number of bridges must be monitored. Widespread monitoring of bridges will be needed to maximize bridge performance and maintenance scheduling. This calls for a simple test regime that can be conducted by technicians with only minor training, as having enough technical personnel to conduct the required surveys would be too costly.
- *Accurate.* Any test used by different districts should give meaningful and accurate results to allow comparisons among the structures surveyed.

Using the above ideas as a guide, the following tests were determined to be the most promising. The tests all serve to evaluate the corrosion condition of the bridge. Performance factors such as skid resistance, rideability, and vehicular capacity were beyond the scope of this study and were not addressed.

4.2 TEST PROCEDURES

4.2.1 Half-Cell Potential Testing

The copper-copper sulfate half-cell test as described by ASTM C876 was developed as a means of detecting corrosion in bridges containing uncoated reinforcing steel. As detailed in Chapter 2, an electrical potential or voltage develops in the structure as the corrosion "driving force" when conditions in the concrete are favorable for corrosion to occur. The half-cell test uses a voltmeter to compare the corrosion potential in the bridge to the known reference potential generated by the copper sulfate solution surrounding the copper rod in the half-cell. These potentials are recorded as "Volts CSE," meaning corrosion potential as referenced to a Copper-Sulfate Electrode (Ref 79). The test is performed by simply placing the half-cell on the concrete surface and reading the voltmeter. The test can be performed on any portion of the concrete structure that contains electrically connected reinforcing steel.

The most common application has been to bridge decks, but the system is equally applicable to all other portions of the bridge. For typical testing of bridge decks, a grid with 4-foot spacing is laid out on the structure to be tested, and readings are taken at each grid point. The readings are then plotted to produce equipotential contour plots or any other graphic representation of the readings. Also, simple block plots such as is shown in Figure 4.1 can be created with commercially available general-purpose plotting software.

An important aspect of the test is its dependence on good electrical connections between the various components of the system. Typically, the electrical connection between the top layer of the reinforcing steel and the voltmeter is made through a core hole drilled in the concrete to the top of the steel. After coring, the concrete surrounding the reinforcing steel is removed with an electric chipping hammer. The connection is then made with a copper alligator clip or some other attachment. The connections be-

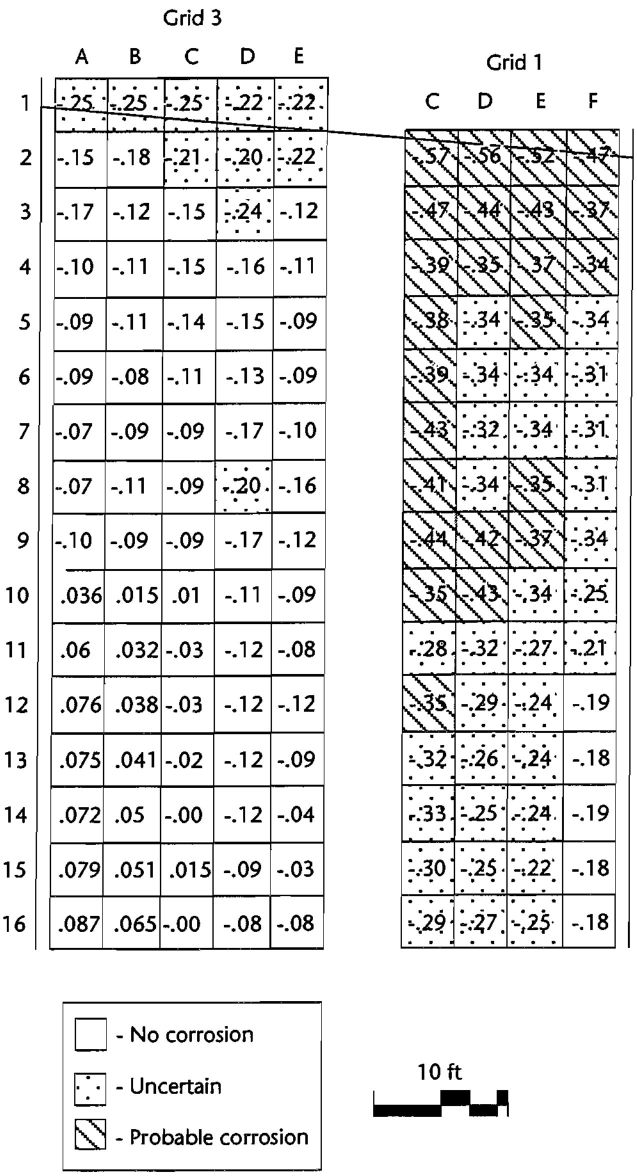


Figure 4.1 Sample half-cell potential plot

tween the various bars are made through the tie wire and the direct bar contact established during the construction of the bridge. For the equipment used in this project, the connection between the half-cell apparatus and the bridge deck reinforcement was achieved through the surfactant solution contained in a bottle attached to the electrode (Figure 4.2).

The sponge slowly releases surfactant solution to the deck during the readings to act as a conductor. Note that because there are no electrical connections between the bars, concrete containing epoxy-coated reinforcing steel cannot be tested using the half-cell potential test. Also, due to the different nature of corrosion of epoxy-coated reinforcement, the test is expressly not applicable to the testing of epoxy-coated reinforcement.

According to ASTM C876, the half-cell potential test is performed as follows:

- Connect the lead from the voltmeter to the top layer of the reinforcing steel.
- Connect the copper-sulfate electrode to the other terminal of the voltmeter.
- Place the electrode and junction device (wet sponge) on the surface and observe the readings for five minutes.
- If the readings are stable (± 0.02 Volts CSE), then proceed to take readings at all points.
- If the readings are not stable, pre-wet the surface by wetting the surface with water or with sponges until stability is attained.
- If the readings cannot be made stable, the concrete is too dry for readings, or external stray currents are interfering. In neither case should readings be continued.

During the field testing portion of the study reported herein, the ASTM procedure was followed, and the accuracy and stability of the half-cell readings were confirmed at each bridge. To this end, the researchers conducted accuracy and stability confirmation tests. The accuracy test called for testing ten random points before and after testing the entire grid and then comparing the results. If any readings differed by more than 0.02 V CSE, the entire process was repeated. The stability tests called for the continuous monitoring of the readings at two random spots for five minutes. This test was used to confirm the five-minute stability required by the ASTM specification.

According to ASTM C876, the results of the half-cell testing can be interpreted as follows:

- readings more negative than -0.35 V CSE indicate a "90 percent probability of corrosion activity"
- readings less negative than -0.20 V CSE indicate a "90 percent probability of no corrosion activity"
- readings between -0.20 and -0.35 V CSE indicate inconclusive results

In addition to the specialty half-cell and surfactant bottle, a number of other special equipment items were used to facilitate the gathering of the potential values. Most important was a hand-held portable single-channel data acquisition unit. This allowed the readings to be electronically recorded with the push of a button by the operator. This greatly sped up the testing, as no cumbersome hand recording was needed. Also used was an extension handle attached to

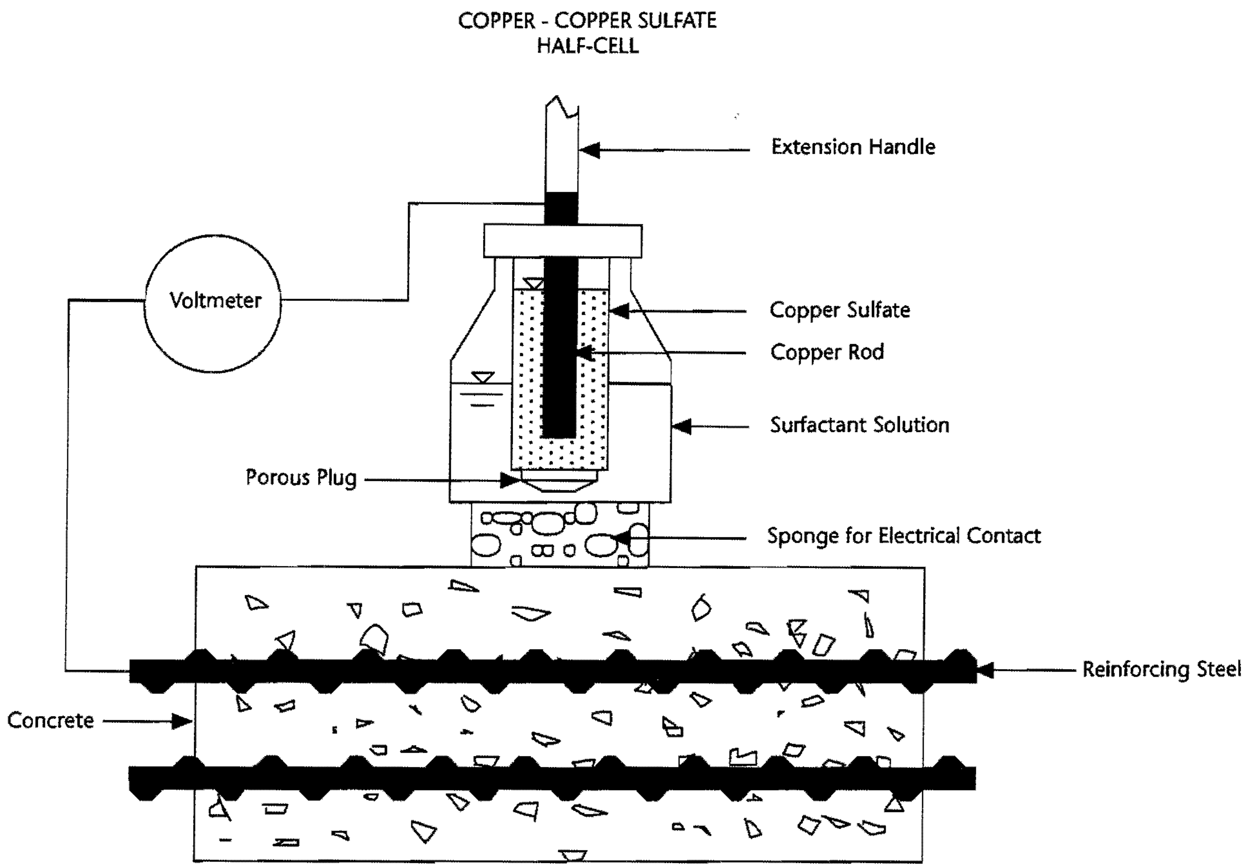


Figure 4.2 Schematic of half-cell test apparatus

the electrode. This allowed the operator to take readings without stooping over, again increasing the speed of testing.

The results of the half-cell test can be used to plan maintenance and repair of the structures, as the existence of corrosion can be determined well before external symptoms such as cracking and staining become apparent. In fact, the Alberta Department of Transportation and Utilities, along with many state DOT's, uses half-cell monitoring for long-term bridge maintenance planning (Refs 79, 135, 126).

4.2.2 Chloride Content Testing

The extent of chloride ion penetration into the bridge concrete is an important indicator of the effectiveness of a corrosion protection strategy. To determine the extent of chloride contamination of the concrete the testing program called for determining the chloride content of various components of the bridges inspected. This required taking concrete powder samples from the bents, caps, rails, and deck of the structure for analysis using commercially available testing equipment. With an electric rotary-hammer, powder samples were taken from the concrete at four depths: 0 to 1/2, 1/2 to 1, 1 to 1-1/2, and 1-1/2 to 2 inches. To provide a wide representation of exposures, samples were taken from areas visually determined to have "good" and "poor" protection, where "good" and "poor" represented a somewhat arbitrary performance difference as determined by visual inspections at the site. To include the various traffic exposures, samples were taken from centerline, wheel path, and shoulder areas.

Once the powder samples were removed from the deck, they were carefully placed into labeled 20-ml airtight vials for transport. At the laboratory, the powder samples were mixed with an acetic acid extraction fluid and tested to determine the acid-soluble chloride content. The testing equipment meets the requirements of ASTM C1152 test for acid soluble chloride content.

The results are obtained in percent chloride by sample weight. To convert these values to pounds of chloride per cubic yard, a concrete density of 4,000 pounds per cubic yard was assumed. Results are typically presented as chloride penetration profiles and in tabular form.

The results of this test can be used to plan future and specific repair strategies, as the chloride content of a deck is considered a good indicator of future performance. Once the chloride content has reached the threshold value for corrosion, damage and deterioration can be expected, and repair planning should begin. Chloride contents can also be used when determining the best course of action when a structure is being repaired. If the concrete is not badly contaminated with chlorides, a less extensive repair can be performed. When the concrete is badly contaminated, a simple overlay or sealer application cannot be expected to prevent recurrence of the problem. This test is well suited to monitoring applications because the samples are small and the resulting holes are easily repaired.

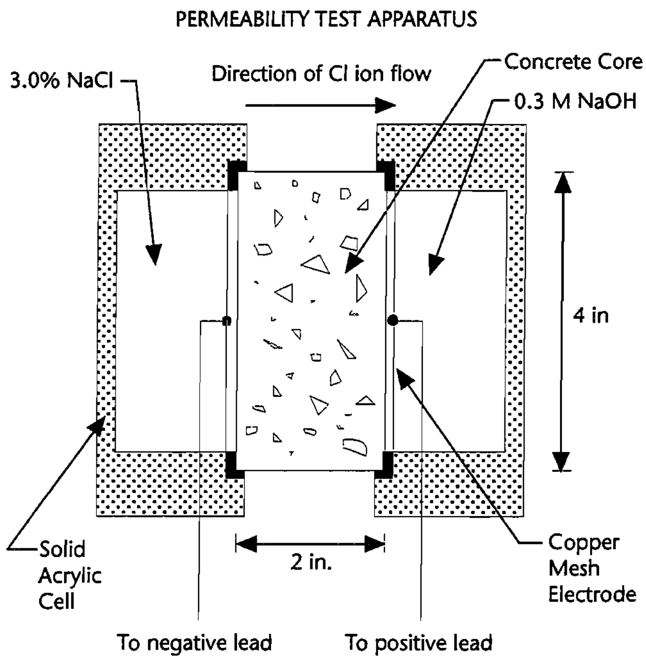
4.2.3 Permeability Testing

The water and chloride permeability of the bridge concrete is an important factor in bridge performance and can be used as a basis for comparison of different materials and protection strategies. The AASHTO T277 Rapid Chloride Ion Permeability Test (RCIPT) was chosen for characterizing the permeabilities of the concretes at the sites to be visited. The permeabilities can be used to indicate the relative effectiveness of the different concretes in resisting chloride ion penetration. The RCIPT works by impressing an external potential across the sample, forcing the chloride ions at one end of the sample to pass through the concrete (Figure 4.3). The test measures the permeability in terms of coulombs, or volt-seconds. This represents the area under the voltage-time curve. The results of different RCIPT tests can be directly compared as relative indicators of the concrete's permeability.

The test requires taking 4-inch-diameter cores from the structure. This was usually done by TxDOT personnel. Project researchers chose the core locations based on:

- Field selected areas of "good" and "poor" protection system
- Base concrete type and age
- Wheel path and non-wheel path exposure
- Core machinery accessibility
- Any special structure history such as widening, resurfacing, or rehabilitation

For the purposes of this project, some of the samples were not tested in strict adherence to the AASHTO test procedure, which calls for the removal of the top 1/2 inch from the cores prior to testing. Some of the cores were tested with the surface intact to determine the relative effectiveness of any surface sealers or special surface conditions in preventing chloride penetration. During testing, all specimens with the top surface left intact were placed in the apparatus with the intact surface facing the NaCl reservoir. This exposed the existing surface to chloride penetration to best represent actual exposure conditions.



From: *A History of the "Rapid Chloride Permeability Test"* by D. Whiting and T.M. Mitchell

Figure 4.3 ***Schematic of rapid chloride permeability cell***

The T277 permeability cannot be directly correlated to any other measured concrete performance characteristic. The results are grouped in terms of relative permeability. T277 defines 2,000 to 4,000 coulombs as "moderate" permeability, less than 1,000 coulombs as "low" permeability, and greater than 4,000 coulombs as "high."

The RCIPT can be used as an acceptance specification test part of a quality control program and as a monitoring test. Used in specifications, the test can ensure that the corrosion protection system installed is comparable to those described by standard tests. By monitoring a concrete sealer with the test, reapplication needs can be determined and performed when necessary.

4.2.4 Crack Mapping

The test program called for the mapping of the type, size, and location of surface cracking, delaminations, and any other surface defects. This was done to determine any correlation among crack patterns, deck performance, chloride content, half-cell potential, and concrete permeability. The cracking surveys include mapping all areas of cracking and noting the type and frequency of the cracking. The project personnel conducted the surveys visually while recording the information on survey maps drawn beforehand from the supplied half-scale drawings of the structure. Crack widths were determined using optical length comparators. All crack patterns were referenced to the *Guide for Making a Condition Survey of Concrete in Service*, ACI Manual of Concrete Practice, ACI 201.1R-68. The data aretypically reported by mapping the cracking in a final presentation format.

Crack mapping can be used in monitoring of the bridge performance with time. By determining the timing and relative severity of crack development, potential problems can be detected before they reach catastrophic size. Also, the close monitoring of cracking and the related corrosion characteristics such as half-cell potential and chloride content may lead to a better understanding of the exact role and sequencing of crack effect in corrosion performance.

4.2.5 Strength Testing

The strength of concrete serves as a relative indicator of the concrete quality, water-cement ratio, and curing, all of which have an effect on the resistance to chloride intrusion of the concrete. Cores for strength testing should have a minimum of a 1:1 height-diameter ratio and should contain no reinforcement. Many bridge decks are only 4inches thick, so the retrieval of a core of sufficient length is somewhat difficult to accomplish in practice. Also compounding the problem is the use of overlays as part of many corrosion protection systems. Since the compression testing of composite overlay-base cores would reveal little information about either of the two particular concretes, project personnel decided not to test any of the composite cores for strength. The problem of reinforcing steel in the cores is best prevented by the use of pachometers, or reinforcing steel locators, to locate the reinforcing steel in the bridge deck prior to coring.

Cores were tested following the ASTM C42 guidelines. All cores were tested using unbonded neoprene caps.

4.2.6 Delamination Testing

In order to correlate half-cell, permeability, and chloride content data with actual performance, delamination surveys were performed. This non-destructive test finds delaminations of the deck concrete by acoustical means. As described by ASTM D4580, delamination surveys are performed using a chain drag "broom." Intact and delaminated areas of concrete are indicated by the distinctly different sounds produced by the chains as they are dragged across the concrete surface. The only equipment required is the chain broom described in the test specification. The results of the test are best presented in the form of sketches showing the areas of the deck exhibiting delamination.

The test methodology is easily learned and performed by unskilled personnel after little training. The repeatability of the test is considered good. The delamination sounding is easily incorporated into a bridge monitoring program, and the information regarding areas affected can be used in planning the timing and extent of repairs to be made.

CHAPTER 5. TxDOT EXPERIENCE

5.1 HISTORICAL EXPERIENCE

5.1.1 Introduction

In response to the adverse effect of salt on reinforcement corrosion, TxDOT developed a variety of protection methods for new construction. Construction practices were improved by increasing curing time, lowering the concrete's water-cement ratio, using cleaner aggregate, adding air entrainment, increasing concrete cover, and improving placement techniques. The Texas Protection System, an asphaltic overlay system, was developed in an effort to prevent water penetration of the deck. The Texas Protection System was later supplemented by epoxy-coated reinforcing steel. Lastly, boiled linseed oil was specified as a surface treatment for all bridges not utilizing the Texas Protection System.

At the same time, a number of remedial measures were developed for repairing damaged structures still in service. In general, the decision to mill and overlay a bridge deck with dense concrete is made on the basis of both chloride content at the level of the reinforcement and the percentage of deck showing distress. Concrete overlays were used if the chloride content of the deck was less than 2 lb/yd^3. The concrete overlays were placed after removing the damaged portion of the deck above the reinforcing steel. If the damaged structure would be exposed to future heavy use of deicers, a "low-slump" dense concrete overlay would be used. When the damage was not extensive or when the only desire was to restore rideability, an asphalt overlay was used for repair.

5.1.2 The Texas Protection System

As mentioned above, the Texas Protection System (TPS) was developed to protect bridge decks in areas exposed to deicing salts or to sea water. The system can take any one of three forms:

- 2-inch overlay composed of:
  - two courses of asphalt surface treatment
  - asphaltic concrete overlay

- 1-1/2-inch overlay composed of:
  - one course of a latex asphalt
  - lightweight surface treatment
  - latex asphaltic concrete overlay

- 1-inch overlay composed of:
  - three courses of asphaltic surface treatment

The system has fallen into disfavor with TxDOT due to continued problems with deck deterioration despite the widespread use of the system. In many cases, the removal of asphalt overlays for repair of minor surface damage has revealed extensive corrosion and scaling in the underlying concrete decks. This requires expensive and time-consuming changes in the repair contracts. Apparently, any water eventually penetrating the asphaltic overlay is trapped and held next to the concrete by the overlay, causing the damage.

In addition to the corrosion and scaling problems, the concealment of any ongoing concrete degradation by the asphalt is a major drawback of the system. Because of this, minor problems that could be repaired inexpensively escalate into large and expensive repairs. Also, the asphalt overlays require fairly intensive maintenance and repair due to cracking, wear, rutting, and shoving.

5.1.3 Linseed Oil

A boiled linseed oil surface treatment is used on all Texas bridges not protected by asphaltic overlays. The oil is applied before the structure is opened to traffic and is supposed to be reapplied every 3 to 5 years. The performance of the material is thought to be variable and dependent on exposure to ultra-violet (UV) light, application rate, and concrete permeability.

The linseed oil is currently placed on all bridges at the time of construction but is rarely reapplied in practice. This is due to the general feeling that the material does not contribute to deck performance and is a waste of time, effort, and money. The material merits further study, as little work has been performed to determine the field effectiveness of linseed oil. Also, much of the laboratory work that has been performed on the material has not exposed the linseed oil to the UV light required to polymerize the material.

5.1.4 Epoxy-Coated Steel

Since epoxy-coated reinforcing steel has become economically competitive, TxDOT has permitted its use to replace or supplement the Texas Protection System. Depending on location and exposure, the epoxy-coated steel is used in either the top mat of reinforcing or in both mats in the deck. Epoxy-coated steel is also used in the substructures of bridges that are exposed to salt- or sulfate-laden water. Because of the young age of most installations, the effectiveness of the epoxy-coated steel is still under review by TxDOT.

5.2 TxDOT CURRENT PRACTICE

5.2.1 Introduction

In addition to the standard corrosion protection systems mentioned above, TxDOT has implemented a number of other systems on an experimental basis. These systems include polymer impregnation, polymer concrete overlays, cathodic protection systems, silane sealers, methacrylate, and many others. Unfortunately for TxDOT as a whole, these experimental systems have been implemented on an individual district basis, and the resulting data remain scattered throughout the state.

In an effort to create a performance data base for the state as a whole, a survey was distributed to all of the TxDOT districts as part of this study. The survey was developed by the project researchers and supervisors and was approved by the Technical Advisory Committee prior to statewide distribution. The survey was developed to gather the following information:

- Corrosion protection systems installed in Texas
- The problems and merits of the various systems in place
- The relative costs of the systems
- The locations of test bridges for further investigation during the field visits

The survey was designed to gather the maximum amount of information possible. To this end, the survey first asked for general information about what systems were used, what portions of the structure they were used on, and when they were used. Next, a number of sheets were included asking for installation, performance, and maintenance information for the individual corrosion protection systems. Last, the survey asked for the repair and replacement history of the various corrosion protection systems used in the district. A number of sheets describing the various protection systems and offering examples were included. A sample survey is included as Appendix C of this paper.

The survey was sent to the District Engineers of all 22 TxDOT districts. They were requested to circulate the survey through their design, maintenance, and construction divisions for comments. Nineteen surveys were returned. The non-returned surveys were mainly from districts in southern or western Texas where no deicers are used and reinforcement corrosion is not a problem.

5.2.2 General System Use Sheet

The results of the General Use sheet are shown in Table 5.1. Linseed oil and the Texas Protection System were the most frequently used systems, with epoxy-coated reinforcement and dense concrete overlays the second most common group. Other methods had seen limited experimental use in all areas of the structures.

The responses to this portion of the survey indicated the wide range of experimental installations of various corrosion protection systems throughout Texas.

5.2.3 Corrosion Protection System Characterization

5.2.3.1 Introduction

As mentioned above, a portion of the survey was designed to determine construction, performance, maintenance, and cost characteristics of the various systems. The following sections are the compilations of the survey responses for the various systems. It must be emphasized, however, that these are only the opinions of the various respondents and are not necessarily the results of research or testing. For this reason, the responses should be regarded as informed opinions only.

5.2.3.2 Epoxy-Coated Reinforcement

Eight districts indicated widespread use of epoxy-coated reinforcement in the deck, rails,

Table 5.1 System use compilation sheet

	Number of Districts Indicating Use							
	Period of Use					Area of Use		
Protection System	Experimental	Never	Past	Present	Future	Deck	Substructure	Superstructure
None		1	4	3	1	5	5	6
Epoxy-coated rebar		2	8	8	7	7	4	4
Cathodic protection	1	7				1		
Waterproof membranes		7						
Polymer impregnation	3	5	2	1	1	1		
Calcium nitrate		8						
Three-inch cover		7	1	1	1		1	
Texas Protection System		2	9	10	6	9	1	1
Overlays:								
Dense concrete	1	4	5	3	3	5		
Latex modified		8						
Polymer concrete		4						
Sealers:								
Methacrylate	1	6					1	
Linseed oil	1		18	14	13	16		
Silane	1	5		1	1	2		
Siloxane	1	6			1			
Sodium silicate	1	5				1		
Epoxy waterproofing			1	1			1	1
Asphaltic seal coat				1		1		

and substructures of bridges. All of the districts except one indicated that the system would be used in the future. The estimated service life was 20 to 50 years. The only problem noted with the material was the care needed in handling and installation and the associated need to repair damage to the coating. The extra care necessary sometimes resulted in increased placement time for the steel and necessitated the use of contractors familiar with the material. The use of epoxy-coated steel also led to confusion and inconsistent results in projects mixing coated and uncoated steel. One district's cost estimate was \$0.45/lb for the material and \$0.50/lb for the installation. In general, epoxy-coated reinforcement was regarded as a promising method of corrosion protection.

5.2.3.3 Cathodic Protection

One district reported the use of a cathodic protection system. The system was reported as 4 years old and performing well. Although regarded as effective, the system was difficult, expensive, and sophisticated to install. In addition, monitoring the system requires specialized technical expertise. The service life was estimated as 10 years. No cost information was returned. Future use was not anticipated. Performance of this system has been rendered questionable and is currently being studied by Texas Tech University under the sponsorship of TxDOT.

5.2.3.4 Polymer Impregnation

Three districts reported the use of polymer impregnation in bridge decks but only one plans any future use of the system. The system performance was regarded as good. Problems were reported with cracking during the heating and cooling required to impregnate the deck and with determining the actual depth of penetration. The service life was estimated as 25 years. The material was estimated to cost \$1.00/ft^2 for installation. In addition, a two-week closure time of the structure was required.

5.2.3.5 Texas Protection System

Ten districts reported widespread use of the Texas Protection System, but only six plan to continue use in the future. The system was used to prevent water intrusion of the deck, to restore ride quality, and to improve skid resistance. The system was widely used on structures widened to increase capacity and was used to extend the life of degraded surfaces. The system can be used only on bridge decks. Construction problems included variability in performance based on the asphaltic concrete quality, the placement operations, and the cleanliness of the joints and deck prior to application. One district noted that the treatment of expansion joint details can significantly affect the long-term performance. In remedial applications, the increased dead load on the structure due to the overlay can also be a problem.

Quality degradation was the main performance problem with the Texas Protection System. Shoving, rutting, and stripping occurred under heavy truck traffic and where vehicles were routinely turning. Freeze-thaw degradation also occurred. Also noted were ride problems at bridge joints and on thick overlays on bridge approaches. Required maintenance included spot repairs, crack sealing, and milling to restore skid resistance. Major repairs frequently require removal and relaying of the material.

The service life was estimated as 25 years, with a maintenance cycle of 6 to 8 years. Cost estimates ranged from \$0.30 to \$0.50/ft^2 and annual maintenance ranged from \$0.00 to \$0.20/ft^2.

5.2.3.6 Dense Concrete Overlays

Five districts reported the use of dense concrete overlays for bridge deck rehabilitation, with three indicating that the system will be used in the future. The material was described as highly dependent on the installation quality, which was reported as highly variable. Construction was reported as difficult and labor-intensive due to contractor problems with deck preparation and material handling. Performance was variable, with all of the decks experiencing cracking and/or delaminations. Spalling was often a problem, especially at finger joints and expansion joints. Maintenance consists mainly of sealing cracks and repairing spalls. No service life was estimated. The material costs were estimated as \$2.50/ft^2 for materials and \$6.50/ft^2 for a completed deck.

5.2.3.7 Methacrylate Sealers

One district reported use of high-molecular-weight methacrylates for sealing cracks and repairing delaminations. The district plans to use the material in the future. Construction problems were reported with the short shelf life of the material, the required curing time in cold weather, and the volatility of the material. Some spalling and transverse cracking has been noted in most of the structures. No maintenance of the material was required, although future resealing is anticipated. The service life was estimated to be 7 years. The cost estimate was \$0.71/ft^2 for the entire job, including \$0.57/ft^2 for material and \$0.14/ft^2 for labor.

5.2.3.8 Linseed Oil

Eighteen districts reported widespread use of linseed oil on bridge decks, with thirteen of the districts anticipating future use of the material. The placement rate of the material is regarded as an important factor during installation. Problems experienced during placement included low absorption into the deck, extra traffic control time due to excessive curing time, and a loss of skid resistance. Performance was variable, with one district characterizing it as "poor." Planned maintenance includes a reapplication at two-year intervals, but only one district reported that the reapplications were actually performed. The estimated service life was given as 6 months to 3 years. The installation cost was estimated as \$0.08/ft^2 for the material with a \$6.00/ft^2 cost for a complete deck. The installation required twenty minutes for deck preparation, one hour for application, and four hours for curing.

5.2.3.9 Silane and Siloxane Sealers

One district reported use of silane and siloxane sealers on all portions of bridge structures, as well as the planned future use of the materials. No contractor difficulties were reported, and the material was described as "easy to apply." The young age of the structure precluded any opinions on performance or maintenance characteristics. The cost was estimated as \$0.34/ft^2 for the material, and no maintenance costs were anticipated.

5.2.3.10 Sodium Silicate

One district reported use of sodium silicate on all portions of a bridge structure. Again, no contractor difficulties were reported, and the installation was described as "easy to apply." The young age of the structure again precluded any opinions on performance or maintenance characteristics. The material works by expanding to seal the concrete pores with the addition of water, and some doubt as to the material's ability to do this after extended drying was expressed. The cost was estimated as \$0.19/ft^2, including \$0.08/ft^2 for materials and \$0.11/ft^2 for labor.

5.2.3.11 Epoxy Waterproofing

One district reported use of epoxy waterproofing surface treatments in substructure and superstructure applications. Constructability was reported as "average" but dependent on surface preparation and material handling. Construction problems included the difficulty obtaining a material approved for use with adequate lead time. Performance problems include stripping and peeling of the paint-on material when exposed to sunlight, moisture, and salt. Forecast repairs include removal of the material by sandblasting and reapplying the material every 3 to 5 years. Future use is not anticipated. The installed cost was estimated to be \$1.00/ft^2.

5.2.3.12 Asphalt Seal Coat

One district reported using an asphalt seal coat on many structures, but does not plan to use the system on future structures. Constructability was characterized as "poor." Performance was poor due to deterioration, delamination, and spalling of the concrete once water penetrated the seal coat and was trapped next to the deck. Forecast maintenance includes patching of delaminations and cracks. The installation cost was estimated to be \$0.25/ft^2.

CHAPTER 6. FIELD SURVEY METHODOLOGY

6.1 INTRODUCTION

Because a monitoring program is necessary for the effective use of experimental and other selected bridge protection systems, a major effort of the investigation was the development of a bridge corrosion testing and monitoring program. The corrosion-specific inspection and testing regime was developed to compliment the current FHWA Bridge Inspection Program (BRINSAP) data collected regularly for all bridges. By instituting a standard corrosion testing program for experimental and other specially selected bridges, more useful data will be collected as all bridges statewide will be tested in the same way for the same relevant corrosion performance information. This will allow conclusions to be drawn earlier from test installations and past projects, decreasing the time required to improve the current corrosion protection measures.

A rigorous testing program will increase the amount of information that can be obtained from field tests of corrosion protection measures. This will allow the examination of the mechanisms of protection strategies as they apply to actual field performance. Through monitoring of field structures, data regarding the relationships between parameters such as concrete permeability, concrete cracking, corrosion potential, chloride content, and ultimate performance can be developed. Since each individual field installation provides a semi-controlled experiment with all portions of the structure seeing the same weather and traffic exposures, deicer applications, and climatic exposure, much can be learned by monitoring each installation. Also, the monitoring program will result in an extensive corrosion protection performance database with time as more data are collected.

6.2 TEST PROGRAM DEVELOPMENT

As discussed in Chapter 4, a number of tests were investigated for use in the field test program. After conducting the literature search and determining the state-of-the-art in corrosion testing, a number of tests were selected for use. Once the tests were chosen, researchers familiarized themselves with the test methods using controlled field trials conducted both at the Balcones Research Center of the University of Texas at Austin and on the southbound IH-35 frontage road over Onion Creek in Austin. The tests were used to confirm the repeatability of the test methods, as well as to finalize the test methods and the materials and equipment required.

The tests and procedures ultimately selected for use in the field surveys include half-cell corrosion potential (ASTM C876), concrete permeability (AASHTO T277), chloride content (ASTM C1152), delamination detection (ASTM D4580), strength testing (ASTM C42), and crack mapping.

6.3 TEST SITE SELECTION

6.3.1 Introduction

The selection of the testing sites is a crucial step that can increase or limit the amount of information that can be learned from field visits. Successful site selection is accomplished through the coordination of the researchers, District contacts, and the Advisory Committee. Suggestions for investigation sites come from many sources, including the researchers' personal experiences, supervisor experience, District contact, surveys of the districts involved, and past experimental work.

The structures tested during the first portion of the project were chosen to provide performance information about as many different corrosion protection systems as possible and to confirm the test methods' applicabilities to the wide range of bridge conditions that could be expected in the field. The sites were selected primarily from the Survey of Current Practice distributed state-wide during the early stages of the project and discussed in Chapter 5. The protection systems investigated include dense concrete overlays, methacrylate crack sealers, epoxy-coated reinforcement, silane and siloxane surface sealers, washing and sweeping after deicer application, linseed oil, and unprotected controls. The exposures of the structures included deicing salt, sea water, and sulfate and gypsum laden rivers. The conditions of the structures investigated ranged from new to pending demolition.

6.3.2 Site Selection

Test sites should be chosen to accomplish a specific task and to maximize the amount of data to be collected during the site visit. Although site selection is not a factor in cases such as single site test installations or the investigations of corrosion protection system failures, it is often necessary in determining and quantifying the performance of a system installed in many different structures.

A test site should have as many of the following attributes as possible to maximize the information that can be gathered with the given equipment, time, and labor cost:

1. *Multiple Lanes.* Multiple lanes permit staged testing with a minimum of traffic control effort. This allows staged inspections with common or overlapping reference or connection points, as well as allowing investigation of an entire structure without requiring complete closure or flagging.
2. *Continuous Spans.* Continuous spans allow one half-cell corrosion potential electrical connection to be used for multiple spans. This eliminates the time required to reconnect and reconfirm the half-cell equipment. It also reduces the amount of steel to be exposed to allow connecting the half-cell.
3. *Plain Reinforcing.* Uncoated reinforcing allows the use of the half-cell corrosion potential test. This non-destructive test can be used to determine the corrosion state of a deck with no visible damage and can thus be used as an early indicator of corrosion performance. Bridges with epoxy-coated steel cannot be surveyed with the half-cell.
4. *Past Data.* If data has previously been collected for a structure's corrosion performance, another survey offers a chance to determine the time-history performance of the structure and the protection system.
5. *Test Installations.* Any structure built as a test installation with comparison sections or control sections is a prime candidate for study. Investigating such a structure allows a semi-controlled field test of the corrosion protection systems incorporated into the structure.
6. *Representative Exposure.* Structures should be chosen with representative exposure to freeze/thaw cycles, traffic, chlorides, and other environmental factors. The performance of a protection system installed on a low volume secondary road probably is not a good indicator of the system's performance on a high-volume interstate bridge.
7. *Accessibility.* If possible, a structure should be chosen that allows easy access to the substructure and the underside of the deck. This allows substructure samples to be taken and underside observations to be made, increasing the information gained from the inspection.
8. *Past Performance.* Structures having areas of both good and poor performance for a given system should be investigated. By comparing the information gathered for two areas, the reasons for the different performance may be determined.

6.3.3 Test Site Background Determination

Once the investigation site is decided upon, a background determination should be performed. The information is determined through the BRINSAP records, highway department contacts, and any documents or test reports concerning the structure. The background check is used to plan the site testing regime and to determine a plan of action for the investigation of the structure. The information can also be used to create a site data-base for possible future monitoring of the structure.

Figure 6.1 shows the Bridge information sheet developed during the project to identify, describe, and track the structures visited. The sheet also serves as an input card for a site database. By using this prepared sheet, all investigated bridges are described according to similar parameters, and more information is compiled.

6.4 PRE-VISIT PLANNING

6.4.1 Introduction

It is necessary to plan each site survey before arriving at the site in order to maximize the information gained while minimizing the effort required. The planning must begin early enough to collect the necessary materials and equipment and to prepare the necessary data recording sheets.

6.4.2 Coordination and Communication

Once a specific site has been chosen for investigation, coordination and planning for the investigation of that specific site should begin in earnest. Scheduling should begin as soon as possible to ensure that adequate District and researcher resources are available.

Equipment and support required from the District includes:

1. *Traffic Control.* All flagging, signing, and other traffic control necessary to ensure the safety of the researchers and motorists are best handled by the individual districts.
2. *Access Equipment.* Special equipment such as ladders, bucket trucks, bridge inspection "snoopers," boats, and edge buckets may be necessary for inspecting some structures.
3. *Coring Equipment.* Using District coring equipment simplifies and reduces the cost of the researchers' travel by eliminating the need to tow coring equipment.
4. *Wash Water.* In addition to the water required for coring the structure, water is needed to wet the deck surface for the half-cell corrosion potential tests. Either a tank truck or a truck with a water tank and spray bar can be used.
5. *Bridge Layout Plans.* Layout plan and elevation drawings are necessary for preparing survey drawings and for developing the testing plan. Preparing survey plans before reaching the site greatly facilitates the surveying process.

6.4.3 Testing Planning

Once the survey site is finalized and the preliminary information has been gathered, the site testing should be determined. Although the same basic tests are performed at all locations, special circumstance may merit alteration of the testing program. For example, the investigation of a structure suffering from extreme substructure deterioration should be modified to concentrate on the substructure in order to best determine the nature of the problem. The emphasis of the site survey can be adjusted by changing the number of samples and the extent of the testing.

The first step in developing the test program is to determine what special information is needed from the survey, if any. Next, any special circumstances should be considered. Items to be considered include:

1. *Special Needs.* If a structure is being surveyed for specific information, the tests applicable to that information should be emphasized. For example, if a structure is being used to provide chloride contents for dosing calcium nitrite additives, chloride content testing should be emphasized.
2. *Existing Information.* Structures that have been previously investigated should be tested in a manner that will allow comparison of the different data groups. This will allow comparisons over time and will expand the information that can be gained.
3. *Age.* Early in the life of a structure, only the minimum testing required to establish a baseline for future monitoring should be performed. Once the baseline has been established, a minimum of testing should be performed until the limited testing detects a change in the structure's corrosion status. This minimizes the cumulative damage done through the testing.
4. *Closure Windows.* If a structure can be closed only at night or during limited hours, some modification may be required to allow the survey to be completed under the imposed time restraints.
5. *Parallel Structures.* If the two directions of traffic are carried by identical parallel structures, a typical assumption is that the exposures and materials will be similar, and that only one of the structures need be tested.
6. *Special Conditions.* Test installations or other unique structures may require changes in the test regime to maximize the information gained through the testing.

Once any outstanding special circumstances have been considered, the routine testing planning inherent to all surveys should be started. The major decision to be made is the number of lane-spans to be surveyed. This depends primarily on the length of the individual spans and the time available for the survey. As a rule, eight lane-spans per day can be surveyed by an experienced three person crew. The number will decrease if the spans are very long or if a wide shoulder or narrow breakdown lane is included in the testing of the adjacent lane. Time requirements of changing the traffic control layout should also be considered.

After determining the number of lane-spans that can be tested by the available personnel, the individual lane-spans to be tested are chosen. A number of factors should be considered when choosing the spans, including:

1. *Location.* Different portions of the same bridge can be expected to experience different exposures. For example, the approach and first span of a structure would have higher exposure to deicers than the middle spans as cars carry them from the main roadway. For

BRIDGE INSPECTION SHEET

Control Number: ________

Location: ______________________________
____
____
____

Deck Type: ______________________________
____

Substructure Type: ______________________________
____

Protection System
Deck ______________________________
____

Substructure ______________________________
____

Superstructure ______________________________
____

Year Built: Length: No. of Lanes: No. of Spans: ADT:

Exposure Condition ______________________________

Structure History ______________________________
____
____

Problems to Date ______________________________
____
____

TxDOT Future Plans ______________________________
____
____

Project Action ______________________________
____
____

Contact Person: ________________ Division: ________________
Address: ________________ Phone: ________________
________________ FAX: ________________

Figure 6.1 Sample Bridge Information Sheet

this reason, lane-spans chosen should include both of the end spans as well as central spans for this reason.
2. *Exposure.* Portions of a bridge over water are exposed to more water vapor and may perform differently than portions over land. For this reason, both portions over the land and portions over water should be tested.
3. *Traffic Patterns.* Certain structures may experience radically different traffic exposures for different portions of the structure due to layout or location. For example, if a quarry is on one side of a bridge and the stone processing plant is on the other, the heavily laden trucks traveling to the plant will influence the deck differently than the empty trucks returning to the quarry.
4. *Performance.* If one portion of a structure is performing differently than another, both portions should be investigated in an effort to determine the reason for the disparity.

Once the lane-spans to be tested are chosen, specific preparation can begin. First, the bridge is assigned a number for record keeping. For the eight bridges detailed in this report, a simple numbering system of 1, 2, 3, . . . 8 was used. The individual lane-spans are also assigned sequential identifying numbers.

Most important is the preparation of individual lane-span working drawings. Single-page drawings of each lane-span are drawn using the plans supplied by the District liaison. The 4-foot grid for the half-cell testing is drawn on the maps to reduce the effort required to map the delaminations and cracking. This is very useful as the grid will be painted on the structure during the survey and locations can be determined by simply matching grid points. Copies of the drawings are used during the survey to record the delamination and cracking data. They will also be used later for plotting the half-cell data and the sampling locations. The drawings are easily made by hand or with general purpose drawing software, as shown in Figure 6.2.

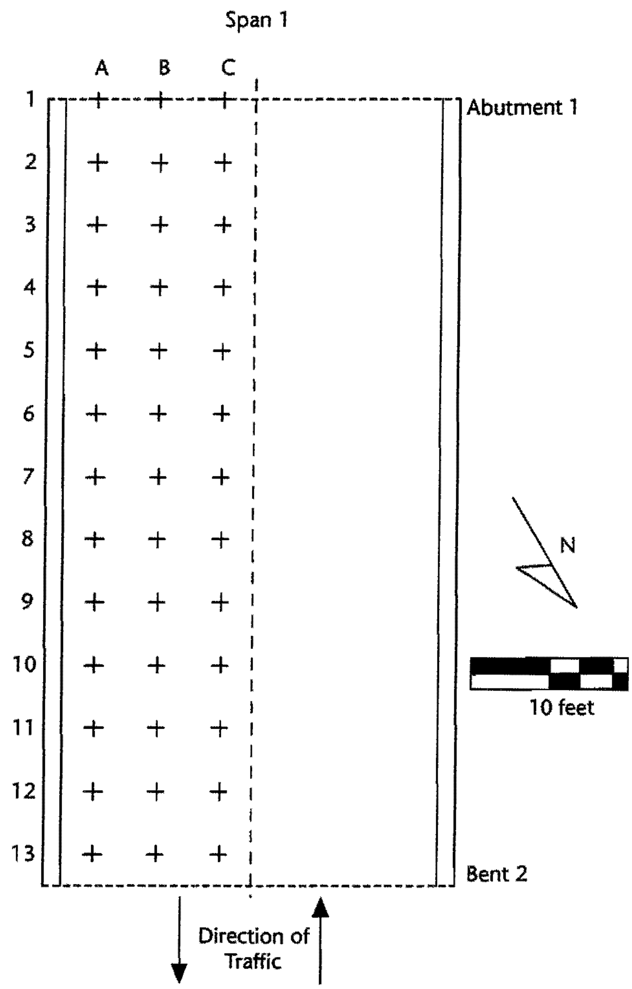


Figure 6.2 Example of lane-span drawings

6.4.4 Sample Location Determination

The general testing program calls for the following number of samples to be taken:

1. Chloride Content Determination
   deck - 5 samples/lane/span
   bent caps - 2 samples/cap
   columns - 2 samples/column
   curb and rail - 2 samples/span
2. Rapid Chloride Permeability Testing
   2 cores/lane/span
3. Strength Testing Core
   1 core/lane/span
4. Petrographic Analysis Core
   1/2 core/lane/span

These numbers were determined by the trial surveys and Technical Advisory Committee input. These numbers can be modified to reflect any special testing emphasis as decided in section 6.4.3. Once any special modifications of the testing program are made and the location of the lane-spans to be tested are determined, the location of the samples should be determined.

The preliminary sample locations can be determined by a random point generator. The use of random points provides for a representative sample that avoids any possible unintentional tester bias. Random points should be used for any new structures or other structures exhibiting no abnormal performance. If a specific attribute such as abnormal cracking is being investigated, sampling points are best chosen in the field to ensure that both problem points and suitable comparison points are included in the testing. Also, sampling points may need to be changed in the field to adjust for unanticipated interference problems with tar sealing or patches. If sample locations must be chosen in the field, it is important to keep the goals and objectives of the survey in mind. Only the sampling points that can best achieve those goals should be used.

6.5 ON-SITE INVESTIGATION

6.5.1 Introduction

After the survey site has been chosen and all possible preparation has taken place, the site survey can take place. When on site, it is important to remain flexible and to keep the survey objectives in mind. It is important to note that everything may not go according to plan, but that by remaining flexible the proper adjustments can be made.

Before traveling to the site, all equipment should be collected and tested. A sample equipment checklist is included as Appendix B of this paper.

6.5.2 Site Coordination

On the morning of the site inspection, the investigators should meet with the District liaison at the District headquarters. At the meeting, all last minute details can be worked out and the traffic control planning can be finalized and confirmed. The surveyors should coordinate with the operators of the coring machine, ensuring that the proper bits are available and that everyone understands the testing plans. The water supply should be confirmed at this time. Lastly, the inspection team should ensure that an adequate supply of drinking water will be available for everyone working on the site.

6.5.3 Site Conduct

After the traffic control has been set up, the survey can begin. Throughout the survey, an emphasis should be placed on traffic safety. No information is worth any personal risk, and work should proceed accordingly. All inspectors should remain aware of changing traffic conditions and should request adjustments in the traffic control accordingly.

6.5.3.1 Orientation

The first action on the site is to confirm the accuracy of the supplied drawings and plans. Any discrepancies should be noted and documented to prevent confusion after the data have been collected. If any changes in grid layout or sampling are made, they should be recorded in detail. Extra effort in documenting field changes from the testing plan is worthwhile and worth any delay. The bridge orientation should be determined and understood by all of the people performing the inspections, as should the nomenclature for the northbound/eastbound and southbound/westbound traffic lanes. The individual grid numbering system should be reviewed at this time as well.

The purpose of the above is to ensure that all records will be consistent. By ensuring that all nomenclature is the same, later confusion while reducing the data can be avoided.

6.5.3.2 Grid Layout

After everyone has been oriented, the testing grids should be laid out. The grid is primarily for the corrosion potential testing but is extremely useful during the crack mapping and the delamination sounding.

A grid consists of points spaced four feet on center and extends the full length and width of the test area. The grids are set up based on a reference point chosen for its easy reproducibility during future visits. Typically, the reference point is used as the A1 grid point and is located two feet from the bridge rail at the armor or expansion joint marking an end of the grid (Figure 6.3). The grid should be laid out so that the A1 point is in the upper left hand corner of the grid, with the grid numbers running the length of the grid and the grid letters across the width of the grid. This facilitates the data manipulation by creating a grid that is easily reproduced in any spreadsheet program.

The grid is easily laid out with two 100 ft tape measures. First, the "A" column is marked on the deck using dots of spray paint. Next, the tape used to mark the "A" column is moved across the deck to the approximate location of the outermost column and the second tape is placed across the deck between the first tape and the "A" column using the marks and the second tape to square the line. The row points are then marked. It is helpful to mark the otherwise blank back sides of the tape measures at four-foot intervals and to layout the grid using these marks. This greatly reduces the chance of field errors in marking. The grids are typically laid out to follow any skew of the bridge.

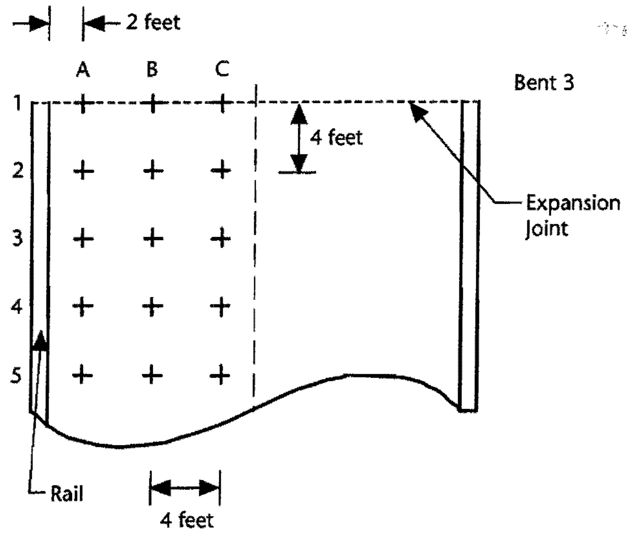


Figure 6.3 Sample grid layout

Each test section should have its own grid. The test sections typically begin and end at expansion joints, boundaries of experimental test sections, the ends of the bridge, and any other discontinuities of the underlying reinforcing steel. These boundaries stem mainly from the corrosion potential test's requirement of a continuous underlying reinforcing steel mat.

Lastly, it has been found convenient to mark the grid row numbers on the curb, rail, or parapet to allow quick, easy reference in the field. As marks in these locations are highly visible to the public, the marks should be made with non-permanent lumber crayon so that they can be removed.

6.5.3.3 Chloride Sampling

As soon as the grids have been marked on the bridge, the concrete powder sampling for the chloride content determination should begin. The chloride sampling can take a great deal of time if only one person takes samples, so equipment for two simultaneous samplings should be available. The sampling equipment includes:

1. *Collection Cups.* These are "cups" with a hole in the middle through which the hole is drilled in the deck. They collect the powder produced during drilling and easily transfer the powder to the sample vials.
2. *Sample Scoops.* Stainless steel sample scoops are helpful for retrieving material from the bottom of the drilled holes.
3. *Air Pumps and Paintbrushes.* These are used to clean the hole, collection cup, and drill bit between sampling depths. The air pumps are simple hand-operated bellows similar to those used to clean photographic equipment.
4. *Sample Vials.* Commercially available 20 ml scintillation vials are very effective for storing the collected samples. The vials have airtight lids to prevent moisture from reaching the samples and come in trays of 100 that can be used for easy storage and transport. It is helpful to assign each bottle to a specific location in the tray to ensure that they are correctly replaced in the tray every time they are removed. This tray location should be marked on the bottom of each bottle.
5. *Self-Stick Labels.* Peel and stick labels are needed for marking the individual vials.

In addition to the equipment required for collecting, labeling, and storing the samples, some equipment is needed for drilling the holes. Multiple pieces of this equipment need not be available, as two samplers can efficiently share single units. This equipment includes:

6. *Rotary-Hammer.* Also known as a "hammer-drill," this tool is used to drill the hole, creating the powder to be sampled. A 3/4-inch bit is needed for the sampling. Multiple spare bits should be carried. The rotary-hammer should be equipped with a depth rod that allows the depth of the hole to be controlled. As all samples will be taken from the same four depth regions, it is convenient to have the suitable marks for the four depths machined into the rod to eliminate the need for frequently remeasuring the rod placement.
7. *Electric Generator.* A portable electric generator for powering the rotary hammer should be available, as well as the necessary gasoline and oil.
8. *Extension Cords.* Two hundred feet of heavy-gage extension cord has been found sufficient to sample a structure without moving the generator excessively.

The first step in sampling is determining the number of samples and the sample locations. This should be done according to the pre-planned sampling pattern. If necessary, the locations may be altered to reflect any field conditions or to better achieve the testing objectives. For example, if an area of particular distress is noted at the site, the sampling should be modified to include samples from that specific area. In addition to the deck, the areas sampled should include bents, curbs, rails, wingwalls, and caps according to accessibility. The sample locations should also include areas that typify the structure's exposure. These

CHLORIDE SAMPLE SHEET______ OF ______

PROJECT	1300
DATE	
WEATHER CONDITION	
BRIDGE NO. & NAME	
BRIDGE LOCATION	
SAMPLE #: **BOTTLE #:**	**CHLORIDE SAMPLE LOCATION:** DISTANCE FROM RAIL:
SPECIAL FEATURES:	ZERO POINT DISTANCE: GRID #: LANE (NB, SB, EB, WB . . .): GRID COORDINATE:
	GENERAL COMMENTS:

HALF-CELL POTENTIAL MEASUREMENT: _____ V CSE

CONCRETE SURFACE APPEARANCE: _____SOUND _____ SPALLED _____SCALED

Figure 6.4 Sample chloride sample data sheet include areas of good or poor coating, cracking, wheelpath wear, gutters, and sound concrete.

Once the locations are chosen, the sampling can begin. For record keeping, each sample is assigned a sample data sheet in a field notebook (Fig 6.4). The data sheets ensure that all necessary information will be recorded. The data collection is designed to ensure redundant information regarding sample location, sample storage location, and sample identity.

The data sheet is first referenced to the structure being surveyed and then to the sampling grid location on that particular structure. This makes it easier to find the sample hole while on the bridge. The grid location is recorded on the sample sheet in the GRID NUMBER and GRID COORDINATE boxes. The GRID NUMBER refers to the identification number of the testing grid where the sample was taken. The GRID COORDINATE references the location on the testing grid. Later, the sample locations are also located exactly by measuring the exact locations of the holes. These measurements are recorded as the DISTANCE FROM RAIL and ZERO POINT DISTANCE. It has been found to be convenient to wait until the end of the sampling and to measure all of the holes at one time. Also, for general information, the lane's traffic direction is recorded.

The sample's location is incorporated into the sample's identification number. Since the number is recorded on the sample vial itself, this provides a backup copy of the sample information should the original data sheets be lost or destroyed. The general sample identification coding developed through the project is shown in Figure 6.5.

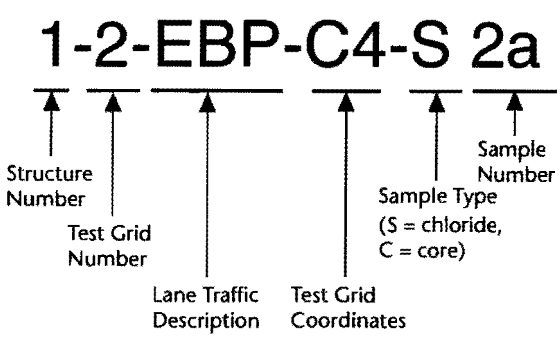


Figure 6.5 Sample numbering system

This sample number is written on the stick-on labels for each sample vial, as well as on the data recording sheets in the SAMPLE # space. As another safeguard against losing track of samples or the structure location from which they were taken, the location of the vial holding the individual samples within the storage tray is recorded on the data sheet in the BOTTLE # space. This ensures that if the labels fall off of the vials, the vial location can be matched to a data sheet.

After the sample location is determined and the sample identification number is developed, four sample stick-on labels are written, one for each half-inch sample. Since there are four different depths sampled from each hole, each sample is composed of separate "a," "b," "c," and "d" portions. Thus, for the first sample for a bridge, the four labels would be 1-1-EB-A6-S1a, 1-1-EB-A6-S1b, 1-1-EB-A6-S1c, and 1-1-EB-A6-S1d. The labels are placed on the bottles and the bottle locations in the storage tray are noted on the data sheet.

After the vials are prepared and the data sheet completed, the sampling can begin. The rotary-hammer depth-stop is set to stop the drilling at 1/2 inch below the surface, the sample collection cup is placed on the surface, and the hole is drilled until the depth rod touches the surface. The collection cup is then picked up and the collected dust is poured into the "a" vial. Any powder remaining in the hole is collected with the scoops and is placed into the vial as well. The hole and drill bit are then cleaned with the blower and the paint brush, the depth stop is moved to the 1-inch mark, and the process is repeated until the full 2-inch depth has been sampled.

This process is repeated until all of the chloride samples have been taken. As a side note, it has been found effective to place all of the vials in the tray upside down before sampling and right side up after sampling to identify the filled bottles and to prevent accidentally spilling or filling a bottle with a sample already in it.

6.5.3.4 Coring

While one or two people are conducting the chloride sampling, another can work with the district coring crew. The required equipment for coring includes:

1. *Coring Machine.* The coring machine to be supplied the individual districts should be equipped with a four-inch inside diameter bit and should carry a spare bit. It is also necessary to have core removal tools such as chisels and hammers available as the cores will not extend through the entire deck.
2. *Pachometer.* A pachometer, also called a "rebar locator," is needed to determine the locations of the reinforcing steel in the deck. For most of the cores, it is desirable to avoid including reinforcing steel in the core.

3. *Data Sheets.* A data sheet will be required for every core removed from the deck.
4. *Plastic Bags.* Plastic bags have been found to work well for storing and transporting the cores.
5. *Stick-on Labels.* Labels are used to identify the cores.

Similarly to the chloride samples, the sample locations for the cores are determined using the plans, site conditions, and knowledge of the testing objectives. The first core on each span should be the half-cell connection. This ensures that a serviceable connection can be made and allows the connection to be established and confirmed while the coring continues, speeding the entire survey. This core should be located to hit the top layer of reinforcing steel. Using the pachometer, the steel is located and the estimated depth to the steel is determined. By using the depth information and watching the cutting debris for metal shavings, the core drill is stopped at the top of the steel to prevent unnecessary damage to the reinforcement. The bar must not be completely cut, as its electrical connection to the other bars will be destroyed. The core is removed with a hammer and chisel.

While the core is being drilled, the surveyor should fill out a data sheet, shown in Figure 6.6, and prepare a sample label. The sheet is similar to the chloride sampling sheet and should be filled out in the same way. The sample identification number is determined in the same way as those of the chloride samples, except that a "C" precedes the sample number instead of an "S." Like the chloride samples, the sample numbers should be continuous and should not be repeated for each test grid. In other words, if test grid one has samples X-1-XXX-XX-C1 through X-1-XXX-XX-C5, the grid two samples should start with X-2-XXX-XX-C6. This avoids the possible confusion between multiple C1's on the same bridge while preserving the information contained in the full sample identification number. The other information is filled in as on the chloride sample sheets.

The other sample cores are drilled once the half-cell connection core is finished. They proceed exactly as the half-cell cores except that the pachometer is used to avoid coring into steel, and that the cores are drilled to a minimum four inch and one-half inch depth. It is helpful to have the project personnel precede the core machine and mark the core locations on the concrete using a lumber crayon.

As with the half-cell cores, the standard cores are recorded on a data sheet while the core is being drilled. The recording is exactly as detailed above. When the cores are removed from the deck, they are placed into plastic bags that are in turn identified with stick-on labels. The plastic bags facilitate the sample identification by allowing the fresh wet cores to be labeled immediately. They also provide for clean storage and transport.

As in the chloride samples, the sample location is referenced to the grid marked on the structure in addition to being measured exactly to the zero point and the curb. The redundant measurements protect against losing the sample location and provide a means for checking the sample locations.

Space is left on all of the forms to allow the recording of any special information about the sample or information about why the particular location was chosen. This space should be used to record information such as the depth of the reinforcing steel, particular surface features, or sampling problem descriptions.

6.5.3.5 Corrosion Potential Testing

After the coring or chloride sampling has been completed, the corrosion potential testing can begin. The basis for the test is described in Chapter 4, Section 2.1. This section describes the field testing procedure. The equipment required for the test includes:

1. *Copper-Copper Sulfate Half-Cell.* The half-cell itself is simply a copper rod in a saturated solution of copper sulfate. The cell ends with a porous plug that allows the passage of electrons and ions while preventing the copper sulfate from escaping (Figure 4.2). The cell produces a known constant potential, and thus serves as a reference to which the bridge reinforcement's potential is compared.
2. *Data Acquisition Unit.* The recording of data was greatly simplified by using a computerized data acquisition system to store the measured potentials. This eliminated the need to record the readings by hand and allowed the readings to be transferred directly into computer spreadsheets.
3. *Wire and Connectors.* A copper alligator clip and approximately 200 feet of wire are needed to connect the data acquisition unit to the reinforcing steel.
4. *Thermometer.* A thermometer is needed as the potential readings are temperature sensitive and must be corrected to standard temperature.
5. *Copper Sulfate Crystals and Distilled Water.* As the copper sulfate in the cell must be saturated at all times, spare copper sulfate crystals

and extra distilled water for refilling the cell should be taken on all surveys.

6. *Cold Chisel, Hammer, and Wire Brushes.* When the connection to the reinforcing steel is established, these toolsare needed to clean the reinforcing steel. This ensures a low resistanc connection.
7. *Surfactant.* A surfactant such as dish soap must be added to the conducting fluid connecting the cell to the deck. This facilitates the water penetration of the deck.

The half-cell testing starts with the cell itself. First, the level of distilled water and undissolved copper sulfate in the cell are checked and are replenished as necessary. Next, the cell is connected to the negative terminal of the data acquisition unit and the wire attached to the reinforcing steel is connected to the positive terminal of the unit. This results in negative voltage readings at the data acquisition unit. Negative readings are desired as convention dictates recording half-cell potentials as negative. By connecting the unit to read negative units directly, a positive to negative conversion is not required later. The surfactant temperature should also be measured, as the cell temperature is important because the readings must be adjusted to a standard temperature before reporting.

Next, all of the grid points laid out earlier should be wet to increase the stabilization speed. This can be accomplished either by wetting the individual points be hand or by having a water truck with a spray bar make three passes over the deck. The points must be kept wet during the testing.

The connection to the reinforcing steel is made by chipping the concrete away from the reinforcing bars in the hole using a hammer and chisel. The alligator clip is then attached to the steel and a multimeter is used to ensure that there is negligible resistance across the connection. The connection is then detailed on a Half-Cell Connection Data Sheet (Figure 6.7).

After the connection is established and the half-cell is prepared, the stability and accuracy confirmation can begin. The stability readings are the first tests performed. The test investigates the time required for the readings to effectively stabilize. ASTM C876 requires that the readings be stable within 0.02 V CSE over a five minute time span. The test is performed by placing the cell on the surface to be tested at a random point and monitoring the readings for five minutes without removing the cell from the surface. If the readings are within the 0.02 V CSE allowed, the testing continues with the accuracy test. If the readings do not sufficiently stabilize, the deck should be rewet or allowed to soak for a longer time. If the readings cannot be stabilized, the testing should not be continued until the cause of the problems is determined and corrected.

The accuracy test is performed next. It compares readings taken before the main survey to readings taken after the main survey to determine the repeatability of the readings. Ten points are chosen randomly and readings are taken before the readings on the main grid are taken. If any readings are positive or otherwise questionable, the back-up reference cell is used to check them. Later, after all of the grid points have been tested, the ten points are retested. If the readings are different from the original readings by more than the 0.02 V CSE allowed by ASTM C876, the testing of the entire grid and the 10 points should be repeated until the 0.02 V CSE criteria is satisfied.

After the first portion of the accuracy and the entire stability test have been performed satisfactorily, the testing of the main grid can be performed. The readings should be taken in order of grid point, starting with A1 and continuing along each "column" before reading B1. This facilitates the data transfer from the data unit into a personal computer. The half-cell needs only to be placed on each point for as much time as required to reach a reading consistent with its 5-minute reading. For most of the bridges tested while developing this program, the required time was 10-seconds or less. In other words, if the stability tests showed that the 10-second reading was as good as the 5-minute reading, the cell needs only to be left on the bridge deck for 10 seconds.

During the development of the test program, a number of points to heed during testing were noted. It is best to have only one operator take readings on a given bridge to eliminate the small inter-operator differences in readings due to different reading techniques. It is very helpful to have a person helping the operator. The helper can record readings on the ten random points and can keep all of the grid points wet throughout the testing.

A problem that is sometimes encountered during testing is the return of positive potential readings from the test equipment instead of the negative potentials expected. Positive readings indicate a problem with the test conditions. Some of the possible causes of positive potentials are: (a) poor interconnections within the reinforcing mat, (b) an excessively dry deck, (c) a poor connection to the reinforcing mat, (d) stray current interference, or (e) improper connections to the voltmeter. When positive readings are encountered, all of the above possible problems should be checked and remedied. For example, if

CORE SHEET_____ OF _____

PROJECT		1300
DATE		
WEATHER CONDITION		
BRIDGE NO. & NAME		
BRIDGE LOCATION		
CORE #		**CORE LOCATION:** DISTANCE FROM RAIL: ZERO POINT DISTANCE: GRID #: LANE (NB, SB, EB, WB . . .): GRID COORDINATE:
SPECIAL FEATURES:		**GENERAL COMMENTS:**

DEFECTIVE AS SOUNDED BY CHAIN DRAG: _____YES _____NO

HALF-CELL POTENTIAL MEASUREMENT: _____V CSE

CONCRETE SURFACE APPEARANCE: _____SOUND _____SPALLED _____SCALED

Figure 6.6 Sample core data sheet

positive readings are encountered, the deck should be soaked for an extended period, the connection to the reinforcing should be checked or changed, and the connections to the voltmeter should be checked for proper polarity. If the problem is a result of stray current interference or poor interconnections of the reinforcing mat, nothing can be done. If the positive readings cannot be corrected, the test results cannot be regarded as accurate and should not be reported.

During the development of the testing program, numerous accuracy and repeatability tests were conducted on a bridge on the IH-35 frontage road at Onion Creek in Austin, Texas. Through the cooperation of the Austin District Engineer, test connections were built into the bridge to allow half-cell monitoring without the need to core to the reinforcing to make a connection. The testing showed that the readings on any given site visit were within the 0.02 V CSE allowed by the test specification. The changes in the readings from visit to visit were not so consistant. The readings showed a fairly steady decrease with time, becoming less negative on each visit. As the bridge was cast only three weeks before readings were begun, this may only be a "settling in" phenomenon as the bars that were lightly rusted at installation become passivated by the concrete alkalinity. If so, the readings should stabilize with time. More information on this structure will be included in the final report of this project.

6.5.3.6 Crack Mapping

To investigate the effect of cracking on the corrosion performance of the structure, crack maps should be drawn for all structures investigated. The equipment required for properly conducting the crack mapping includes:

1. *Prepared Drawings of the Structure.* The drawings of the individual test grids prepared before the visit are particularly useful in mapping the crack condition of the structure.
2. *Crack Comparator.* These optical crack measuring devices can be simple clear sheets of plastic with crack widths marked on one side or more complicated magnifying devices.
3. *ACI Manual of Concrete Practice.* Committee Report 201.1 in The Manual of Concrete Practice contains photographs and definitions of typical types of cracking and serves as an excellent reference for crack identification.
4. *Tape Measure.* A tape measure is required for determining crack lengths and spacing.
5. *Camera.* Many types of cracking are best recorded through photographs.

The crack mapping is performed by simply starting at one end of a test grid and closely inspecting the concrete. The locations and types of the cracks are recorded on the drawing of the grid, noting spacing and crack sizes. The grid eliminates the need to measure each crack back to the zero and rail points. All unusual or peculiar area of cracking should be photographed. For photographing, it is helpful to wet the surface first and take the picture after the surface has dried and the cracks are still wet. The locations of any encroaching overlays or seal coats should be noted on the maps as well. If possible, the underside of the deck should be observed for any signs of efflorescence, cracking, or staining. Lastly, the location, size, and frequency of any popouts should be noted. The survey results in a map similar to Figure 6.8.

It must be stated that some discretion is necessary in preparing the crack maps. Obviously, every crack on an extensively damaged deck could not be located, sized, and plotted. Once again, by keeping the surveying objective in mind, the most reasonable solution can be easily determined.

6.5.3.7 Delamination Mapping

Delaminations are recorded in conjunction with the crack mapping. There are many means of detecting delaminations, ranging from simple sounding rods to impact-echo testing devices. Because of cost, accuracy, and testing ease, a chain broom was used. The only equipment required is:

1. *Chain Broom.* As described in ASTM D4580, a chain broom consists of a handle attached to a number of chains, each approximately two feet in length. The broom tests a strip of the structure approximately two feet wide.
2. *Prepared Maps.* The maps of the individual test grids that were prepared before reaching the test site are used to record the delamination information.

The testing is performed by dragging the chain broom along the surface while listening to the sound produced by the chains. When dragged over sound concrete, a high-pitched tinkling sound is produced as opposed to the dull hollow sound produced over delaminated areas. When a delaminated area is found, its location is recorded on the prepared maps using the reference grid marked on the bridge and the maps. Sections can be retested as necessary to confirm questionable or unsure readings. For small areas, it has been found helpful to lift the

HALF-CELL CONNECTION SHEET______ OF ______

PROJECT		1300
DATE		
WEATHER CONDITION		
BRIDGE NO. & NAME		
BRIDGE LOCATION		
CONNECTION NUMBER		**CONNECTION LOCATION:** DISTANCE FROM RAIL: ZERO POINT DISTANCE: GRID #: LANE (SB, NB, EB, WB . . .): GRID COORDINATE:
SPECIAL FEATURES:		**GENERAL COMMENTS:**

SURFACE CONDITION:

SURFACE TEMPERATURE:

Figure 6.7 Sample half-cell connection data sheet broom over the areas and rapidly lower the chains onto the surface. The sounds produced are the same as when dragging the chains, but can be easily repeated and can be attributed to an exact area.

As mentioned above, there are many different tests for delaminations. The chain drag was chosen based on its low initial cost, the speed of testing, and its relative simplicity. However, there are some drawbacks to the test. In high traffic situations, the chain noise can be masked by the traffic noise. Also, the test may not detect small delamination areas.

6.5.3.8 Photography

The testing of each structure should be thoroughly documented with photographs. Slide film should be used, as slides can be used in presentations and can be made into prints. It is much more difficult to make slides from prints. General conditions, cracking patterns, efflorescence, deterioration, and any other visible features should be documented. A few photographs can clear much of the confusion that can develop during the data reduction after the visit.

6.5.3.9 Miscellaneous

Any other tests or observations not mentioned above, but that contribute to achieving the test objectives, should be performed as necessary. These tests may include sealer penetration testing, skid resistance determination, or overlay bond strength, to name a few. Again, these tests should be used if logic and the testing plan so dictate.

6.5.3.10 Patching

The last task performed at the structure under investigation is the patching of the sample and core holes. The exact patching method should be determined in cooperation with the District contact person before reaching the test site. The patching plan should minimize future degradation of the bridge and should ensure that the corrosion testing performed will not jeopardize the future performance of the structure. Successfully applied patching materials include quick-setting concrete patching material, polymer concretes, and "cold-patch" material. Again, any patching method agreed upon by both the District contact and the surveyors is acceptable.

6.6 POST-VISIT REQUIREMENTS

6.6.1 Introduction

After the field visit is completed, and before the next field visit is performed, the data and samples from the visit must be collected, cataloged, and stored. Care must be exercised during this portion of the testing program, as this step is where valuable information can be easily lost or misplaced through carelessness.

6.6.2 Data

The first task after conducting a field survey is to collect the data from the field notebooks and store it in a safe place until it can be returned to the lab. This is also done to provide space in the field notebooks for the next survey. In addition to the paper data sheets, the corrosion potential data stored in the data acquisition unit should be transferred to more permanent storage. This is done by downloading the unit into a portable computer and storing it both on the internal drive and a removable floppy disk. The floppy disk should be stored away from the computer for extra security.

6.6.3 Samples

The chloride samples and the cores should be stored for transport. The chloride samples may be left in the bottle trays and simply moved to a safe location. The cores should be left in their plastic bags and collected in boxes. The boxes should then be stored in an out of the way location where they are protected from excessive vibration and shock during transport.

6.6.4 Miscellaneous

In addition to caring for the data and samples, all of the associated support equipment should be checked and repaired as necessary. This includes checking the generator oil and gasoline levels, cleaning the rotary-hammer, and recharging all of the battery-powered equipment used during the testing. Extra vigilance in performing these tasks will reduce the number of problems experienced during testing.

6.7 LABORATORY TESTING

6.7.1 Introduction

Once all of the data and samples have been brought back to the laboratory, the analysis can

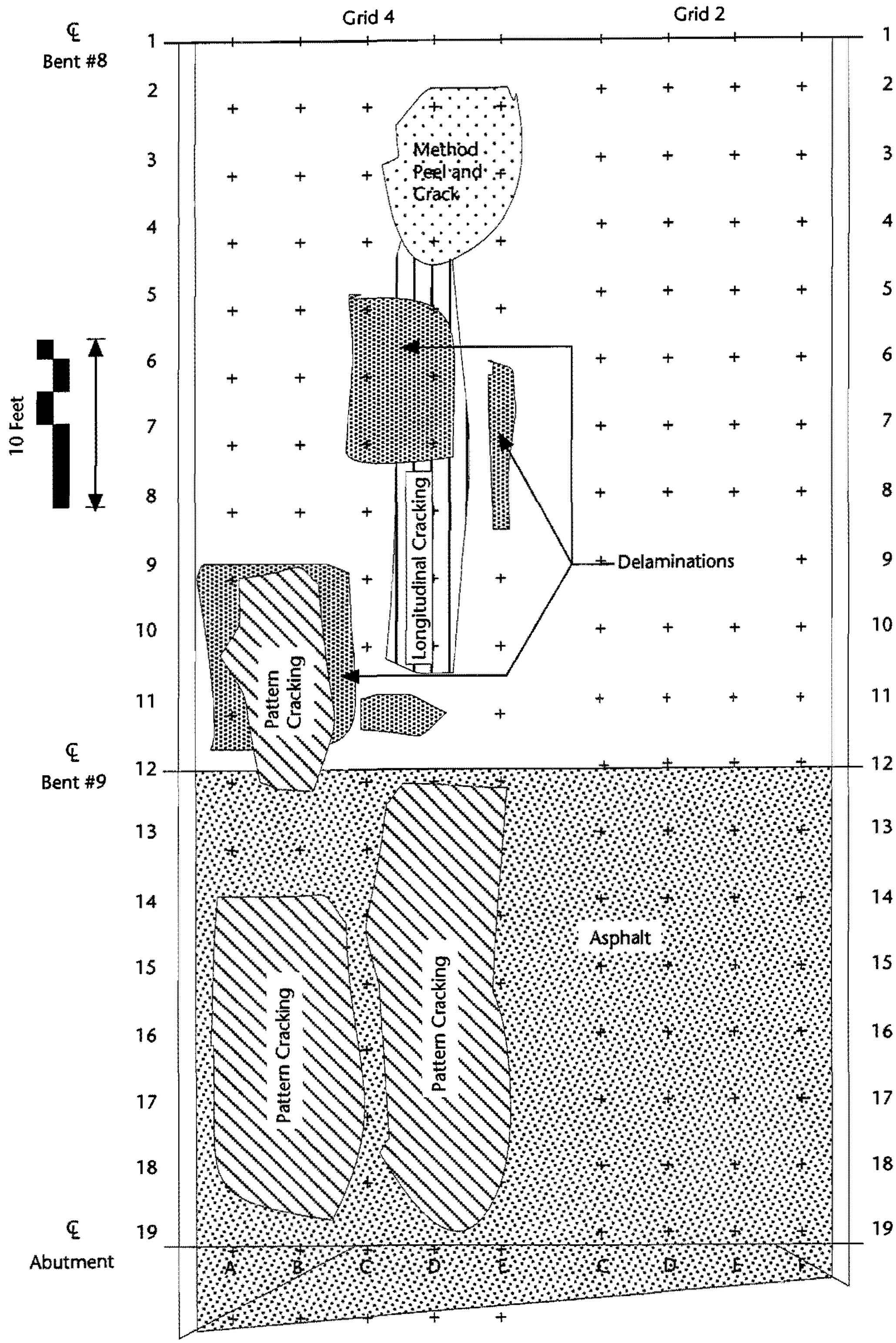


Figure 6.8 Sample crack map

begin. This stage of the testing is where the information is developed and combined to answer the survey's testing objectives. All of the tests were performed following the ASTM or AASHTO test specifications referenced, unless noted in the following sections.

6.7.2 Core Cataloging

The first step in testing and analyzing the cores brought back from the test site is to thoroughly catalog the cores. This will help to determine which cores should be used for the different tests to be performed. The cores are cataloged on a sheet such as that shown in Figure 6.9. The core cataloging sheet includes spaces for information such as core length, steel reinforcement depth, crack widths and depths, and surface chipping. Once the cores have been cataloged, the sheet can be used to develop a plan for cutting various portions of the cores for the various tests. The testing and cutting plan is recorded in the appropriate boxes on the worksheet and can be given to the technicians cutting the cores for guidance.

6.7.3 Concrete Permeability Testing

As described in Chapter 4, Section 2.3 of this report, the AASHTO T277 Rapid Chloride Ion Permeability Test (RCIPT) is used to compare the relative permeabilities of concretes. The first step in testing is the sample determination. For the purposes of corrosion protection system performance testing, various sample types were used, including:

1. *Treated Surface Specimens.* For bridges treated with surface sealers such as linseed oil, silane, or methacrylate, some cores were tested with the treated surface of the core left intact. This can indicate the effectiveness of the treatment when compared to untreated control specimens. Note that leaving the top surface intact deviates from the AASHTO procedure, which calls for removing 1/2 inch of concrete from the top of the specimen.
2. *Untreated Surface Specimens.* For all bridges, some cores were tested with the finished surface left intact. This was done to provide information about the effect of various surface finishes such as tining or saw grooving on concrete permeability. If possible on structures treated with surface sealers, control specimens with untreated surfaces were tested to allow a comparison of treated to untreated surfaces. It is necessary to use a sample with an untreated surface for comparison with surface treated specimens because specimens with the top 1/2 inch of concrete removed typically exhibited higher permeabilities than specimens with the top 1/2 inch left intact.
3. *Standard Specimens.* Specimens from all structures were tested in accordance with the AASHTO procedure. Because the top 1/2 inch of concrete is removed from these specimens, comparisons can be made between the concretes of different structures or different portions of the same structure without the effect of surface finish influencing the results.
4. *Overlay Material Specimens.* When overlays were used in original or remedial applications, the overlay concrete wastested to provide an indication of the overlay performance.
5. *Base Material Specimens.* For bridges protected with some type of overlay, the underlying base concrete was tested. This served two purposes. First, it allowed a comparison of the overlay and base concretes. Second, in remedial overlay applications, it provided information regarding the original concrete that failed and necessitated the overlay application.

Permeability testing should be performed according to AASHTO T277. To aid in following the test procedure and in tracking individual samples through the process, the data sheet shown in Figure 6.10 was developed. The sheet is designed for use with a test apparatus designed for testing eight samples at a time, but could be easily modified to accomodate the equipment available.

It is worthwhile to save all specimens after testing. This allows for investigation of the samples should abnormal or unexpected results be found.

The results should be reported in tabular form, indicating the total charge passed in coulombs, any special testing circumstances, and the type of specimen as detailed above.

6.7.4 Strength Testing

The core catalogue sheet can also be used to determine which cores should be used for strength testing. The core needs to be in excess of four inches long. This is necessary as a four-inch-diameter core must be four inches long after the ends have been cut in order to preserve the 1:1 minimum height-diameter ratio specified in ASTM C42 for core strength testing. Also, the cores chosen for strength testing should not contain any reinforcing steel as the strength results will be adversely affected. As mentioned above, the ends of the core should be cut to provide plane, parallel loading surfaces. The strength testing is facilitated by the use of neoprene end caps rather than sulfur capping compound and can be performed in any concrete testing machine of sufficient capacity.

The strength test results should be reported in tabular form, including the actual sample lengths and diameters, failure load, computed failure stress, and method of testing.

6.7.5 Petrography

A number of cores should be prepared for petrographic examination. The air content and paste system analysis serve to characterize the concrete and are relative indicators of concrete placement quality. The sulfate and alkali-aggregate investigations should be performed only when there is a question regarding the reason for deterioration in an existing structure. In these cases, the information gained from these two tests can help to determine the cause of the deterioration.

The preparation of the samples consists of cutting and polishing the core in preparation for optical investigation using a petrographic microscope. In many number of cases, knowledge of the extent and severity of cracking can prove helpful in understanding the corrosion state of the structure. This can be accomplished by soaking representative cores in an unpromoted methacrylate containing a dye that fluoresces in ultraviolet light. After soaking the cores for one day, they are placed in plastic bags and put in a 170° F oven to promote the methacrylate, causing it to harden. The cores can then be sliced and viewed under UV light to reveal the extent and depth of cracking.

6.7.6 Chloride Content Analysis

The chloride content analysis is performed according to the ASTM C1152 procedure or with commercially available equipment that provides equivalent results. The percentage chloride can be converted into pounds per cubic yard of concrete by assuming a given density of concrete and multiplying. For most work, the assumption of 4,000 pounds of concrete per cubic yard is acceptable.

The results of the test are entered into a spreadsheet for storage and graphing. The results for all the samples at the four sampling depths are best recorded in tabular form.

6.7.7 Miscellaneous

Any other tests or investigations performed should be performed and reported as necessary to fulfill the testing objectives. These could include overlay bond strengths, overlay composition, or other such tests.

6.8 REPORT

6.8.1 Description

A complete report should be written for each structure investigated. The report should include:

1. *Cover Page.* The cover page should include the names of the investigators, the District contact person, the name and location of the structure tested, the tests performed, the date tested, and the weather during testing.
2. *Brief Report.* A brief report detailing the structure history, the tests performed, problems with testing, and the conclusions drawn should be included in the report.
3. *Structural Plans.* A full set of plan and elevation drawings for the structure should be included. The plans should show the location of the structure, and all components tested.
4. *Sample Location Maps.* Maps should be included that show the locations of all samples taken from the structure. This allows the cracking, delamination, half-cell, chloride content, and strength information to be combined by location. This allows an investigation of the combinations of the different factors in overall corrosion performance.
5. *All Test Results.* All of the test results should be reported as described in Chapter 6, Section 7.

6.8.2 Distribution

In addition to a copy to files for the report originator, copies of the reports generated for each structure should be distributed to the district contact person and the sponsoring authority. The reports should also be used for developing a corrosion performance database in some central location.

Core Log Sheet
CTR Project 1300

For Bridge: 7 - Business 281 Overpass - Wichita Falls
Wash and Sweep Maintenance Strategy

Legend
X - Not Possible
1 - Selected
0 - Possible, Not Selected

Core Number	Length (in.)	Overlay Thickness	Base Thickness	Surface Cracks (Yes/No)	Crack Widths (mm)	Crack Depths (16ths)	Chipped (Yes/No)	Surface Texture	Groove Depths (16ths)	Reinforcing Depth	Surface Treatment	With Top	Without Top	Base	Petrography	Structural	Notes
7-1-WBT-C3-C1	2.50	NA	NA	No	–	–	Yes			2.25		X	0	NA	0	X	Half-cell Connection
7-1-WBT-B6-C2	4.25	NA	NA	Yes	0.06	TR	No			NA		1	1	NA	0	0	Surface Map/Pattern Cracking
7-1-WBT-C13-C3	4.50	NA	NA	Yes	0.06	TR	No			5.00		1	1	NA	0	X	Centerline of Lane, Hit Steel
7-2-WBT-C2-C4	2.50	NA	NA	Yes	0.06	TR	No			2.50		0	0	NA	0	0	1/2 Cell, Light Cracking, in Wheelpath
7-2-WBT-B9-C5	4.75	NA	NA	No	–	–	No			4.75		1	1	NA	0	0	Sound Surf., in Breakdown Lane
7-2-WBT-B15-C6	3.00	NA	NA	Yes	0.08	TR	No			NA		1	0	NA	0	X	Pattern Cracking, Voids Near Surface
7-3-WBT-C2-C7	2.00	NA	NA	Yes	0.06	TR	Yes			2.00		0	X	NA	0	X	1/2 Cell, Centerline, No Cracks
7-3-WBT-B8-C8	4.25	NA	NA	Yes	0.06	TR	No			NA		0	0	NA	0	1	Slight Cracking, Some Entrapped Air
7-3-WBT-B12-C9	4.75	NA	NA	No	–	–	No			NA		1	0	NA	0	0	No Cracks, Breakdown Lane, Large Aggregate
7-4-WBP-F3-C10	2.00	NA	NA	Yes	TR	TR	No			2.50		1	X	NA	0	X	1/2 Cell Connection
7-5-WBP-2G-C11	2.75	NA	NA	Yes	0.06	TR	No			2.50		0	0	NA	1	X	1/2 Cell Connection, Large Void at 2 in. Down
7-6-WBP-4F-C12	2.00	NA	NA	Yes	0.15	0.25	No			2.00		0	X	NA	0	X	1/2 Cell Connection
										Totals		6	3	0	1	1	

Figure 6.9 Sample core log sheet

PERMEABILITY INFORMATION SHEET

TEST SET:

HELIOS FILE NAME:

CORE NUMBERS

Time Vacuum Started

vacuum time

Time Vacuum Soak Started

vacuum soak time

Time Regular Soak Started

soak time

Time Siliconed

set time

Time Testing Started

SPECIMEN LAYOUT

Back

Front

Figure 6.10 Permeability data sheet

CHAPTER 7. RESULTS

7.1 INTRODUCTION

During the eight field surveys conducted during June and August 1992, information was gathered concerning the testing regime, the individual corrosion protection systems, and the general corrosion protection performance of concrete in bridges. This chapter presents a summary of the test results. It should be emphasized that the purpose of the field testing in this phase of the study was not necessarily to evaluate the performance of different corrosion protection system, but to evaluate the adequacy of the proposed test regime. (For detailed information on the performance of the individual bridges visited during the surveys, refer to Appendices A1 through A8 of this report.)

7.2 TESTING REGIME

7.2.1 Introduction

The eight site visits conducted during this portion of the project served to develop and verify the testing regime as well as to characterize the bridges visited. The usefulness and accuracy of the test program were confirmed, as was its applicability to the field testing of bridges.

In this chapter, the eight bridges visited are identified by a number referring to the order in which the bridges were visited. The numbers are repeated below for clarity.

1. IH-20 at Morgan Creek in the Abilene District
2. SH 361 Redfish Bay Bridge in the Corpus Christi District
3. FM 610 over Nicholson Creek in the Abilene District
4. FM 1835 over the Salt Fork of the Brazos River in the Abilene District
5. 34th Street overpass at IH-27 in the Amarillo District
6. IH-40 at the A.T. & S.F. Railroad overpass in the Amarillo District
7. Business 281 overpass in the Wichita Falls District
8. SH 67 at the Brazos River in the Wichita Falls District

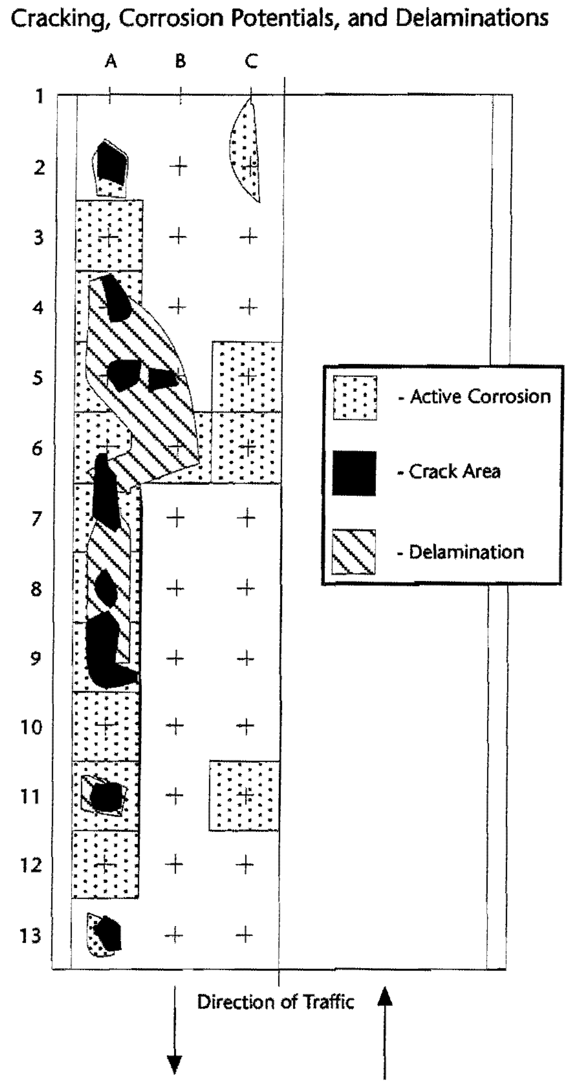


Figure 7.1 ***Correlation of half-cell, cracking, and delamination data for structure #3***

7.2.2 Half-Cell Corrosion Potential Test

Five of the eight bridges visited contained uncoated reinforcement and were evaluated using the half-cell corrosion potential test. The half-cell data agreed well with the results of the chloride content testing, the crack maps, and the delamination soundings. Figure 7.1 shows this for structure #3. The test was found to be quick and easy to perform once the connection to the top mat of reinforcement was made. At structure #7 and one span of structure #8, problems were experienced with the test. The readings either would not stabilize or were otherwise unusable according to ASTM C876. The exact reasons for the unacceptable readings are not known. The problems are discussed in detail in Appendix A.

The half-cell test was found to be an essential portion of the test program on all the bridges where it was applicable. For bridges constructed with uncoated reinforcement, the test is an extremely useful tool for characterizing the corrosion condition of the structures. It can determine the corrosion state of a structure before any visible manifestations of damage, allowing it to be used as an indicator of future performance. This allows the test to be used for long-term repair and replacement planning.

For new construction, its use would be facilitated by the inclusion of a built-in connection to the reinforcing steel in all bridges. The simple connection detail could be similar to the connection detailed in Figure 7.2 that was used at the IH-35 frontage road test site in Austin, Texas. The connection requires no special equipment and can be performed by non-technical personnel.

7.2.3 Chloride Content Testing

All eight bridges were tested for chloride content at various locations in the deck. The sampling was simply performed and the samples were easily tested. Although the sampling techniques and testing procedure are very simple, special consideration should be given to ensure consistent results. It was found that special care during sampling, including frequent cleaning of the sampling equipment and prevention of cross-sample contamination were essential to ensure reliable test results.

The chloride contents correlated well with the condition of the bridges, such as for structure #6, where high potential areas matched high chloride areas as shown in Figure 7.3. Also, the generally accepted relationship between chloride contamination and occurrence of corrosion was confirmed by the high chloride contents observed in structures #2 and #3, where deterioration was evident,

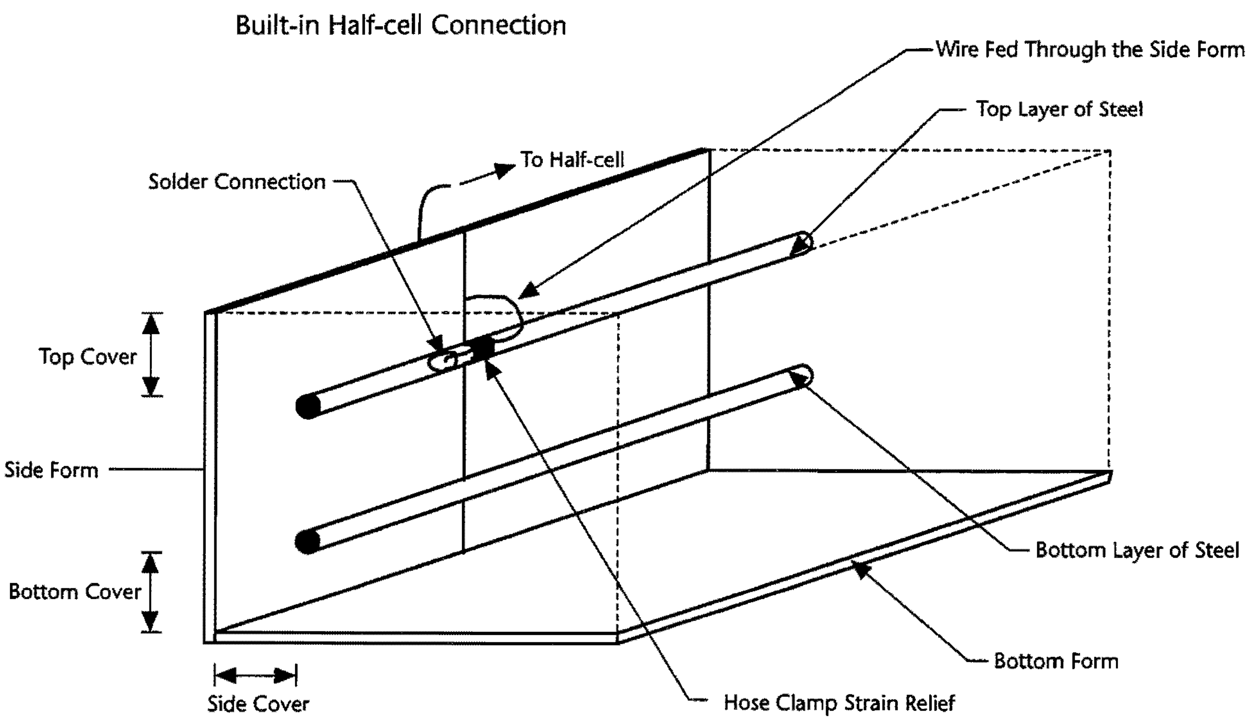


Figure 7.2 Sample half-cell connection detail

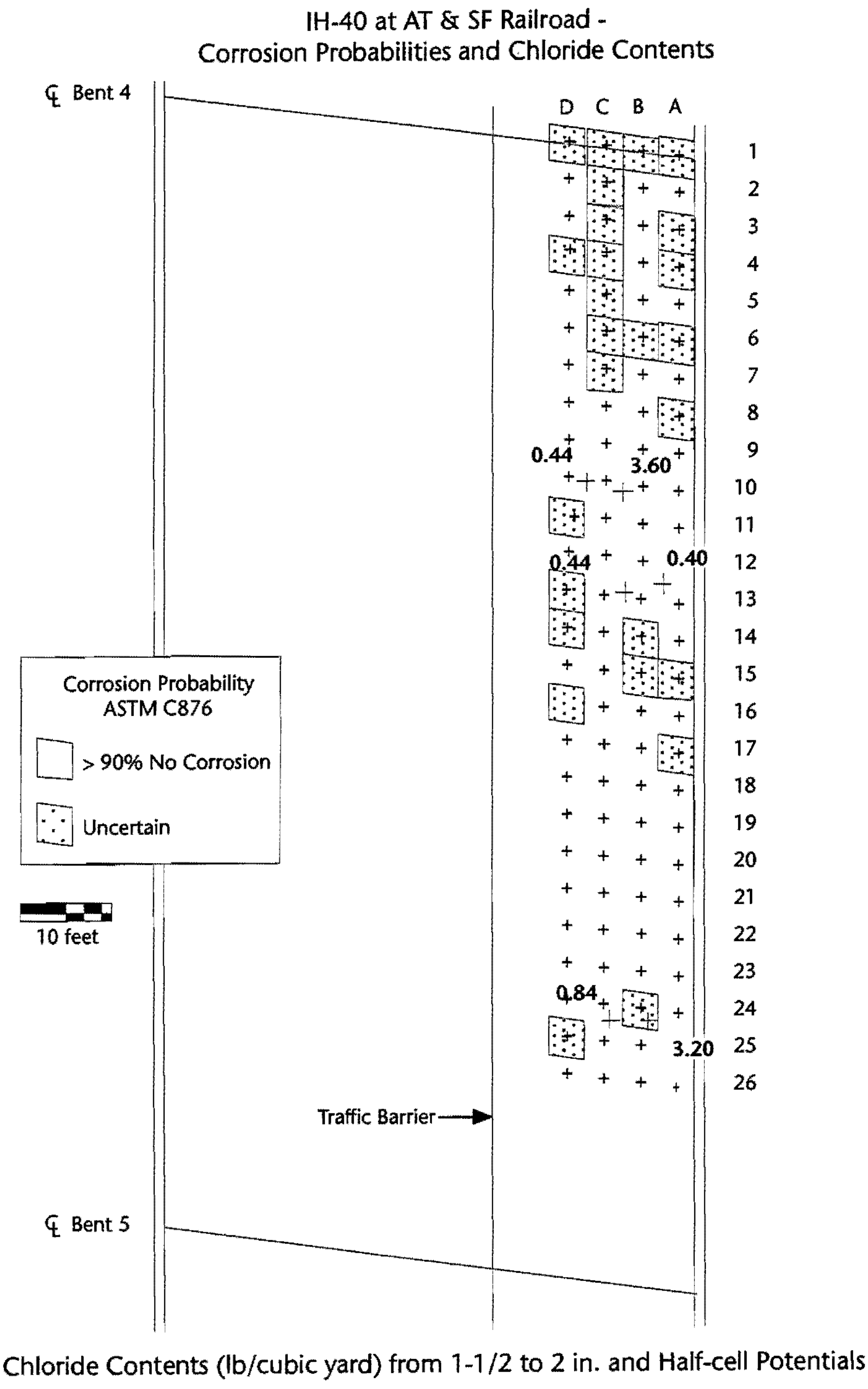


Figure 7.3 Chloride content and half-cell potential correlation

and by the low chloride contamination of structures #1, #4, #5, #7, and #8 that exhibited little corrosion-related damage. Lastly, on structures #5 and #6, the test confirmed that the chloride penetration of a deck was higher at cracks.

The test should be included in any monitoring or evaluation program, as the chloride contents can serve as indicators of corrosion state or relative resistance to chloride penetration of various surface protection methods.

7.2.4 Permeability Testing

Permeability testing was performed on all structures except structure #2 where no cores were retrieved. The tests worked well to allow cross-structure comparisons of concrete permeabilities as well as to characterize the permeability of the concrete an any particular bridge.

It was found that cracking did not affect the permeability of concrete samples as tested according to AASHTO T277. When the samples were visually examined after testing, it was found that the cracks were all sealed below the surface of the concrete with dirt, silt, and other materials. This applied even in the case of samples with cracks completely through the samples. As a result, there is a concern regarding the effect of accumulation of debris in an existing crack in a deck from the AASHTO T277 test results. The presence of debris within a crack, however, was not found to prevent the migration of chloride into the concrete deck.

7.2.5 Crack Mapping

The crack mapping was the most inexact of all the tests performed. Because of the widely varying condition of the bridges visited, the quality and usefulness of the crack maps varied considerably. Because of the volume of cracking on many of the bridges, it was nearly impossible to accurately account for the many cracks on heavily damaged bridges. In general, the crack maps often proved useful or were found to correlated well visually with half-cell corrosion potentials or chloride content. For example, on structure #6, the areas of higher potential matched the large observed cracks very well. This also held true on structure #8, where high potentials were noted in cracked areas.

Despite their inherent inaccuracies, crack maps should be included in all corrosion surveys. They served well to characterize the overall condition of the structure and identify areas of particular interest.

7.2.6 Strength Testing

Although the strength testing does not supply any information directly related to corrosion performance, it does serve to characterize the concrete used in the structure. This in turn related qualitatively to corrosion performance through factors such as water-cement ratio and paste fraction.

No relationship between strength and corrosion performance could be determined because of the extremely limited number of acceptable strength cores retrieved from the bridges.

7.2.7 Delamination Testing

Delamination surveys were performed on all of the decks investigated during the field trials. The delaminations matched the areas of high corrosion potentials on structures #3 and #8, the only structures where substantial areas of delaminations were encountered. The only problem with the test was the difficulty in conducting the test in high traffic areas where the traffic noise drowned out the noise of the chains.

The test should be included in any corrosion survey, as it quickly locates any trouble spots that probably merit further attention. The test is rapidly conducted, with little experience or expertise required.

7.2.8 Petrographic Analysis

Petrographic analysis was conducted on cores from structures #1, #3, #4, #5, #6, #7, and #8. The analysis revealed that all of the concrete air systems were marginally sufficient or slightly insufficient. The analysis revealed insufficient air systems in the dense concrete overlays of structures #5 and #6, reinforcing the general perception that it is difficult to place dense concrete overlays with sufficient air system characteristics. Only the older, original base concrete of structure #1 was tested, and its air system was found to be sufficient.

In addition to air system analysis, a number of cores were visually inspected to determine other characteristics of the samples such as crack depth, sulfate attack, and freeze-thaw damage. Because structures #2 and #3 were exposed to sulfates, samples from structures #2 and #3 were also investigated with a scanning electron microscope to determine if sulfate attack had occurred in the concrete. No sulfate attack was found in structure #2. Structure #3 contained limited quantities of ettringite, the sulfate attack product. The ettringite, however, was found to be negligible and not the cause of the deterioration of the concrete. This was determined based on the formation pattern of the ettringite and the quantity of ettringite found.

7.3 CORROSION PROTECTION SYSTEMS

7.3.1 Introduction

The information learned during the summer field studies should be regarded as preliminary at best. More information will be gained through visiting new sites and subsequent visits to the structures detailed in this report as the performance database is increased.

7.3.2 Epoxy-Coated Reinforcement

No corrosion damage was found at bridges #4, #5, and #8, which were protected with epoxy-coated reinforcement. It must be noted, however, that the oldest of these structures was only 7 years old. Structures this young would not typically show any corrosion damage regardless of their protection systems.

7.3.3 Dense Concrete Overlays

Structures #1, #5, and #6 were protected through use of dense concrete overlays and showed generally good performance. The 4-year-old remedial overlay on the 35 years old structure #1 was found to be in the early stages of corrosion, while the 10-year-old remedial overlay on structure #6 exhibited negligible corrosion. Structure #5 was overlaid in 1989 as part of the original construction. It is also protected with epoxy-coated reinforcement and showed no signs of corrosions.

All of the structures with dense concrete overlays exhibited low permeabilities. The chloride penetration of the overlays varied considerably, even on individual bridges. The main cause of the variability in the chloride penetration was the cracking of the deck generally characteristic of this type of overlay. In the immediate vicinity of cracks, the chloride contents at the level of the reinforcement were often above the corrosion threshold of 1 to 1.5 lb/yd^3. At uncracked locations, the chloride contents at the depth of the reinforcement were low despite high chloride contents in the upper portion of the deck. The air void systems of the dense concrete overlay systems were typically below standard, reflecting the construction difficulties associated with this type of overlay.

This protection system appears to be working with limited damage being noted. Structure #1, the only structure indicating active corrosion, was overlaid because of previous deterioration of the deck. The evaluation of the system should be continued to determine the performances of both original construction and remedial use of overlays.

7.3.4 Wash and Sweep Maintenance Strategy

Structure #7 was protected by a "wash and sweep" maintenance strategy that calls for the washing and power-sweeping of the bridge as soon as possible after any deicer applications. The structure was 24 years old when surveyed for the project and showed no corrosion related damage. The half-cell potential test did not function properly at this bridge, so detailed corrosion performance could not be determined. The test equipment indicated positive potentials despite repeated surface wettings, so according to ASTM C876, the results were regarded as erroneous. Although the exact cause is unknown, the problems could be due to poor connections between the reinforcing bars or to stray currents resulting from any electrical lines carried by the structure.

The structure showed significant traffic wear and surface cracking, but only one small region of delamination was detected. The chloride contents at the reinforcement were generally low but somewhat variable. The permeabilities of the concrete were also variable, varying from ,1857 to 4,537 coulombs for the surface concrete. The underlying concrete was found to be quite permeable. As previously mentioned, no corrosion damage was noted in spite of the low permeabilities and the high chloride contents.

This protection system should be further investigated to determine the merits of this low-cost alternative for corrosion protection.

7.3.5 Methacrylates

Structures #1 and #3 were protected with high-molecular-weight methacrylate overlays with seeded sand for skid resistance. Structure #8 included two methacrylate test spans as well. Structures #1 and #8, which are 5 and 7 years old respectively, show no corrosion damage. Span 2 of the 42-year-old structure #3 had a one-year-old methacrylate overlay, showed extensive corrosion damage, and was demolished because of through-holes developing in the deck.

In structure #8, comparisons between the span protected with methacrylate and those protected with linseed oil or other sealers showed little difference in chloride penetration for the two systems. These samples were taken from uncracked locations. Permeability tests were conducted on all of the bridges. The tests were performed on specimens with the methacrylate both intact and removed. The samples with the methacrylate described as "INTACT" were from the surface of the bridge, and the samples described as "REMOVED" either had the methacrylate-coated top 1/2 inch removed or were taken from the portions of the cores below the top 2-inch specimen. As shown in Table 7.1, the permeabilities of the untreated specimens were not significantly higher than those of the treated specimens for structures #1 and #8. Structure #2 performed differently, with substantial differences between the specimen types.

Table 7.1 ***Comparison of permeabilities for methacrylate-treated cores***

Structure	Methacrylate*	
	Intact	Removed
1	923 (6)	1,436 (3)
2	2,103 (5)	6,592 (4)
8	2,843 (3)	3,090 (1)

*The number of samples averaged to determine this value is included in parenthesis

For structure #8, the chloride content of the methacrylate-treated span could be compared to that of the other spans of the structure that were treated with other systems. As shown in Figure 7.4, the chloride content of the methacrylate protected spans was slightly higher than that of the linseed oil treated spans. Grid 5, which showed high chloride penetration, had a notably different surface appearance, with coarse aggregate exposed throughout, and some samples from grid 1 were taken from cracked locations.

7.3.6 Sealers

Structures #4 and #8 were both test structures with a variety of surface sealers. Structure #4 utilized silane, siloxane, sodium silicate, and linseed oil. Structure #8 was protected with methacrylate, linseed oil, or a calcium nitrite admixture. Both of the structures were less than 8 years old when visited. The results of the permeability and chloride content testing of the two structures are contained in this section for comparison. As before, some of the permeability specimens were tested with the sealed surface intact and some were tested without a sealed surface. For these two structures, the "TREATED SURFACE INTACT" samples came from the top two inches of the retrieved cores, while the untreated "SURFACE REMOVED" specimens were taken from the 2-to 4-inch portions of those same cores. As shown by the individual sample results presented in Table 7.2, in structure #4 the linseed oil and 20 percent silane in a water carrier had the lowest permeabilities, followed by the sodium silicates, siloxanes, and silanes in an isopropanol or mineral spirit carrier. As can be seen in Figure 7.5, the linseed oil treated span showed the lowest chloride contents, followed by the 20 percent silane in water. The silanes in mineral spirits and isopropanol spans had higher chloride contents. The two spans treated with sodium silicate showed highly variable chloride intrusion.

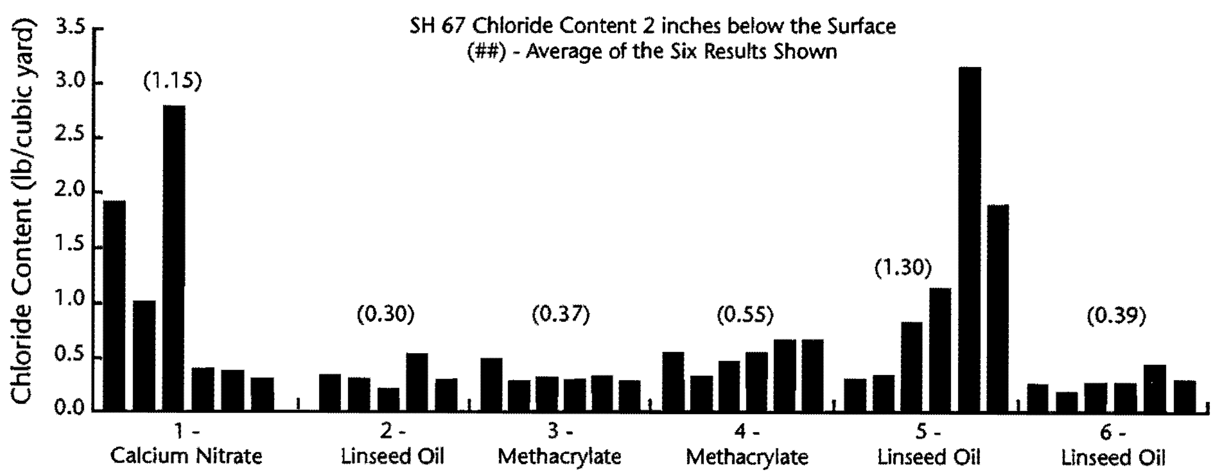


Figure 7.4 ***Chloride contents for the structure #8 test spans***

Table 7.2 FM 1835 core permeabilities

FM 1835 at the Salt Fork of the Brazos Multiple Test Systems Concrete Permeabilities (Coulombs)				
		Treated Surface Intact		Surface Removed
Sample	Surface Treatment	Cracked	Uncracked	Uncracked
4-1-C1	Linseed oil			12,321
4-1-C2	Linseed oil	10,943*		
4-2-C3A	Sodium Silicate		4,469	
4-2-C3B	Sodium Silicate			7,609
4-2-C4	Sodium Silicate		4,432	
4-3-C5	40% Silane in isopropanol		5,071	
4-4-C8A	20% Silane in isopropanol		6,109	
4-5-C10A	Sodium Silicate		4,639	
4-5-C10B	Sodium Silicate			6,366
4-6-C12	20% Silane in water		3,329	
4-7-C13A	Silane in mineral spirits		5,500	
4-7-C13B	Silane in mineral spirits			9,115
4-7-C14A	Silane in mineral spirits		4,986	
4-7-C14B	Silane in mineral spirits			8,366
4-8-C16	Linseed oil, siloxane		5,027	
4-9-C17	Linseed oil		3,919	
4-9-C18	Linseed oil		3,836	

*This specimen had a large chip in the surface exposed to the NaCl during testing, possibly affecting the results

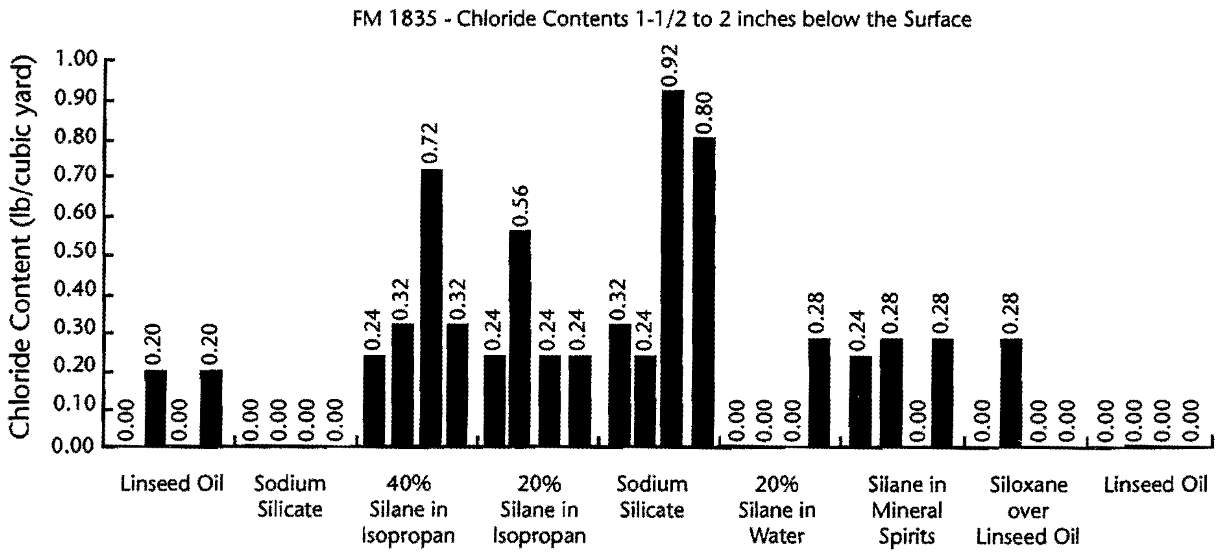


Figure 7.5 FM 1835 chloride contents

Table 7.3 Structure #8 concrete permeabilities

SH 67 at the Brazos River Multiple Test Systems Concrete Permeabilities (Coulombs)					
			Treated Surface Intact		Surface Removed
Span	Sample	Surface Treatment	Cracked	Uncracked	Uncracked
9	8-1-NB-8D-C2	Calcium Nitrate*		4,169	
9	8-1-NB-D15-C3A	Calcium Nitrate*	3,939		
9	8-1-NB-D15-C3B	Calcium Nitrate*			2,312
8	8-2-NB-D8-C5	Linseed Oil	3,402		
7	8-3NB-10D-C8	Methacrylate	2,584		
7	8-3-NB-18D-C9A	Methacrylate		3,662	
7	8-3-NB-18D-C9B	Methacrylate			3,090
6	8-4-NB-7D-C10	Methacrylate		2,283	
5	8-5-NB-2D-C12A	Linseed Oil		4,406	
5	8-5-NB-9-C12B	Linseed Oil			11,638
5	8-5-NB-9-C13A	Linseed Oil		5,100	
5	8-5-NB-9-C13B	Linseed Oil			8,027
5	8-5-NB-22-C14	Linseed Oil		1,976	
4	8-6-NB-11D-C16	Linseed Oil (Control)		3,300	
4	8-6-NB-21C-C17	Linseed Oil (Control)			2,777

*Although not a surface treatment, calcium nitrate is listed in this column for convenience.

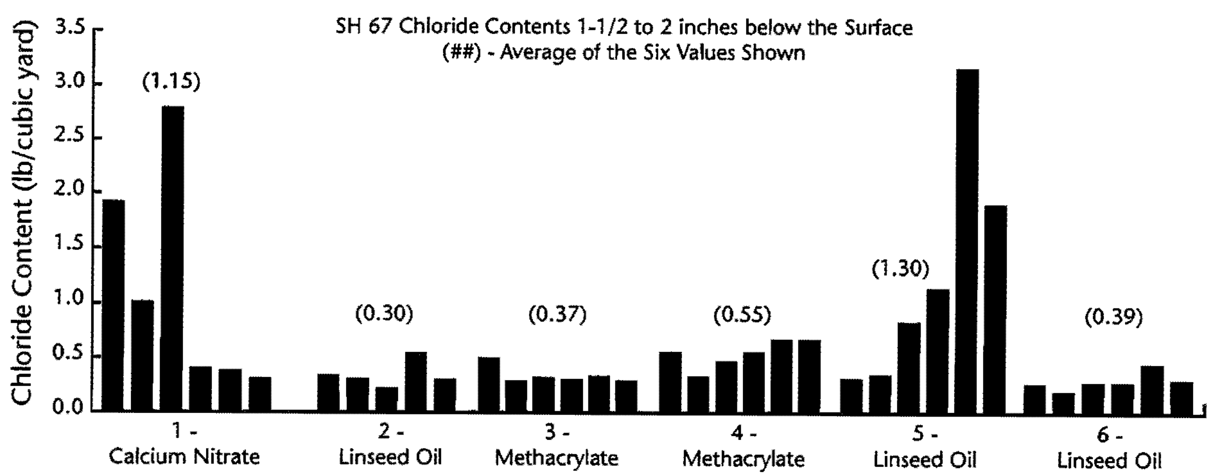


Figure 7.6 Structure #8 chloride contents

In structure #8, the spans treated with methacrylate and linseed oil had approximately the same permeabilities, with the linseed oil samples being slightly more variable, as shown in Table 7.3. Figure 7.6 shows that the linseed oil and methacrylate treated spans had approximately the same amount of chloride intrusion in the 1-1/2 to 2-inch sampling region. Test section 5 of structure #8 had greatly different surface characteristics than those of the other spans and may not be comparable to the other spans.

Since all of these structures are relatively young, monitoring should be continued until time-effectiveness histories have been developed.

7.4 GENERAL CORROSION PERFORMANCE

7.4.1 Introduction

In the course of the field trials of the test regime, some basic knowledge was gained of the field performance of corrosion protection measures. This insight was mainly into the cracking and wheelpath-wear aspects of corrosion performance.

7.4.2 Cracking

As discussed above, the permeability and chloride testing portions of the test programs may have revealed the reason for the variable effect of cracking on corrosion performance. Cracking has always been a concern for corrosion protection systems such as protective overlays and surface sealers, but some reports (Ref 113) have found that cracking was not an important factor in corrosion performance.

After testing showed no difference in the permeabilities of cracked and uncracked cores, some of the cracked cores were soaked in a dyed methacrylate for two days after oven drying for one day. When viewed under an ultra-violet light that caused the dye to fluoresce, it was found that the cracks were completely sealed with dirt or silt 1/4 to 1/2 inch below the surface of the concrete. As a result of this, the wider cracks sealed with dirt did not get filled with methacrylate while the concrete paste system fluoresced, indicating that the methacrylate had penetrated the capillary system of the concrete. This agrees with the observed high chloride penetration at cracks. Although the cracked samples exhibit normal permeability, materials in solution, such as chloride ions and the dye, are able to penetrate the cracks to a greater extent than the surrounding concrete. This shows that the susceptibility of cracked deck concrete to chloride penetration is not effectively indicated by the AASHTO T277 permeability test, as a low permeability does not necessarily indicate a low chloride content.

The chloride content analysis, however, showed cracking to be an important factor. Structures #5 and #6 were sampled at both cracked and uncracked regions. As shown in Figure 7.7 for structure #5, the chloride content was markedly higher in cracked locations. The same pattern holds true for structure #6. This again indicates that chlorides can penetrate the sealed cracks faster than uncracked concrete.

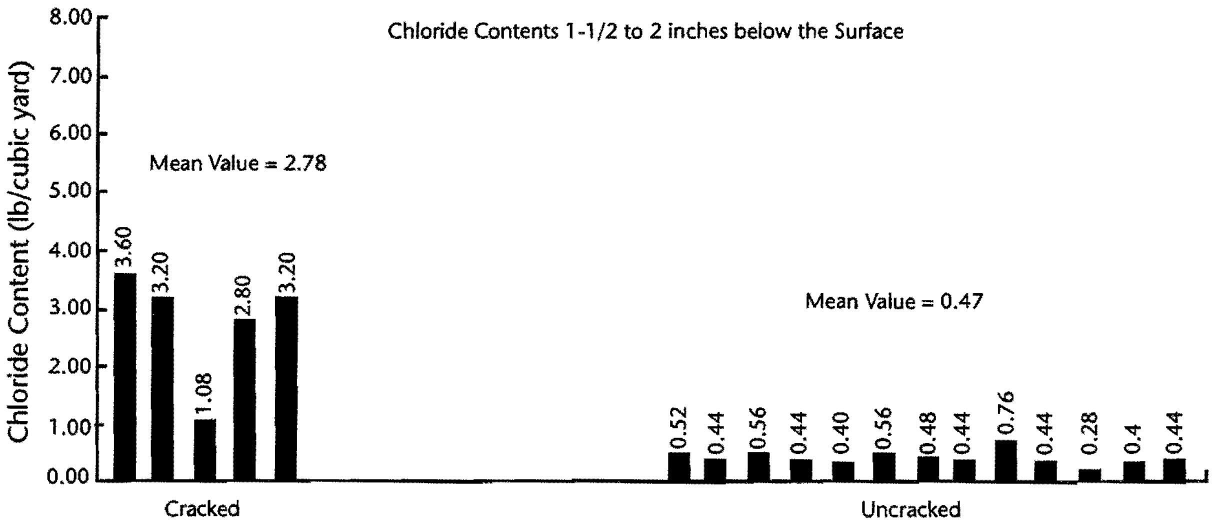


Figure 7.7 Effect of cracking on chloride penetration for structure #6

7.4.3 Wear Exposure

Similarly to the role of cracking, surface wear's effect on surface sealers is not completely understood. At structures #4, #5, and #8, both traffic and non-traffic locations were sampled. There was no apparent effect of wear exposure on chloride contamination of the deck. Figure 7.8 shows the chloride contents in the 1-1/2 to 2-inch sampling depth range for structure #8. Structures #4 and #5 exhibit similar behavior.

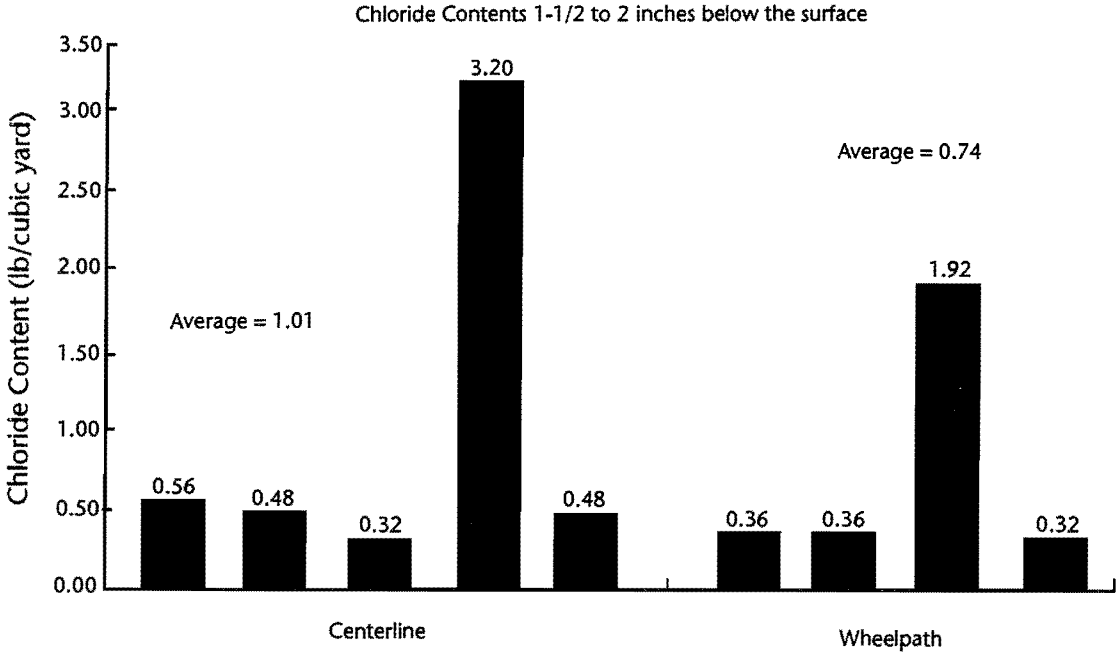


Figure 7.8 Effect of wheelpath wear on chloride penetration for structure #8

CHAPTER 8. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

8.1 SUMMARY

Despite over 25 years of experience with the problem of bridge reinforcement corrosion due to chlorides from deicers or sea water exposure, the solution to the problem is still unknown. Many different corrosion protection systems have shown promise in laboratory and preliminary field studies, but their performance on the basis of long-term field tests either has not been evaluated or has been questionable. The development of a testing program to increase the amount of information learned from test installations of various corrosion protection systems in Texas was the scope of the experimental program reported herein.

Preliminary questionnaire surveys distributed to all the TxDOT districts revealed that many different types of corrosion protection systems have been installed in Texas, but that little information on their performance has been collected from the structures since their completion. In this study, a field testing program was developed and used to investigate eight preexisting bridges representing different protection systems and various overall exposure and service conditions.

The test program, which included determination of half-cell potentials, concrete permeability, chloride content, cracking patterns, and delaminations, was found to accurately reflect the corrosion condition of bridges protected by a variety of corrosion protection measures. The tests selected for use in the field surveys were generally successful and accurate, with different tests confirming each other's results.

Some limited information about the performance of different corrosion protection systems in use in Texas was also learned. Dense concrete overlays were found to work well in both remedial and original installation applications, exhibiting very low permeabilities and low chloride contents even after ten winters with deicer exposure. The dense concrete overlays did, however, show a high incidence of cracking and associated chloride penetration at the cracks. Sealers were found to decrease concrete permeability, with water-carried silanes and linseed oil performing the best in terms of reducing permeability and chloride contents.

8.2 CONCLUSIONS

8.2.1 Introduction

The main objective of this study was to identify and evaluate current corrosion protection methods in use in Texas and nationwide, and to determine their effectiveness and applicability in Texas. To this end, an extensive literature search was conducted to identify corrosion protection systems and field test methods. In addition, a survey of TxDOT districts was conducted to identify corrosion protection systems used in Texas and to determine the districts' experiences concerning the different systems. In the part of the study reported herein, a field testing program was developed and conducted for evaluating and characterizing the bridge corrosion performance of eight bridges in Texas.

8.2.2 Findings

1. A large number of corrosion protection strategies have been developed for protecting highway bridge structures and have been used in Texas and nationwide, but relatively little field performance categorization and evaluation have been conducted.
2. Many field test procedures exist for general and corrosion-related performance evaluation, but little correlation or comparison among the methods has been performed.
3. The field testing program developed through this work accurately indicates the corrosion condition of bridges and provides performance information about the corrosion protection systems. In particular:
   - The results of the non-destructive half-cell potential test were confirmed by the chloride content, delamination, and

crack mapping results on the spans where the half-cell testing was used.
- The structures not characterized using the half-cell potential test were accurately represented through the chloride content, permeability, delamination, and crack tests performed.
- The half-cell test was an accurate indicator of corrosion, even in areas where visual inspections did not yet reveal any physical manifestations of damage.
- When modified to leave the treated surface of uncracked specimens intact, the AASHTO T277 test can be used to test and compare the effectiveness of different surface overlays or sealers applied to field structures.
- Chloride content testing can be used to directly compare the chloride penetration of different areas of a bridge utilizing different corrosion protection strategies.
- The test regime of half-cell potential testing, delamination determination, chloride content analysis, crack mapping, and permeability testing accurately reflected the corrosion condition of the bridges tested.

4. The field tests performed through the project have expanded the knowledge of the protection systems and can be used to augment the laboratory-determined characteristics of the protection systems.

8.3 RECOMMENDATIONS

1. The work performed to date has provided an outline of the information that can be gained through an extensive and comprehensive corrosion protection testing program. A field testing program should be implemented to allow TxDOT to better monitor both its test installations and its conventional structures. This will serve as an early indicator of the performance of the protection systems and as an indicator of corrosion damage to come. The information gained through a monitoring program would also assist TxDOT in financial planning and budgeting, as the information would allow more accurate predictions of ultimate structure life and necessary repair timetables. Specific details of the evaluation program for any bridge should be developed on an individual basis considering urgency, cost, usage, aesthetics, and expected service life.
2. The monitoring and evaluation program implemented should be patterned as follows:
   - *Half-cell corrosion potential tests should be performed at least every two years.*
     The half-cell test quickly, easily, and nondestructively determines the actual corrosion condition of the structure and will indicate upcoming corrosion problems before any physical manifestations of damage occurs. The use of the test would be facilitated by the addition of a built-in connection such as that shown in Figure 7.2 to all future bridges using uncoated reinforcement. Similar permanent connections should also be retrofitted to existing structures.
   - *Chloride content analysis should be performed when the half-cell potentials indicate the possibility of corrosion.*
     The chloride analysis should be performed once corrosion potentials indicate possible corrosion. The chloride contents supplement the half-cell tests by serving as an indicator of the possible extent and severity of the corrosion damage. Although the test requires drilling small holes in the concrete deck, the information gained is worth the small amount of damage to the deck.
   - *Delamination and crack mapping should be performed if the half-cell potentials and chloride content determination show the possibility for corrosion damage.*
     By performing the more time-intensive delamination and crack mapping as soon as the potential for corrosion damage is determined, a baseline for future comparison is created. This helps determine the extent of any repairs needed, as well as provides information about the deterioration process itself.
   - *Once started, the half-cell testing, crack mapping, and delamination testing should be repeated at least every two years to determine the progression of the deterioration and to increase the accuracy of the condition assessment of the structure.*
     The repeated testing would refine the estimates of structure condition. This increased knowledge of the structure's actual condition will increase the effectiveness of any repair or replacement planning, as well as providing a time history of the deterioration for further study.
   - *Permeability testing and petrographic analysis should be performed when quantitative characteristics of the protection system are needed.*
     Permeability testing and petrographic analysis do not yield any information

about the corrosion state of the structure, instead offering information about the protection system or the concrete itself. Permeability testing should not be used to provide estimates of the chloride content of the concrete, as the testing was found to not reveal the susceptibility of cracked deck concrete to chloride penetration. The permeability test was found to not be influenced by cracking, while the cracking was found to influence chloride penetration and corrosion activity. The test, however, is useful for comparing systems or as an acceptance criterion for some protection systems. As it measures permanent parameters such as air system characteristics, petrographic analysis does not lend itself to use in periodic monitoring. Petrographic analysis, however, should be used if the cause of structural deterioration is not known or if specific problems such as an inadequate air system or sulfate attack are suspected.

3. A laboratory testing program should be implemented to allow direct, accelerated, controlled comparisons of the various protection systems. The direct laboratory comparisons will subject all of the systems to the same exposure, as opposed to the different exposures of the structures visited to date. This will allow rapid quantitative comparisons of the systems in terms of their experimentally determined corrosion potentials, permeabilities, and chloride penetration.

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APPENDIX A

INDIVIDUAL BRIDGE TEST RESULTS

APPENDIX A1
IH-20 AT MORGAN CREEK

A1.1 BACKGROUND

The IH-20 bridge at Morgan Creek is located in Mitchell County, west of Colorado City. The structure was built in 1957 and was protected with an asphaltic overlay. In 1988, as part of a widening project, the overlay was removed due to delamination and spalling of the deck. The deck was milled and overlaid with a dense concrete overlay. The overlay cracked extensively, and a methacrylate crack sealer with seeded sand was applied over the entire bridge to seal the cracks. The 300-foot-long bridge carries two lanes of traffic over Morgan Creek. The 1990 average daily traffic was 4,650 vehicles, 31 percent of which was truck traffic. The structure is exposed to deicers during the winters and to water vapor from the river year round.

A1.2 TESTING

A1.2.1 Half-Cell Potential Testing

The half-cell accuracy confirmation tests were generally within the 0.02 V CSE allowed by ASTM 876, although some locations containing epoxy-coated reinforcement did not meet the requirement. The locations exceeding the allowable 0.02 V CSE variability were generally in the areas of the bridge added during the widening. Epoxy-coated bars were discovered in the widened portions of the structure during the investigation. For this reason, readings taken in the breakdown-lane region of the bridge should be viewed with some skepticism, as the condition of epoxy-coated reinforcement cannot be characterized with the half-cell potential test as described in Chapter 4, Section 4.2.1. The half-cell stability confirmation tests showed stability within a range of 0.005 and 0.013 V CSE, well within the allowable 0.02 V CSE.

Overall, 15 percent of the deck can be classified as "90 percent probability of corrosion activity," 40 percent of the deck can be classified as "90 percent probability of no corrosion activity," and the remaining 45 percent falls into the uncertain category as defined in ASTM 876. Complete results are attached. There was no apparent correlation between the half-cell potentials and any other factors such as cracking or methacrylate appearance.

A1.2.2 Chloride Content Testing

The chloride content testing showed some variability through the 2-inch depths. This may be due to unrefined sampling techniques in the early part of the testing program as the variability decreased in bridges tested later. Chloride content testing could not be performed on the substructure due to high water.

As can be seen from the data, the chloride contents were quite variable over the bridge, often showing higher chloride levels beneath the surface than at the surface. The reason for these suspect readings is not known at this time, although "pumping" and washing from wet-dry cycling are suspect. The relative inexperience with sampling during the testing of this bridge may also have been a factor. As Figure A1.1 shows, there was no apparent performance relationship between areas of good and poor methacrylate and the different regions of the deck.

A1.2.3 Permeability Testing

Cores take from areas of visually "good" and "poor" methacrylate condition were tested with the methacrylate surfaces intact and removed. Samples of the underlying base concrete of the base concrete were also tested to determine the effectiveness of the overlay concrete as compared to the base concrete. These base samples were taken from both the original portion of the bridge and the sections added during the widening.

Tests were performed on 15 samples including: 3 samples each of specimens with "good" and "poor" methacrylate; 3 specimens of the overlay concrete with the methacrylate removed by sawing off the top of the core; and 3 samples each of the original and widening base concretes. The tests indicated lower permeabilities for the overlay concretes than for the base concretes, but showed only minor differences between the "good" and "poor" methacrylate. The methacrylate coated cores had lower permeabilities than the overlay cores with the methacrylate removed, but only marginally so with a 400-coulomb difference between the averages. The base concretes had noticeably higher permeabilities than the overlay cores.

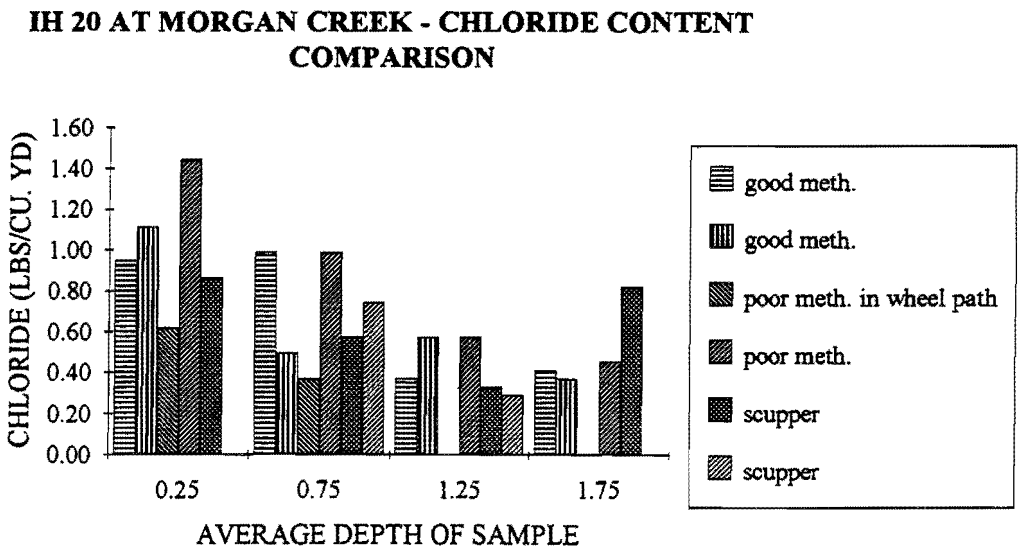


Figure A1.1 Chloride contents of "Good" and "Poor" Methacrylate regions

A1.2.4 Crack Mapping

Thorough crack mapping surveys were only completed for test grids 3 and 4 because of time constraints. Brief sketches of the crack condition of test grids 1 and 2 were made.

Little correlation between cracking patterns and any other corrosion characteristics was found through the survey; however, the crack mapping was not as thorough as that conducted on later bridges.

A1.2.5 Strength Testing

No compressive strength testing was performed as all cores retrieved were composites of overlay and base concrete. The strength testing of the composite cores would provide little reliable information about either of the two concretes.

A1.2.6 Delamination Testing

Delamination tests were performed on the bridge but were hampered by the large amount of traffic noise. The traffic noise prevented the investigators from hearing the chains clearly.

As only the eastern end of the eastbound travel lane was sounded with the chain drag apparatus, limited information can be drawn from the testing. No apparent correlation was found between the delamination information and any other data in the region tested.

A1.2.7 Petrography

Petrographic analysis was conducted on one core to determine the characteristics of the air void system of the overlay concrete.

The analysis revealed that the overlay had the following air system characteristics (recommended values from the American Concrete Institute Manual of Concrete Practice (216) are included in parenthesis after the results):

• Air Content	4.7%	(>5.0% ± 1.5%)
• Voids per Inch	8.6	(>7.5)
• Specific Surface	734 in^2/in^3	(>600 in^2/in^3)
• Spacing Factor	0.007	(<0.008)

All of these values meet currently accepted standards.

A1.3 CONCLUSIONS

The bridge was found to be in the early stages of corrosion, with corrosion starting to occur but without external manifestations of damage. The test regime was found to work well and a number of refinements were made in the test program. This structure represents a clear example of the case in which performance testing would provide an early warning of the occurrence of corrosion long before external evidence of damage is apparent.

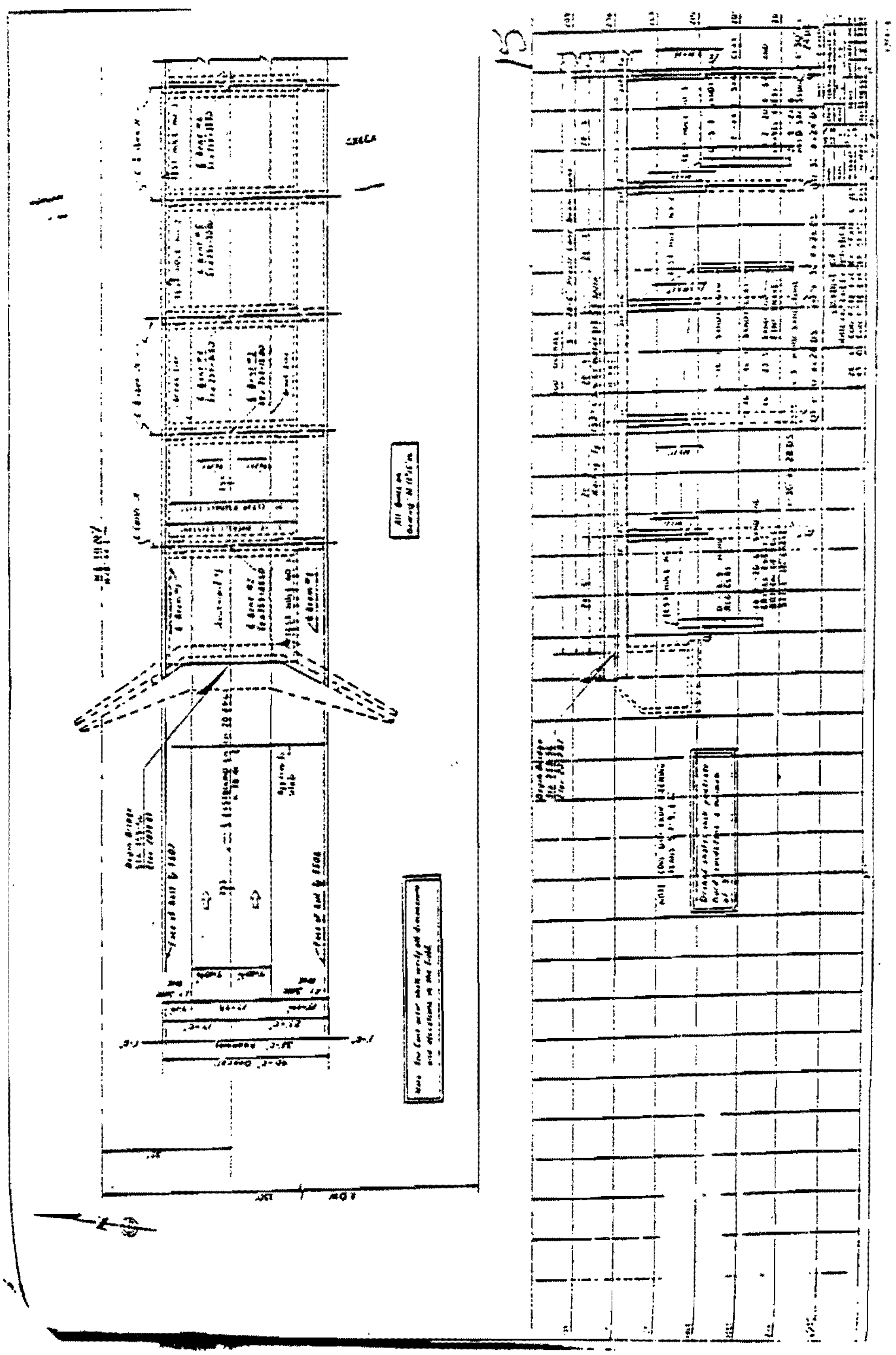


Figure A1.2

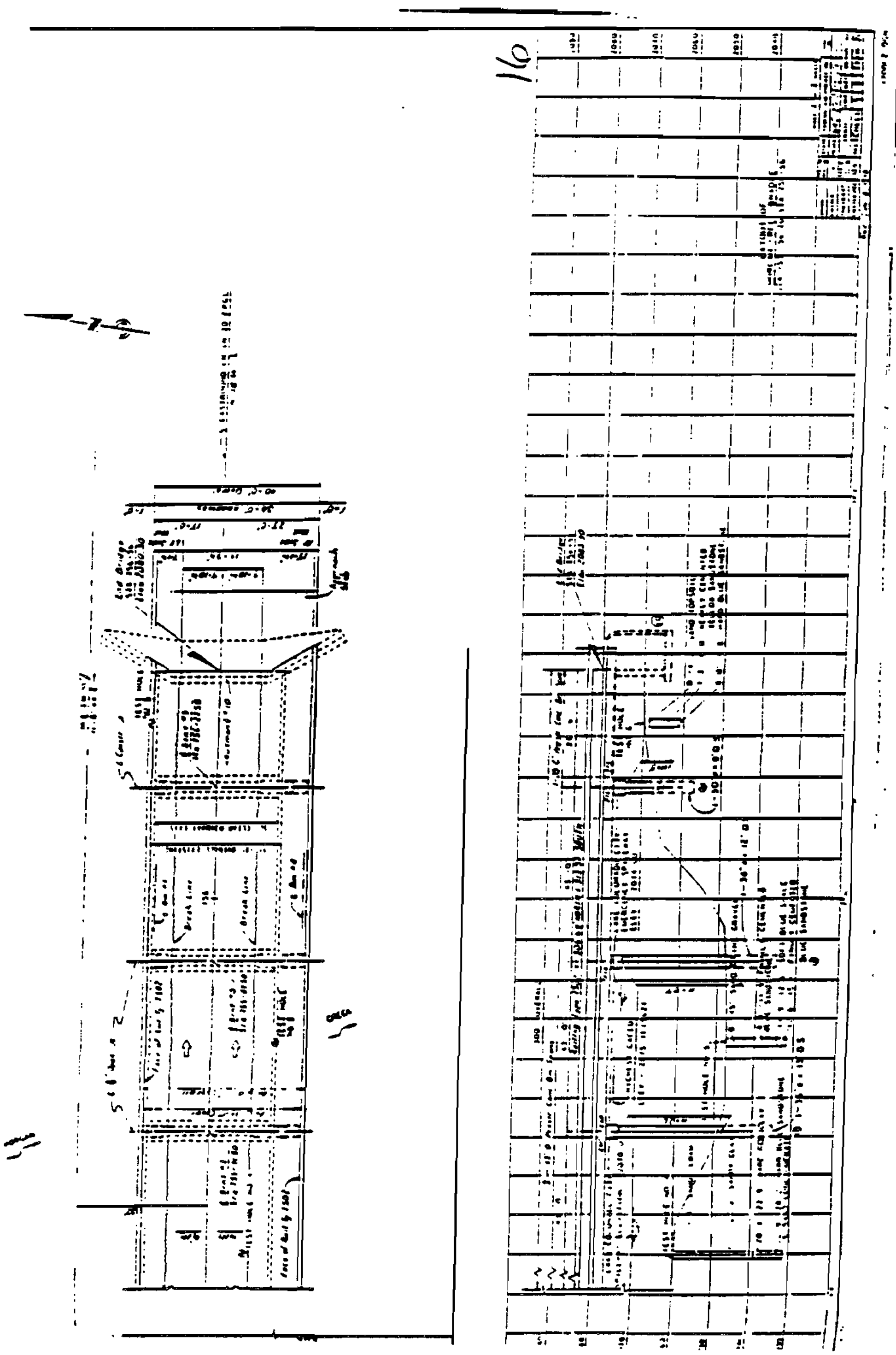


Figure A1.3

IH-20 AT MORGAN CREEK - SAMPLE LOCATIONS

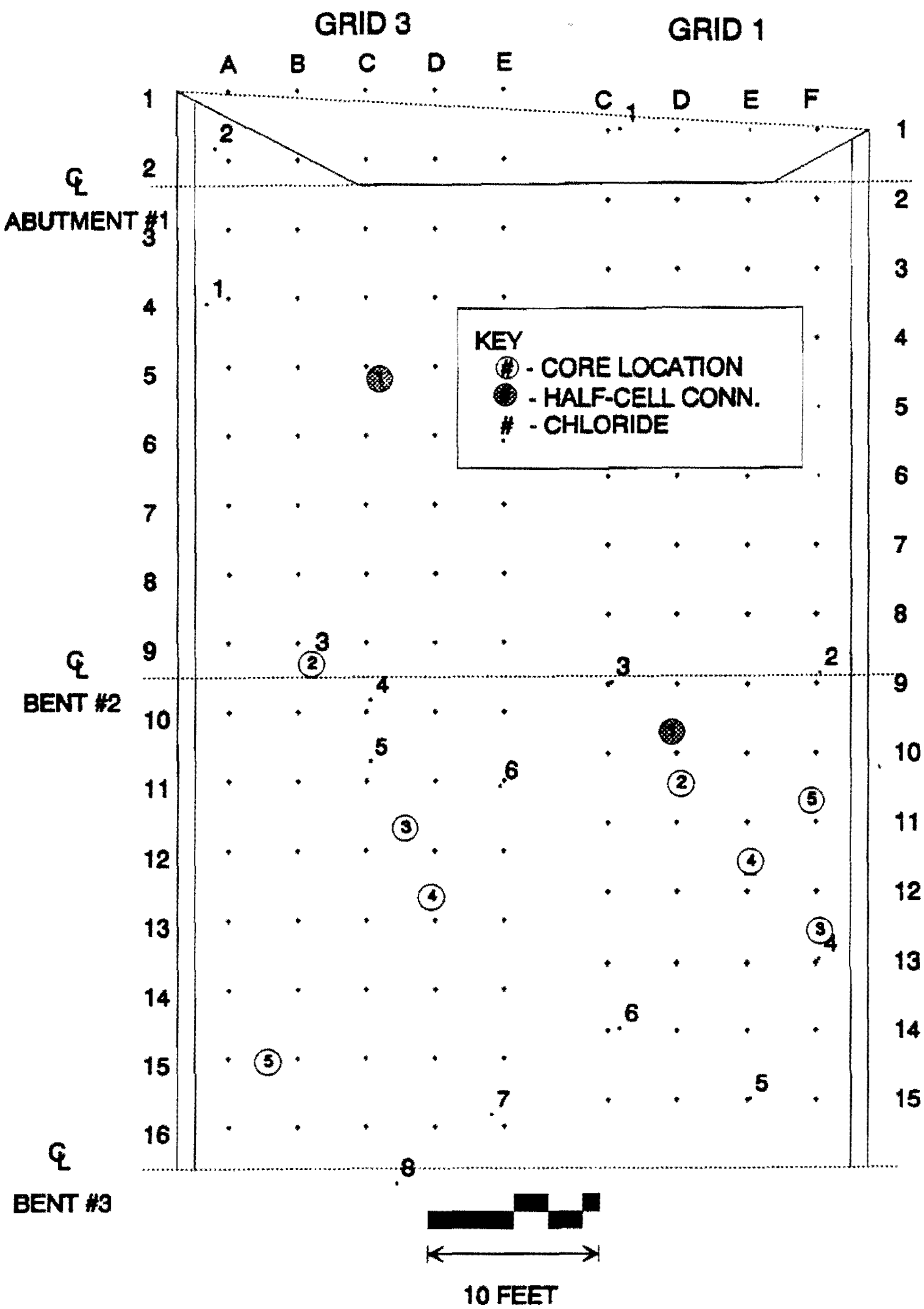


Figure A1.4

IH-20 at MORGAN CREEK CRACK MAPPING

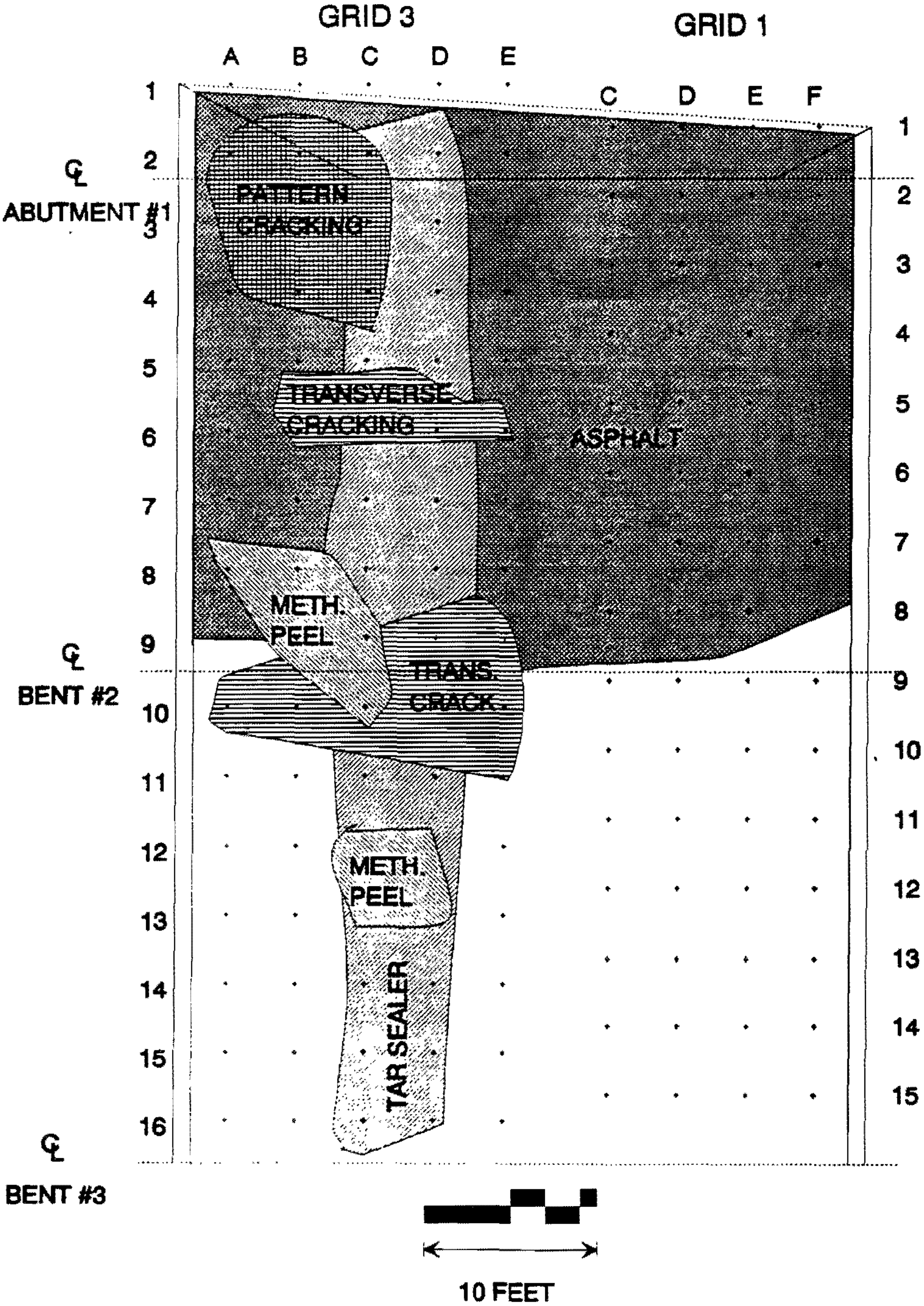


Figure A1.5

IH-20 AT MORGAN CREEK - SAMPLE LOCATIONS

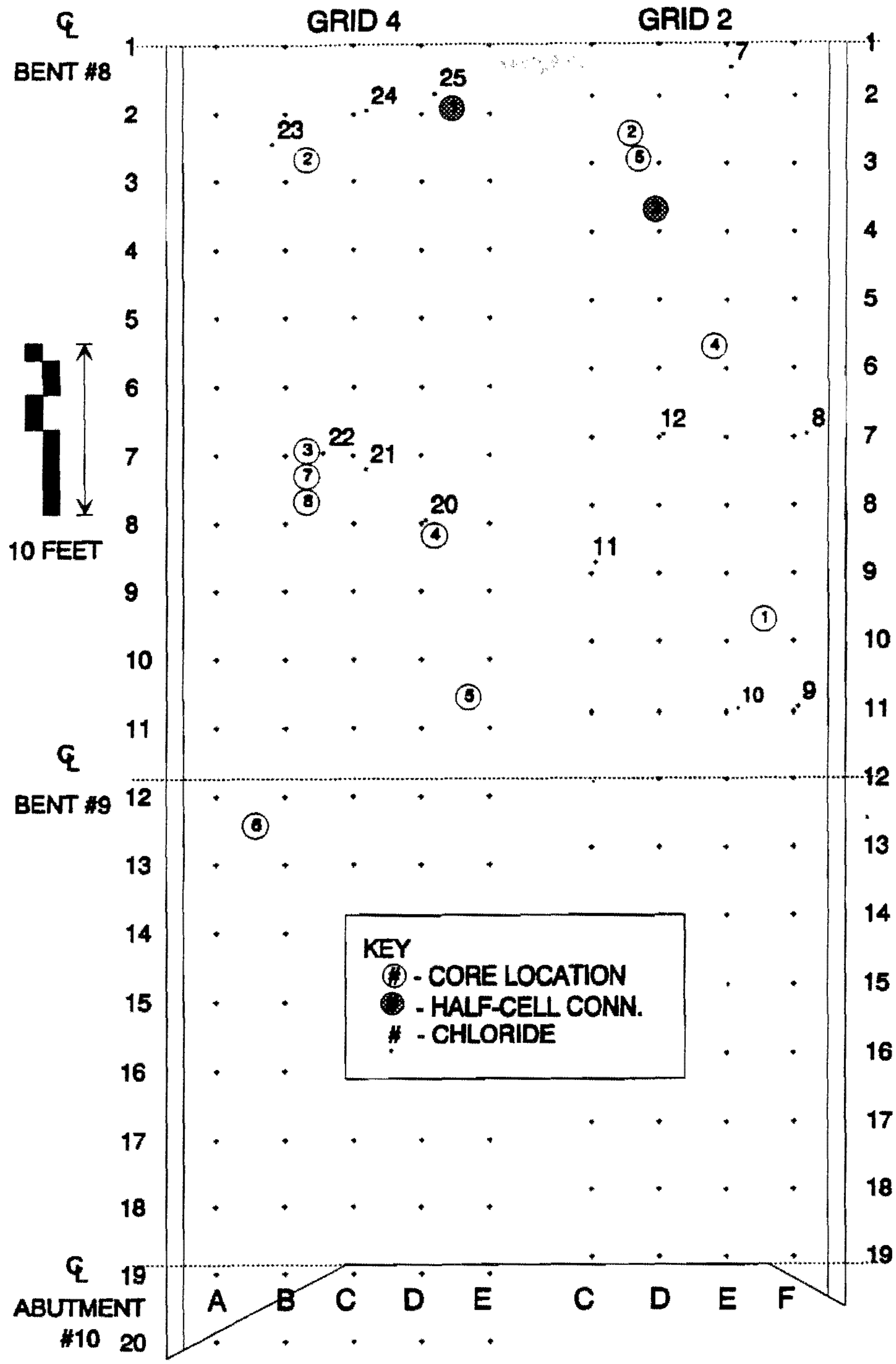


Figure A1.6

IH-20 at MORGAN CREEK CRACK MAPPING

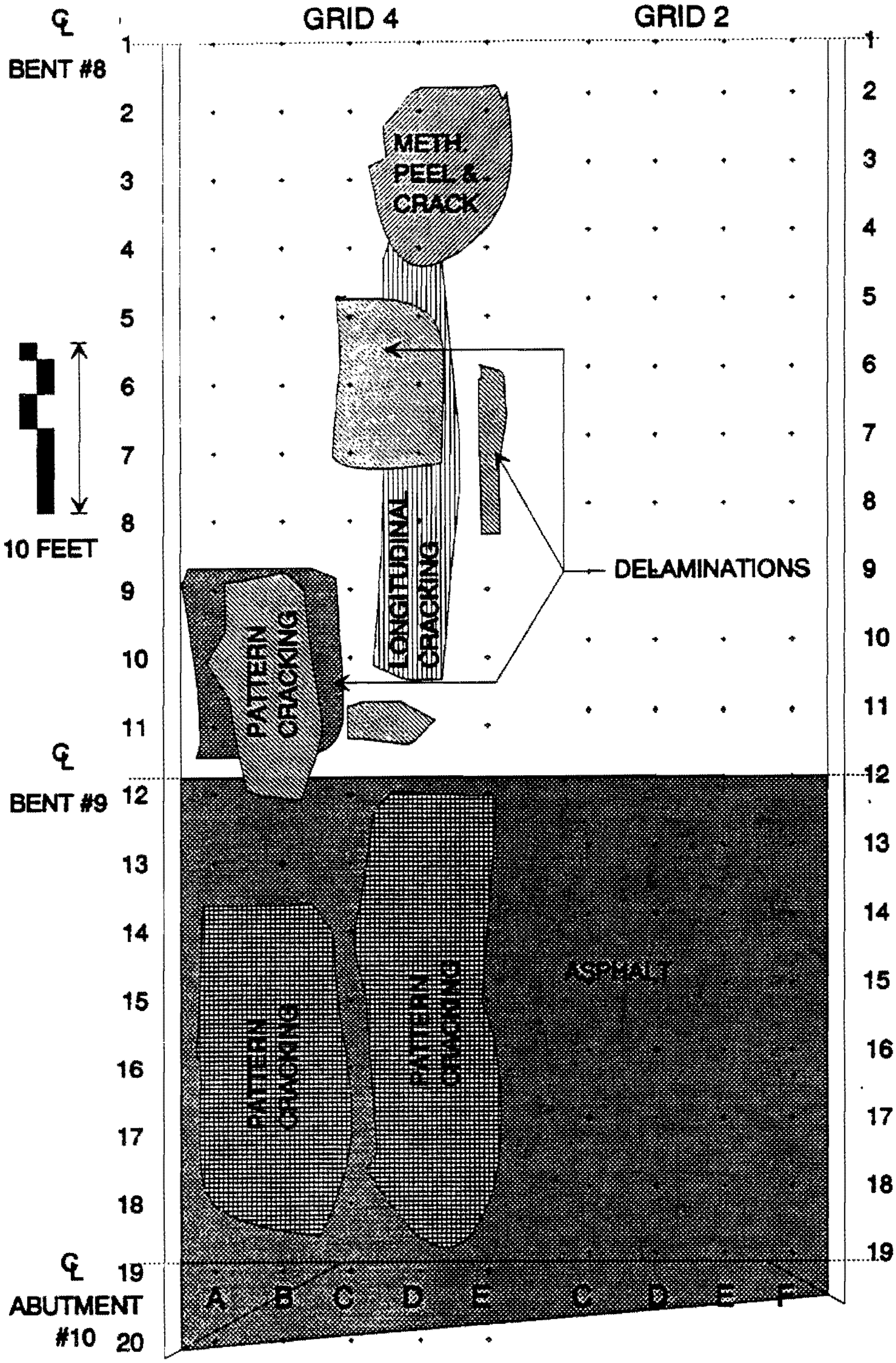


Figure A1.7

IH-20 AT MORGAN CREEK - HALF CELL POTENTIAL MAP

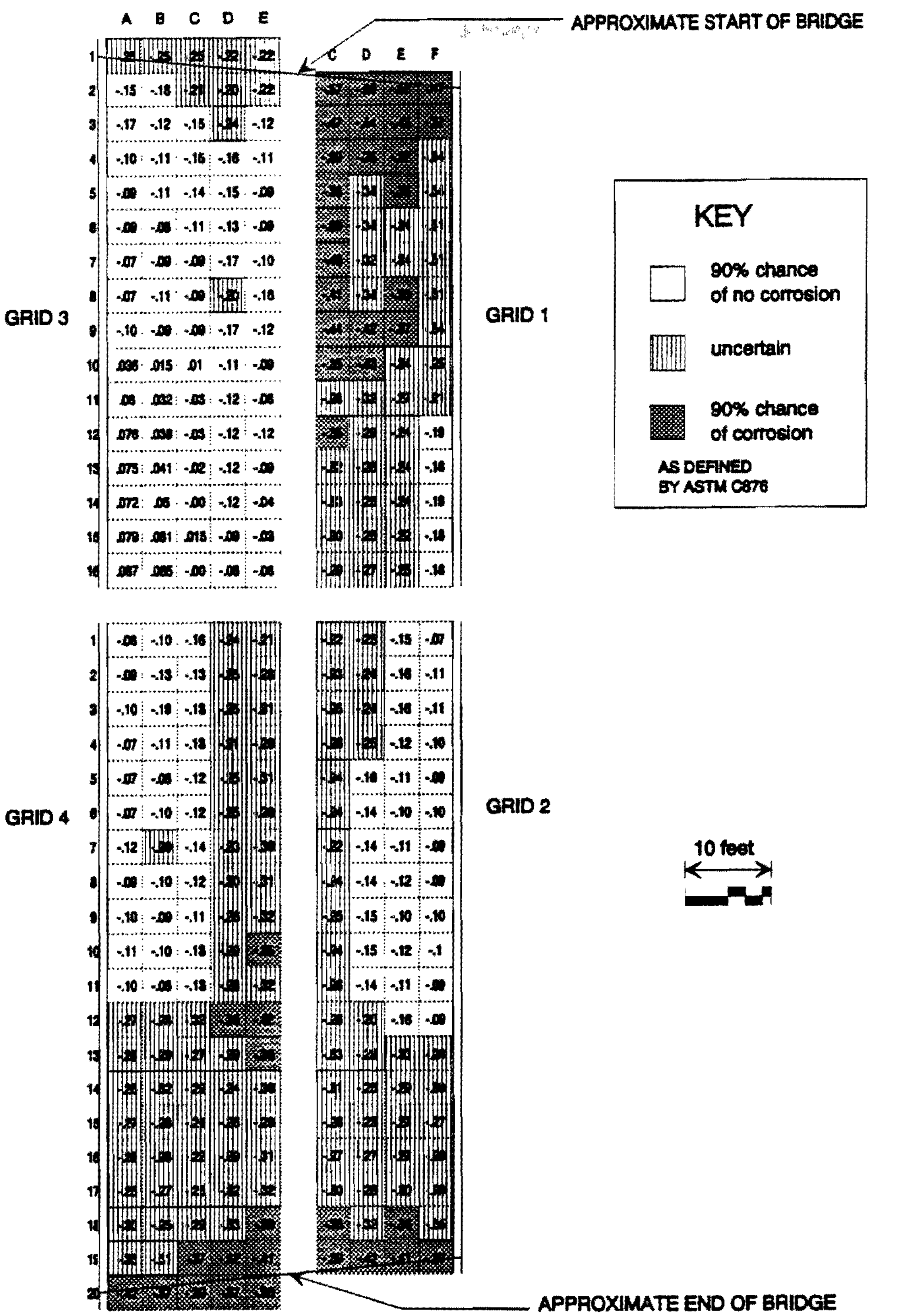


Figure A1.8

Table A1.1

IH 20 at Morgan Creek
Stabilization Runs

	STABILIZATION READING, VOLTS CSE					
time (min)	Grid 1 Point E2	Grid 2 Point E2	Grid 3 Point B2	Grid 3 Point D3	Grid 4 Point B2	Grid 4 Point C3
0.0	-0.389	-0.176	-0.19	-0.239	-0.137	-0.137
0.5	-0.388	-0.177	-0.188	-0.241	-0.14	-0.137
1.0	-0.387	-0.177	-0.186	-0.243	-0.143	-0.135
1.5	-0.386	-0.176	-0.185	-0.244	-0.143	-0.138
2.0	-0.386	-0.177	-0.184	-0.244	-0.144	-0.138
2.5	-0.385	-0.179	-0.182	-0.244	-0.145	-0.139
3.0	-0.386	-0.177	-0.181	-0.244	-0.145	-0.139
3.5	-0.383	-0.176	-0.179	-0.244	-0.145	-0.141
4.0	-0.384	-0.177	-0.177	-0.244	-0.145	-0.145
4.5	-0.384	-0.175	-0.179	-0.244	-0.146	-0.145
5.0	-0.385	-0.174	N/A	-0.244	N/A	-0.145
Minimum	-0.383	-0.174	-0.177	-0.239	-0.137	-0.135
Maximum	-0.389	-0.179	-0.190	-0.244	-0.146	-0.145
Range	0.006	0.005	0.013	0.005	0.009	0.010

Table A1.2

IH 20 at Morgan Creek
Methacrylate

Concrete Permeabilities (Coulombs)

Concrete Type	Surface Condition	Sample Number 1	2	3	Average
Overlay	"Good" Methacrylate	975	639	744	786
	"Poor" Methacrylate	799	1046	1333	1059
	Methacrylate Removed	1643	1541	1123	1436
Base	Original Base Concrete	6092	4235	1354	3894
	New Base Concrete	6073	4604	6189	5622

Table A1.3

IH 20 AT MORGAN CREEK
CHLORIDE CONTENTS (LB/CU.YD)

Sample number	SAMPLE DEPTH (IN.) 0 - 0.5	0.5 - 1.0	1.0 - 1.5	1.5 - 2.0	REMARKS
1-1-EBP-0C-S1	0.00	0.91	0.74	0.37	
1-1-EBP-F8-S2	0.45	0.21	0.33	0.29	
1-1-EBP-8C-S3	1.65	1.65	0.37	0.41	good methacrylate
1-1-EBP-13F-S4	0.70	0.25	0.00	0.70	
1-1-EBP-15E-S5	0.00	0.33	0.25	0.25	medium methacrylate
1-1-EBP-C14-S6	0.78	0.33	0.25	0.25	
1-2-EBP-1E-S7	1.36	0.62	0.58	0.37	
1-2-EBP-7F-S8	3.30	1.11	0.99	0.58	good methacrylate
1-2-EBP-11F-S9	1.44	0.99	0.58	0.45	poor methacrylate
1-2-EBP-11E-S10	2.06	1.24	0.66	0.49	
1-2-EBP-9C-S11	0.95	0.99	0.37	0.41	good methacrylate
1-2-EBP-7D-S12	1.11	0.49	0.58	0.37	good methacrylate
1-3-EBT-4A-S1	0.87	0.58	0.33	0.82	at scupper
1-3-EBT-2A-S2	0.00	0.74	0.29	0.00	at scupper
1-3-EBT-8-9BC-S3	0.00	1.32	0.33	0.37	no asphalt
1-3-EBT-9C-S4	0.58	0.78	0.33	0.33	poor asphalt, near wheelpath
1-3-EBT-9-10C-S5	0.00	0.49	0.25	1.36	poor asphalt, near wheelpath
1-3-EBT-11E-S6	0.78	0.45	0.21	0.25	medium methacrylate in wheel path
1-3-EBT-15E-S7	0.62	0.37	0.00	0.00	poor methacrylate in wheel path
1-3-EBT-15CD-S8	0.99	0.37	0.29	0.25	medium methacrylate in wheel path
1-4-EBT-8D-S20	0.58	0.33	0.25	0.33	
1-4-EBT-7C-S21	0.33	0.00	0.37	0.25	good methacrylate in wheel path
1-4-EBT-7C-S22	2.47	1.32	1.11	0.66	in wheelpath
1-4-EBT-2.5A-B-S2	0.62	0.25	0.00	0.00	heavy methacrylate
1-4-EBT-2C-S24	0.70	0.21	0.00	0.37	in wheelpath
1-4-EBT-2D-S25	0.78	0.21	0.21	0.00	

APPENDIX A2
REDFISH BAY BRIDGE, CORPUS CHRISTI TEXAS

A2.1 BACKGROUND

The 2002-foot SH 361 bridge over Redfish Bay between Port Aransas and Aransas Pass was built in 1959. The structure carries an ADT of 4500 through its two lanes. The bridge is composed of pan-form girders with an integral deck spanning between concrete bents on concrete piles. The structure's substructure is exposed to sea water and the superstructure is exposed to salt laden sea spray and water vapor. The substructure of the bridge is showing corrosion and spalling. Many repairs such as the jacketing of piles and girders and resurfacing of the pan form girders have been performed to date. The deck of the structure shows little deterioration and has needed no unusual maintenance or repairs.

A2.2 TESTING

A2.2.1 Half-Cell Potential Testing

As the deck had not deteriorated and as the investigation was concentrating on the substructure deterioration, half-cell testing was not performed at this bridge. This decision was also based on the fact that the bridge was composed of numerous short independent spans. The testing would have required extensive coring and confirmation testing of each span, requiring too much time to complete during the availiable testing window.

A2.2.2 Chloride Content Testing

Forty-nine locations were tested for chloride content. Samples were taken from the exterior rail sections, the pan form girders, the bent caps, and the concrete piles. The samples were taken from either the outside surface of the bridge using a bridge inspection "snooper" or from the water using a small boat. At a depth of 1.5 to 2.0 inches, the chloride content was generally below 1.5 lb/yd^3, with a few exceptions. The 1.5- to 2.0-inch samples from the piles and the wingwall showed high chloride contents ranging from 4.0 to 20.4 lb/yd^3. These are quite high and are near the upper end of the accuracy range of the testing equipment used.

The chloride contents are often higher below the surface than at the surface of the concrete. This seems to indicate that the cyclic exposure of alternate wetting and drying periods is driving the chlorides into the concrete. This would happen when chlorides dissoved in the sea water enter the concrete during wetting and are left behind when the water evaporates. The process would continue with the deposited chloride redissolving and migrating further into the concrete with subsequent wetting and drying cycles.

The chloride contents in the piles and the wingwall were all well above the corrosion threshold level, as were many locations in the bent caps and the girders. This agrees well with the observed distress in the substructure. Apparently, the build up of chloride in the concrete due to the cyclic wetting and drying has accelerated the corrosion and deterioration processes. Also indicating the severity of this type of exposure are the relatively constant chloride contents through the different sampling depths. This structure does not show the marked decrease in chloride content with depth that is typically shown in the other structures investigated.

A2.2.3 Permeability Testing

No cores could be taken from the portions of the bridge under investigations, so no permeability testing could be performed.

A2.2.4 Crack Mapping

No crack mapping was performed because of time and accessibility constraints on the substructure elements under investigation.

A2.2.5 Strength Testing

No strength testing was performed as no cores were removed from the structure.

A2.2.6 Delamination Testing

As the deck was not under investigation, no delamination surveys were made during the testing.

A2.2.7 *Petrographic analysis*

No petrographic analysis of the concrete was performed as no cores were removed from the structure. Samples from the structure, however, were examined with a scanning electron microscope (SEM) for evidence of possible sulfate attack. Analysis revealed that sulfate attack was not a cause of deterioration in this structure.

A2.3 CONCLUSIONS

This bridge is undergoing severe deterioration of the substructure. The deterioration is such that the district is already planning its replacement. Monitoring of the structure should continue. The sea exposure is apparently an extreme exposure, with chlorides being forced deep into the concrete.

Redfish Bay Causeway
Corpus Christi

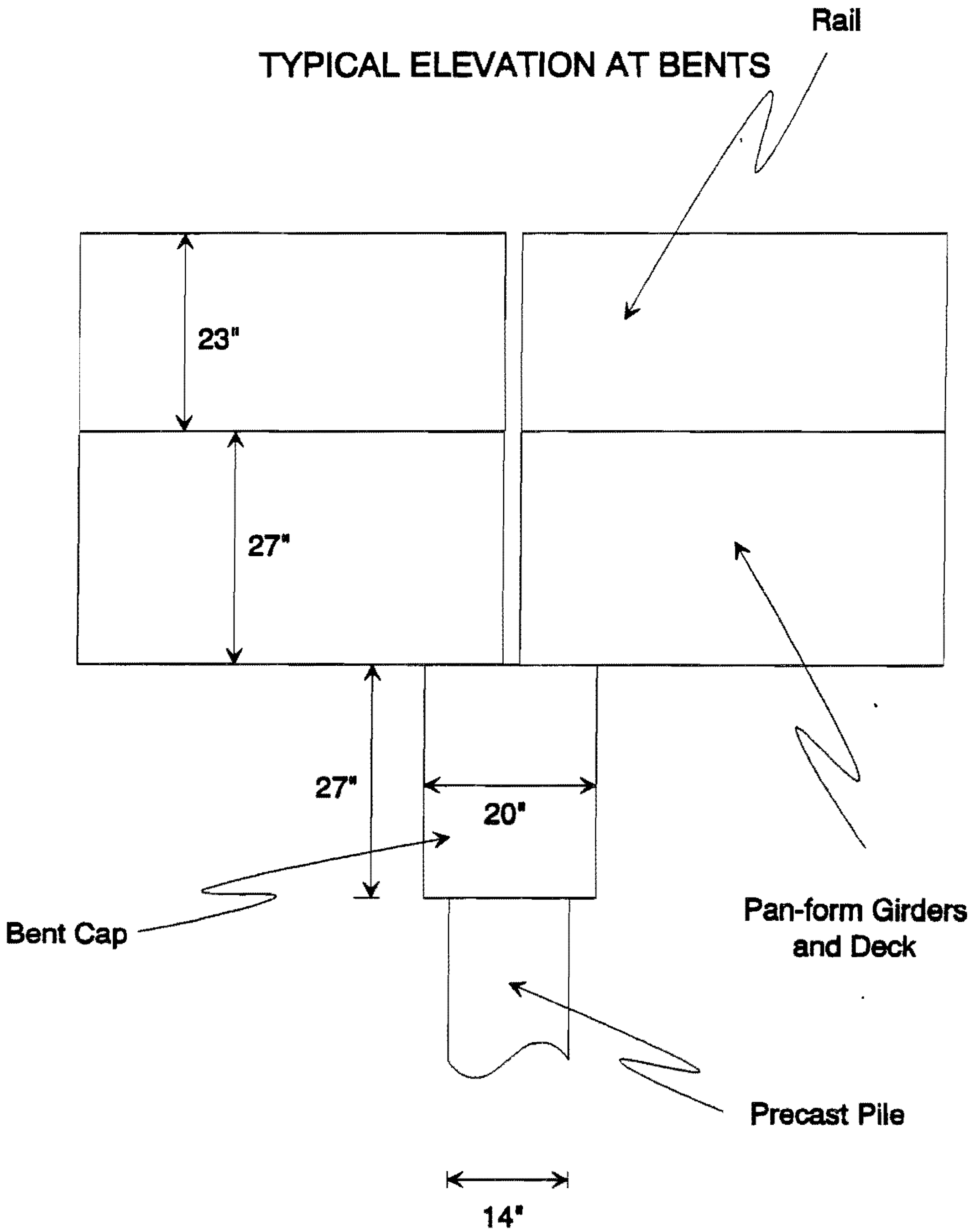
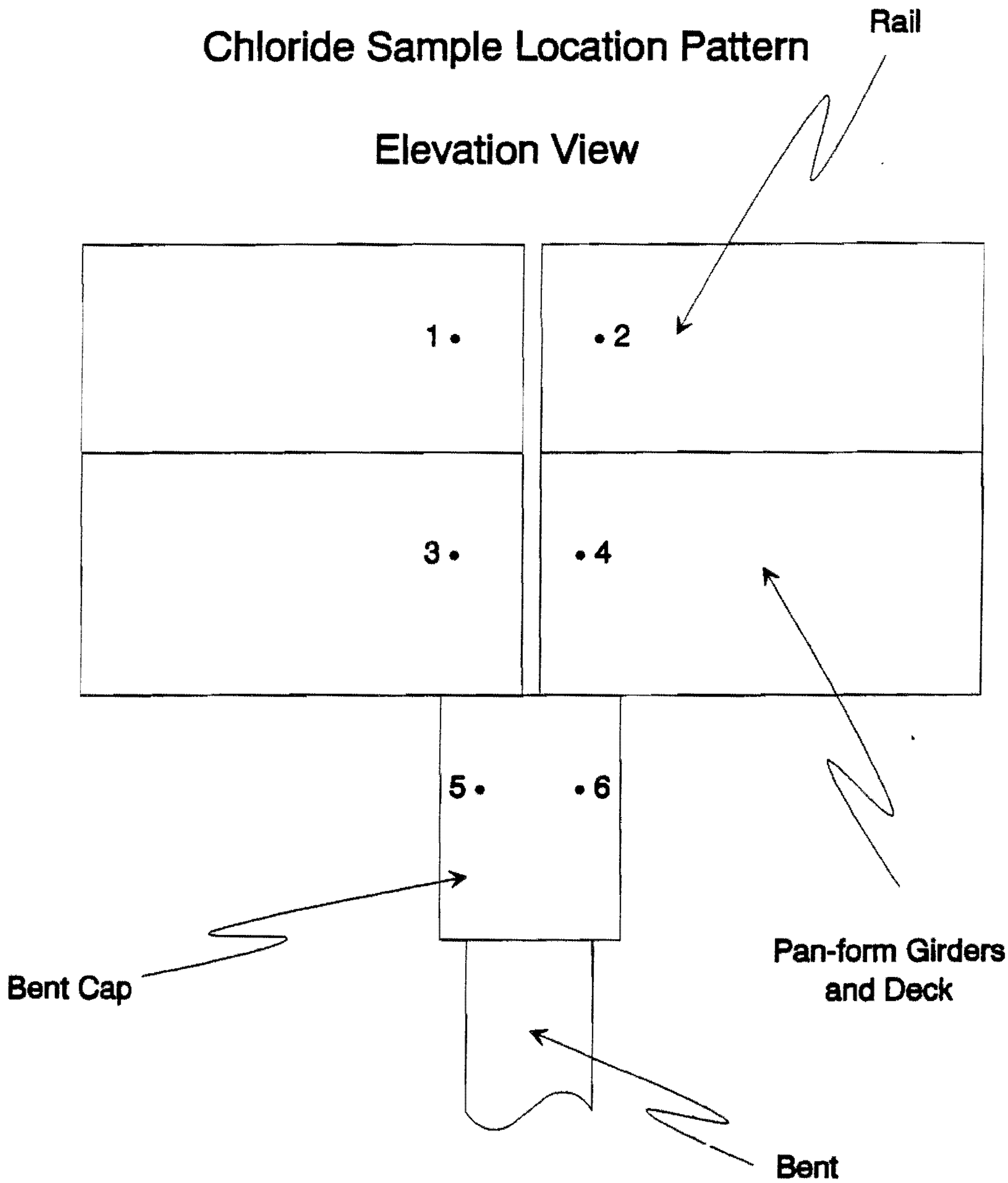


Figure A2.1

Redfish Bay Causeway
Corpus Christi

Chloride Sample Location Pattern

Elevation View



Note: All sampling areas were indexed using this six number sequence

Figure A2.2

Table A2.1

REDFISH BAY BRIDGE - CORPUS CHRISTI
CHLORIDE CONTENTS (LB/CU. YD)

SAMPLE		SAMPLE DEPTH (IN)				
Location	Number	0.0 - 0.5	0.5 - 1.0	1.0 - 1.5	1.5 - 2.0	Remarks
Rail Samples						
Bent 2	1	0.84	2.00	1.92	0.96	
	2	0.88	1.40	0.76	0.56	
Bent 3	7	0.96	1.04	0.64	0.56	
	8	0.60	0.56	0.56	0.48	
Bent 4	13	1.40	1.40	0.80	1.20	
	14	0.64	0.68	0.88	0.72	
Bent 18	19	1.04	1.88	1.72	1.20	
	20	0.68	0.68	1.04	0.76	
Bent 19	25	1.44	1.20	2.40	1.32	
	26	1.88	1.40	1.20	0.84	
Bent 20	31	0.56	1.08	1.40	1.28	
	32	0.60	1.16	0.96	0.64	
Girder Samples						
Bent 2	3	0.36	0.68	N/A	2.00	
	4	0.76	0.84	1.04	0.56	
Bent 3	9	0.44	0.56	1.16	0.64	
	10	0.64	1.32	0.92	1.20	
Bent 4	15	0.64	0.44	0.92	0.92	
	16	1.20	1.72	0.88	0.64	
Bent 18	21	1.60	1.08	1.44	1.16	painted black
	22	3.60	1.76	0.96	0.68	
Bent 19	27	2.40	1.44	1.08	0.96	
	28	0.32	0.52	1.92	1.40	painted grey
Bent 20	33	1.40	2.00	1.88	0.92	labeled "Test 2"
	34	0.36	0.20	1.20	1.40	
Bent Cap						
Bent 2	5	1.44	3.20	3.60	3.60	
	6	0.92	1.20	1.60	0.96	
Bent 3	11	0.60	1.72	2.80	1.92	
	12	1.32	2.80	3.20	2.40	
Bent 4	17	2.40	2.40	1.92	0.72	
	18	2.80	2.80	3.60	2.00	
Bent 18	23	0.68	1.88	1.20	0.56	
	24	0.36	1.72	1.16	0.72	
Bent 19	29	0.96	1.88	2.00	0.36	
	30	0.24	2.00	1.60	1.28	
Bent 20	35	0.88	6.80	7.20	3.60	
	36	1.44	2.80	3.20	1.32	

REDFISH BAY BRIDGE - CORPUS CHRISTI
CHLORIDE CONTENTS (LB/CU. YD)

SAMPLE		SAMPLE DEPTH (IN)				
Location	Number	0.0 - 0.5	0.5 - 1.0	1.0 - 1.5	1.5 - 2.0	Remarks
Piers						
Bent 3	37	9.60	9.20	10.40	6.80	in splash zone
	38	10.40	13.20	6.40	4.00	in splash zone
Bent 4	39	11.60	18.80	12.00	9.20	in splash zone
	40	9.20	11.60	8.80	8.40	in splash zone
Bent 19	41	16.00	17.60	12.80	8.00	in splash zone
	42	18.80	16.00	12.80	8.80	in splash zone
Abutment 1						
	44	20.40	14.00	17.20	14.40	waterline 1
	45	16.00	20.40	14.40	20.40	waterline 2
	46	18.80	20.40	N/A	N/A	waterline 3
	47	6.80	17.60	20.40	17.60	midway 1
	48	3.20	9.60	18.80	12.00	midway 2
	43	3.60	8.00	12.80	8.80	high 1
	49	7.20	5.20	16.00	20.40	high 2

APPENDIX A3
FM 610 AT NICHOLSON CREEK, ABILENE DISTRICT

A3.1 BACKGROUND

This bridge, which carries the two lanes of FM 610 over Nicholson Creek in Stonewall county, was built in 1950. The bridge originally had a two-inch asphalt overlay. Due to deterioration, the overlay was removed in November 1991 and a high-molecular-weight methacrylate sealer with seeded aggregate was applied to seal the deck.

The structure is exposed to deicing chemicals as well as to sulfates and gypsum carried by Nicholson Creek below. The average daily traffic is 550 vehicles, 12 percent of which are trucks. The truck traffic is composed mainly of heavily loaded trailer trucks carrying raw gypsum from a quarry to a processing plant.

The bridge suffered from extreme deterioration, with large delaminations and a full-depth local failure. It was demolished and replaced with a new three span structure in November 1992. During the demolition, the investigators were able to retrieve samples of the steel and concrete to confirm the corrosion condition of the structure.

A3.2 TESTING RESULTS

A3.2.1 Half-Cell Potential Testing

The half-cell test was performed on all three spans tested. The repeatability confirmation tests were generally within the acceptable 0.002 V CSE tolerance range. The stability confirmation readings, however, had variabilities of 0.045 and 0.028 V CSE, outside of the accepted range. In accordance with the steps outlined in the test standard, the deck was repeatedly soaked to saturate the underlying concrete, but the soakings had no effect. The problem may have been related to the generally thick layer of methacrylate, which may have prevented uniform saturation of the deck, allowing some areas to soak while keeping others dry. The problems may have also resulted from the advanced corrosion condition of the bridge. Areas of the bridge were found to be extensively corroded during an inspection during the demolition of the bridge, and this extreme corrosion may have influenced the results.

Despite the variability problem, the half-cell testing revealed that approximately 98 percent of the deck was exhibiting possible corrosion. Of that 98 percent, over 50 percent was categorized as having probable corrosion.

The areas of probable corrosion correlated well with the areas experiencing delaminations for all three spans. The areas of heavy cracking on span 1 also correlated well to the areas of high corrosion potential. The cracking on spans 2 and 3 did not correlate as well to the half-cell, as there were large areas of indicated corrosion with no widespread cracking. This, however, is to be expected, and does not reflect poorly on the half-cell testing as it is possible to have corrosion without cracking. The important relationship puts cracking in the causality condition. In other words, the testing shows that there is corrosion at all the cracked areas but there is not necessarily cracking at all corroding areas. This is to be expected due to the early detection characteristics of the testing procedure.

A3.2.2 Chloride Content Testing

Thirty-one chloride samples were taken from the curb, rail, deck, piers, and bent caps of the structure. Areas of visually good and poor methacrylate surface condition were sampled to determine the effectiveness of the coating.

The chlorides are somewhat variable, especially in the bents. The levels are generally high, with approximately 68 percent of the samples taken from 1-1/2 to 2 inch range at over 1 lb/yd^3. This matches the large amount of corrosion and deterioration indicated by the other tests. The data also shows there is no apparent correlation between visually determined "good" or "poor" condition of the methacrylate and chloride penetration. Apparently, visual inspection is not effective in characterizing the performance of methacrylate surface treatments. This is probably because of the methacrylate's ability to penetrate into the surface of the concrete.

A3.2.3 Permeability Testing

Nine samples were tested to determine the chloride permeability of the concrete. Unfortunately, a problem with the test data acquisition equipment prevented any determination of the permeabilities of six of the specimens. Six replacement specimens were retrieved during the demolition of the structure, but the original locations of the replacement samples in the bridge could not be determined as the samples were taken from large pieces of debris left after the demolition of the deck. Samples were tested both with the methacrylate surface intact and removed to determine the effectiveness of the material.

A3.2.4 Crack Mapping

Thorough crack mapping was performed for all of the spans tested. There were multiple areas of cracked and spalled methacrylate and transverse cracking.

A3.2.5 Strength Testing

No cores were removed from the structure that were long enough to allow testing in accordance with the 1:1 height:diameter ratio limit recommended in the ASTM C42 standard as discussed in Chapter 4, Section 2.5.

A3.2.6 Delamination Testing

All three test spans were sounded for delaminations with the chain drag apparatus. Approximately 45 percent of the deck area was delaminated, with the damage concentrated in span 2 which was over 70 percent delaminated. As mentioned above, the delaminations closely matched the areas categorized as having active corrosion by the half-cell test.

A3.2.7 Petrographic analysis

One core from the bridge was examined to determine the parameters of the air void system. The results are shown below:

Note: Recommended values from the American Concrete Institute Manual of Concrete Practice (216) are included in parenthesis after the results.

- Air Content 3.5% (>5.0% ± 1.5%)
- Voids per Inch 5.6 (>7.5)
- Specific Surface 638 in^2/in^3 (>600 in^2/in^3)
- Spacing Factor 0.008 (<0.008)

These values are below currently accepted standards, but it must be noted that no evidence of free-thaw damage was observed during the inspection.

In addition one sample from this bridge was examined using a scanning electron microscope for evidence of sulfate attack. A limited amount of ettringite, the sulfate attack product, was found. The ettringite, however, was not the cause of the deterioration observed in the structure and formed as a secondary product only.

A3.2.8 Visual Analysis

After the bridge was demolished, a number of samples of the reinforcing bars were retrieved from the demolition debris. There was substantial corrosion and loss of cross section in many of the bars observed. Also, most of the corrosion was noted on bars from the top layer of the deck reinforcing in the region corresponding to the gutter. All observations confirmed the extensive corrosion damage indicated by the testing outlined above.

A3.3 CONCLUSIONS

This bridge was suffering from extreme corrosion, especially in the second span. The different tests seemed to corroborate with each other well and the entire testing regime worked well to describe the overall corrosion condition of the bridge. The damage indicated by the field survey was confirmed by the post-mortem observation of the structure during demolition.

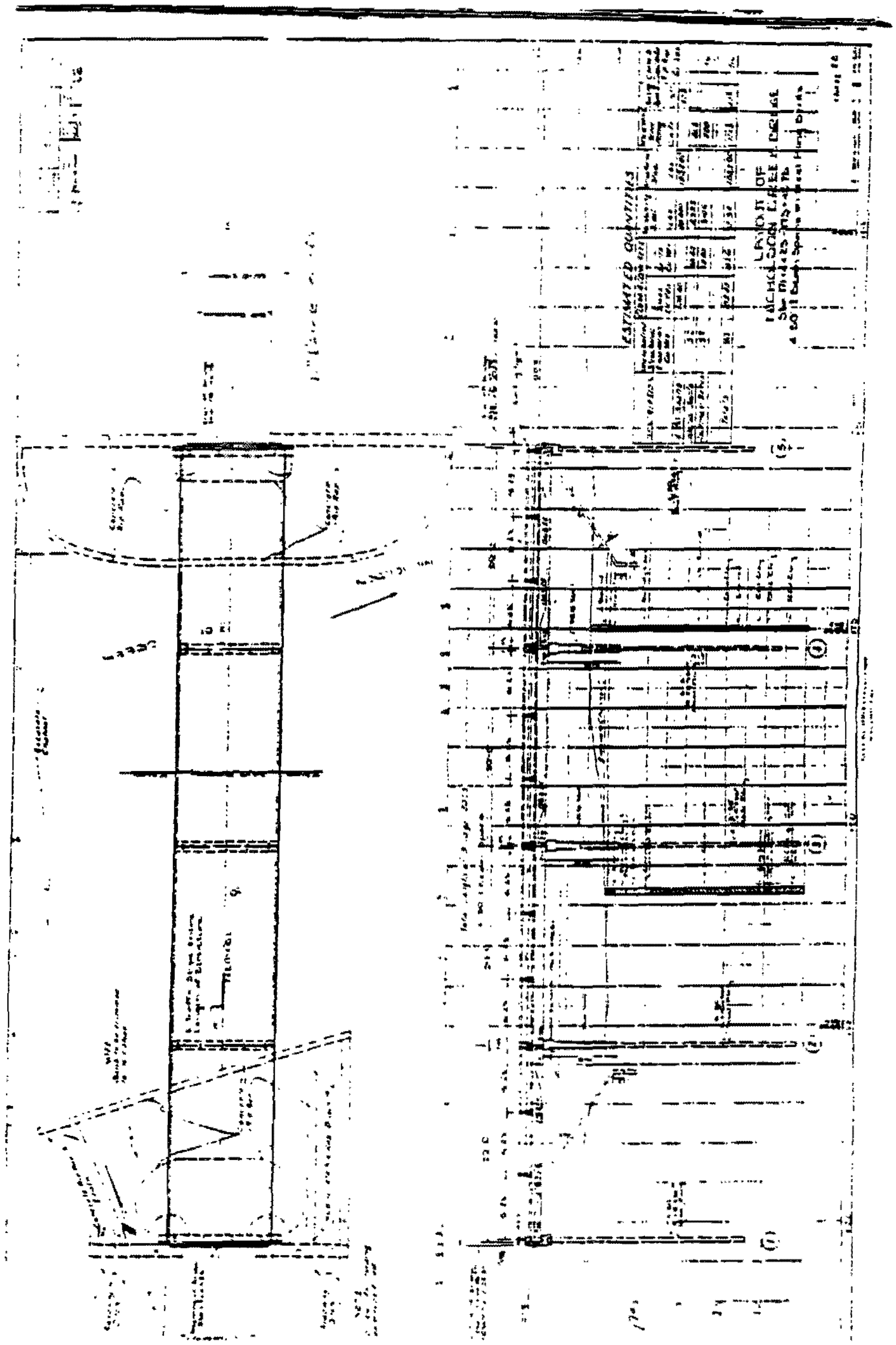


Figure A3.1

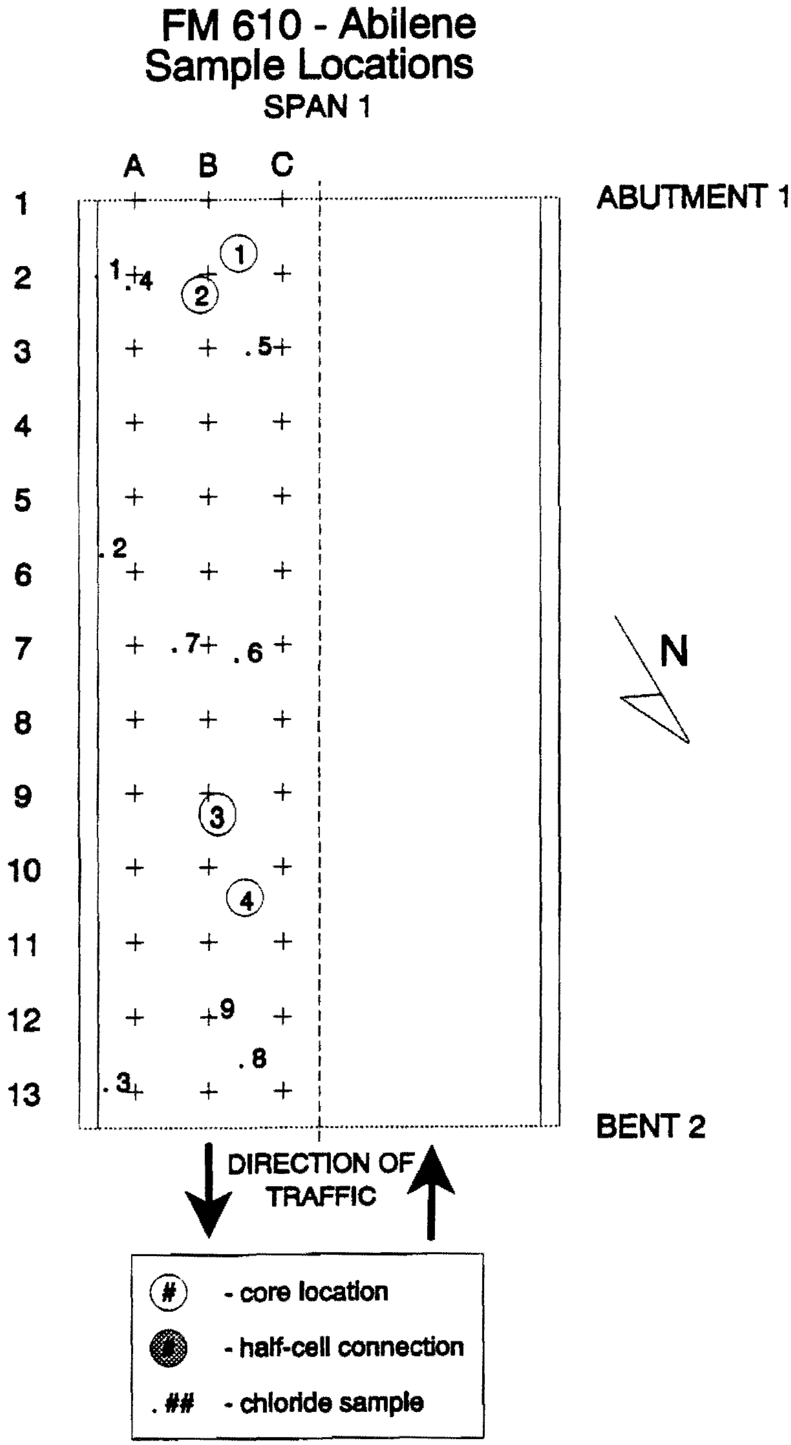
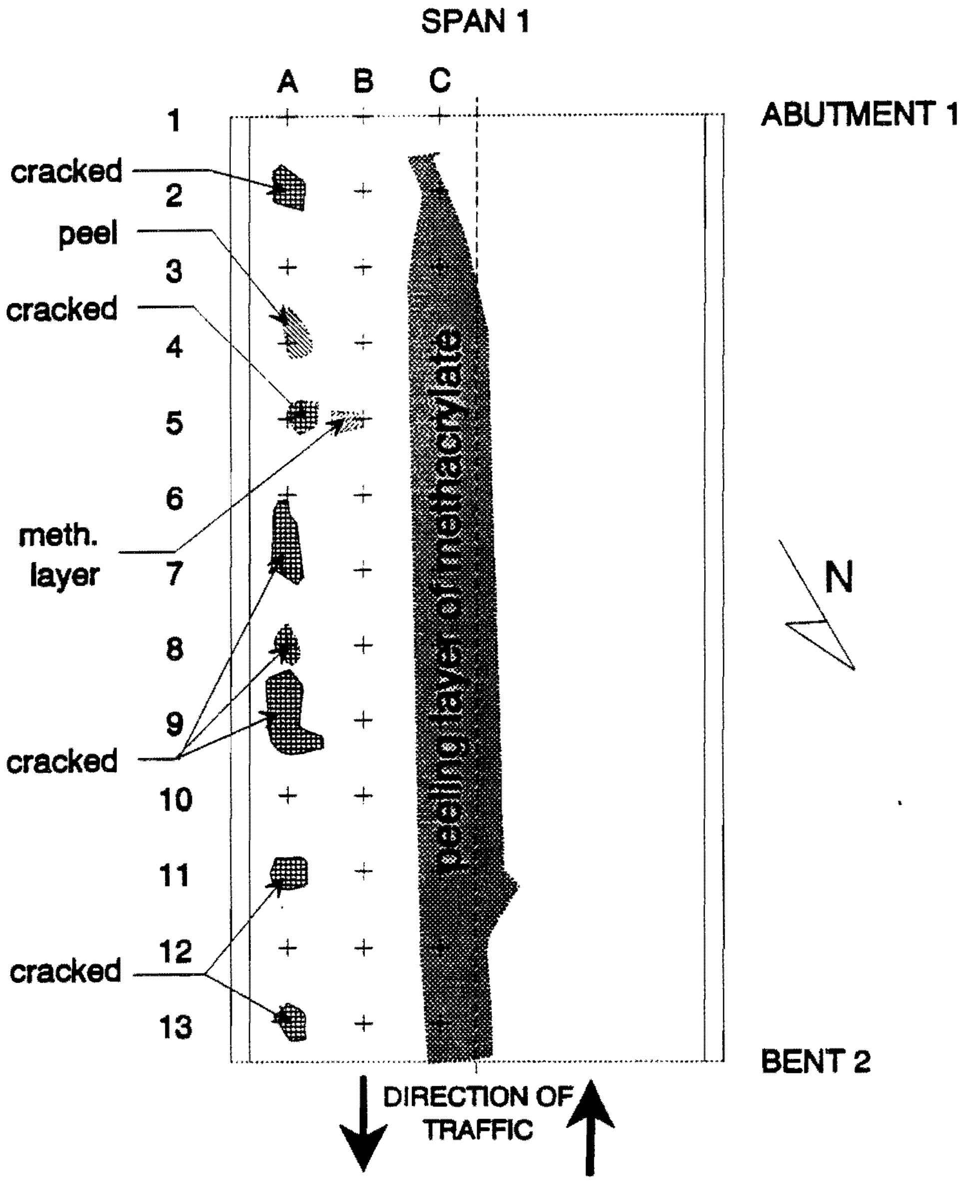

FM 610 - Abilene
Sample Locations
SPAN 1
A B C
1
2
3
4
5
6
7
8
9
10
11
12
13
ABUTMENT 1
BENT 2
N
1
2
3
4
5
6
7
8
9
DIRECTION OF TRAFFIC
- core location
- half-cell connection
. ## - chloride sample

Figure A3.2

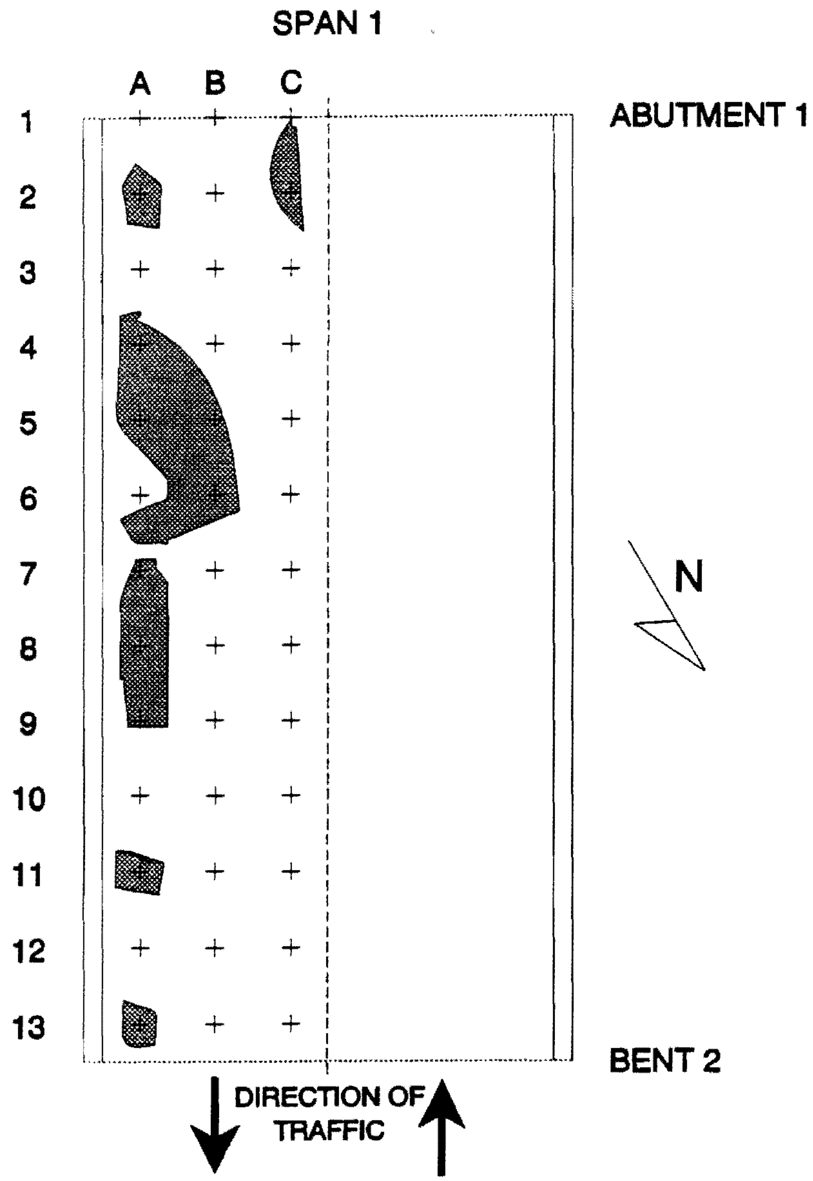
FM 610 at Nicholson Creek - Abilene



Crack Mapping

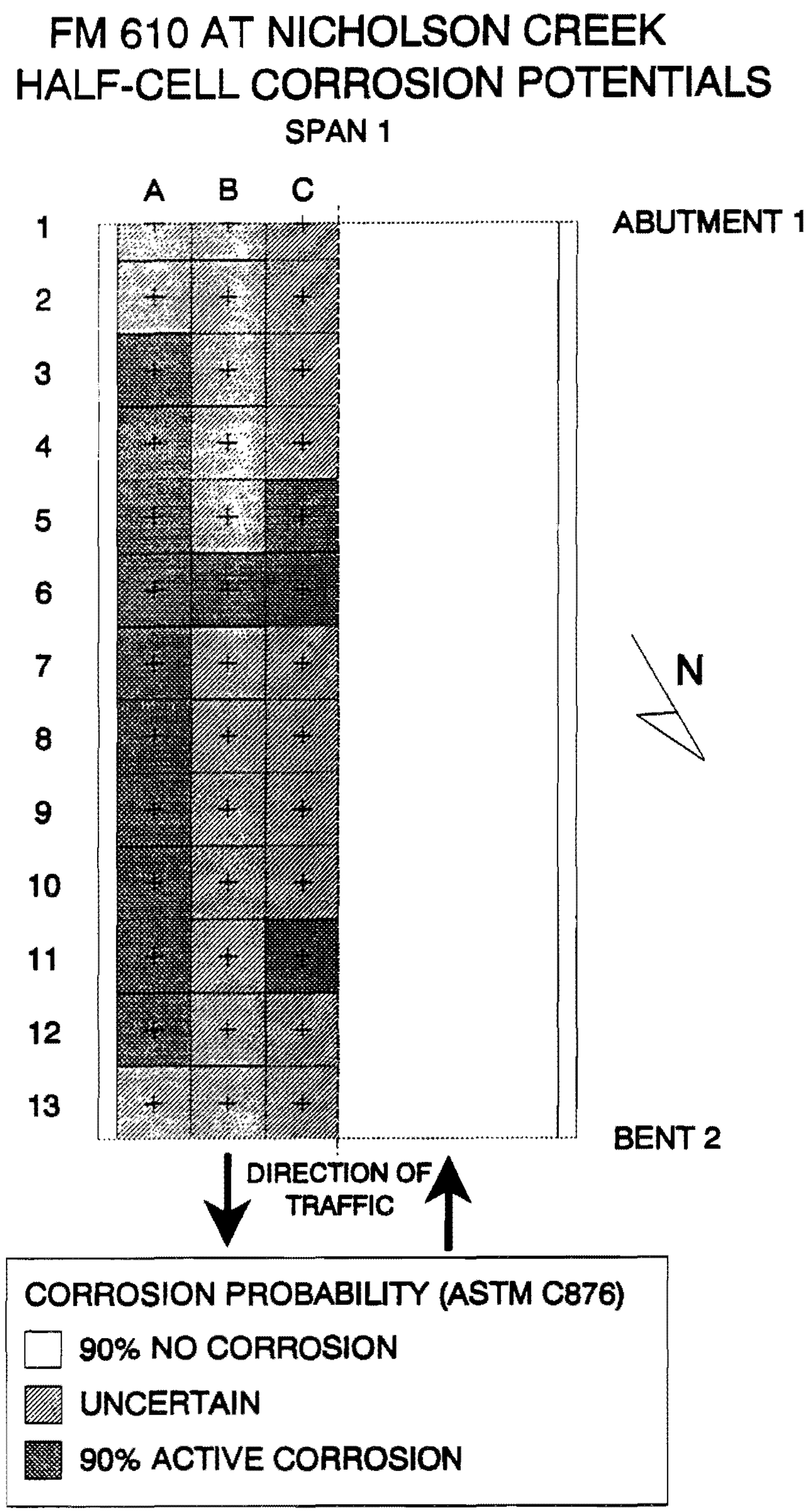
Figure A3.3

FM 610 at Nicholson Creek - Abilene



Delamination Surveys

Figure A3.4



FM 610 AT NICHOLSON CREEK
HALF-CELL CORROSION POTENTIALS
SPAN 1
A
B
C
1
2
3
4
5
6
7
8
9
10
11
12
13
ABUTMENT 1
N
BENT 2
DIRECTION OF TRAFFIC
CORROSION PROBABILITY (ASTM C876)
90% NO CORROSION
UNCERTAIN
90% ACTIVE CORROSION

Figure A3.5

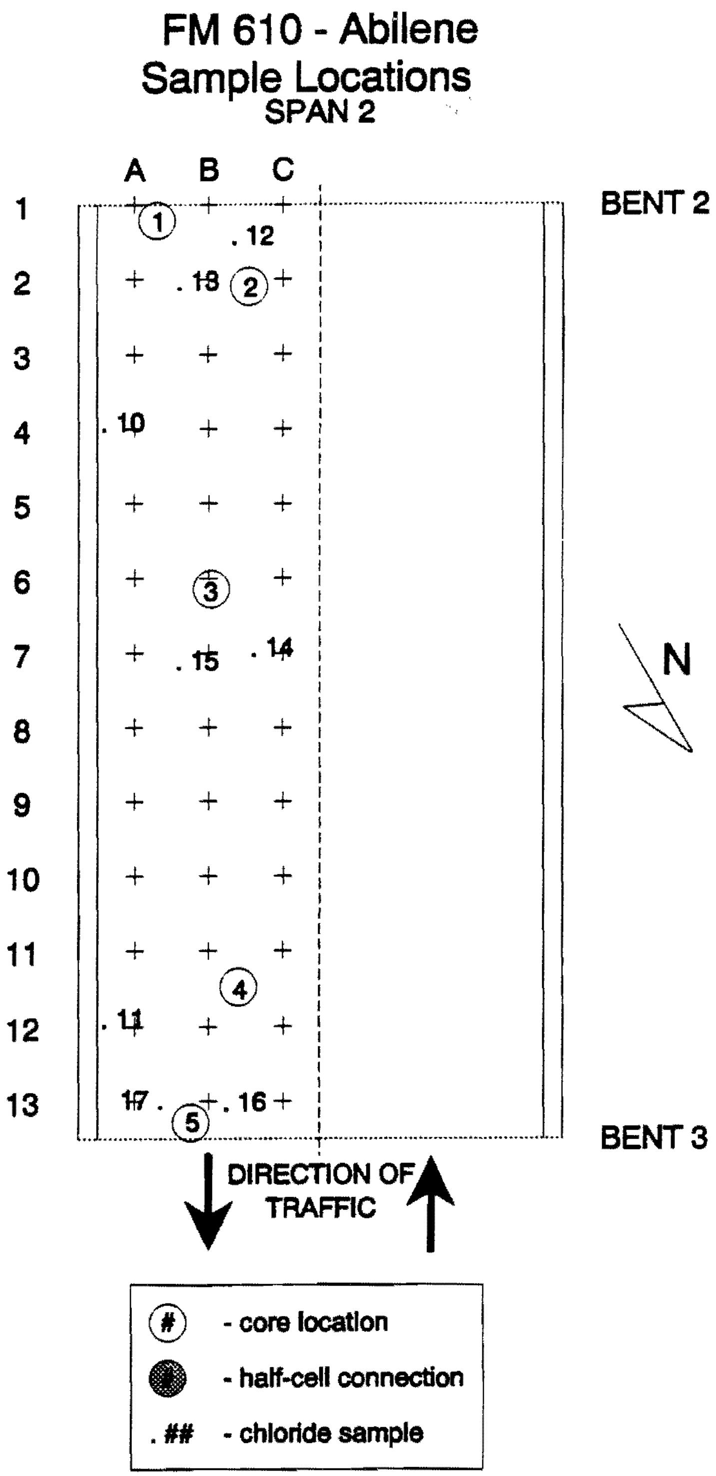
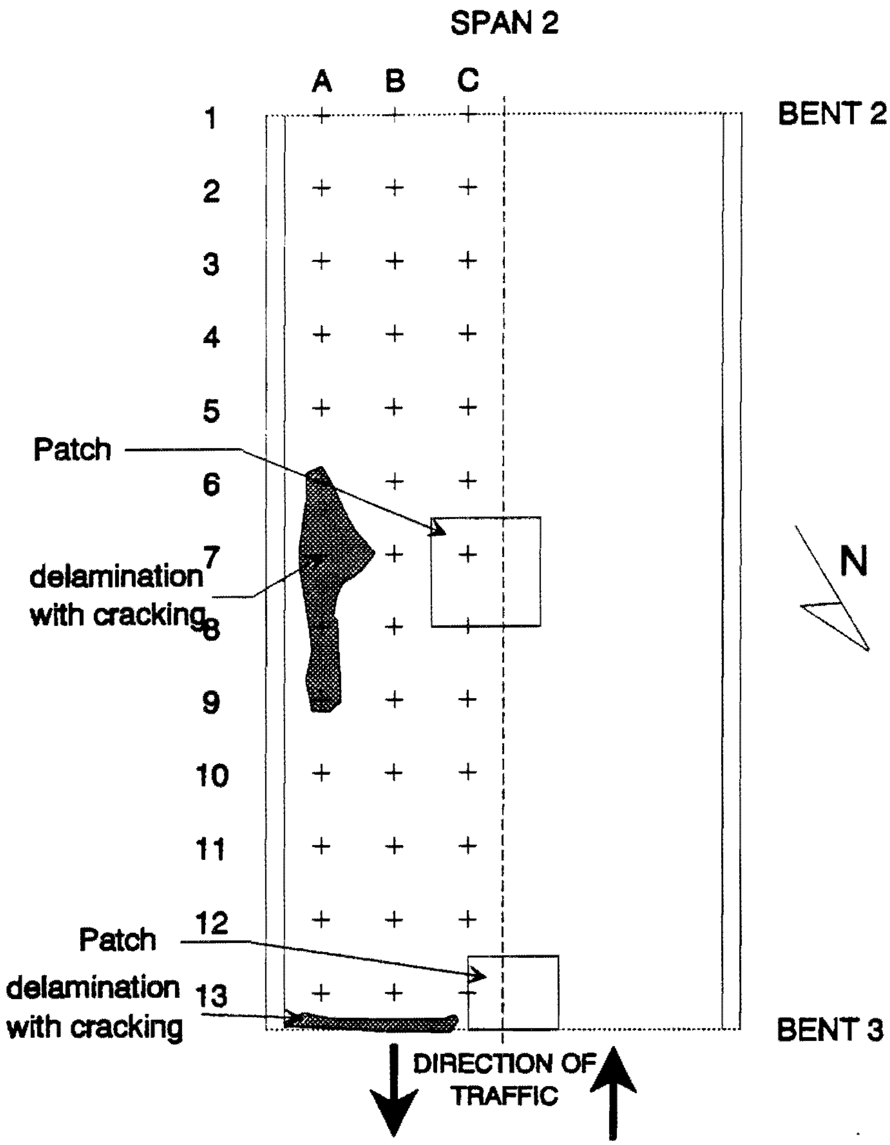

FM 610 - Abilene
Sample Locations
SPAN 2
A
B
C
1
2
3
4
5
6
7
8
9
10
11
12
13
BENT 2
BENT 3
1
2
3
4
5
. 12
. 13
. 10
. 15
. 14
. 11
17 .
. 16
N
DIRECTION OF
TRAFFIC
- core location
- half-cell connection
. ## - chloride sample

Figure A3.6

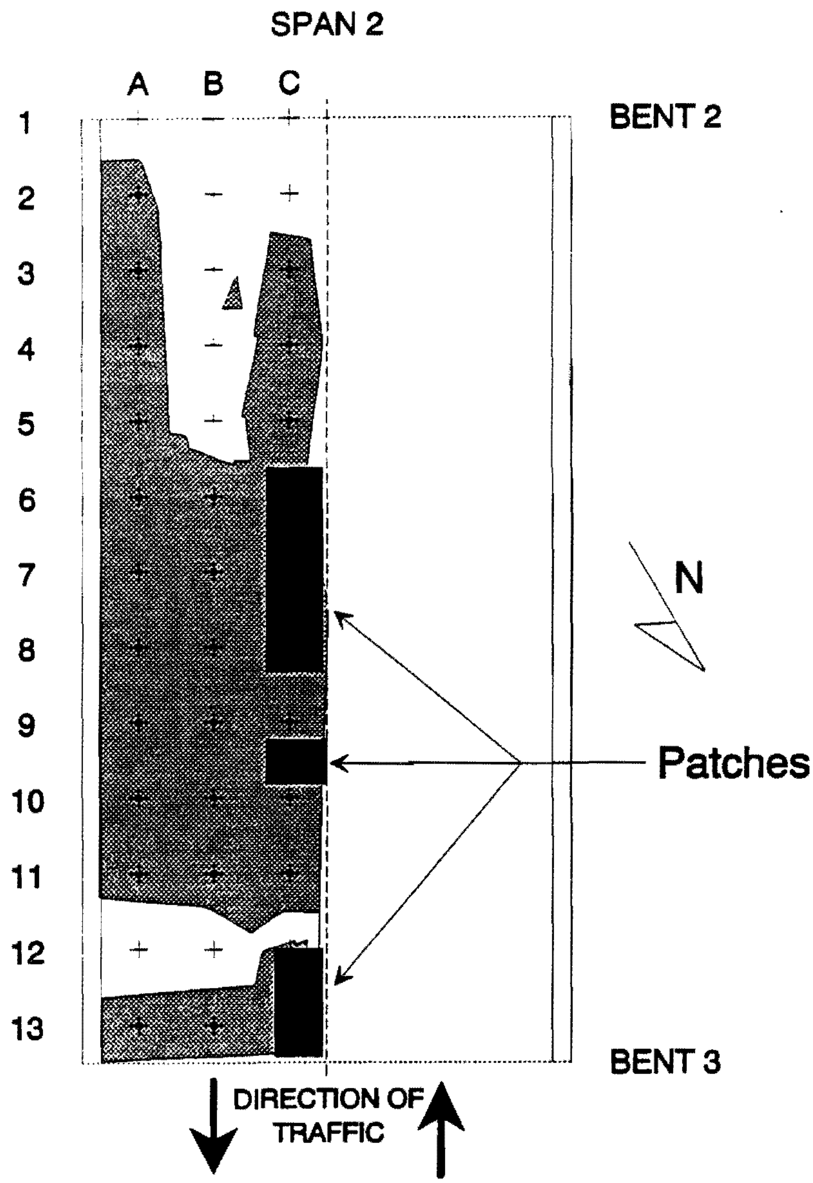
FM 610 at Nicholson Creek - Abilene



Transverse cracks all across span

Figure A3.7

FM 610 at Nicholson Creek - Abilene



Delamination Surveys

Figure A3.8

FM 610 AT NICHOLSON CREEK
HALF-CELL CORROSION POTENTIALS
SPAN 2

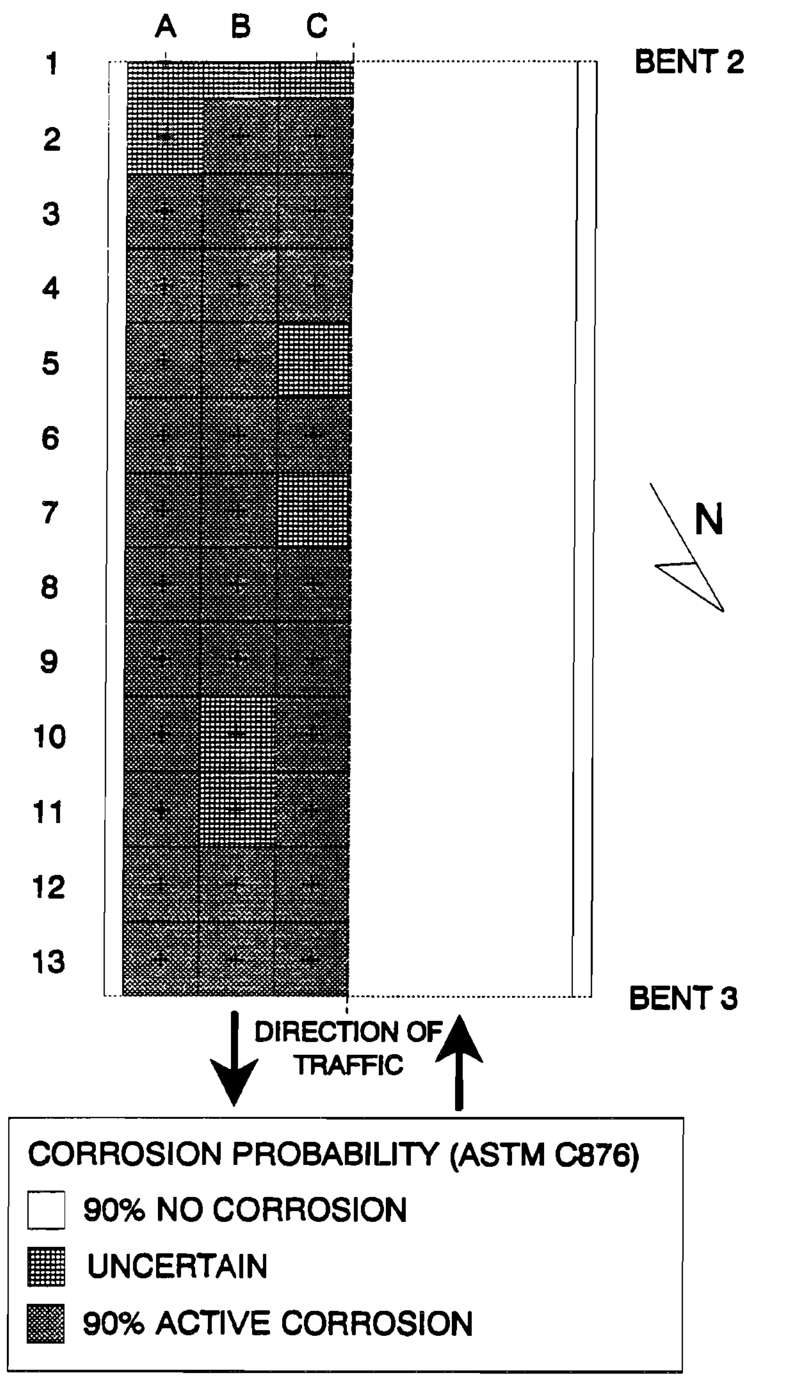


Figure A3.9

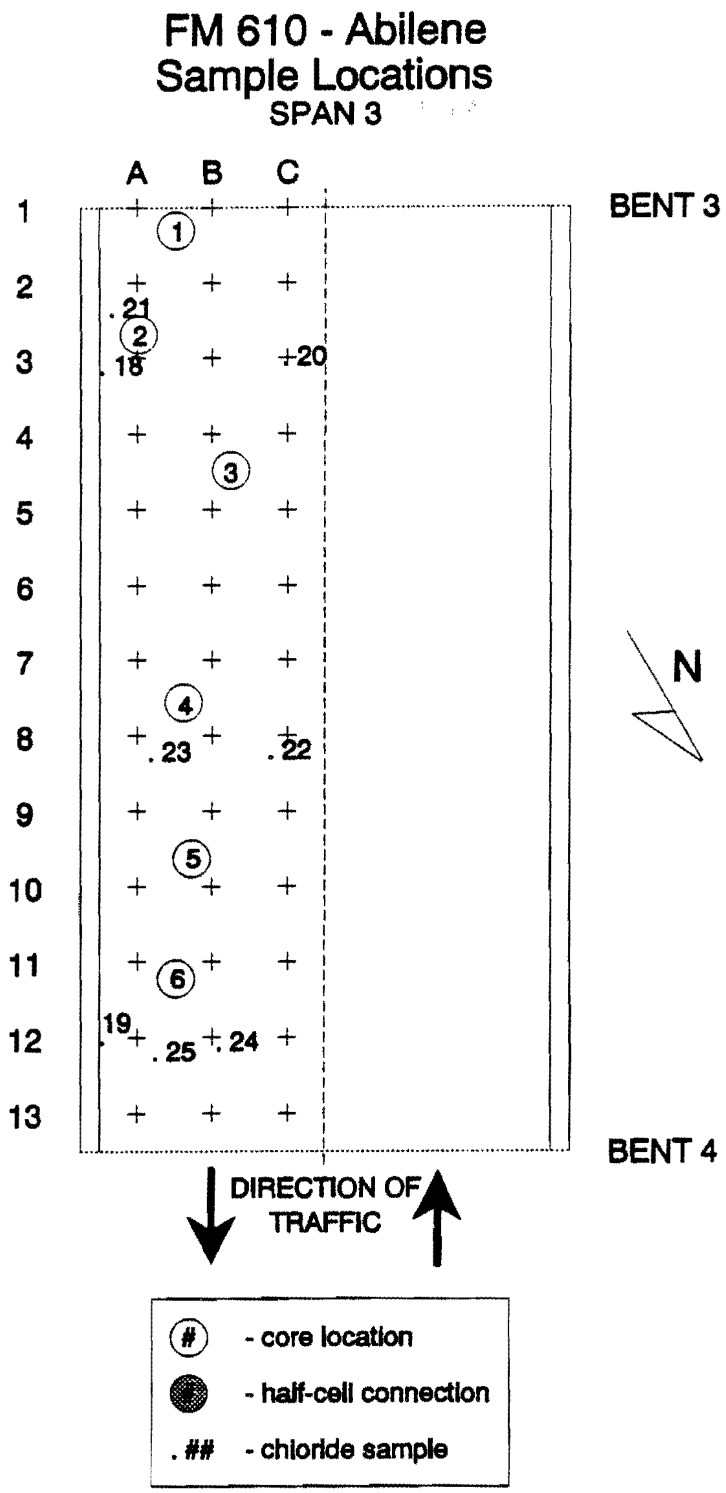
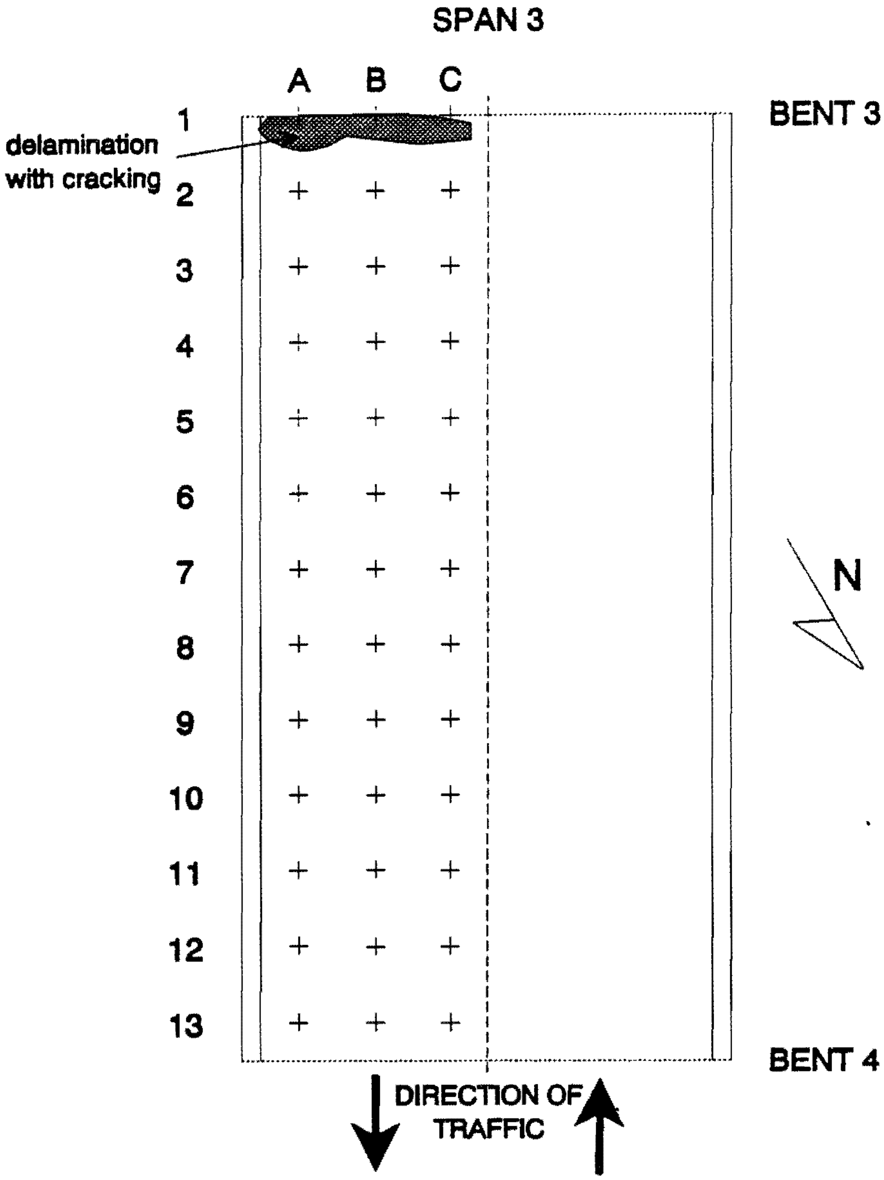

FM 610 - Abilene
Sample Locations
SPAN 3
A
B
C
1
2
3
4
5
6
7
8
9
10
11
12
13
BENT 3
BENT 4
N
21
18
20
23
22
19
25
24
DIRECTION OF
TRAFFIC
- core location
- half-cell connection
. ##
- chloride sample

Figure A3.10

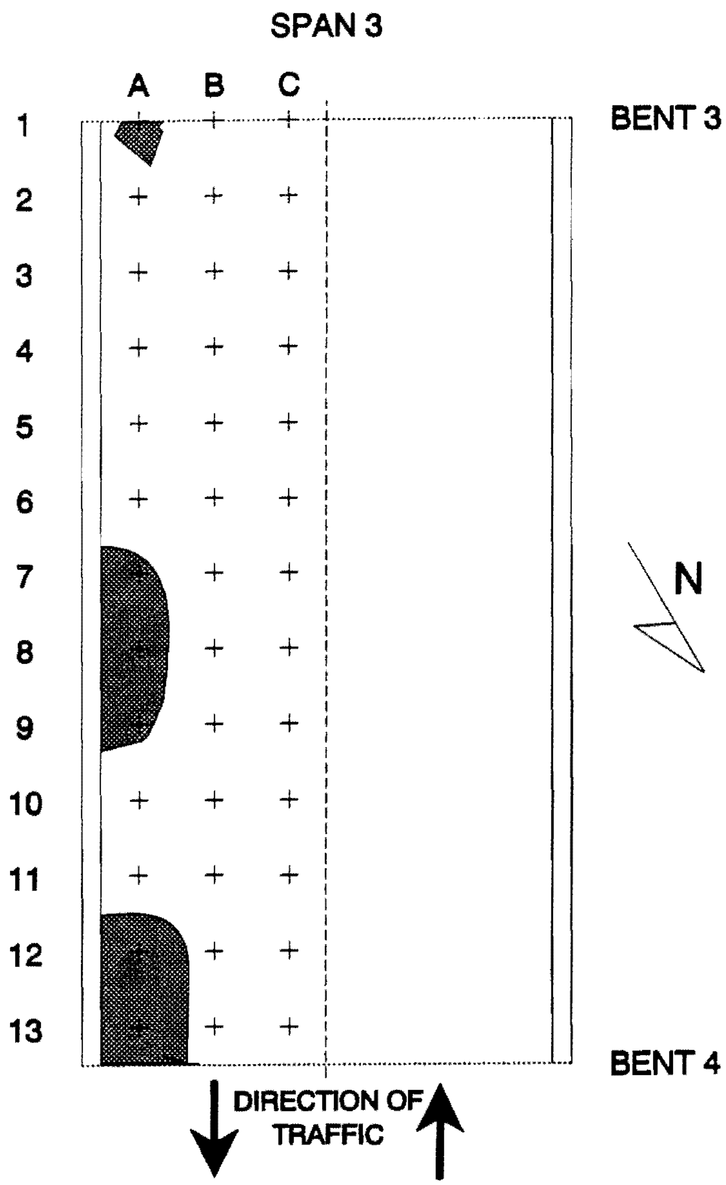
FM 610 at Nicholson Creek - Abilene



Transverse cracks all across span

Figure A3.11

FM 610 at Nicholson Creek - Abilene



Delamination Surveys

Figure A2.13

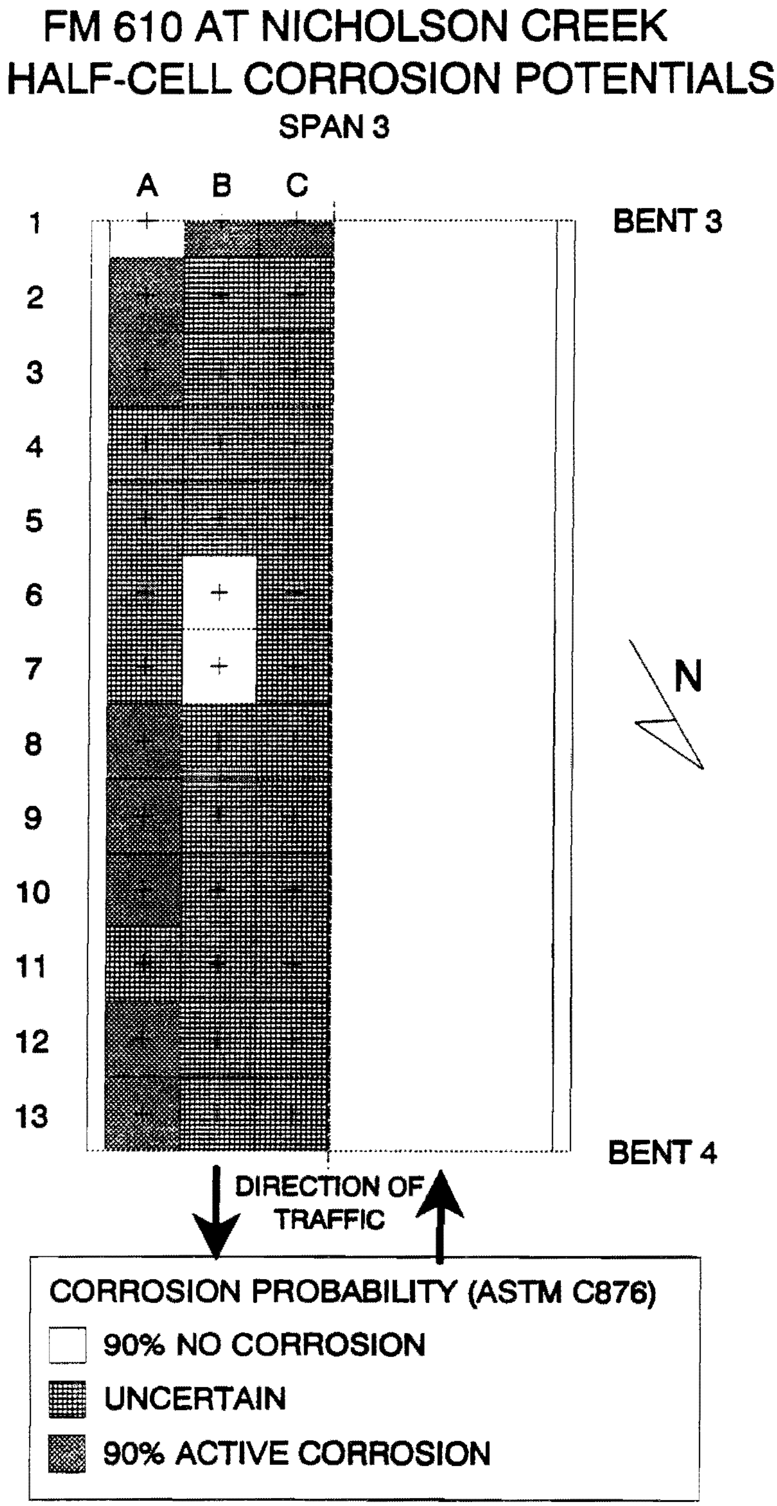

FM 610 AT NICHOLSON CREEK
HALF-CELL CORROSION POTENTIALS
SPAN 3
A
B
C
1
2
3
4
5
6
7
8
9
10
11
12
13
BENT 3
BENT 4
N
DIRECTION OF
TRAFFIC
CORROSION PROBABILITY (ASTM C876)
90% NO CORROSION
UNCERTAIN
90% ACTIVE CORROSION

Figure A3.13

Table A3.1

FM 610 - GRID 3

Grid 3- Five Minute Stabilization				
Time(min.)	2B	2A	2B MIN:	-0.246
0	-0.246	-0.361	2B MAX:	-0.201
0.5	-0.243	-0.359	RANGE:	0.045
1	-0.240	-0.352		
1.5	-0.235	-0.348	2A MIN:	-0.361
2	-0.232	-0.344	2A MAX:	-0.333
2.5	-0.227	-0.338	RANGE:	0.028
3	-0.220	-0.340		
3.5	-0.215	-0.337		
4	-0.210	-0.335		
4.5	-0.205	-0.333		
5	-0.201	-0.338		

GRID 3- TEN POINT RANDOM SURVEY			
	BEFORE	AFTER	DIFF.
2A	0.391	0.392	0.001
3B	0.294	0.302	0.008
3C	0.258	0.256	0.002
5B	0.211	0.230	0.019
6A	0.318	0.327	0.009
8C	0.302	0.306	0.004
9A	0.360	0.365	0.005
11A	0.328	0.331	0.003
12B	0.248	0.247	0.001
13C	0.202	0.212	0.010

GRID 3 - MAIN SURVEY			
	Col A	Col B	Col C
1	-0.342	-0.420	-0.386
2	-0.386	-0.300	-0.271
3	-0.384	-0.303	-0.256
4	-0.347	-0.265	-0.237
5	-0.283	-0.228	-0.214
6	-0.319	-0.194	-0.254
7	-0.300	-0.185	-0.311
8	-0.393	-0.289	-0.295
9	-0.372	-0.253	-0.240
10	-0.377	-0.267	-0.238
11	-0.333	-0.291	-0.259
12	-0.372	-0.248	-0.264
13	-0.357	-0.282	-0.202

FM 610 - GRID 2

GRID 2- TEN POINT RANDOM SURVEY			
	BEFORE	AFTER	DIFFERENCE
2A	0.348	0.340	0.008
3C	0.435	0.425	0.010
5A	0.393	0.395	0.002
6B	0.488	0.491	0.003
6C	0.417	0.422	0.005
7B	0.477	0.491	0.014
10C	0.449	0.445	0.004
10A	0.411	0.432	0.021
12A	0.528	0.543	0.015
13B	0.407	0.409	0.002

GRID 2 - MAIN SURVEY			
	Col A	Col B	Col C
1	-0.299	-0.286	-0.291
2	-0.335	-0.414	-0.358
3	-0.392	-0.422	-0.419
4	-0.393	-0.482	-0.399
5	-0.370	-0.436	-0.344
6	-0.421	-0.475	-0.416
7	-0.407	-0.498	-0.349
8	-0.368	-0.520	-0.423
9	-0.402	-0.462	-0.528
10	-0.420	-0.303	-0.450
11	-0.393	-0.321	-0.422
12	-0.520	-0.470	-0.496
13	-0.387	-0.407	-0.373

FM 610 - GRID 1

GRID 1- TEN POINT RANDOM SURVEY			
	BEFORE	AFTER	DIFFERENCE
1B	0.234	0.258	0.024
3A	0.338	0.369	0.031
5A	0.448	0.462	0.014
5C	0.359	0.364	0.005
6B	0.389	0.423	0.034
8B	0.269	0.301	0.032
9B	0.216	0.223	0.007
10C	0.287	0.297	0.010
12A	0.359	0.381	0.022
13C	0.207	0.220	0.013

GRID 1 - MAIN SURVEY			
	Col A	Col B	Col C
1	-0.283	-0.253	-0.228
2	-0.286	-0.215	-0.251
3	-0.356	-0.273	-0.305
4	-0.489	-0.200	-0.305
5	-0.451	-0.224	-0.362
6	-0.471	-0.406	-0.406
7	-0.461	-0.302	-0.341
8	-0.381	-0.286	-0.271
9	-0.364	-0.203	-0.310
10	-0.380	-0.254	-0.281
11	-0.358	-0.256	-0.374
12	-0.370	-0.237	-0.253
13	-0.323	-0.288	-0.214

Table A3.4

FM 610 at Nicholson Creek

Methacrylate

Concrete Permeabilities (Coulombs)

	Finished Surface	
Sample	Intact	Removed
3-3-C4		7217.1
3-3-C5	1233.9	
3-3-C6		8650.8
Surface 3	2549.7	
Surface 1	1832.4	
Base 1		4805.1
Surface 2	1843.2	
Base 2		5694.3
Surface 4	3056.4	

FM 610 - ABILENE TEXAS
CHLORIDE CONTENTS (LB/CU. YD)

SAMPLE		SAMPLE DEPTH (IN)				
Grid	number	0- 0.5	0.5 - 1.0	1.0 - 1.5	1.5 - 2.0	REMARKS
GRID #1						
	4	8.40	0.56	2.80	5.60	sound conc., hit steel in 1.5-2.0"
	5	3.60	2.40	2.40	2.00	
	6	2.40	1.60	0.72	0.52	sound concrete
	7	3.60	2.80	1.72	0.68	
	8	2.40	1.88	1.92	2.00	poor methacrylate
	9	3.60	3.60	1.04	0.88	good methacrylate
GRID #2						
	1	2.00	1.40	0.84	1.08	on rail near spalls
	2	1.76	1.40	0.72	0.72	on rail, good conc., no spalls
	3	5.60	2.80	0.56	1.08	on rail
	10	0.00	0.00	1.40	2.00	poor concrete on rail
	11	1.72	1.72	0.88	0.68	poor concrete on rail
	12	3.20	4.40	5.60	5.20	medium methacrylate
	13	3.20	3.20	4.00	10.40	medium methacrylate
	14	2.80	4.40	8.00	2.40	medium methacrylate
	15	2.80	4.80	2.00	1.60	near big delamination
	16	3.20	3.20	2.00	1.88	
	17	3.20	5.60	2.40	2.00	
GRID #3						
	18	0.44	1.28	1.72	1.28	on rail, hit steel in 1.5-2.0 sample
	19	0.88	1.40	1.44	0.76	on rail
	20	4.80	3.20	1.88	0.56	wheelpath
	21	2.40	5.60	3.20	4.40	at scupper
	22	3.60	3.20	2.80	0.60	spalled methacrylate
	23	6.80	5.20	3.20	1.72	thick methacrylate
	24	6.00	5.60	3.20	1.60	thick methacrylate
	25	7.20	5.60	7.60	3.60	
BENT 2						
	26	9.20	1.44	3.60	2.80	
	27	2.40	0.92	0.92	1.20	
	28	0.56	0.00	0.00	0.00	
	29	2.40	3.20	1.44	0.60	cap
	30	2.80	2.00	1.32	0.56	cap
	31	0.76	1.28	1.6	0.64	cap

APPENDIX A4
FM 1835 AT THE SALT FORK OF THE BRAZOS RIVER, ABILENE DISTRICT

A4.1 BACKGROUND

The two lanes of FM 1835 in Stonewall County are carried over the Salt Fork of the Brazos River by a 9 span 453 foot bridge. The structure consists of an integral deck on pan form girders supported on concrete bents and drilled shafts. The bridge carries an ADT of 60 vehicles, 14 percent of which are trucks. The structure was built in 1991 and was used as a test installation for a variety of sealers. Sealers included in the testing were: linseed oil, sodium silicate, 40 percent silane in isopropanol, 20 percent silane in isopropanol, 20 percent silane in water, silane in mineral spirits, and siloxane in mineral spirits applied over linseed oil.

According to area maintenance personnel, the structure has been exposed to only one or two deicer applications during its short history. In addition, the Salt Fork of the Brazos has a relatively high chloride content and may expose the structure to some chloride through contact with the substructure and through vapor transmission. The structure was visited mainly to establish a baseline for comparison to data from future visits and to conduct permeability tests for comparisons among the sealer systems.

A4.2 TESTING

A4.2.1 *Half-Cell Potential Testing*

As the structure was constructed using epoxy-coated reinforcement, the half-cell test could not be performed on the structure.

A4.2.2 *Chloride Content Testing*

Forty-two locations were sampled for chloride content. Four locations on each of the nine spans were tested in addition to six substructure locations. Chloride contents were unexpectedly high at the surface of some of the spans, considering the limited exposure that the bridge has seen to date. Also, the chloride contents are quite variable for individual spans and for the structure as a whole. The reason for this is not known although these values occur in a region that could be feasibly influenced by the sealer materials themselves. At concrete depths from 1 to 2 inches, the chloride contents were more consistent within individual spans. This, coupled with the relative inability of any of the sealer substances to penetrate to this depth, may be indicative of the sealers' ability to prevent the intrusion of chloride into the deck.

On the basis of the chloride contents from 1 to 2 inches, the linseed oils, water-borne silanes, and siloxanes are performing well. The sodium silicate spans are performing somewhat variably with some of the best and worst performances. The silanes in isopropanol are not performing as well.

A4.2.3 *Permeability Testing*

All of the cores removed from the deck were tested to determine the AASHTO T277 Rapid Chloride Permeability. Unfortunately, the testing equipment malfunctioned during the testing of six of the samples. This resulted in four of the systems being characterized by the results of the testing of only one core. The specimens were tested with the treated surface intact, except for one sample from the linseed oil section and four extra specimens cut from the lower portion of other cores. Because of usable core lengths of less than four inches, the samples cut from the bottom of other specimens were all shorter than the two inches prescribed in the test specification and ranged from 1-5/8 to 1-7/8 inches in length. This probably artificially increased their resulting permeability values. In addition, these samples typically had imperfect surface faces because of the core removal process. For example, as noted on the data sheet, sample 4-1-C2 had a large chip in the surface that may have influenced its test results.

Ignoring the results of the chipped sample, the specimens treated with linseed oil and water carried silanes had the lowest permeabilities. Their permeabilities ranged from 3329 coulombs for the silane to 3919 for the linseed oil. The sodium silicate samples were next, with permeabilities of 4469, 4432, and 4639 coulombs. The remaining samples all had permeabilities from 4900 to 6109 coulombs. These findings are consistent with those revealed by the chloride content testing. By comparing all the specimens taken from the lower portions of the cores to their respective upper portions, it can be seen that all of the treatments decreased the permeabilities of the concrete by approximately 3500 coulombs.

The apparent variability of the "Surface Removed" samples may be cause for concern. As all of samples came from more than two inches below the surface of the concrete, the surface sealers should have negligible influence on their permeabilities. The variability in these samples may be due to different concretes or to the inconsistent conditions of the samples from the lower portion of the cores. This should be clarified by the next visit that will include sampling to sufficient depth to provide a standard specimen from the lower portion of all cores.

A4.2.4 Crack Mapping

This bridge was in excellent condition when surveyed. Because of this, no crack patterns or delaminations were detected. Any cracking observed was minor and random. No cracking drawings were made at the site.

A4.2.5 Strength Testing

No cores suitable for strength testing were retrieved during the sampling of the bridge.

A4.2.6 Delamination Testing

As mentioned in section A4.2.4, no delaminations were detected during the survey of the bridge.

A4.2.7 Petrographic analysis

One core from the bridge was examined to determine the parameters of the air void system. The results are shown below:

Note: Recommended values from the American Concrete Institute Manual of Concrete Practice (216) are included in parenthesis after the results.

• Air Content	4.6%	(>5.0% ± 1.5%)
• Voids per Inch	8.2	(>7.5)
• Specific Surface	712 in^2/in^3	(>600 in^2/in^3)
• Spacing Factor	0.007	(<0.008)

These values meet all current standards, and no evidence of freeze-thaw damage was observed during the investigation.

A4.3 CONCLUSIONS

This bridge was observed to be in excellent condition, a finding consistent with its young age. These chloride content and permeability results seem to indicate that the best performers were linseed oil and the 20 percent silane in water. In both tests, these two systems showed the most favorable results.

Except for its low traffic volume exposure, this bridge represents a near ideal test of the various sealers. Monitoring should continue to determine the time and exposure effects on the systems.

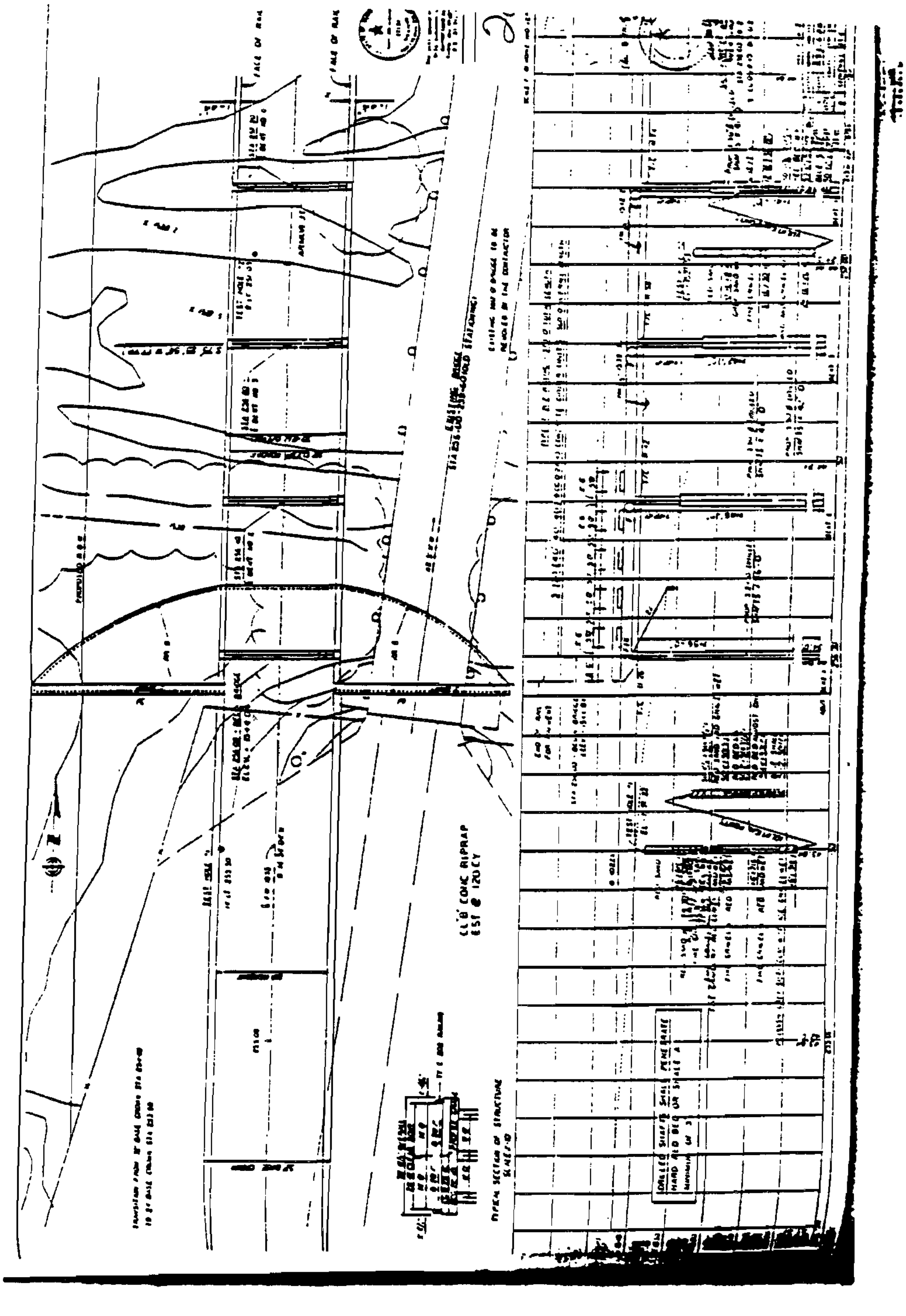


Figure A4.1

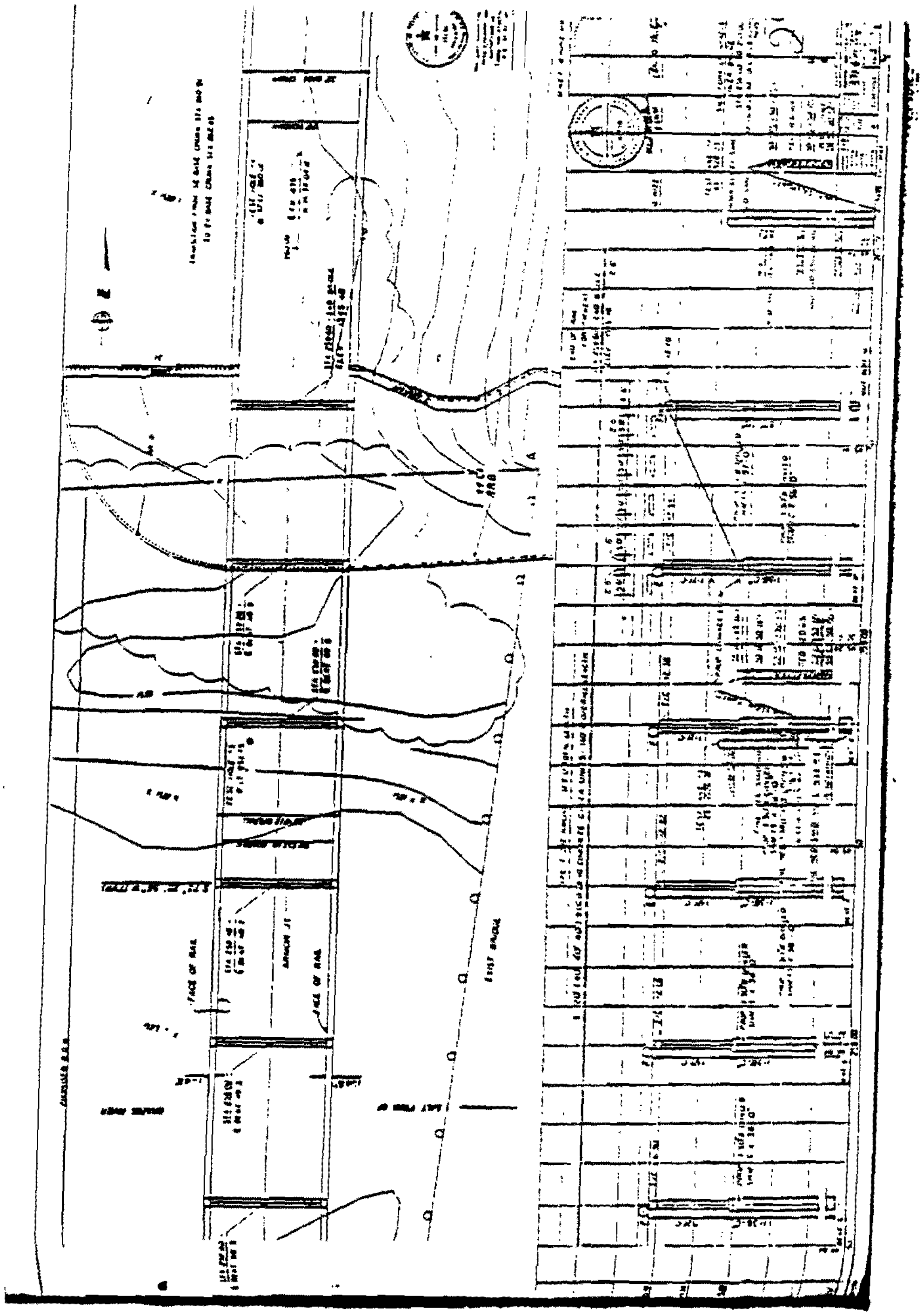


Figure A4.2

FM 1835 at the Salt Fork of the Brazos River

Sample Locations

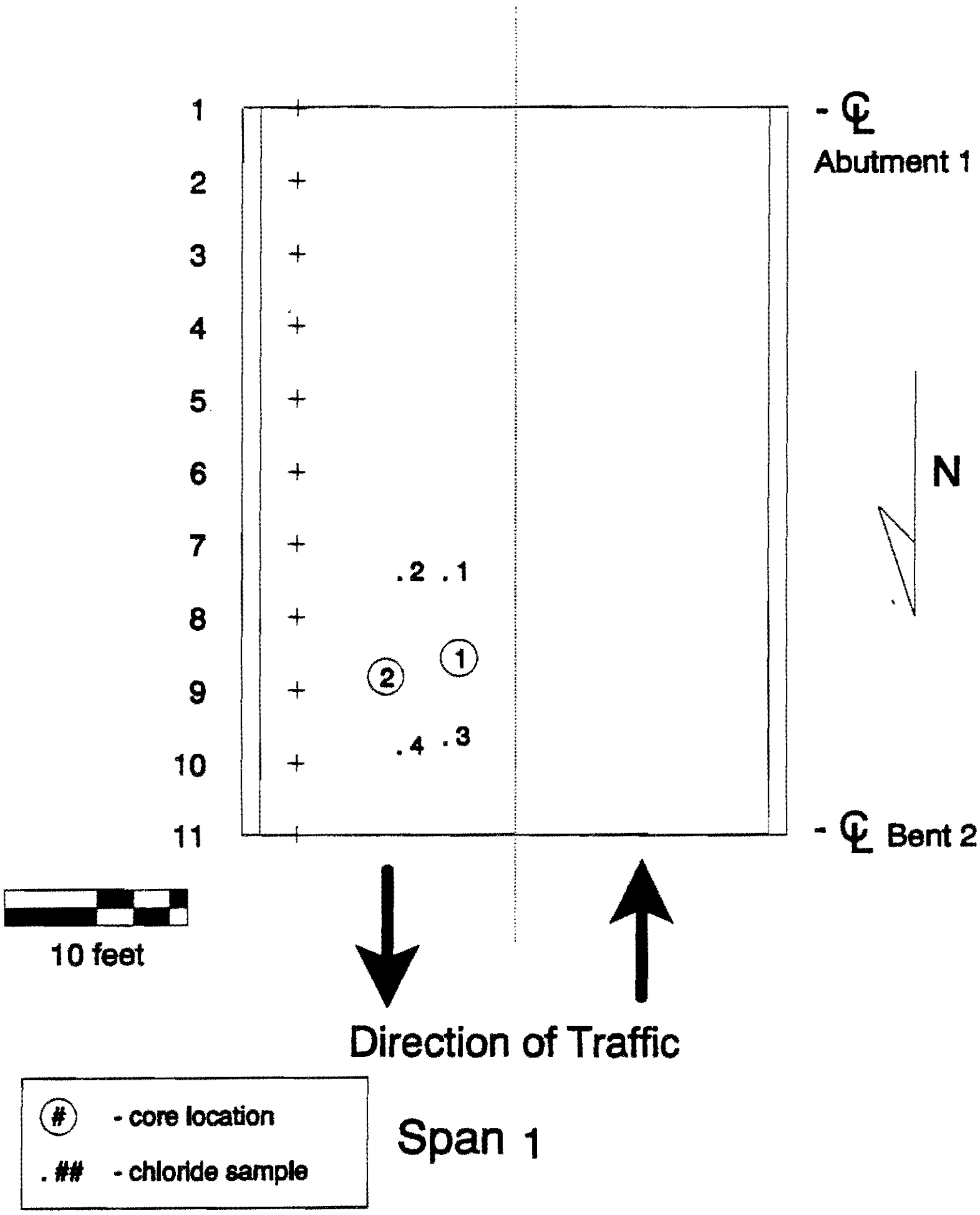


Figure A4.3

FM 1835 at the Salt Fork of the Brazos River

Sample Locations

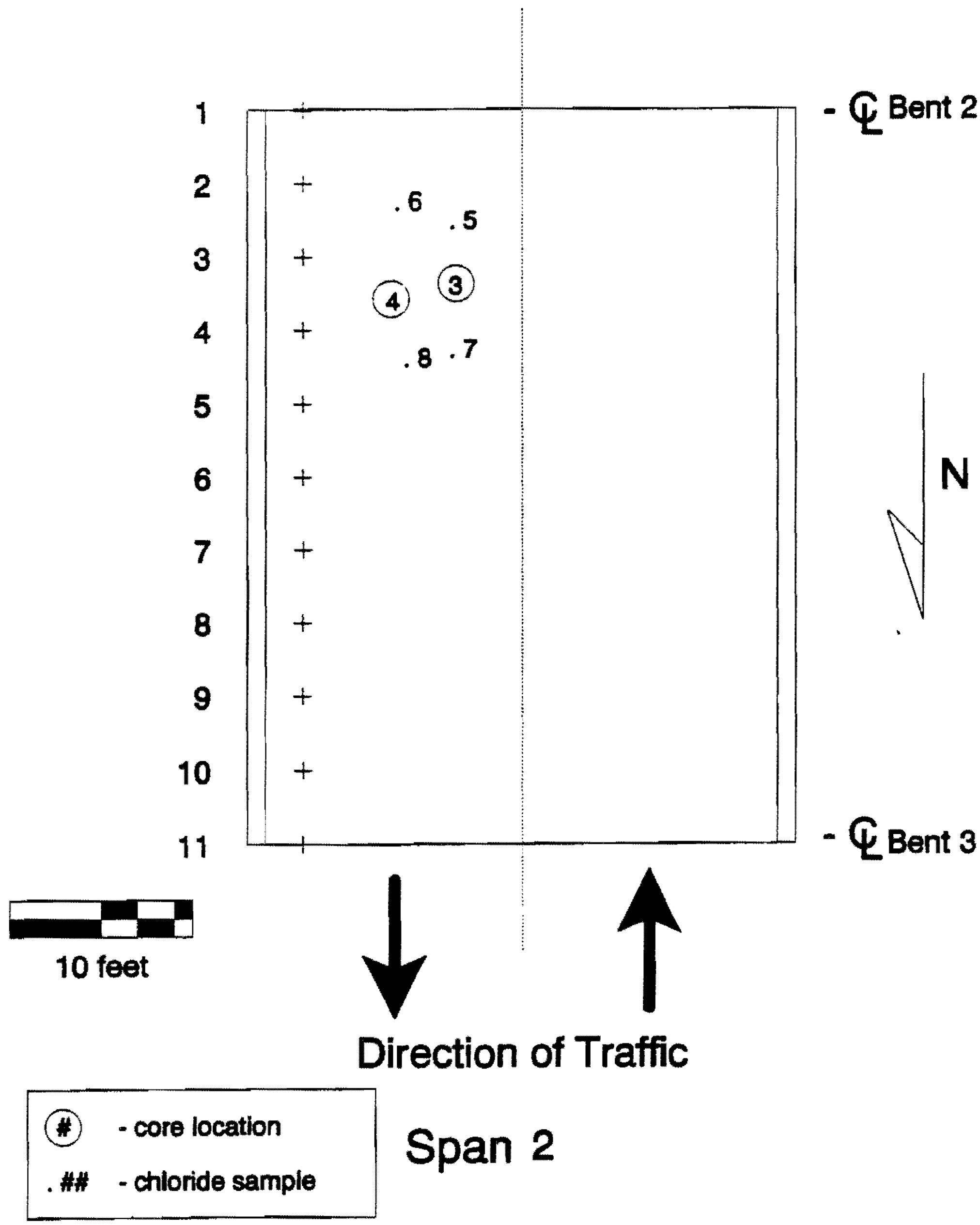


Figure A4.4

FM 1835 at the Salt Fork of the Brazos River

Sample Locations

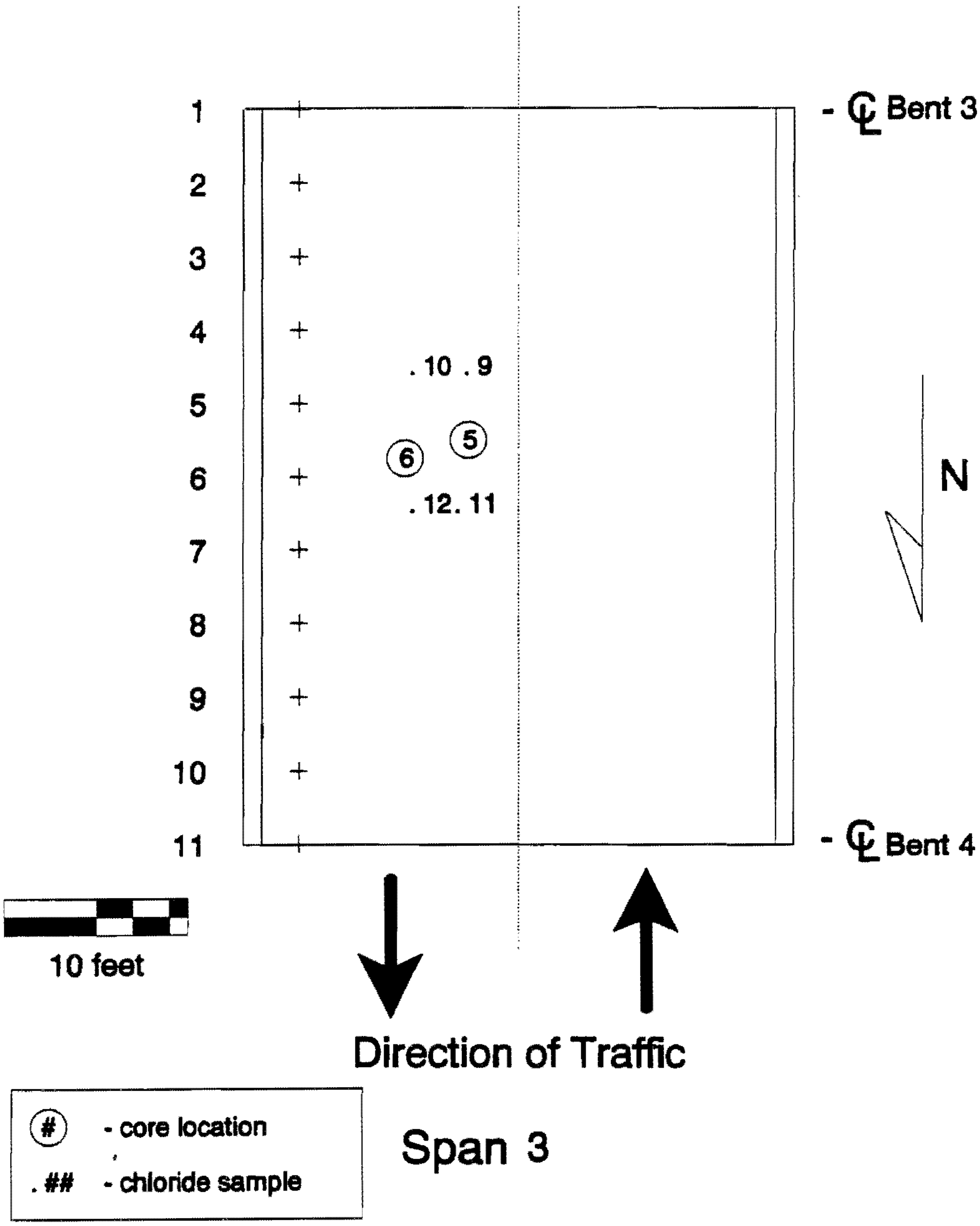


Figure A4.5

FM 1835 at the Salt Fork of the Brazos River

Sample Locations

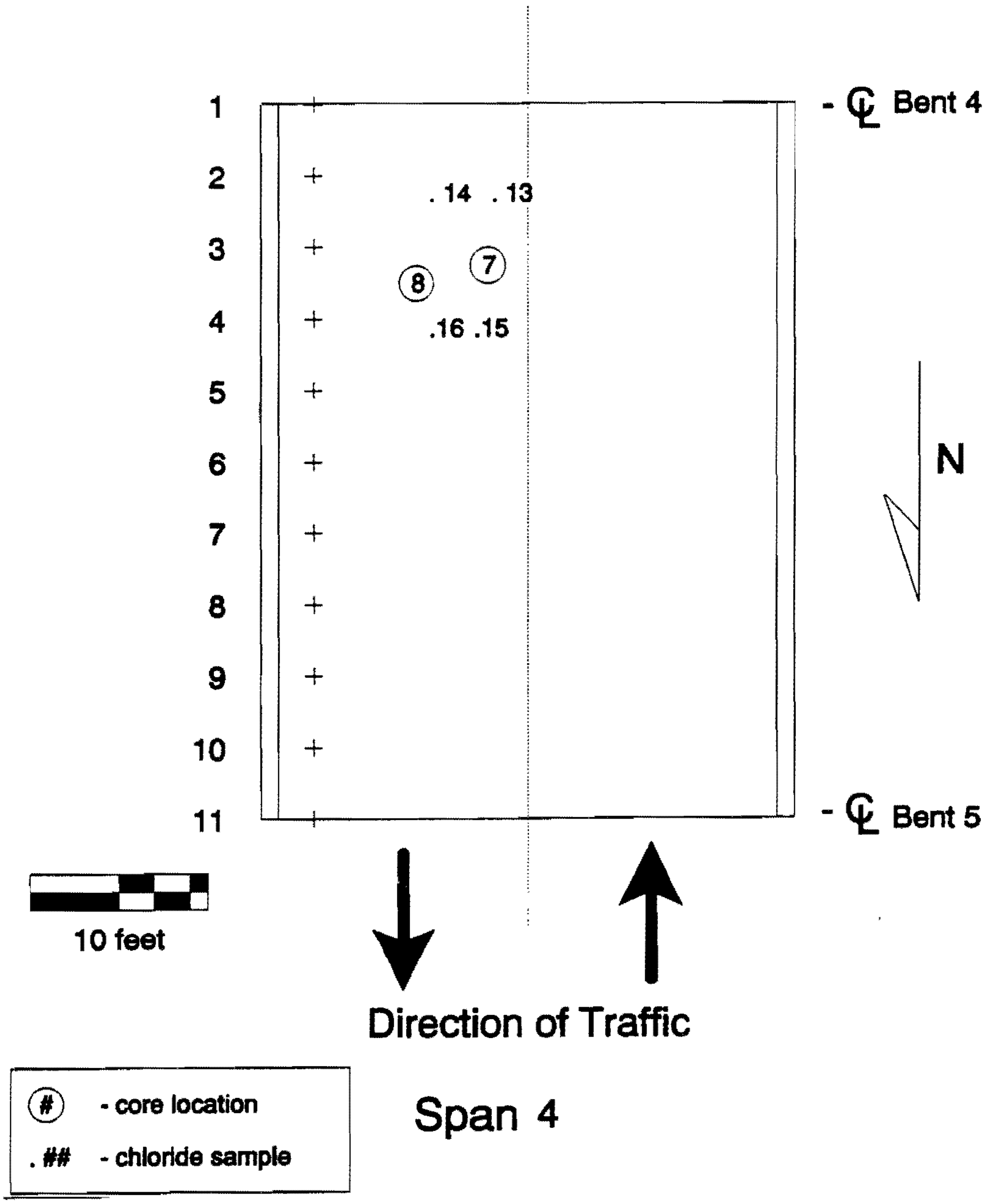


Figure A4.6

FM 1835 at the Salt Fork of the Brazos River

Sample Locations

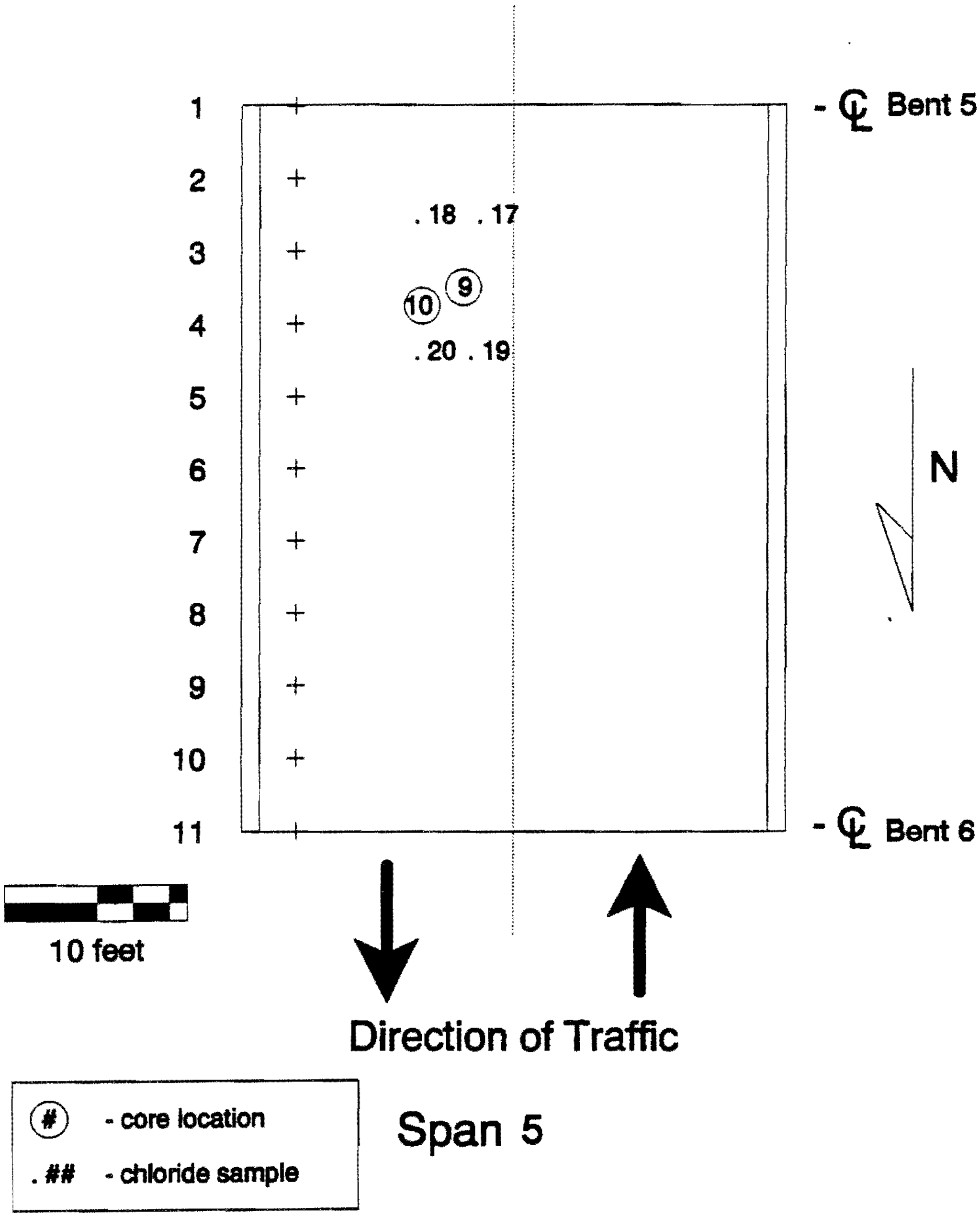


Figure A4.7

FM 1835 at the Salt Fork of the Brazos River

Sample Locations

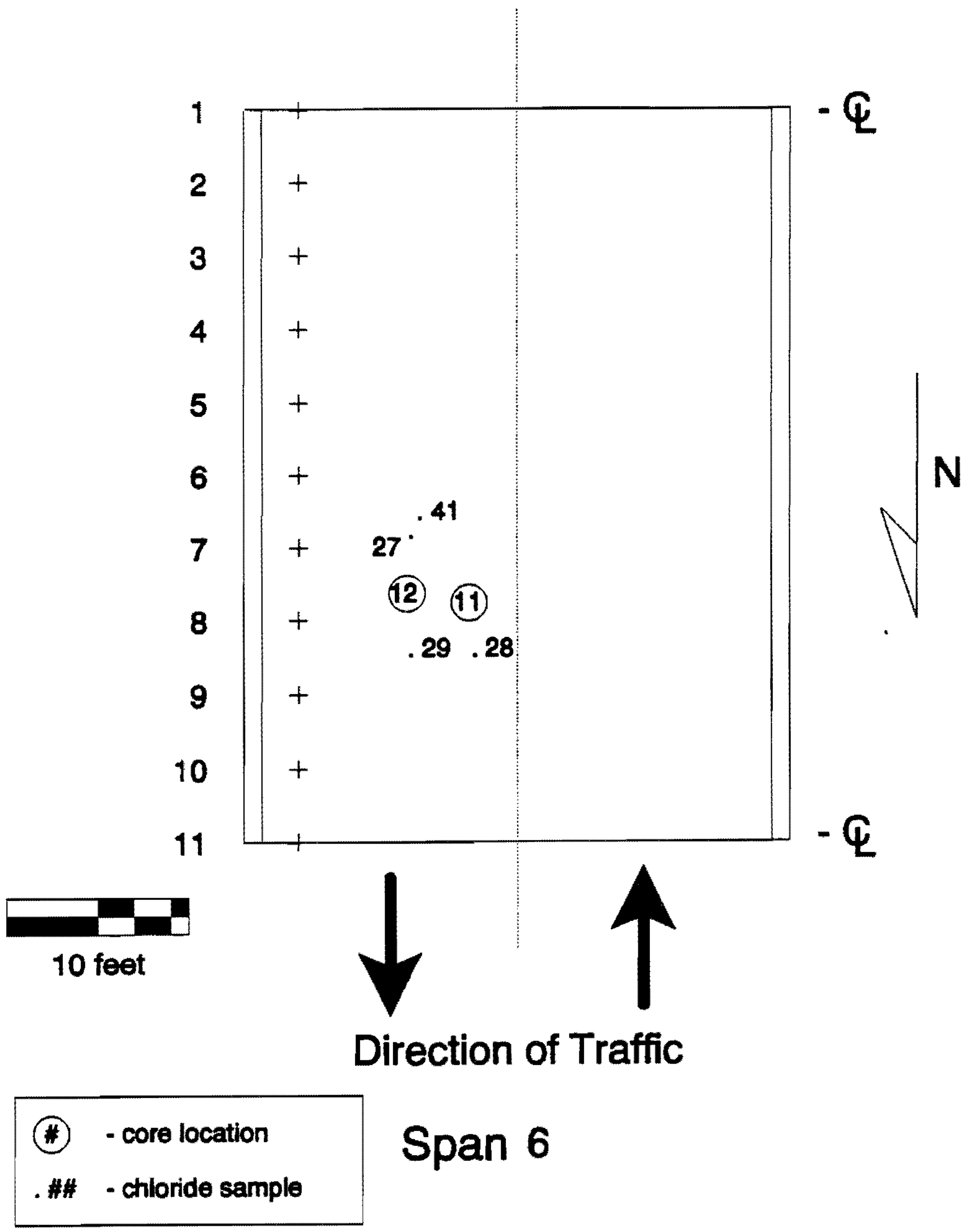


Figure A4.8

FM 1835 at the Salt Fork of the Brazos River

Sample Locations

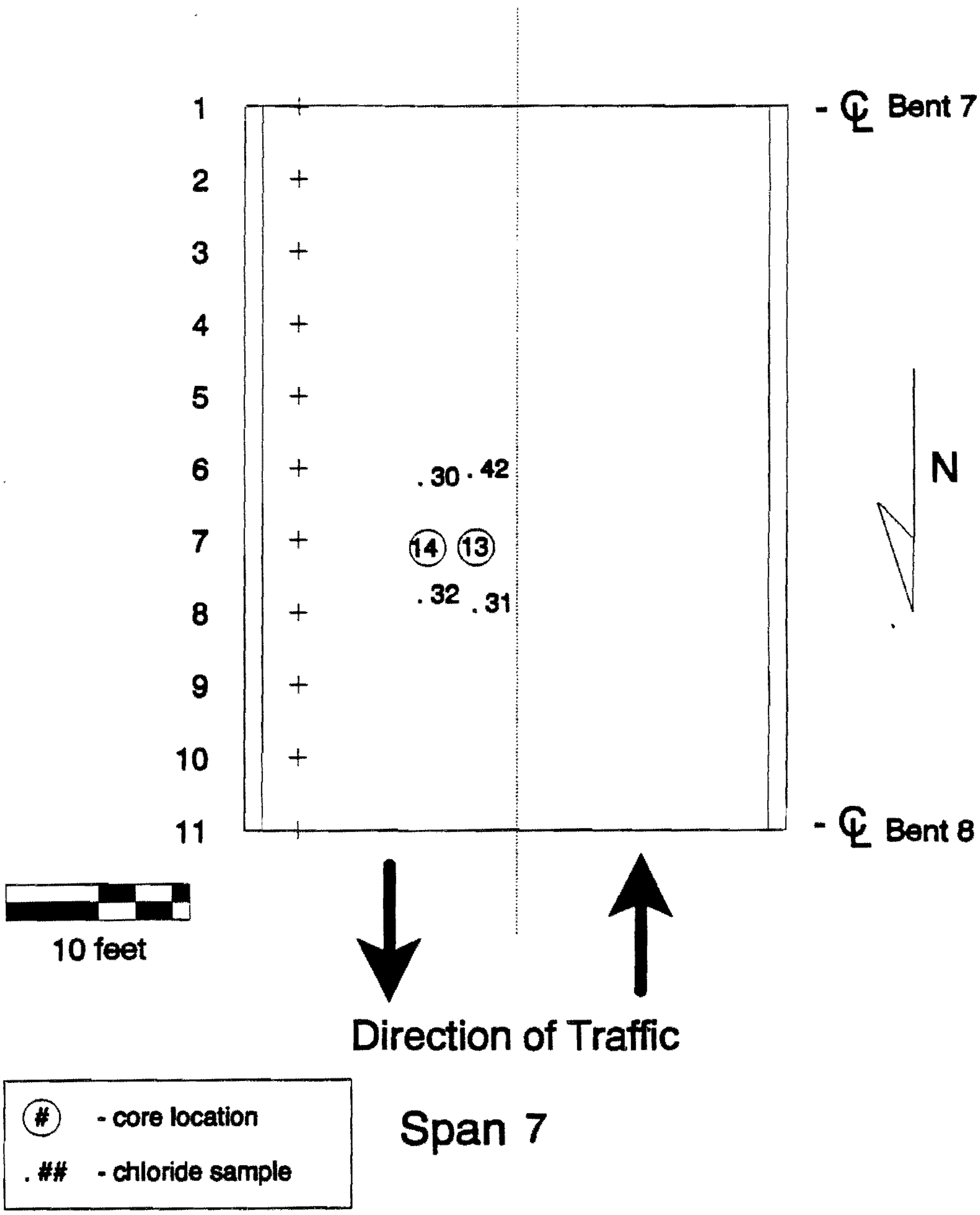


Figure A4.9

FM 1835 at the Salt Fork of the Brazos River

Sample Locations

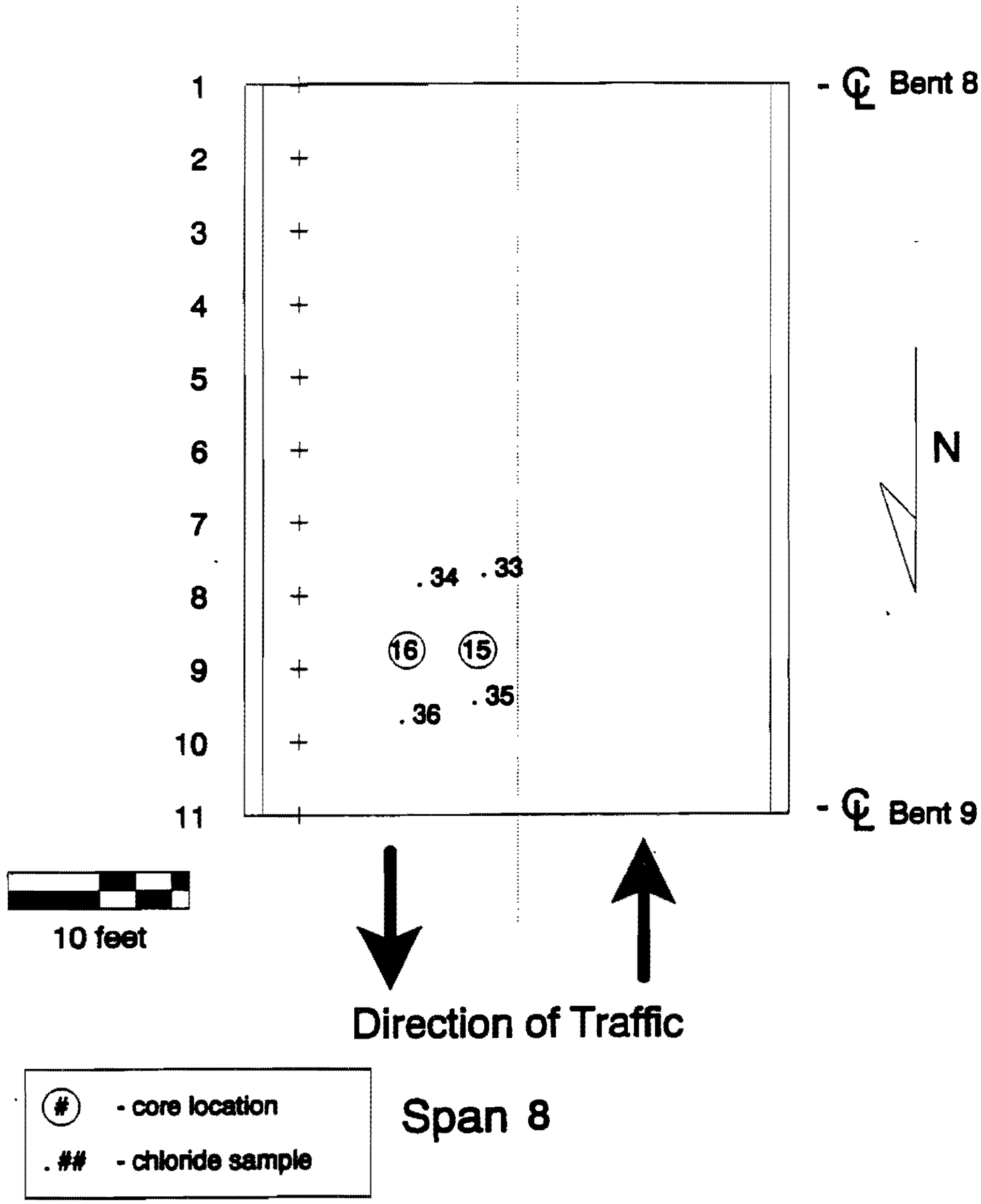


Figure A4.10

FM 1835 at the Salt Fork of the Brazos River

Sample Locations

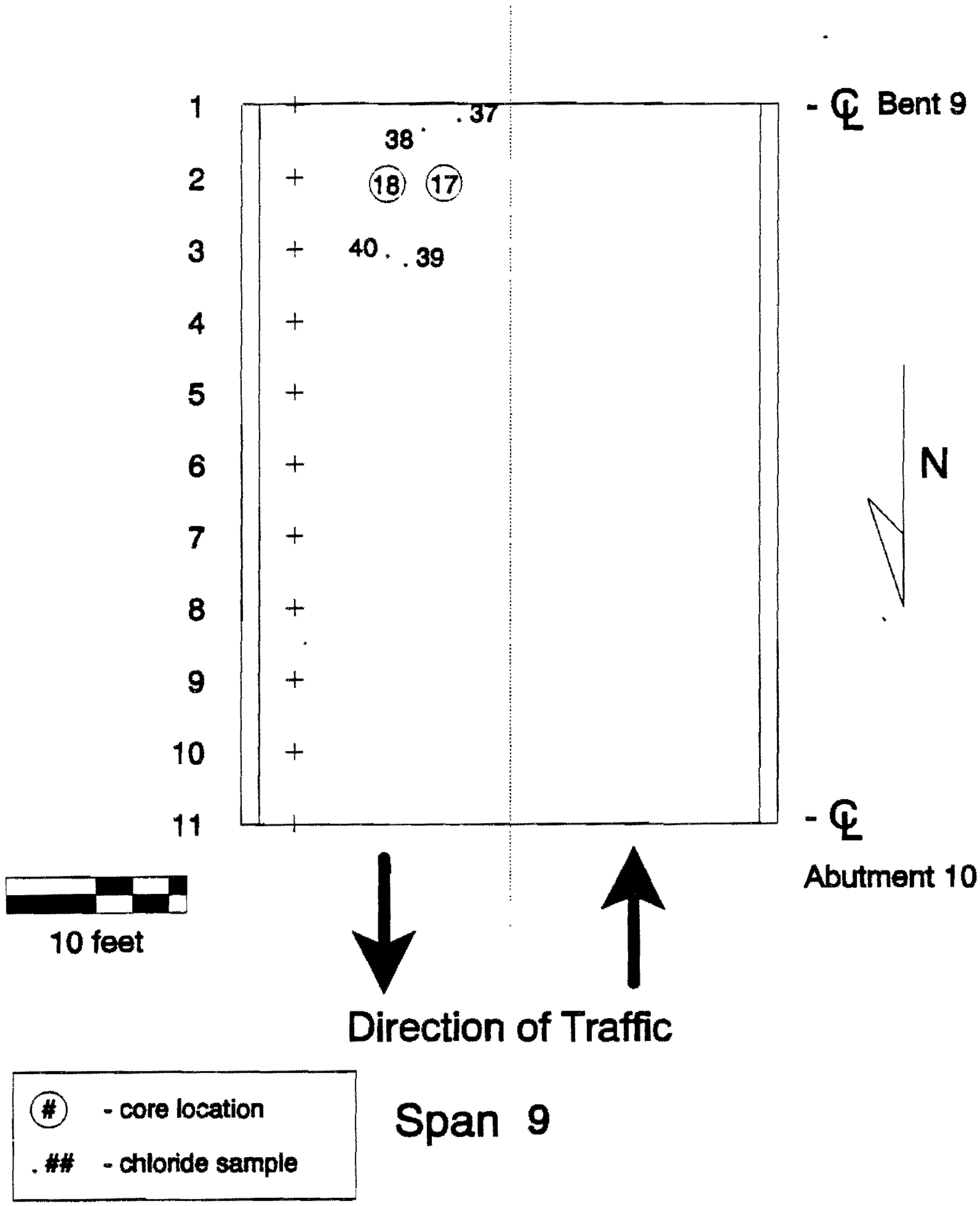


Figure A4.11

FM 1835 at the Salt Fork of the Brazos

Multiple Test Systems

Concrete Permeabilities (Coulombs)

		Treated Surface Intact		Surface Removed
Sample	**Surface Treatment**	**Cracked**	**Uncracked**	**Uncracked**
4-1-C1	Linseed oil			12321
4-1-C2	Linseed oil	10943*		
4-2-C3A	Sodium Silicate		4469	
4-2-C3B	Sodium Silicate			7609
4-2-C4	Sodium Silicate		4432	
4-3-C5	40% Silane in isopropanol		5071	
4-4-C8A	20% Silane in isopropanol		6109	
4-5-C10A	Sodium Silicate		4639	
4-5-C10B	Sodium Silicate			6366
4-6-C12	20% Silane in water		3329	
4-7-C13A	Silane in mineral spirits		5500	
4-7-C13B	Silane in mineral spirits			9115
4-7-C14A	Silane in mineral spirits		4986	
4-7-C14B	Silane in mineral spirits			8366
4-8-C16	Siloxane over linseed oil		5027	
4-9-C17	Linseed oil		3919	
4-9-C18	Linseed oil		3836	

* This specimen had a large chip in the surface exposed to the NaCl during testing, possibly affecting the results

Table A4.2

FM 1835 - ABILENE
CHLORIDE CONTENTS (LB/CU. YD.)

SAMPLE		SAMPLE DEPTH (IN.)				
Grid	Number	0.0 - 0.5	0.5 - 1.0	1.0 - 1.5	1.5 - 2.0	REMARKS
1- Linseed Oil 50-70%						
	1	0.32	0.00	0.00	0.00	
	2	0.48	0.00	0.00	0.20	
	3	0.72	0.24	0.00	0.00	
	4	0.00	0.24	0.32	0.20	
2- CHEM-CRETE Sodium						
Silicate	5	1.28	0.92	0.20	0.00	
	6	1.72	0.68	0.36	0.00	
	7	1.76	1.16	0.36	0.00	
	8	2.00	0.72	0.24	0.00	
3 - SIL-ACT 40% Silane in						
isopropanol	9	1.20	1.20	2.04	0.24	
	10	3.60	2.40	0.60	0.32	
	11	1.20	0.80	0.36	0.72	
	12	1.88	1.92	1.76	0.32	
4-SIL-ACT 20% Silane in						
isopropanol	13	1.20	1.60	0.44	0.24	
	14	2.00	1.40	1.40	0.56	
	15	0.68	0.68	0.24	0.24	
	16	0.76	1.08	0.40	0.24	
5 - Sodium Silicate						
	17	3.20	3.20	0.68	0.32	
	18	1.44	1.20	0.48	0.24	
	19	2.00	2.40	0.72	0.92	
	20	1.04	2.80	1.44	0.80	
6- Alkyl-Silane 20% silane						
in water	41	0.56	0.00	0.00	0.00	
	27	0.96	0.36	0.20	0.00	
	28	0.96	0.68	0.32	0.00	
	29	1.04	0.56	0.24	0.28	
7- Alkyl-Alkoxy Silane in						
mineral spirits	42	1.28	0.28	0.00	0.24	
	30	1.76	1.04	0.28	0.28	
	31	1.60	1.04	0.36	0.00	
	32	2.00	1.08	0.64	0.28	
8- Oligomeric 10%						
Siloxane over	33	0.56	0.00	0.00	0.00	
linseed oil	34	2.00	0.00	0.00	0.28	
	35	0.36	0.00	0.00	0.00	
	36	0.32	0.00	0.00	0.00	

Table A4.3

FM 1835 - ABILENE
CHLORIDE CONTENTS (LB/CU. YD.)

SAMPLE		SAMPLE DEPTH (IN.)				
Grid	Number	0.0 - 0.5	0.5 - 1.0	1.0 - 1.5	1.5 - 2.0	REMARKS
9- Linseed Oil 50-70%						
	37	1.08	1.04	0.00	0.00	
	38	1.16	0.28	0.00	0.00	
	39	0.32	0.00	0.00	0.00	
	40	0.36	0.24	0.00	0.00	
Bent 5- Substructure						
	21	2.80	1.20	0.40	0.28	
	22	2.80	1.44	0.68	0.40	
	23	3.60	3.60	0.72	0.32	
	24	7.60	18.80	3.60	0.84	
	25	5.20	3.60	2.04	1.92	
	26	3.60	3.20	1.76	0.00	

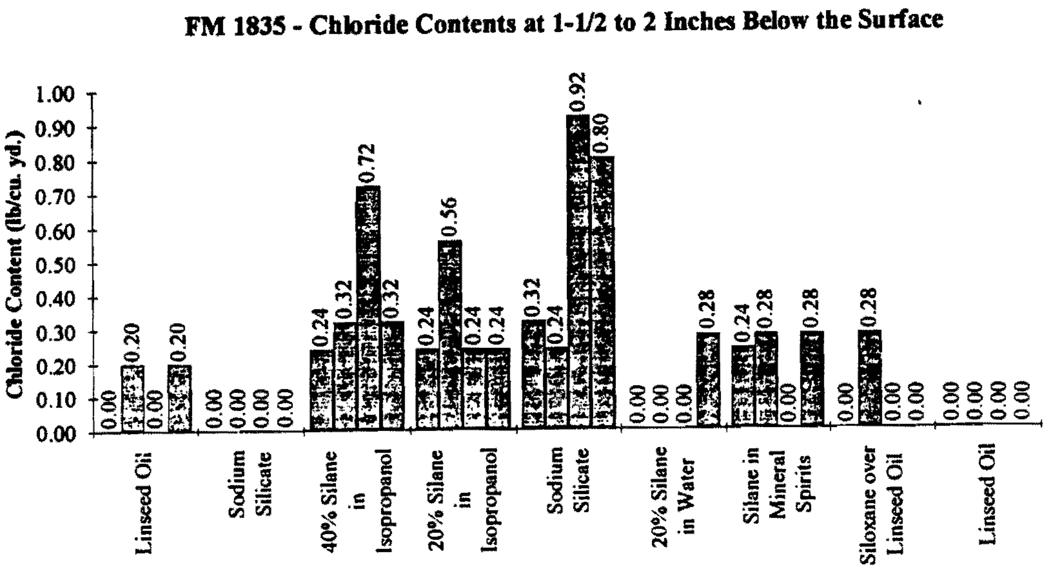


Figure A4.12

APPENDIX A5
34TH STREET OVERPASS AT IH-27, AMARILLO DISTRICT

A5.1 BACKGROUND

The two span bridge carrying the four lanes of 34th Street over IH-27 in Amarillo was built in 1989. The structure consists of a dense concrete overlay reinforced with epoxy-coated reinforcement placed over precast prestressed panels on precast box beams. According to the 1990 Federal BRINSAP data, the average daily traffic is 44000 vehicles, with 9 percent of that volume truck traffic. The structure is exposed to deicers during the winter. Also, linseed oil was applied to the deck during construction as standard procedure.

A5.2 TESTING

A5.2.1 Half-Cell Potential Testing

Because epoxy-coated reinforcement was used in this structure, the half-cell corrosion potential test could not be performed.

A5.2.2 Chloride Content Testing

Twenty-five locations on the bridge were sampled to determine the chloride content of the deck and sidewalk. As the bridge showed much cracking, both cracked and uncracked locations were sampled to determine the effect of the extensive surface cracking on the chloride contamination of the deck. Samples were also taken from both the wheelpath and centerline areas of the deck to determine the effect of the traffic exposure on the chloride penetration.

In general, this deck showed a very high chloride contamination for a structure only three years old. At 1-1/2 to 2 inches from the surface, the chloride content ranged from 0.28 to 3.2 pounds per cubic yard. As shown in Figures A5.1 there was no apparent correlation between centerline or wheelpath exposure and chloride contamination. Figure A5.2 shows, however, that there was a correlation between cracking and chloride contamination. The samples taken at cracks showed higher chloride contents than those taken from uncracked areas. The difference became more pronounced with sample depth. Although the depths of the cracks sampled is not known and could not be determined from surface evaluation, the existence of surface cracks seems to be a significant influence on chloride penetration and thus on corrosion performance.

A5.2.3 Permeability Testing

Ten samples were tested to determine the AASHTO T277 permeability of the concrete. The samples included: three samples of the overlay concrete at crack locations, four samples of the overlay at uncracked locations, and three uncracked samples of the overlay concrete with the top 1/2 inch removed. This selection was used to determine both the effect of the surface cracking and the effect of the finished surface layer on the permeability of the samples.

As the data shows, the permeabilities ranged from 1036 to 2498 coulombs. These values are all classified as "low" by the AASHTO T277 standard, reflecting the use of the dense concrete overlay. The permeabilities appear independent of the existence of cracks in the surface of the concrete overlay. Samples of cores with both full and partial depth cracks were sectioned lengthwise after testing to determine the actual depth of the cracks. The inspection showed that the cracks were internally sealed with dirt and silt at less than 1/4 inch below the surface. This could explain the negligible effect of surface cracking on permeability. Comparison between samples taken from the upper and lower portions of cores shows that the lower portions have permeabilities 100 to 850 coulombs higher than the upper portions. This could be due to the effect of the finishing operations on the surface permeability of the material.

A5.2.4 Crack Mapping

Both tested spans were mapped to show the locations of major cracks. Both spans showed longitudinal and transverse cracking over the entire deck. Using a rebar locator, it was determined that the longitudinal cracks typically occurred over the longitudinal reinforcement. A number of surface defects, worn areas, and small holes were found.

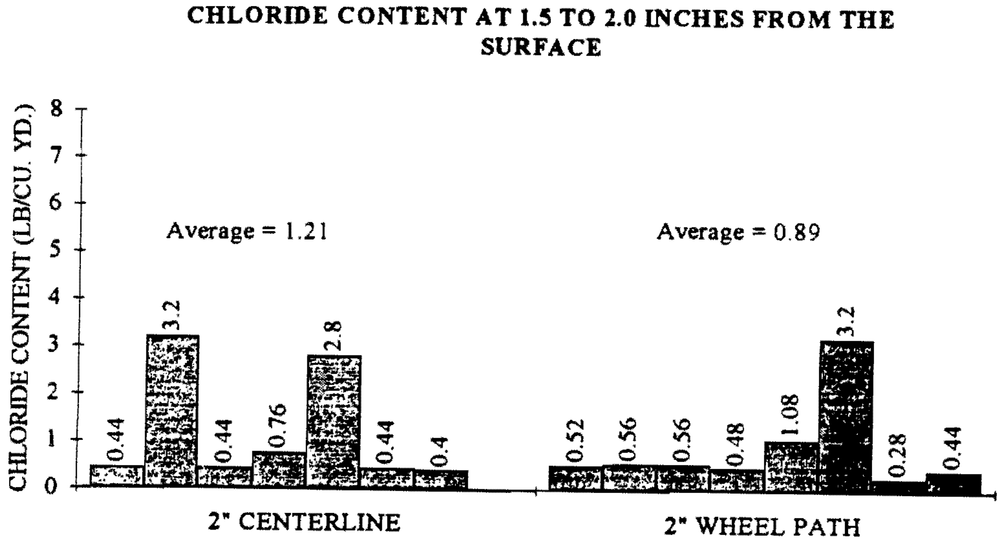


Figure A5.1 Chloride Contents of Wheelpath and Centerline Exposure

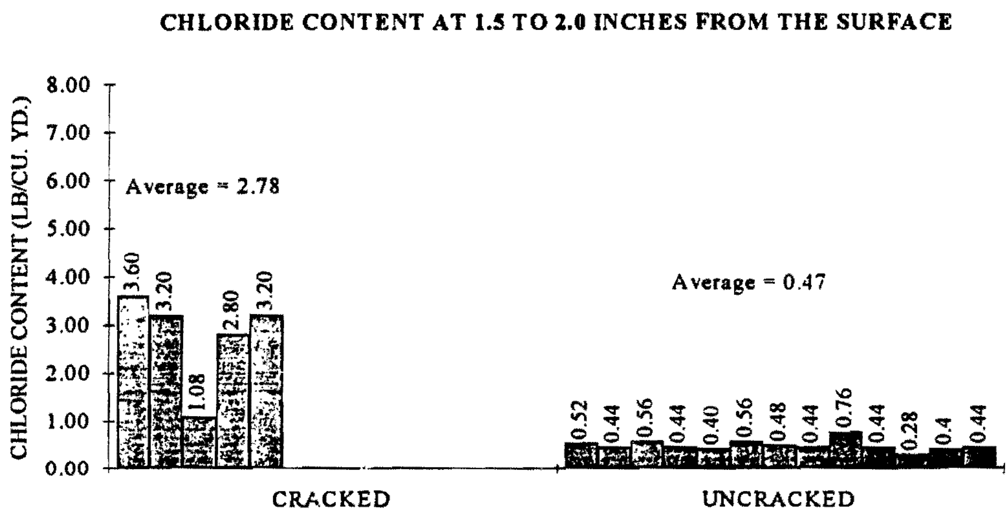


Figure A5.2 Chloride Contents of Cracked and Uncracked Locations

A5.2.5 Strength Testing

A sample removed from this bridge for strength testing had a compressive strength of 4894 psi.

A5.2.6 Delamination Testing

The sounding of the deck for delaminations revealed an approximately 64 ft^2 delaminated area at the east end of span 1. No other delaminations were noted.

A5.2.7 Petrographic analysis

One core from the bridge was examined to determine the characteristics of the air void system. The results are as follows:

Note: Recommended values from the American Concrete Institute Manual of Concrete Practice (216) are included in parenthesis after the results.

- Air Content 5.7% (>5.0% ± 1.5%)
- Voids per Inch 6.6 (>7.5)
- Specific Surface 468 in^2/in^3 (>600 in^2/in^3)
- Spacing Factor 0.011 (<0.008)

Although the air content is acceptable, all of the other parameters of the air void system were substandard. This sample had the lowest air void specific surface of all the bridges visited during this portion of the program. These parameters indicate that although the volume of entrained air in the sample is acceptable, it is concentrated in large voids spaced too far apart to provide adequate protection against freeze-thaw degradation.

A5.3 CONCLUSIONS

Although the dense concrete overlay results in a low permeability for the concrete in this structure, the chloride contents show that the excessive cracking observed in the deck is leading to high chloride penetration into the deck. Areas of the deck already show chloride content at the level of the reinforcement to be above the threshold for corrosion. As such, this bridge offers an excellent opportunity to determine the field effectiveness of the epoxy coating on the reinforcement to protect against corrosion. Monitoring should continue to characterize the corrosion performance of the epoxy-coated reinforcement.

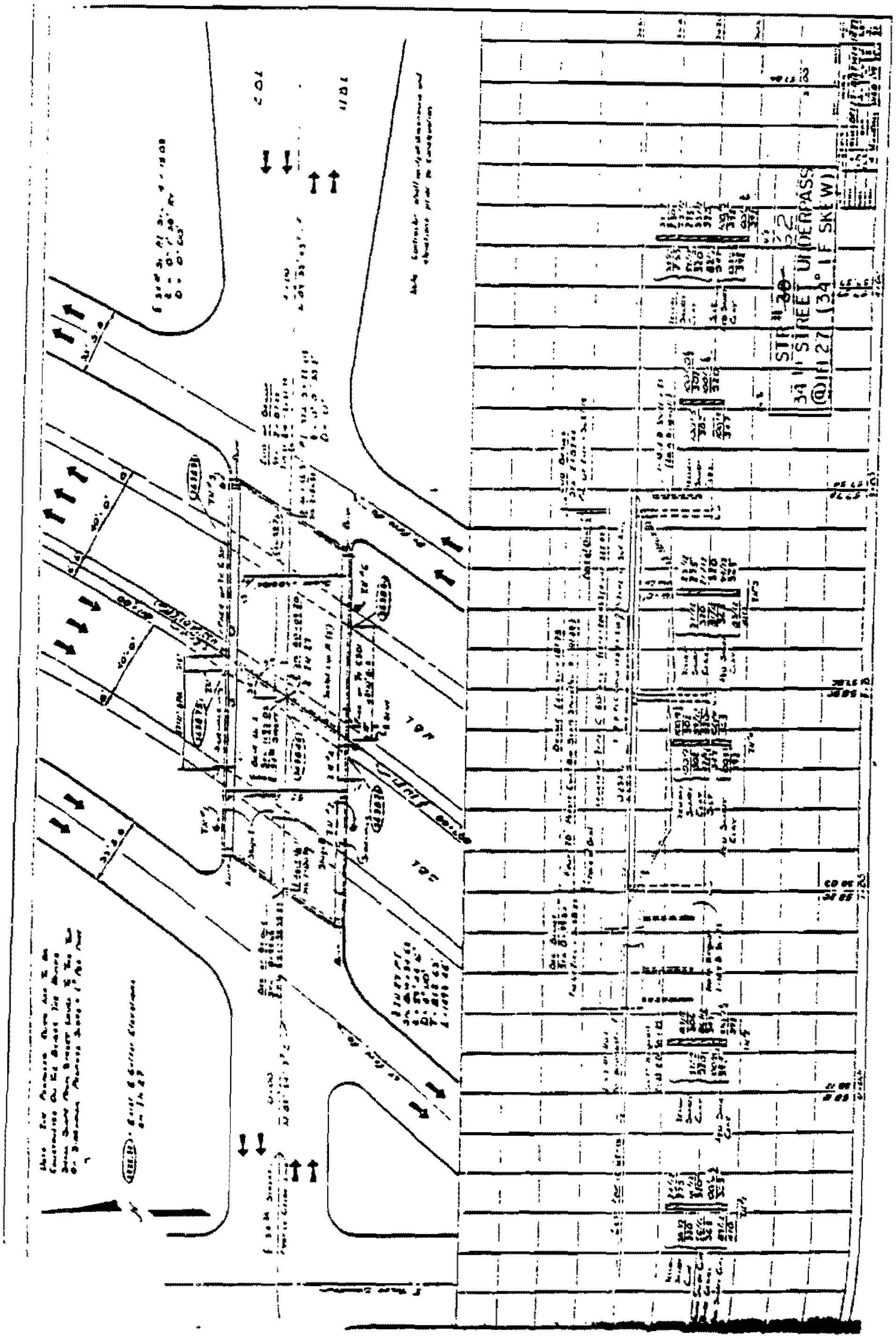
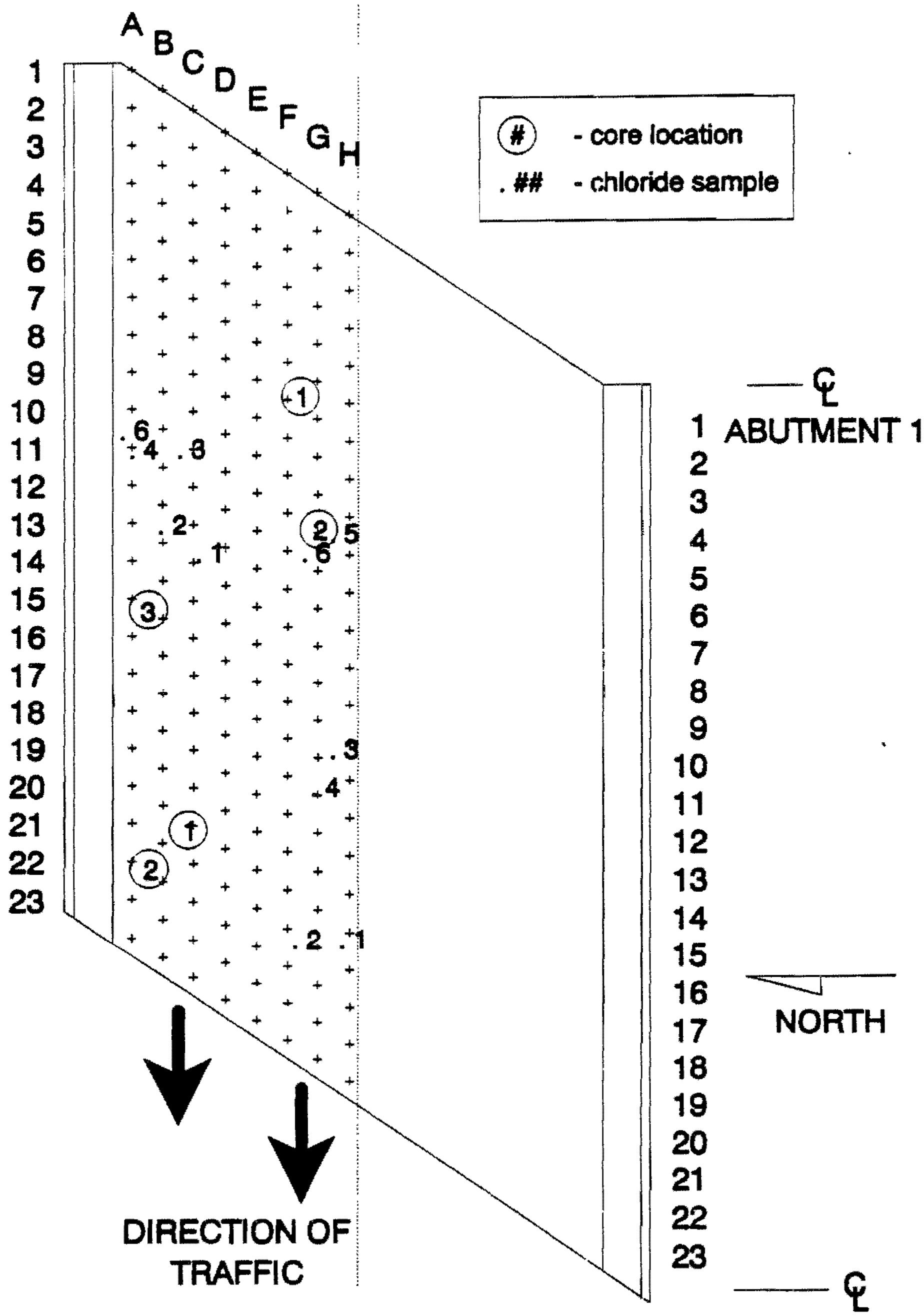


Figure A5.3

34th STREET OVERPASS AT IH-27
SAMPLE LOCATIONS



GRID 1,3

Figure A5.4

34th STREET OVERPASS AT IH-27
CRACK AND DELAMINATION MAPPING

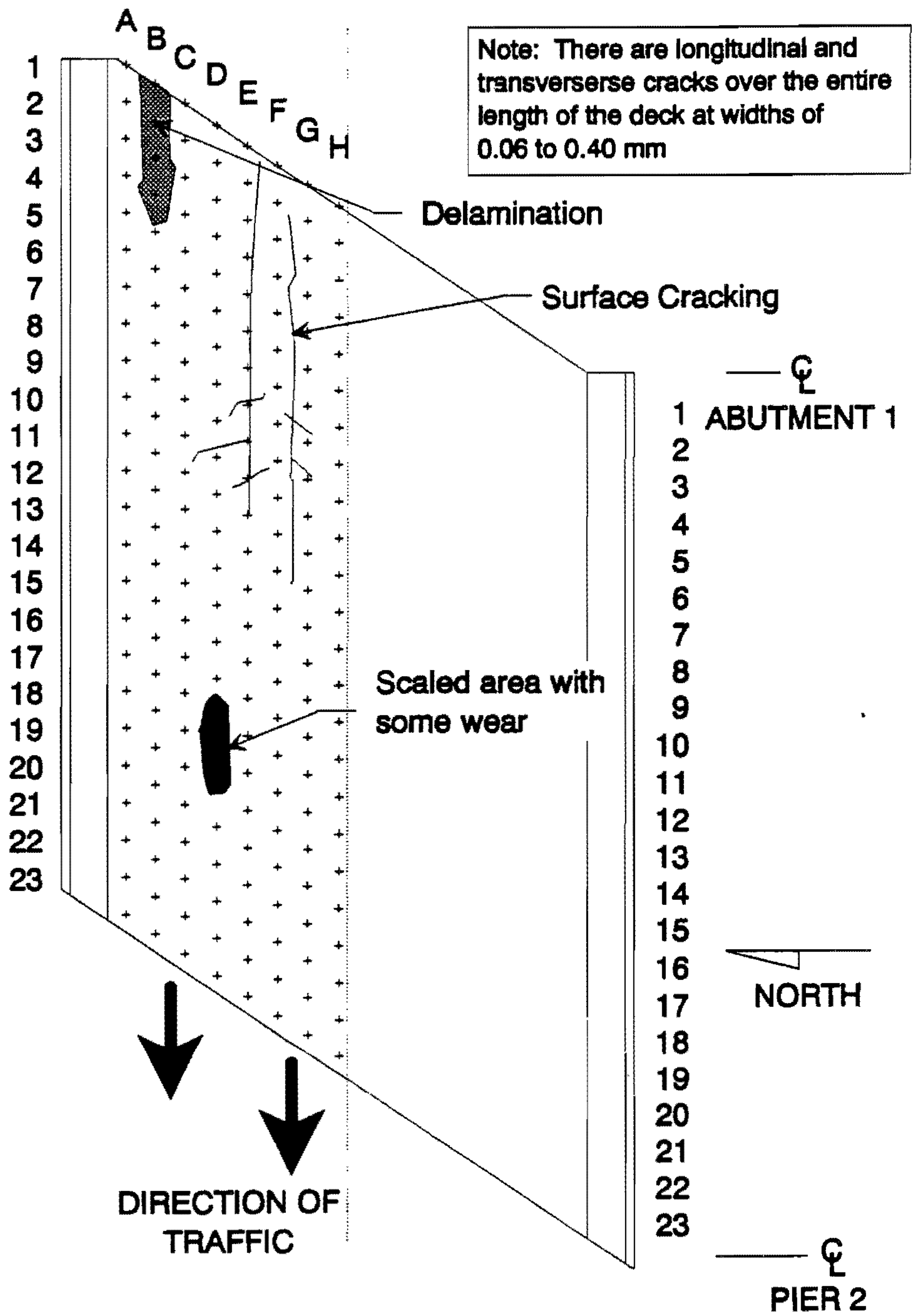


Figure A5.5

34th STREET OVERPASS AT IH-27
SAMPLE LOCATIONS

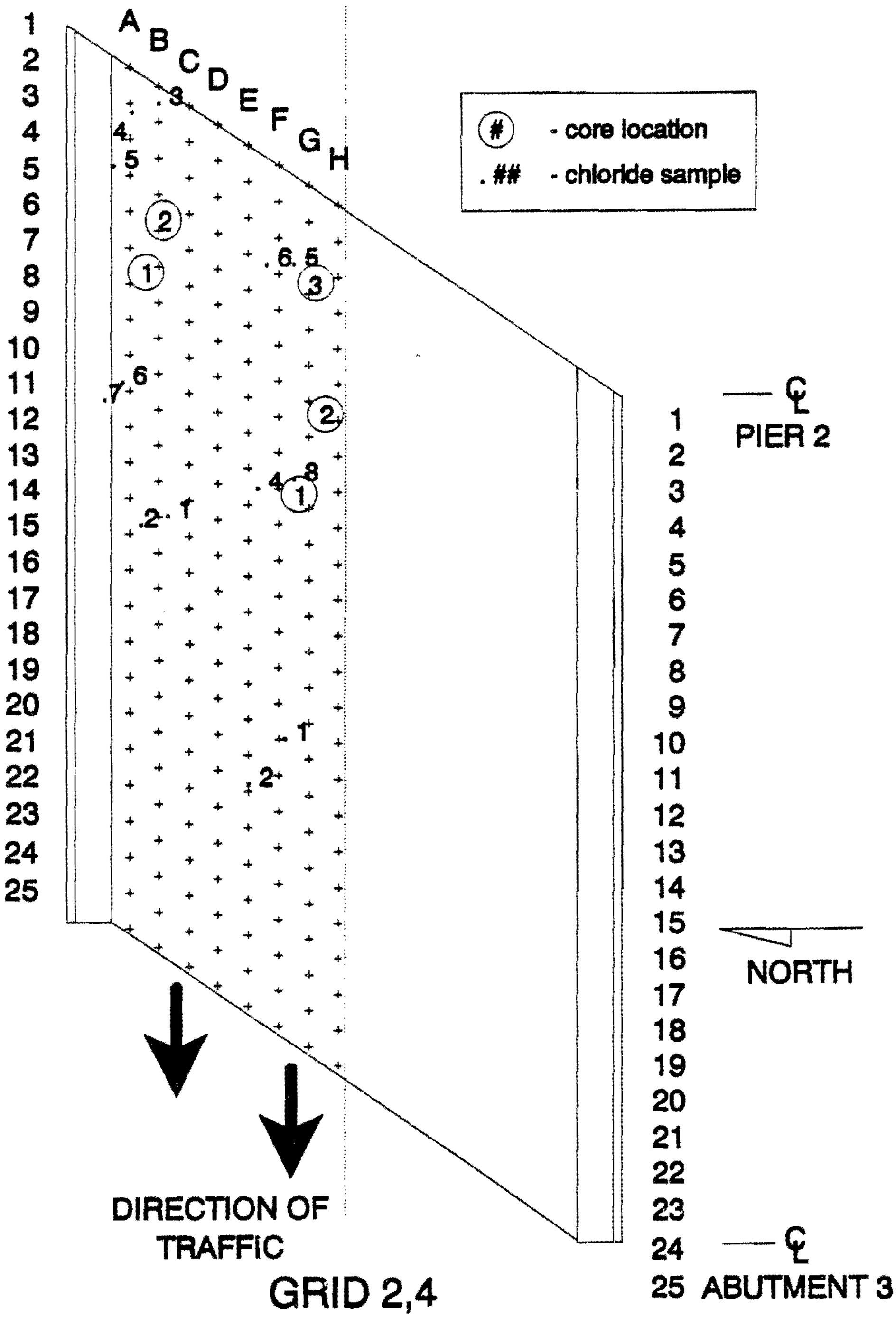


Figure A5.6

34th STREET OVERPASS AT IH-27

CRACK AND DELAMINATION MAPPING

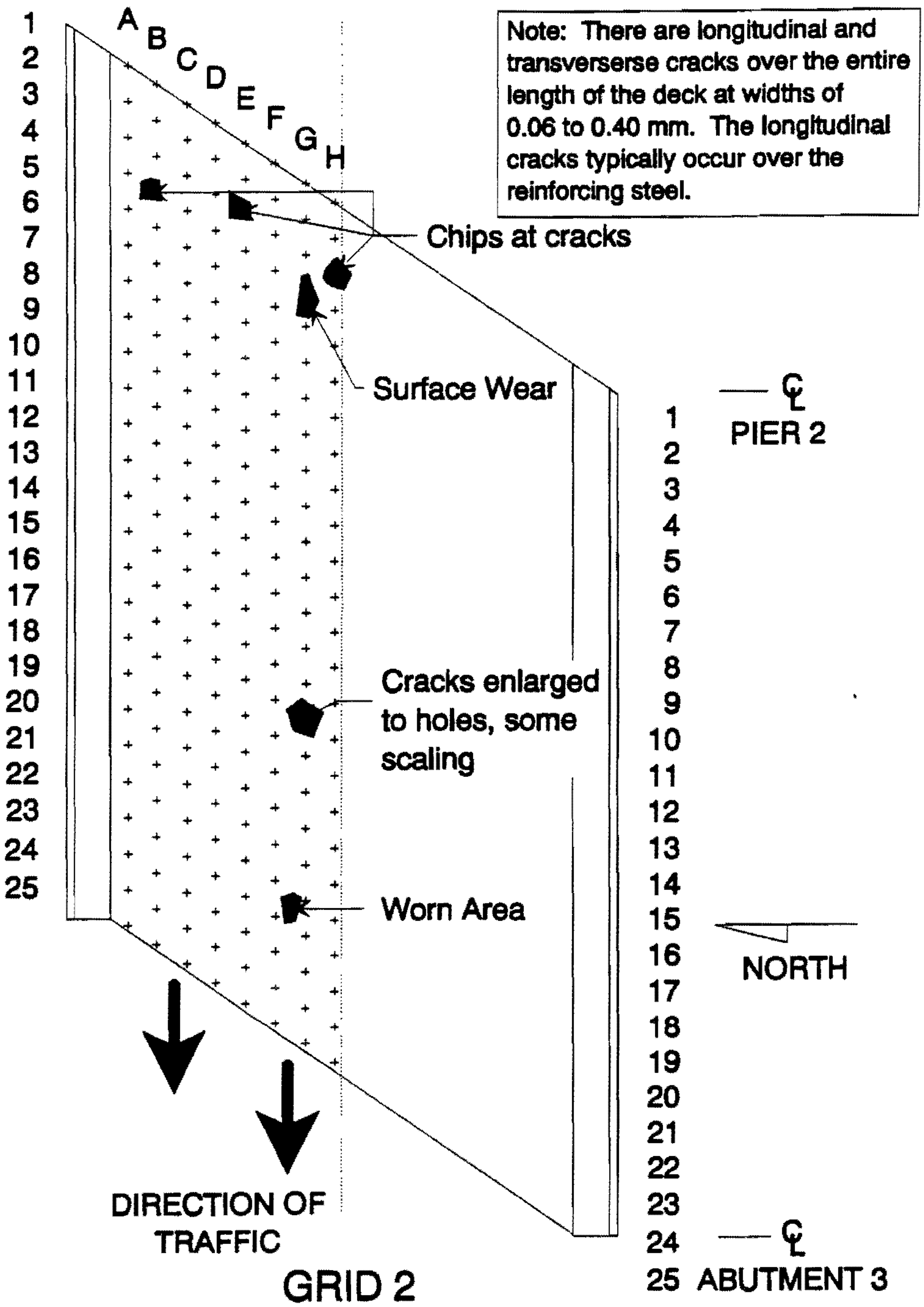


Figure A5.7

Table A5.1

34th Street Overpass at IH27

Epoxy Coated Reinforcing Steel

Concrete Permeabilities (Coulombs)

	Finished Surface Intact		Surface Removed
Sample	**Cracked**	**Uncracked**	**Uncracked**
5-1-EBT-19-C1A		1607	
5-1-EBT-19-C1B			2159
5-1-EBT-21-C2A	1036		
5-1-EBT-21-C2B			1895
5-1-EBT-14-C3	2108		
5-2-EBT-8-C1		2498	
5-2-EBT-6-C2		1041	
5-3-EBP-22-C1A		1757	
5-3-EBP-22-C1B			1633
5-4-EBP-11-C2	1636		

Table A5.2

34TH STREET OVERPASS AT IH 27 - AMARILLO
CHLORIDE CONTENTS (LB/CU. YD)

SAMPLE Number	SAMPLE DEPTH (IN) 0.0 - 0.5	0.5 - 1.0	1.0 - 1.5	1.5 - 2.0	Remarks
GRID # 1					
1	6.80	1.76	0.56	0.52	wheelpath, no cracks
2	2.40	0.80	0.40	0.44	Centerline (CL), no crack
3	3.60	1.44	0.44	0.56	wheelpath, no crack
4	5.20	1.88	0.56	0.44	no crack, shallow depression
5	N/A	N/A	N/A	N/A	
6	2.40	0.88	1.60	0.40	gutter
GRID # 2					
1	10.80	7.20	4.40	3.60	on a crack
2	9.20	3.20	0.76	0.40	on a good section
3	5.60	4.00	3.20	3.20	CL, much cracking
4	7.60	3.20	1.44	0.56	wheelpath, no cracks
5	3.60	3.20	2.00	1.72	in gutter
6	5.20	3.60	2.40	1.40	in gutter
7	5.60	5.60	1.72	0.40	on sidewalk
GRID # 3					
1	6.80	0.92	0.52	0.48	wheelpath, no cracks, worn area
2	5.60	0.56	N/A	0.44	CL worn area, no cracks
3	4.40	1.92	1.92	1.08	wheelpath with a crack
4	2.00	1.60	1.04	0.76	center, worn, no cracks
5	6.80	3.60	1.92	0.56	
6	8.80	4.00	1.32	0.80	
GRID # 4					
1	5.60	4.40	3.20	2.80	CL, on a crack
2	6.00	4.80	4.40	3.20	wheelpath on a crack
3	6.80	0.64	0.36	0.44	center, no cracks
4	7.60	3.60	0.48	0.28	wheelpath, no crack
5	5.20	0.56	0.36	0.40	CL, no crack
6	6.40	0.88	0.52	0.44	wheelpath, no crack, some wear

CHLORIDE CONTENT AT 0.5 TO 1.0 INCHES BELOW THE SURFACE

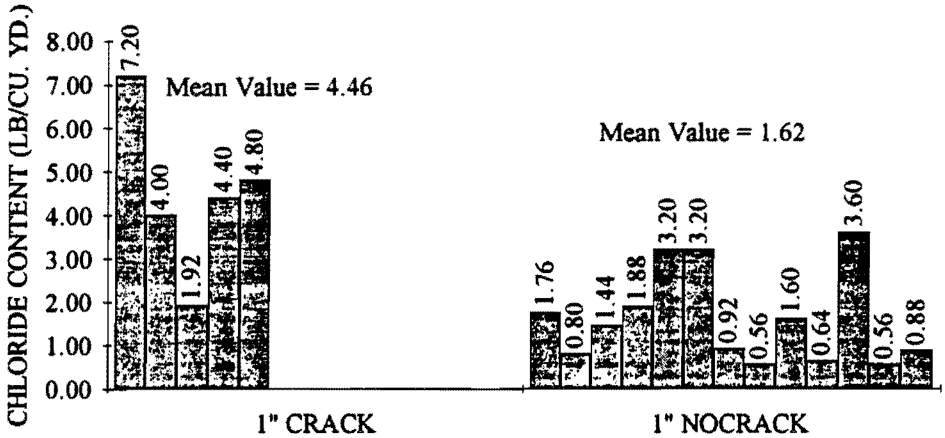


Figure A5.8

CHLORIDE CONTENT AT 1.0 TO 1.5 INCHES FROM THE SURFACE

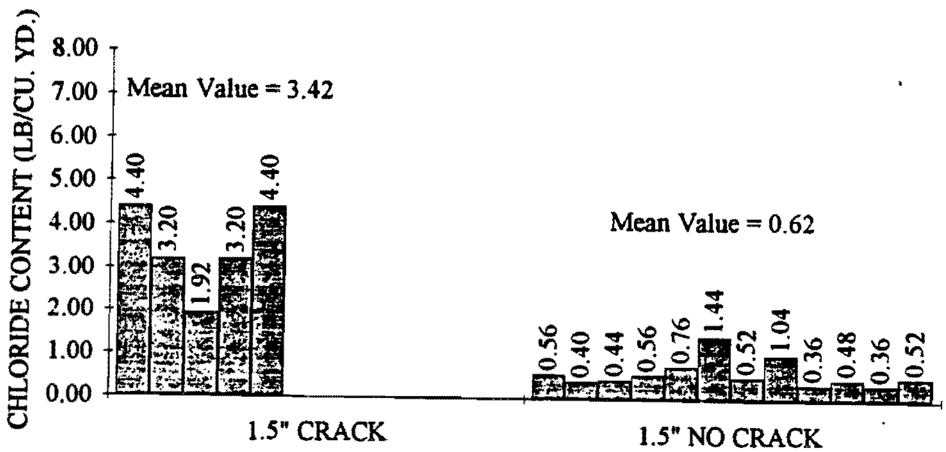


Figure A5.9

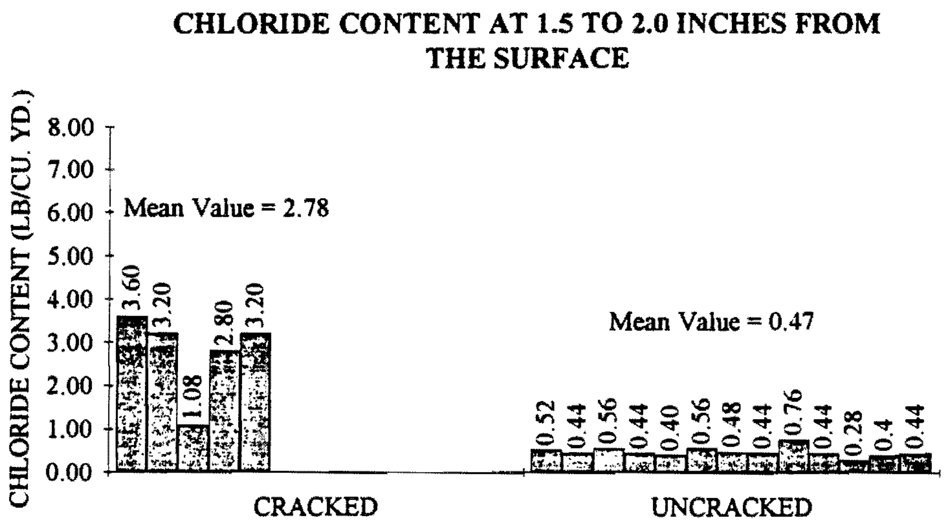


Figure A5.10

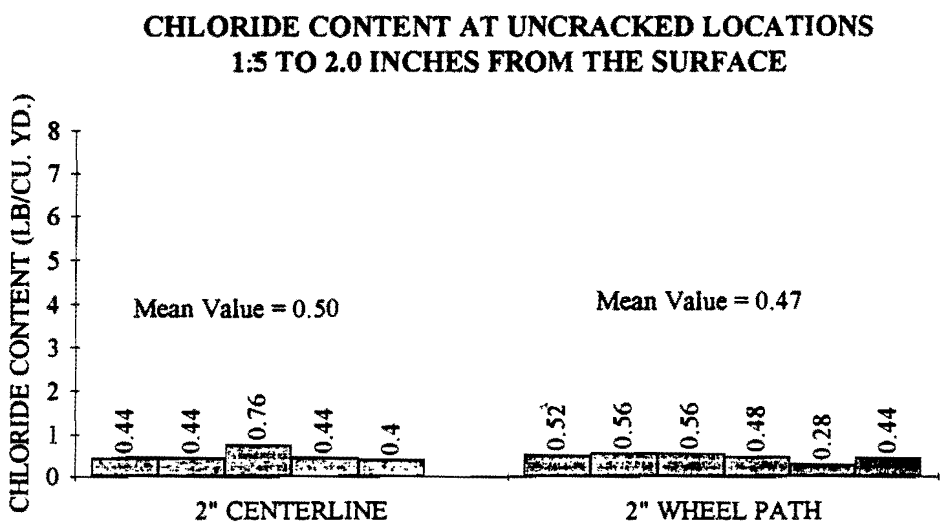


Figure A5.11

APPENDIX A6
IH-40 OVERPASS AT THE A.T. & S.F. RAILROAD, AMARILLO DISTRICT

A6.1 BACKGROUND

The eight lane bridge carrying the east and west bound lanes of IH 40 over the A.T. & S.F. railroad yard in Amarillo was built in 1965. In 1982, the deck was milled and a two-inch dense concrete overlay was placed on the bridge. The 938-foot-long bridge carries an ADT of 35,890 vehicles, of which 10 percent is truck traffic. The structure is composed of a concrete deck on steel plate girders which are in turn supported on concrete piers. The structure is exposed to deicers during the winter months.

When surveyed for this project, the leftmost westbound lane and the shoulder were already closed to traffic because of ramp and interchange work at the westernmost end of the bridge. The survey was conducted entirely within these closed lanes in order to take advantage of the in place traffic control which allowed for a detailed investigation with minimal disruption to traffic.

A6.2 TESTING

A6.2.1 Half-Cell Potential Testing

Half-cell corrosion potentials were measured in all of the spans in the survey. In all, 77 percent of the surveyed area shows 90 percent probability of no corrosion, less than 1 percent shows a 90 percent probability of corrosion, and the remaining 22 percent is classified as having uncertain corrosion activity.

There is an apparent correlation of transverse cracking and intermediate corrosion potentials, especially in grid 3. As before, most of the transverse cracks are in areas of intermediate potentials, but not all areas of intermediate potentials are in cracked regions. This is logical, as a crack is not necessary for corrosion, but corrosion may occur more easily at a crack. It can also be seen that all of the highest chloride contents occurred in areas of intermediate corrosion potential, and that the lowest chloride contents were found in areas of no probable corrosion.

A6.2.2 Chloride Content Testing

Twenty-four locations were sampled for chloride content on this bridge. In general, the upper inch of the deck had high chloride-concentrations. The concentrations in the 1/2 to 1 inch depth sampling region ranged from 0.88 to 8.80 lb/yd^3. Samples from the 1-1/2 to 2 inch depth had concentrations ranging from 0.24 to 5.6 lb/yd^3. It should be noted that the sample locations with high chloride concentrations in the 1-1/2 to 2 inch sampling region were all on or very close to cracks. In the uncracked locations, the chloride concentrations in the 1-1/2 to 2 inch region only ranged from 0.24 to 1.2 lb/yd^3.

A6.2.3 Permeability Testing

Nine samples were tested to determine the permeability of the concrete. Only samples from the dense concrete overlay were tested, as no suitable samples of the original concrete were obtained. Six samples were tested with the finished traffic surface left intact, four of which showed surface cracking and the others showed no visible cracking. Three samples were tested with the finished traffic surface removed. The cracked and uncracked specimens with the finished surface intact had permeabilities ranging from 1050 to 2423 coulombs. There was no obvious effect of surface cracking on permeability, as the uncracked permeabilities were no higher or lower than those of the cracked specimens. The effect of the finished surface, however, was obvious. All of the specimens tested with the finished surface removed had higher permeabilities than the other specimens. The specimens with the finished surface removed had permeabilities ranging from 2803 to 3749 coulombs, which are still classified as low permeability.

A6.2.4 Crack Mapping

All surveyed spans were observed for cracking. Transverse and longitudinal cracks were seen throughout the bridge. Many of the transverse cracks were aligned with the construction joints of the underlying concrete that were visible in the median of the structure. The cracks varied in width from 0.45 mm to greater than 1.4 mm. No areas of deck distress were noted except for small regions of cracking and spalling at the deck finger joints.

A6.2.5 Strength Testing

No cores suitable for strength testing were retrieved during the sampling of the bridge.

A6.2.6 Delamination Testing

No delaminations were found on the deck.

A6.2.7 Petrographic analysis

One core from the bridge was examined to determine the characteristics of the air void system. The results are as follows:
Note: Recommended values from the American Concrete Institute Manual of Concrete Practice (216) are included in parenthesis after the results.

• Air Content	4.5%	(>5.0% ± 1.5%)
• Voids per Inch	6.2	(>7.5)
• Specific Surface	554 in^2/in^3	(>600 in^2/in^3)
• Spacing Factor	0.010	(<0.008)

Although this sample meets the minimum air content requirements, the other values are all below standard. Although no evidence of freeze-thaw damage was observed during the analysis, the air void system should be considered marginal.

In another analysis, some of the permeability samples which exhibited low permeabilities in spite of deep cracks were soaked in a penetrating methacrylate to determine the extent of the crack system. Although the cracks could be seen through the entire depth of the sample, the soaking revealed that the cracks were internally sealed with dirt, silt, and other materials at 1/4 to 1/2 inch below the surface.

A6.3 CONCLUSIONS

This structure was found to be in excellent condition. Very little active corrosion was detected, and no surface damage was noted. The cracks noted during the inspection seem to have no effect on the measured permeability but were seen to increase the amount of chloride that was able to penetrate the deck.

As there is minimal corrosion of the structure at this time, monitoring should continue to better characterize the initiation of corrosion in bridge decks.

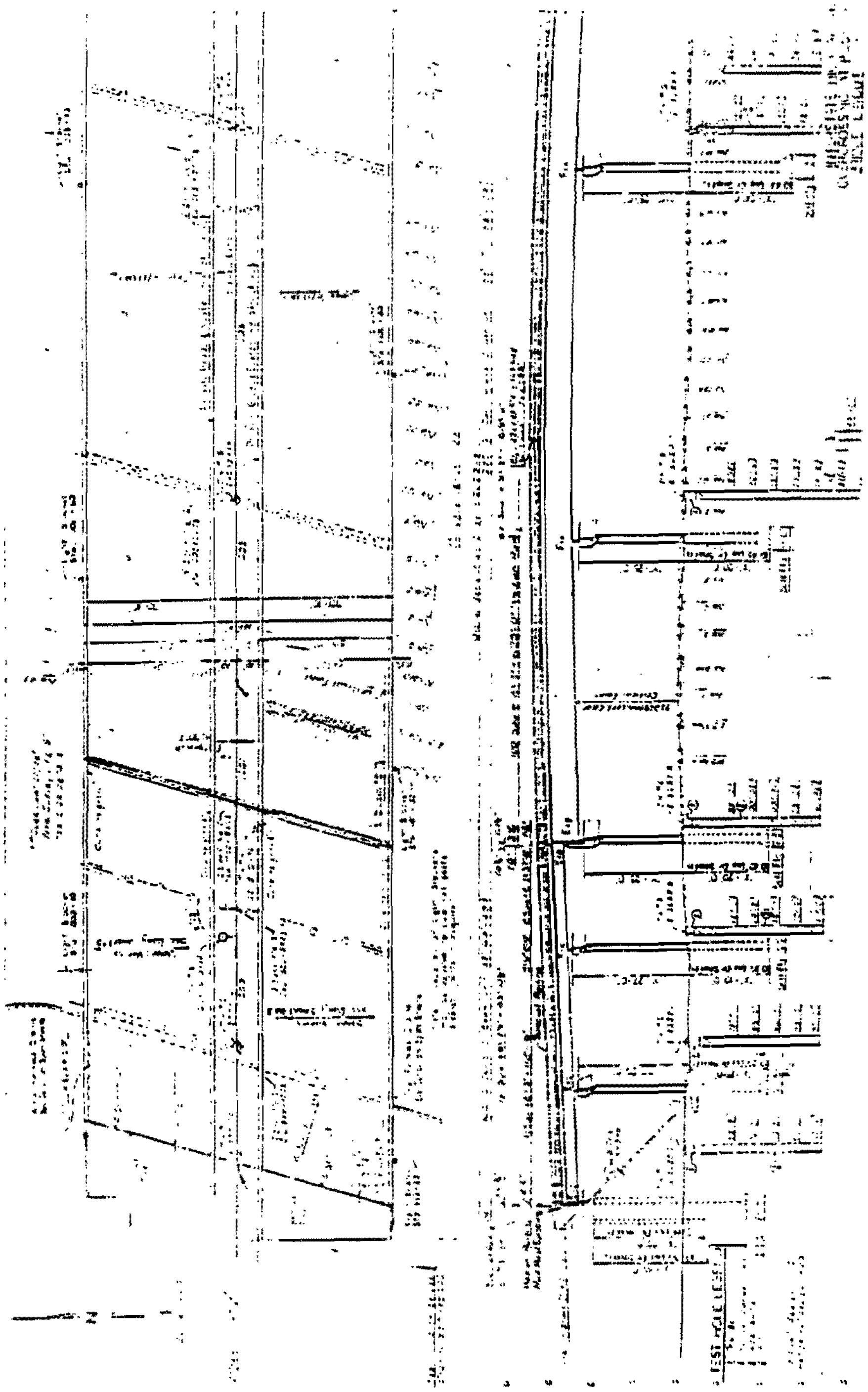


Figure A6.1

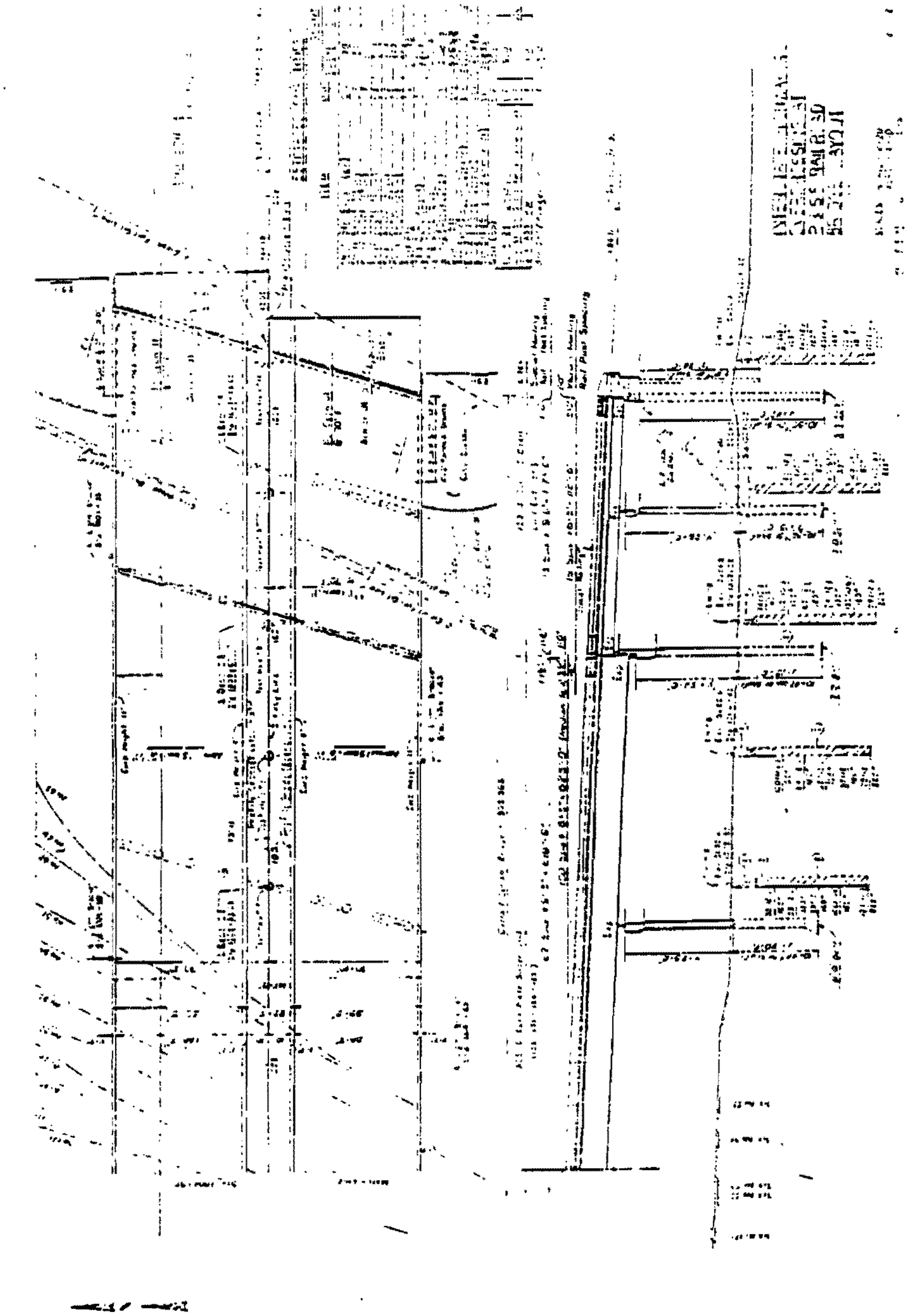


Figure A6.2

IH 40 at the A.T. & S.F. Railroad

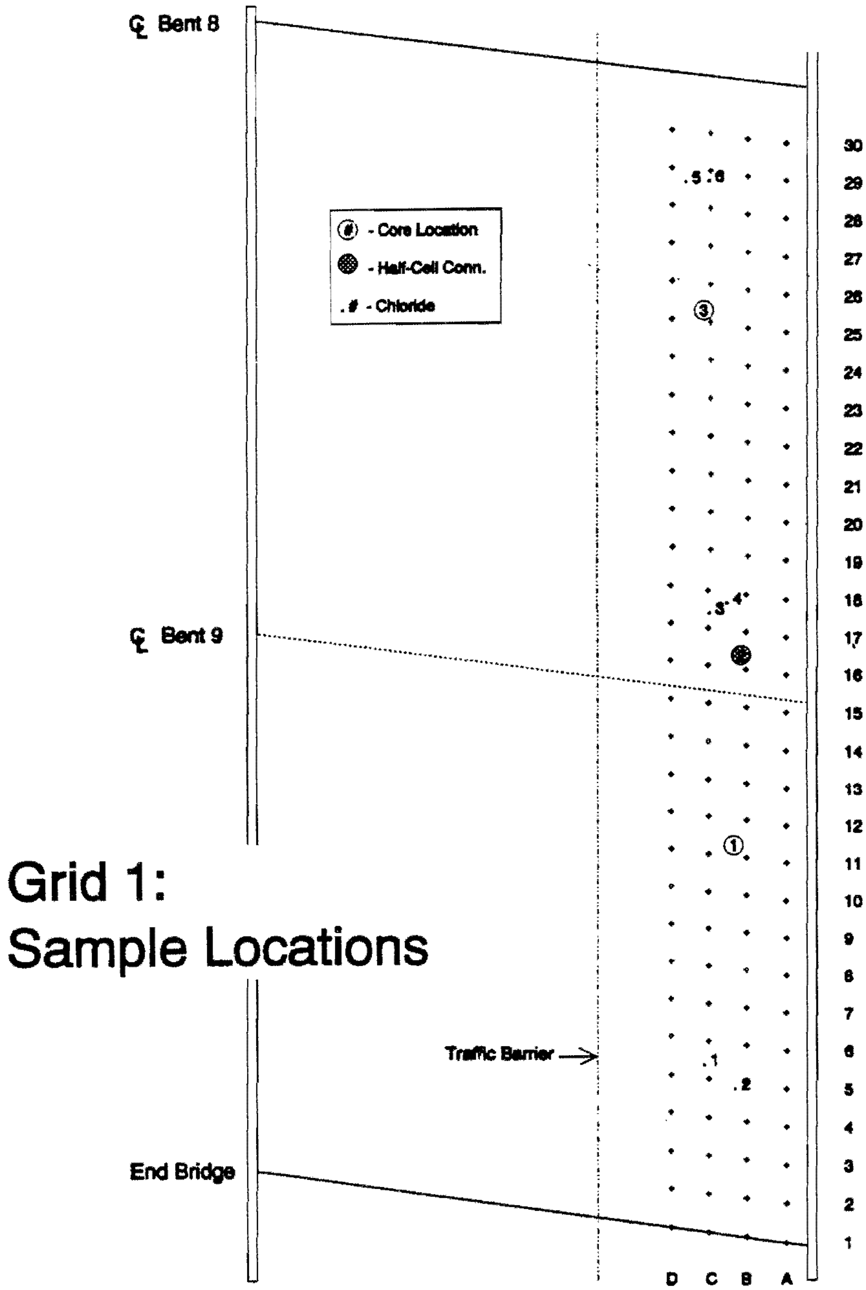


Figure A6.3

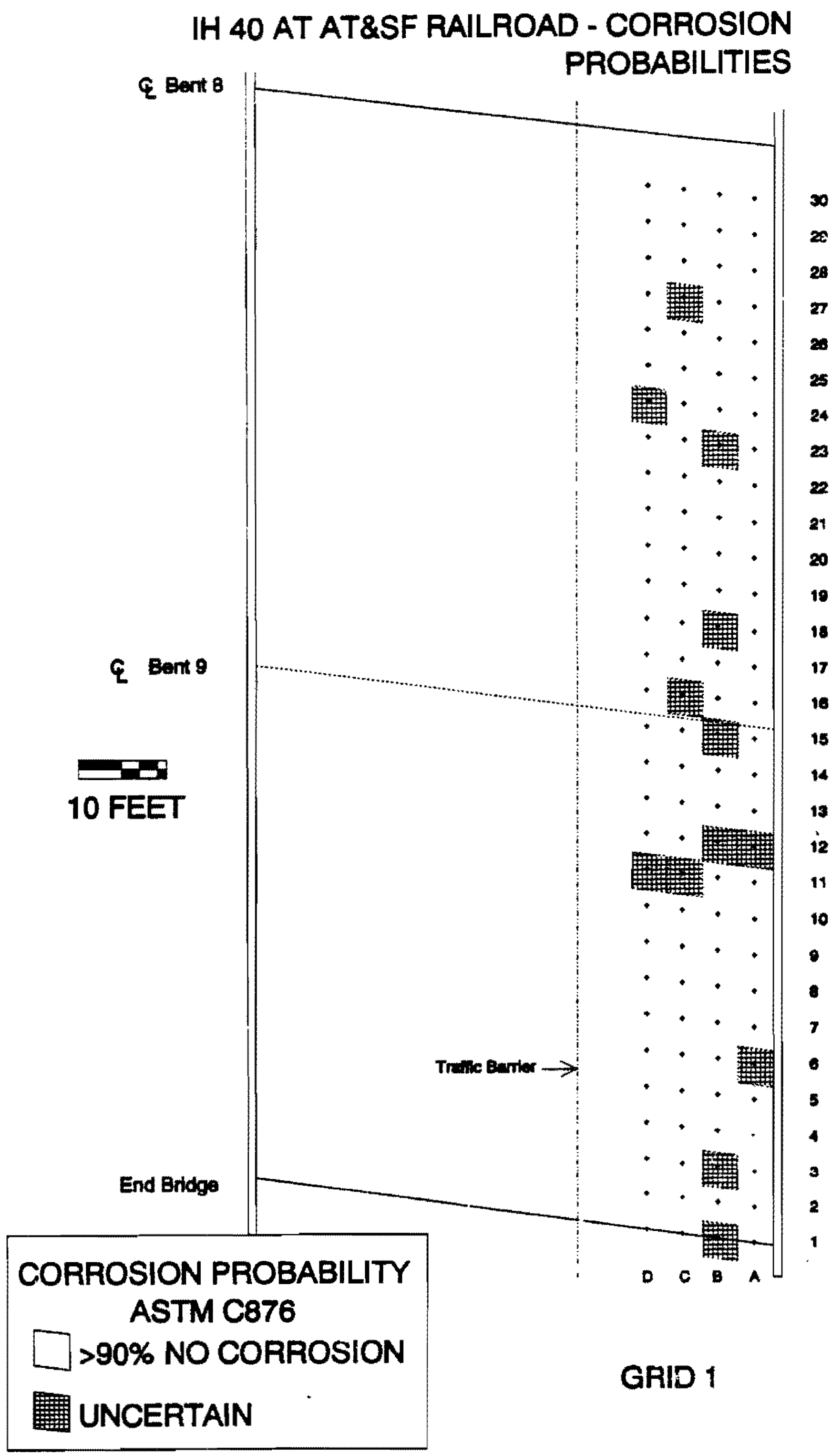

IH 40 AT AT&SF RAILROAD - CORROSION PROBABILITIES
℄ Bent 8
℄ Bent 9
10 FEET
Traffic Barrier
End Bridge
30
28
27
25
24
23
22
21
20
19
17
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
D C B A
CORROSION PROBABILITY ASTM C876
>90% NO CORROSION
UNCERTAIN
GRID 1

Figure A6.5

IH 40 at the A.T. & S.F. Railroad

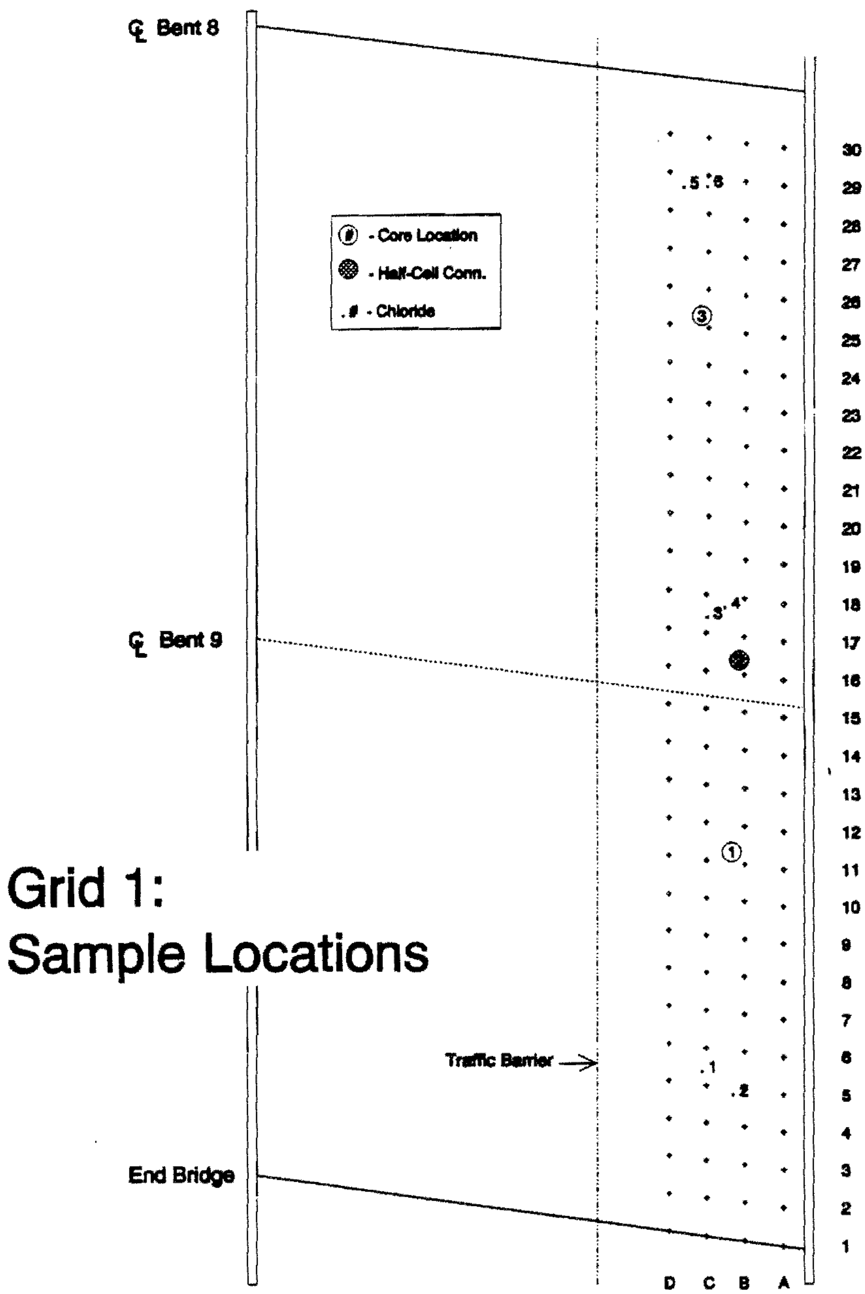


Figure A6.6

IH 40 at the A.T. & S.F. Railroad

Crack patterns and widths

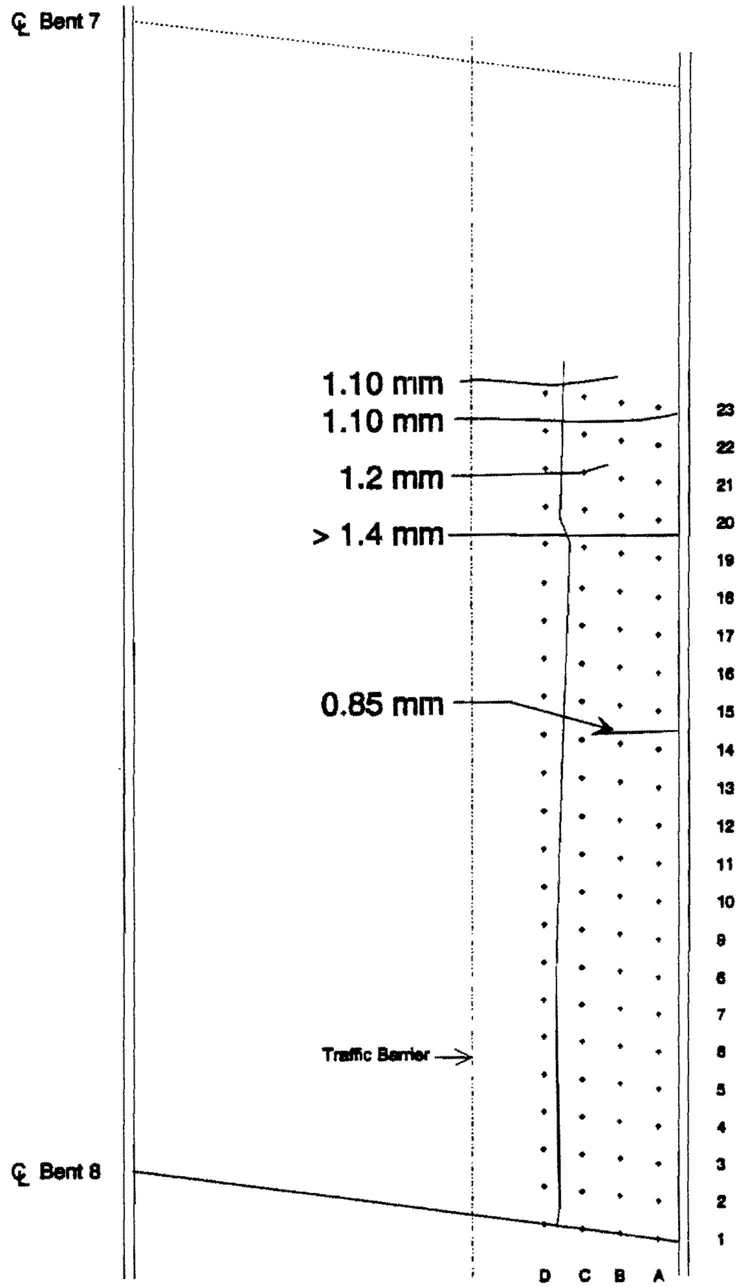


Figure A6.7

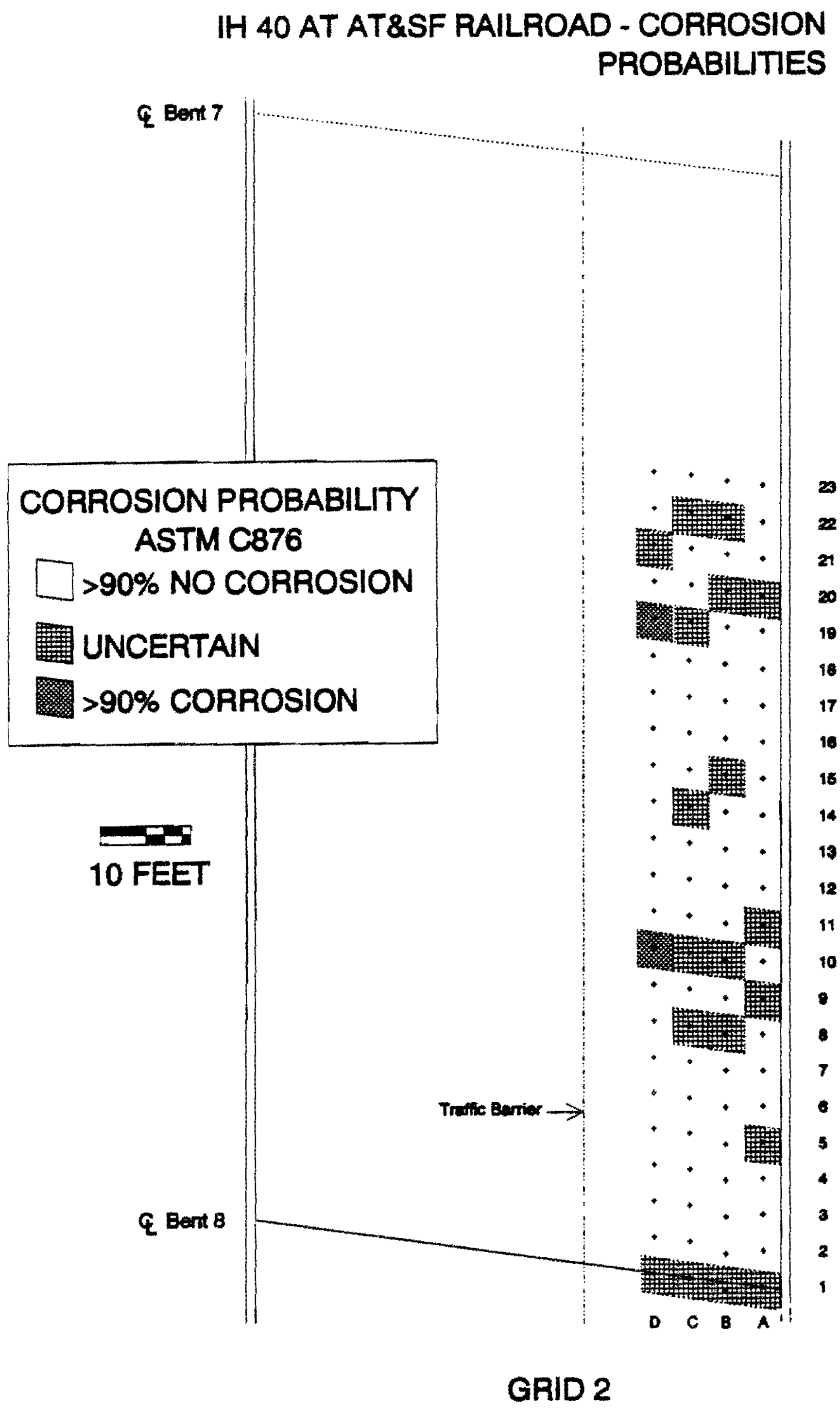

IH 40 AT AT&SF RAILROAD - CORROSION PROBABILITIES
℄ Bent 7
℄ Bent 8
CORROSION PROBABILITY
ASTM C876
>90% NO CORROSION
UNCERTAIN
>90% CORROSION
10 FEET
Traffic Barrier
23
22
21
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
D C B A
GRID 2

Figure A6.8

IH 40 at the A.T. & S.F. Railroad

Grid 3 : Sample Locations

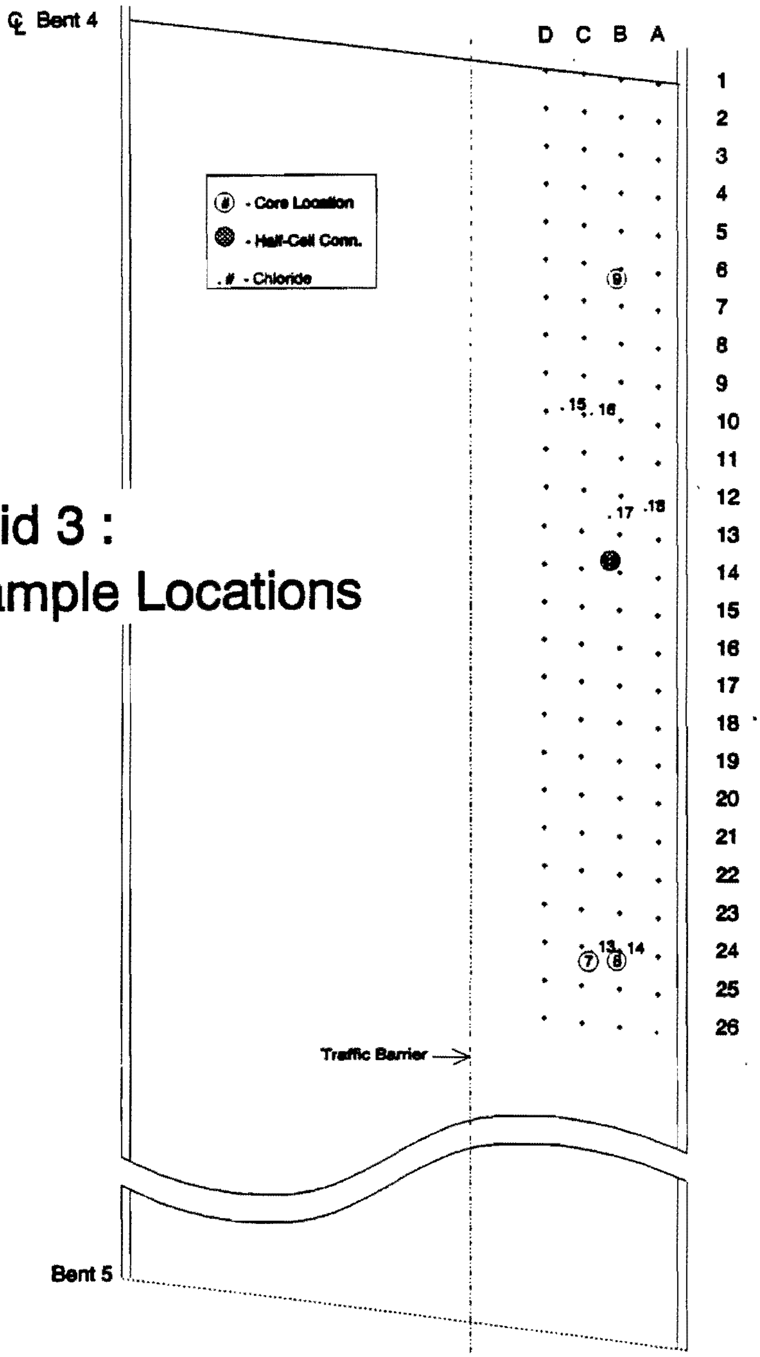


Figure A6.9

IH 40 at the A.T. & S.F. Railroad
Crack patterns and widths

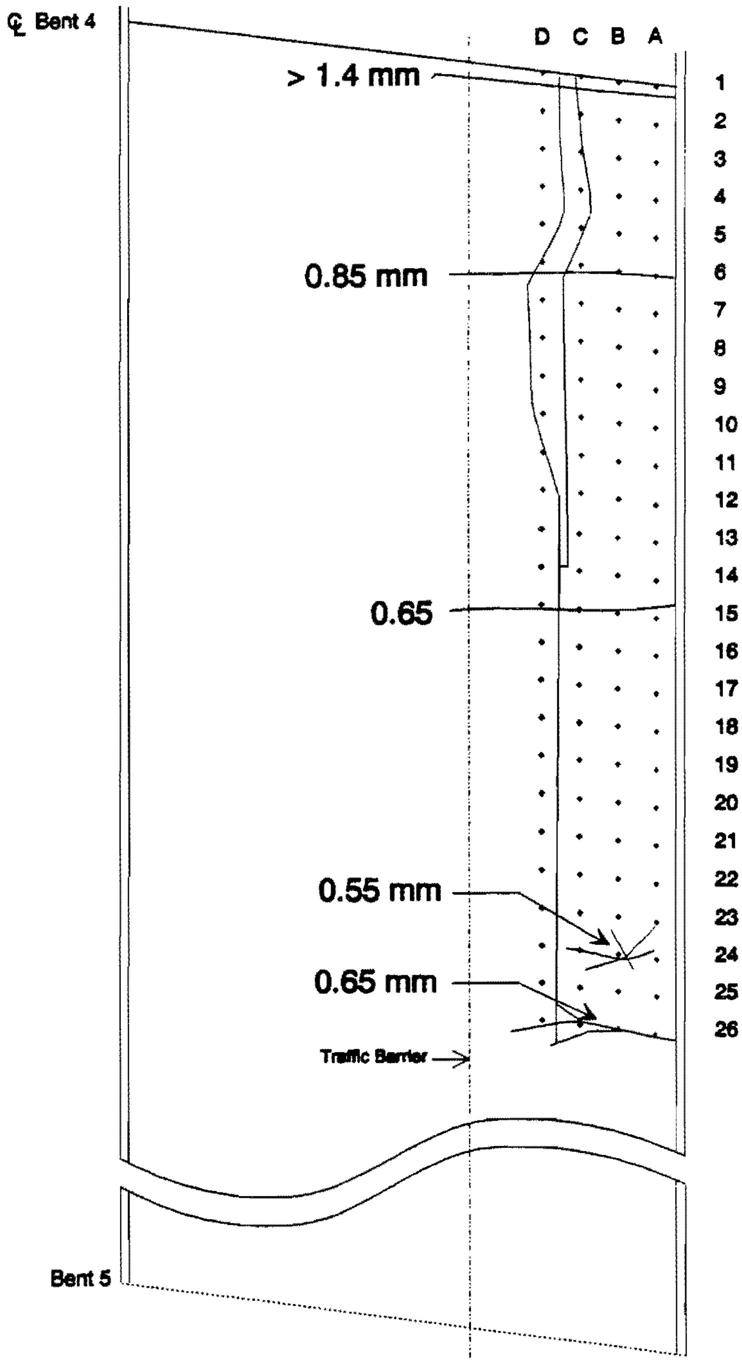


Figure A6.10

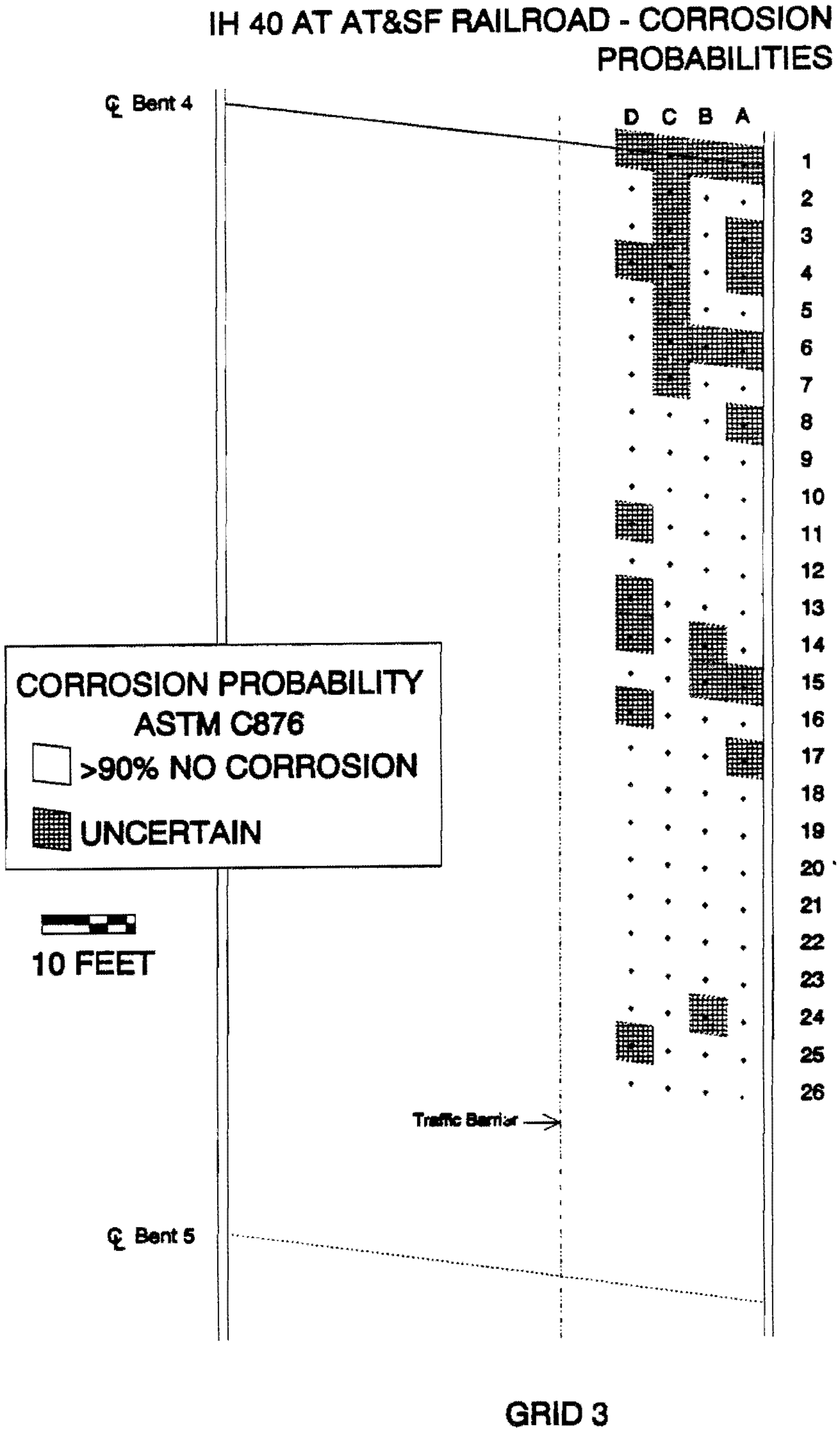
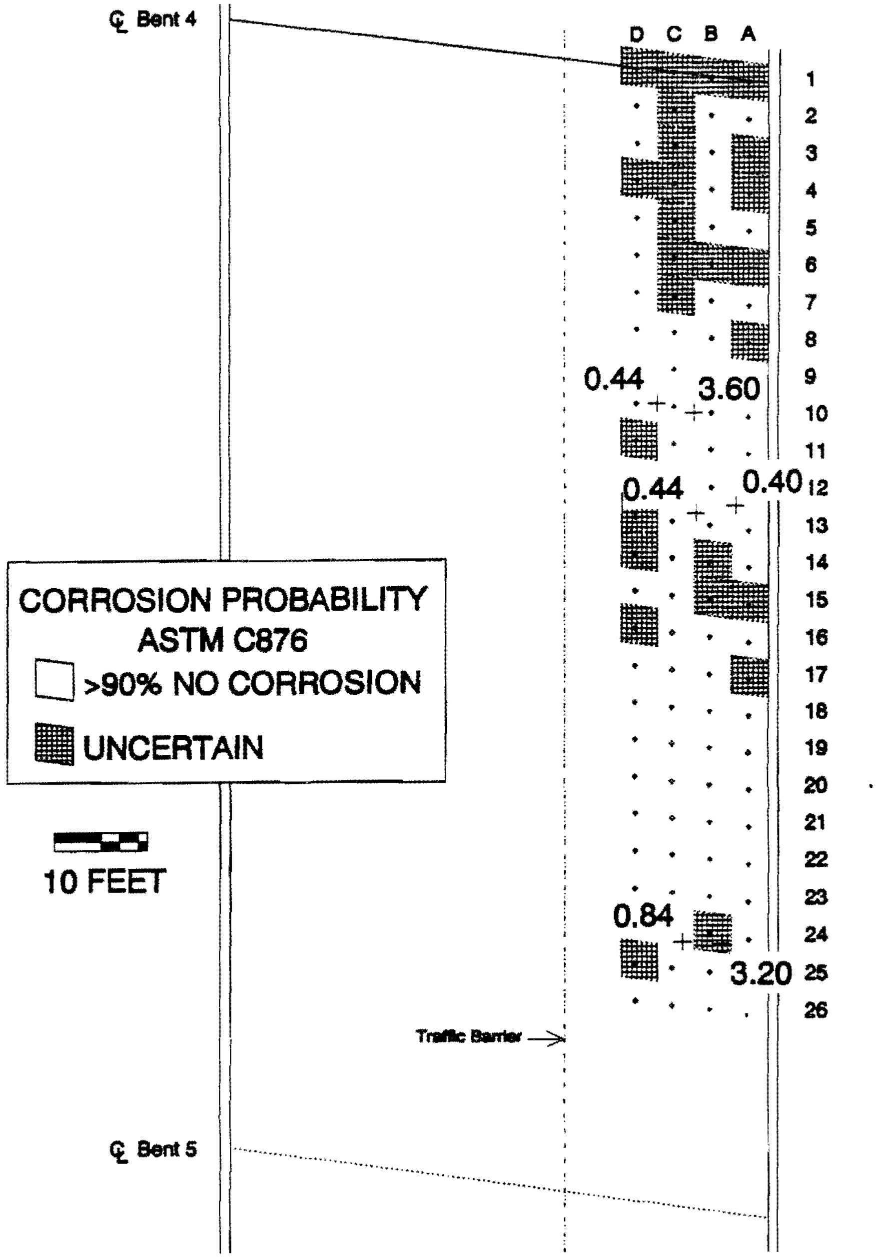

IH 40 AT AT&SF RAILROAD - CORROSION PROBABILITIES
℄ Bent 4
D C B A
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
CORROSION PROBABILITY
ASTM C876
>90% NO CORROSION
UNCERTAIN
10 FEET
Traffic Barrier
℄ Bent 5
GRID 3

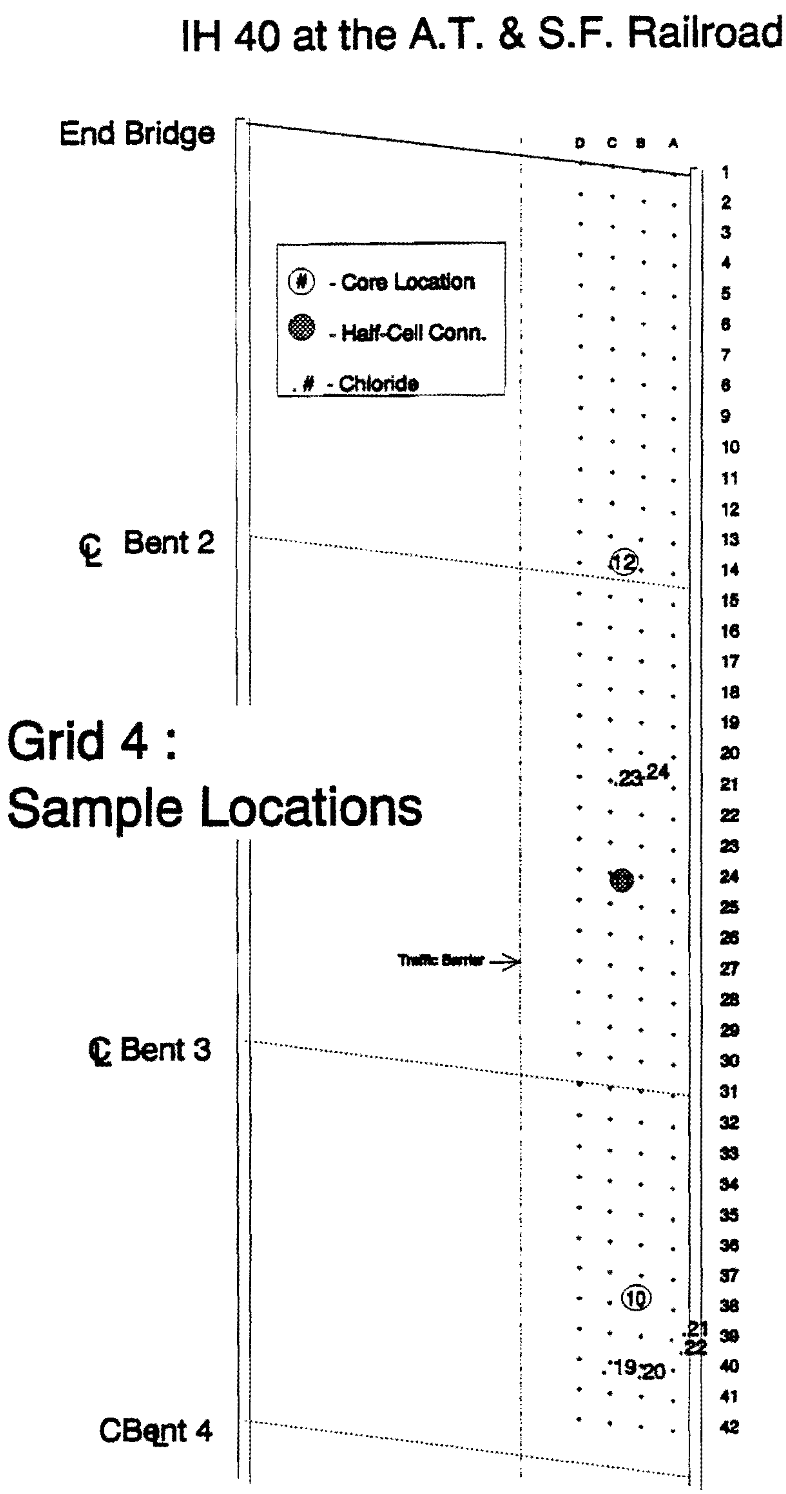
Figure A6.11

IH 40 AT AT&SF RAILROAD - CORROSION PROBABILITIES AND CHLORIDE CONTENTS



CHLORIDE CONTENTS (LB/CU. YD.) FROM 1-1/2 TO 2 INCH AND HALF-CELL POTENTIALS

Figure A6.12


IH 40 at the A.T. & S.F. Railroad
End Bridge
D C B A
(#) - Core Location
- Half-Cell Conn.
. # - Chloride
℄ Bent 2
Grid 4 :
Sample Locations
Traffic Barrier
℄ Bent 3
CBent 4
12
23
24
10
19
20
21
22
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
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41
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Figure A6.13

IH 40 at the A.T. & S.F. Railroad
Crack patterns and widths

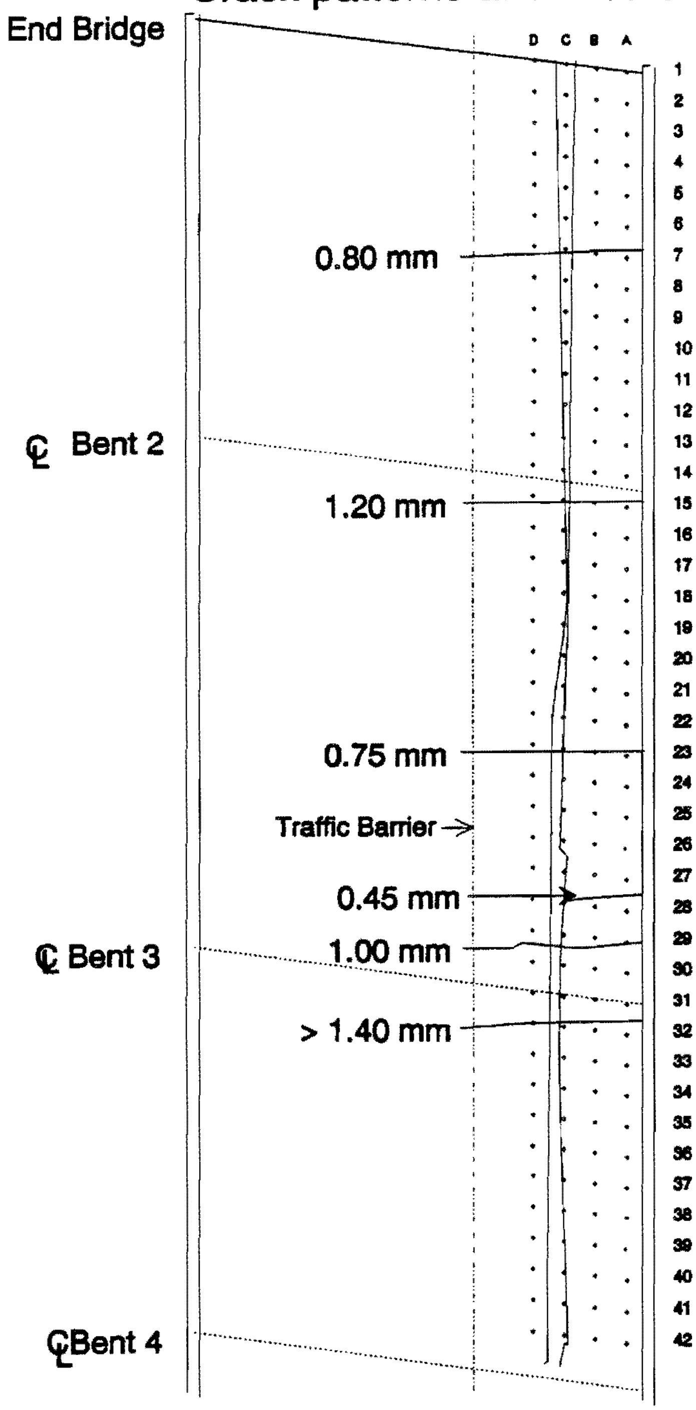
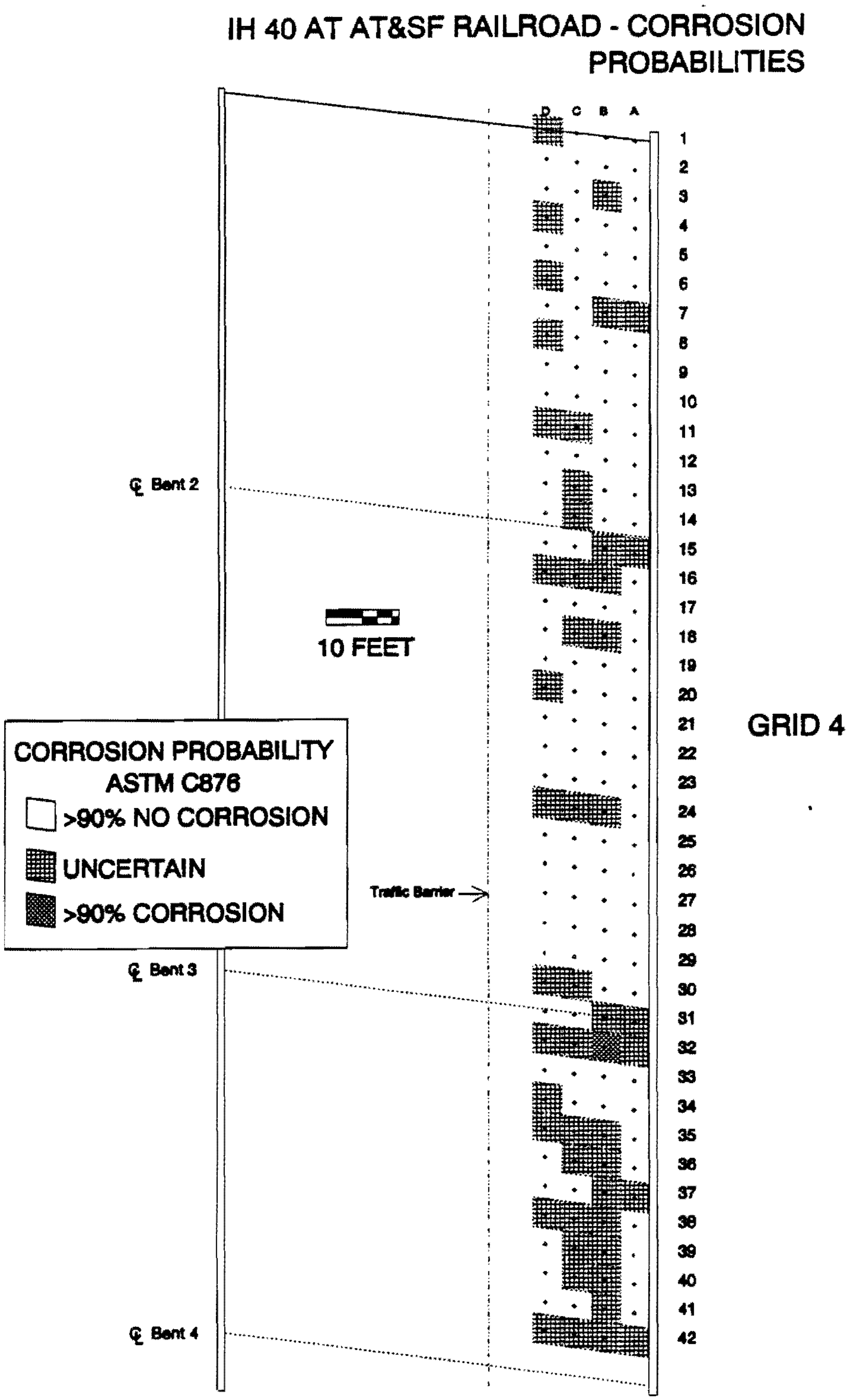


Figure A6.14


IH 40 AT AT&SF RAILROAD - CORROSION PROBABILITIES
D C B A
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
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28
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31
32
33
34
35
36
37
38
39
40
41
42
GRID 4
℄ Bent 2
℄ Bent 3
℄ Bent 4
10 FEET
Traffic Barrier
CORROSION PROBABILITY
ASTM C876
>90% NO CORROSION
UNCERTAIN
>90% CORROSION

Figure A6.15

IH 40 AT THE AT & SF RAILROAD
CHLORIDE CONTENTS (LB/CU. YD)

SAMPLE Number	SAMPLE DEPTH (IN) 0.0 - 0.5	0.5 - 1.0	1.0 - 1.5	1.5 - 2.0	Remarks
Grid #1					
1	2.80	2.40	0.32	0.24	wheelpath, no cracks
2	6.00	3.20	1.08	0.44	breakdown, sound concrete
3	6.80	6.00	3.60	3.20	wheelpath on a crack
4	6.80	6.40	5.60	5.60	breakdown on a crack
5	4.00	1.44	0.32	0.32	center of old traffic lane, no crack
6	7.20	2.80	0.36	0.40	travel lane wheelpath
Grid #2					
7	6.40	1.28	0.32	0.56	wheelpath of travel lane
8	3.20	0.88	0.24	0.32	
9	6.00	2.80	1.04	0.32	breakdown near solid white line
10	3.20	0.84	0.32	0.44	center of breakdown
11	7.20	5.60	2.80	1.32	travel lane, 1" from large crack
12	3.60	1.20	0.40	0.32	CL breakdown
Grid #3					
13	5.60	1.88	0.72	0.84	1" from crack
14	5.60	14.40	3.60	3.20	breakdown lane, 1" from crack
15	13.20	6.80	1.44	0.44	CL travel, good conc.
16	11.60	8.80	5.60	3.60	travel lane, wheelpath
17	4.00	2.40	1.20	0.44	breakdown
18	8.00	4.40	0.76	0.40	breakdown
Grid #4					
19	8.40	6.00	5.60	5.60	travel lane, wheelpath, on crack
20	6.80	7.20	5.60	5.20	breakdown on a crack
21	7.20	3.20	1.32	0.36	in gutter
22	5.20	3.60	1.60	0.44	in gutter
23	5.60	3.20	1.32	1.20	wheelpath travel lane, no cracks
24	8.40	3.60	0.56	0.44	breakdown, no cracks

IH 40 AT THE A.T. & S.F. RAILROAD

Dense Concrete Overlay

Concrete Permeabilities (Coulombs)

	Finished Surface Intact		Surface Removed
Sample	**Cracked**	**Uncracked**	**Uncracked**
6-1-WB-17-C2	1463		
6-1-WB-26-C3			3391
6-2-WB-11-C5		1051	
6-2-WB-21-C6			3749
6-3-WB-23-C7		1613	
6-3-WB-14-C8			2803
6-3-WB-6-C9	2423		
6-4-WB-13-C12	1050		
6-4-WB-24-C11	1445		

Note: All concrete tested was from the dense concrete overlay. No samples of the underlying base concrete were tested

APPENDIX A7
BUSINESS 281 OVERPASS, WICHITA FALLS DISTRICT

A7.1 BACKGROUND

This bridge was built in 1968. The structure carries 5600 vehicles per day, 12 percent of which is truck traffic. The concrete deck is supported by precast prestressed girders on concrete bents. The bridge was visited because it is protected by an alternative "wash and sweep" maintenance strategy. Whenever deicers are applied to the bridge, the deck is washed and cleaned by motorized equipment as soon as possible after the application. This is done in an attempt to remove the salts before they have a chance to penetrate the deck. By removing the chloride from the surface, the chloride buildup within the concrete should be reduced.

A7.2 TESTING

A7.2.1 Half-Cell Potential Testing

The half-cell corrosion testing was not successful on this structure as mostly positive readings were read. The test standard notes that positive potentials should not be regarded as accurate and that they possibly indicate poor interconnections within the reinforcing mat, an excessively dry deck, a poor connection to the reinforcment, poor connections between the reinforcing bars themselves, stray currents, or improper connections to the voltmeter used in the testing.

In response to the positive readings, the deck was continuously soaked, the connection was checked and changed, and the voltmeter connections were checked, all to no avail. Despite these measures, most of the measured potentials remained positive. Only poor connections between the reinforcement and stray currents remained as possible causes for the malfunction. The investigators' inability to control these factors prevented any further use of the half-cell. Thus, no potentials are reported for this structure.

A7.2.2 Chloride Content Testing

Twenty-five locations were sampled for chloride content. The chloride content in the 1/2- to 1-inch-deep portion of the deck ranged from undetectable to 2.4 lb/yd^3. The chloride content in the 1-1/2- to 2-inch depth ranged from undetectable to 1.32 lb/yd^3. The values were smallest in grids 3 and 4, which were travel and passing lanes, respectively. The abutment wingwall had a relatively large chloride content of 1.16 pounds per cubic yard at the 1-1/2- to 2-inch depth. The drilled shafts showed relatively constant chloride contents of approximately 0.55 lb/yd^3 through the entire 2-inch sampling depth.

With the exception of the single 1.32 lb/yd^3 location, all of the samples taken from the 1-1/2- to 2-inch depth had less than 0.64 lb/yd^3. As in other bridges investigated, no correlation is apparent between wheelpath or centerline exposure and chloride concentration of the concrete. Also, samples taken from gutter and scupper areas showed no difference in chlorde content from the main deck.

A7.2.3 Permeability Testing

Nine specimens were tested to determine the permeability of the concrete. Six specimens were tested with the finished wearing surface left intact, four of which had a cracked surface and two without. Three samples were tested with the finished surface removed.

As in the other structures investigated, there ws no correlation between the permeabilities and the surface cracking of the specimens. The samples tested with the surfaces removed had higher coulomb values indicating higher permeabilities than those with the surface intact. In general the permeabilities were very variable, with the specimens with a finished top surface having permeabilities ranging from 1857 to 4537 coulombs. The specimens with the surface removed had permeabilities of 5423 and 9989 coulombs. One of the specimens with the surface removed had a five hour permeability over 11,000 coulombs at the time when it was removed from testing because of excessive heat. The standard six hour permeability test result would have been even higher.

The permeabilities do not seem to correlate to observed surface wear.

A7.2.4 Crack Mapping

All of the spans were inspected for cracking, wear, or other visual characteristics. The structure had light random surface cracking over the entire surface of the deck and had areas of obvious wear, longitudinal cracking, and checking. The wear and checking were most obvious in the wheelpaths. In some areas, the coarse aggregate was exposed.

A7.2.5 Strength Testing

A sample core taken from this structure exhibited a strength of 11,375 psi.

A7.2.6 Delamination Testing

Although the entire deck was surveyed, only one small region of delamination was found in the corner of grid 1 at the joint between the bridge and the approach.

A7.2.7 Petrographic analysis

One core from the bridge was examined to determine the characteristics of the air void system. The results are as follows:

Note: Recommended values from the American Concrete Institute Manual of Concrete Practice (216) are included in parenthesis after the results.

- Air Content 7.2% (>5.0% ± 1.5%)
- Voids per Inch 7.2 (>7.5)
- Specific Surface 520 in^2/in^3 (>600 in^2/in^3)
- Spacing Factor 0.009 (<0.008)

Although the air content meets current standards, the other air system parameters are below accepted values for effective freeze-thaw protection.

Cores soaked in a low viscosity methacrylate showed that the small cracks in the cores were open, but that larger cracks were internally sealed with dirt and silt. This is consistent with observations from other bridges observed.

A7.3 CONCLUSIONS

This bridge showed generally good but variable performance in terms of permeability, chloride content, and delaminations in spite of its age, exposure, and surface deterioration. The half-cell corrosion potential test could not be used to characterize the corrosion performance of this structure for unknown reasons. The wash and sweep maintenance strategy appears to hold some merit and warrants further investigation.

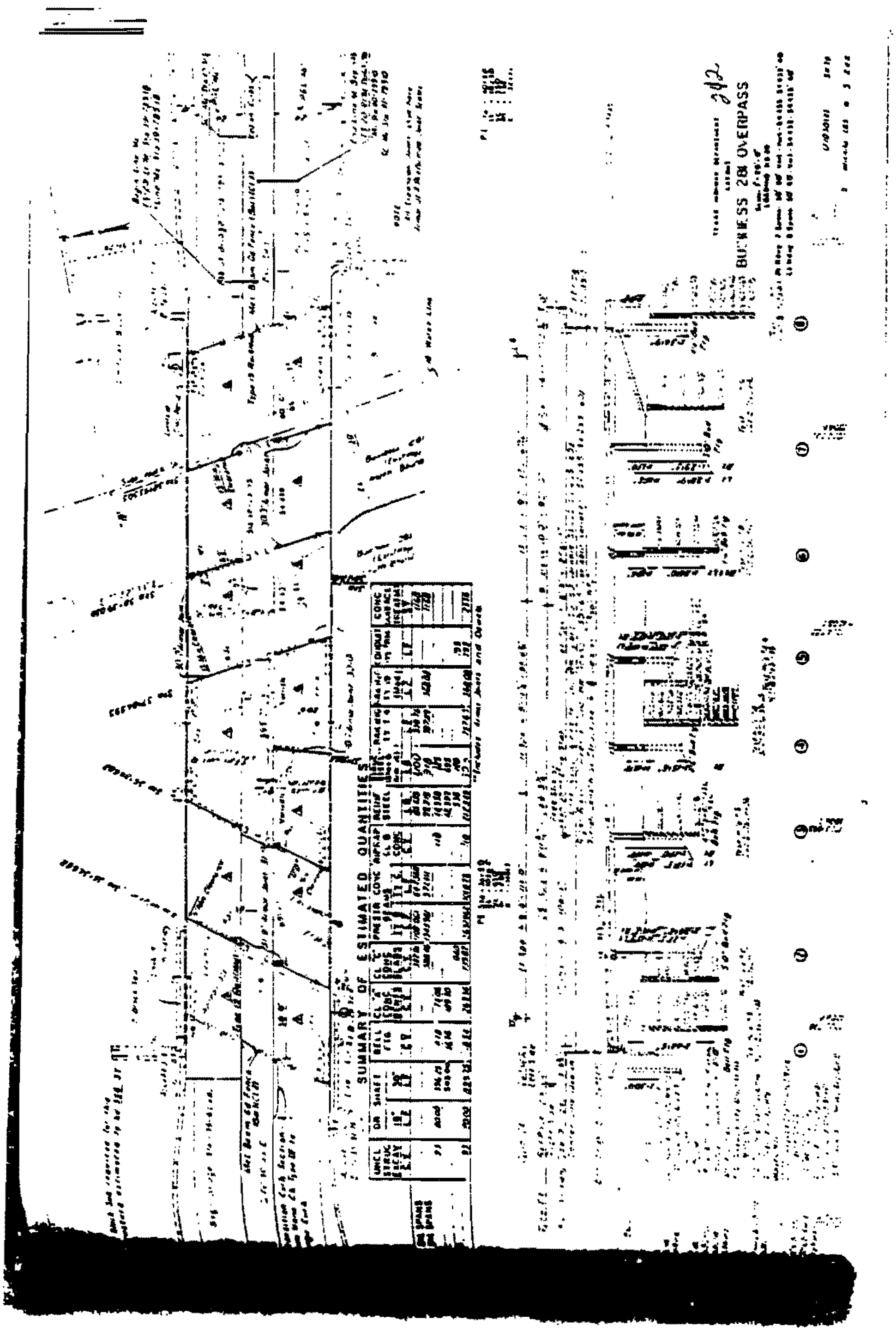


Figure A7.1

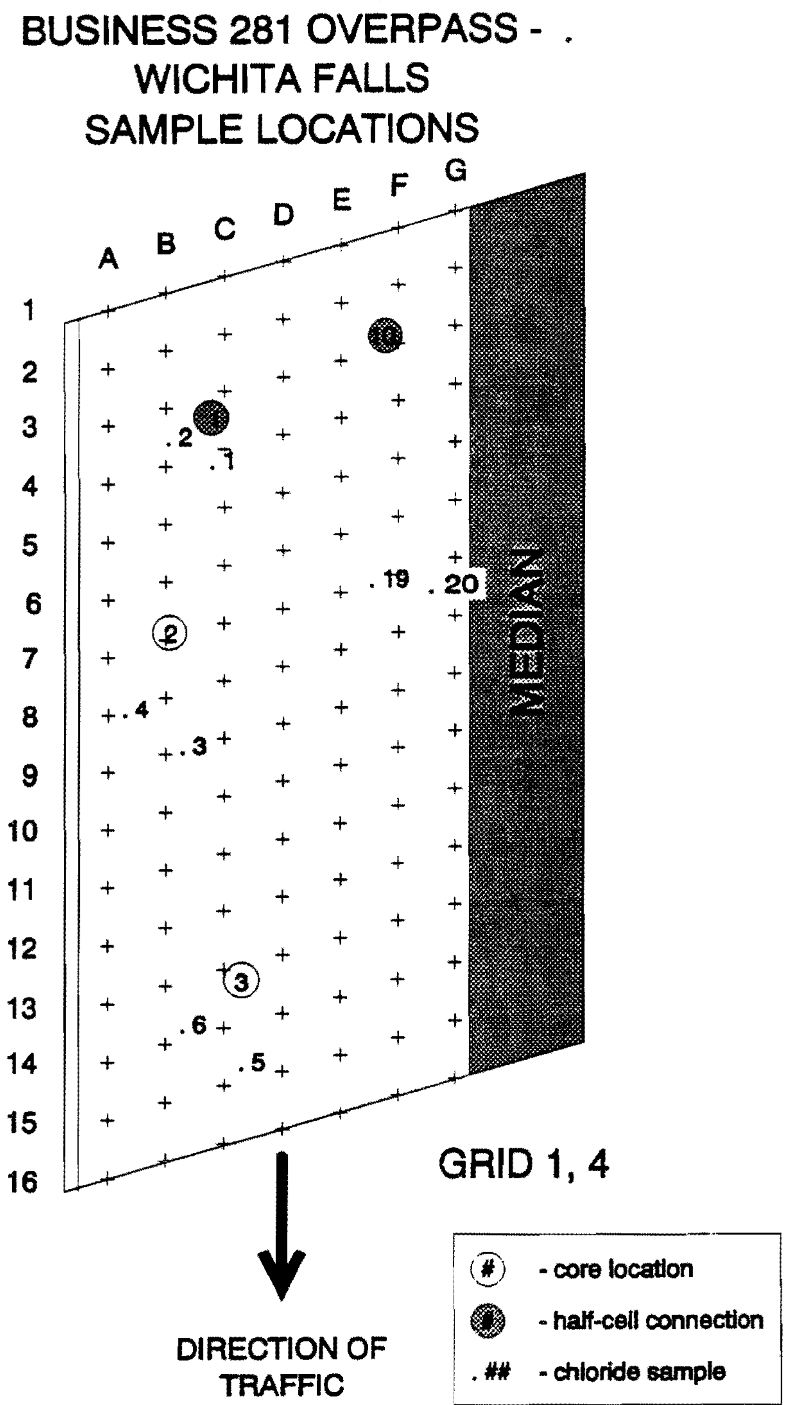

BUSINESS 281 OVERPASS -
WICHITA FALLS
SAMPLE LOCATIONS
A
B
C
D
E
F
G
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
MEDIAN
GRID 1, 4
DIRECTION OF TRAFFIC
- core location
- half-cell connection
. ## - chloride sample

Figure A7.2

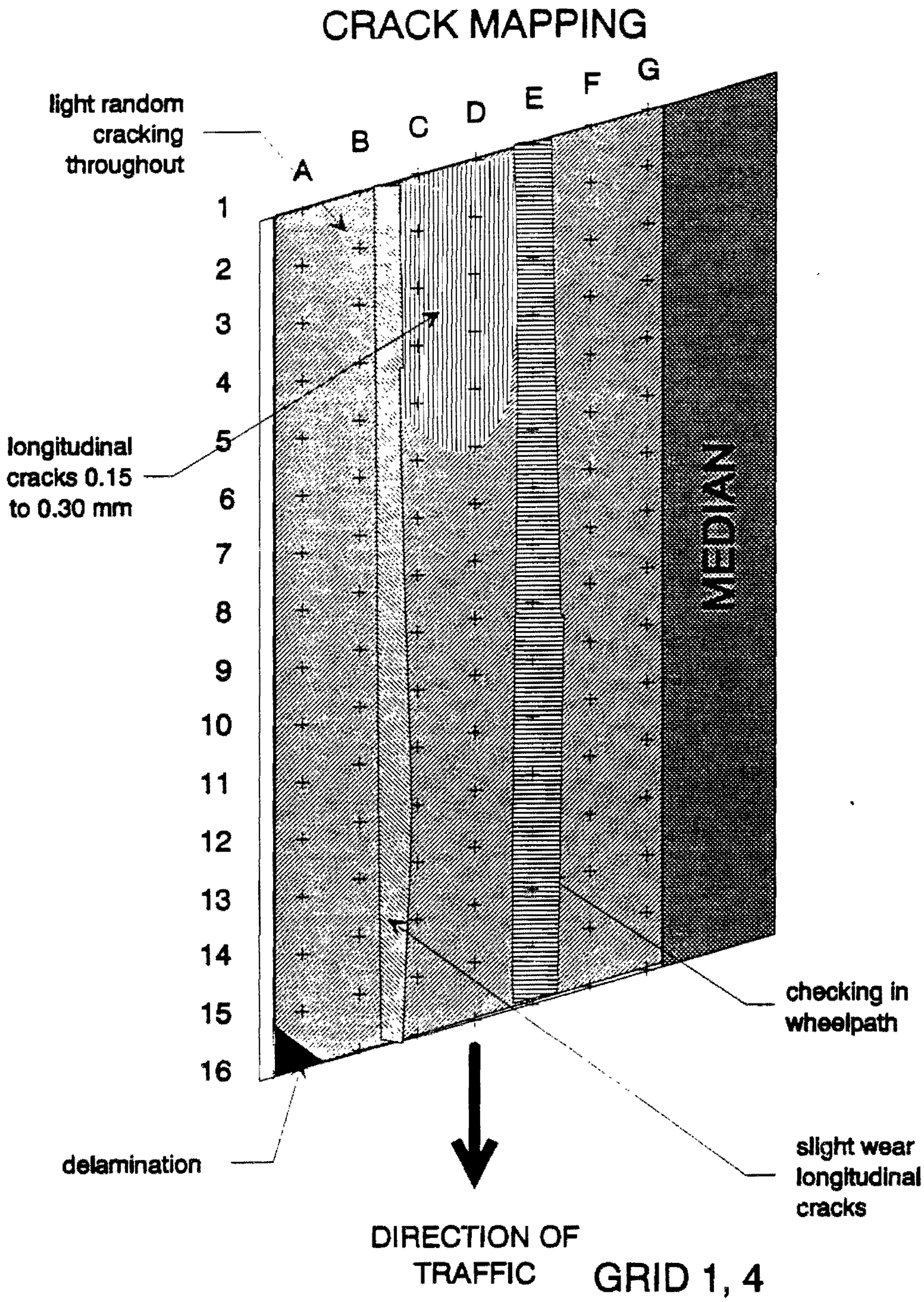
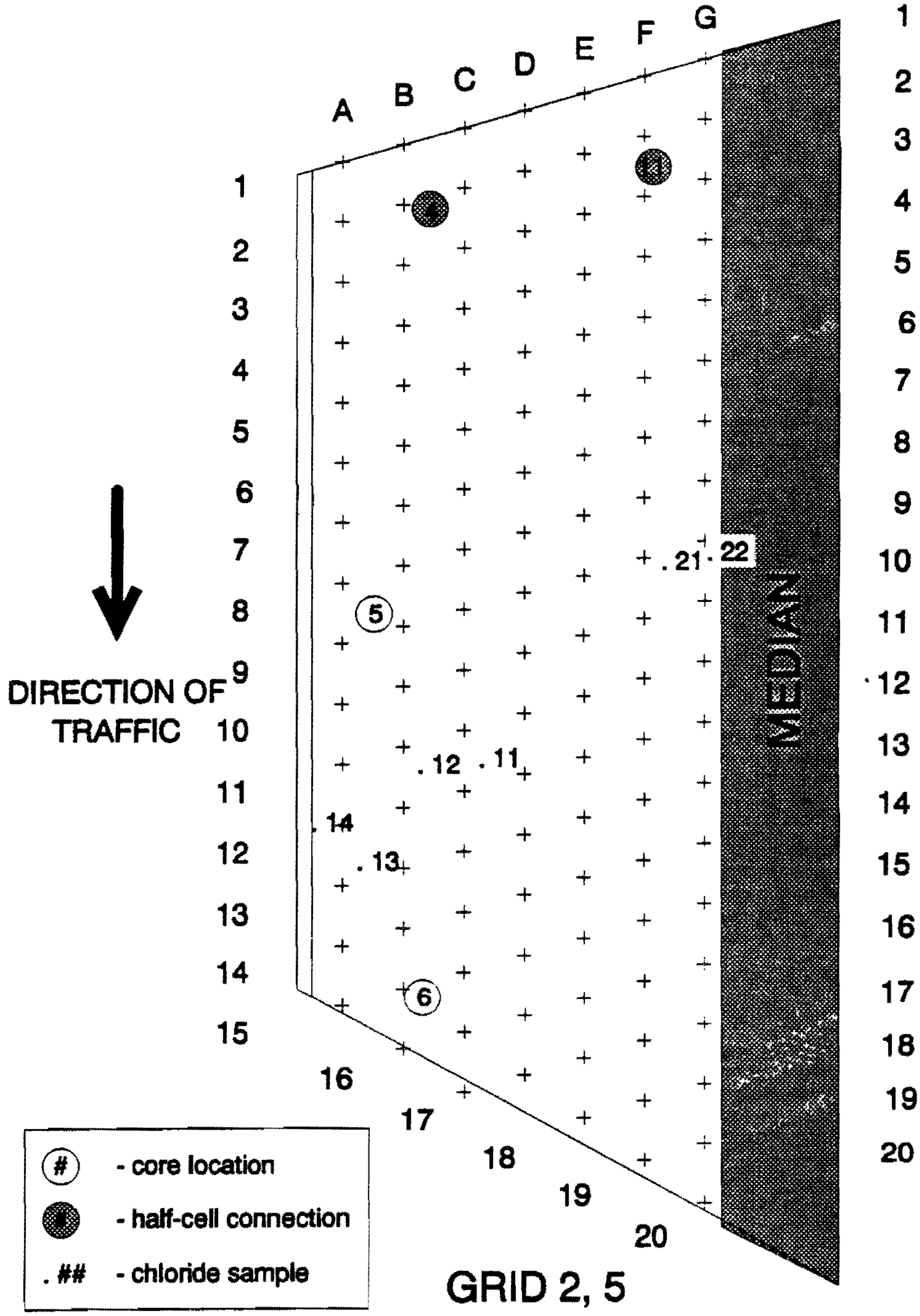

CRACK MAPPING
A
B
C
D
E
F
G
light random cracking throughout
longitudinal cracks 0.15 to 0.30 mm
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
MEDIAN
checking in wheelpath
slight wear longitudinal cracks
delamination
DIRECTION OF TRAFFIC
GRID 1, 4

Figure A7.3

BUSINESS 281 OVERPASS - WICHITA FALLS SAMPLE LOCATIONS



GRID 2, 5

Figure A7.4

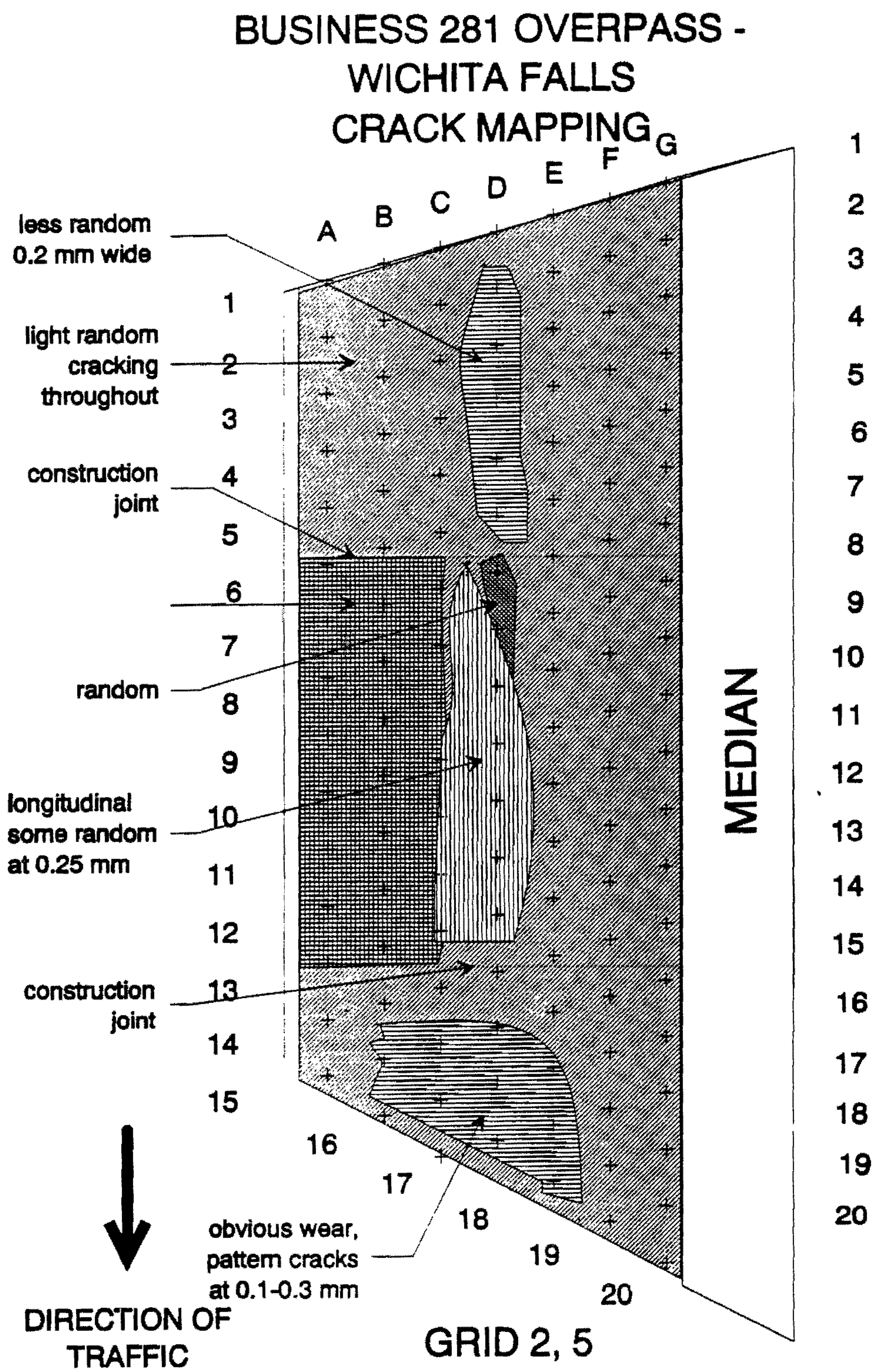

BUSINESS 281 OVERPASS -
WICHITA FALLS
CRACK MAPPING
A B C D E F G
less random
0.2 mm wide
light random
cracking
throughout
construction
joint
random
longitudinal
some random
at 0.25 mm
construction
joint
obvious wear,
pattern cracks
at 0.1-0.3 mm
MEDIAN
DIRECTION OF
TRAFFIC
GRID 2, 5

Figure A7.5

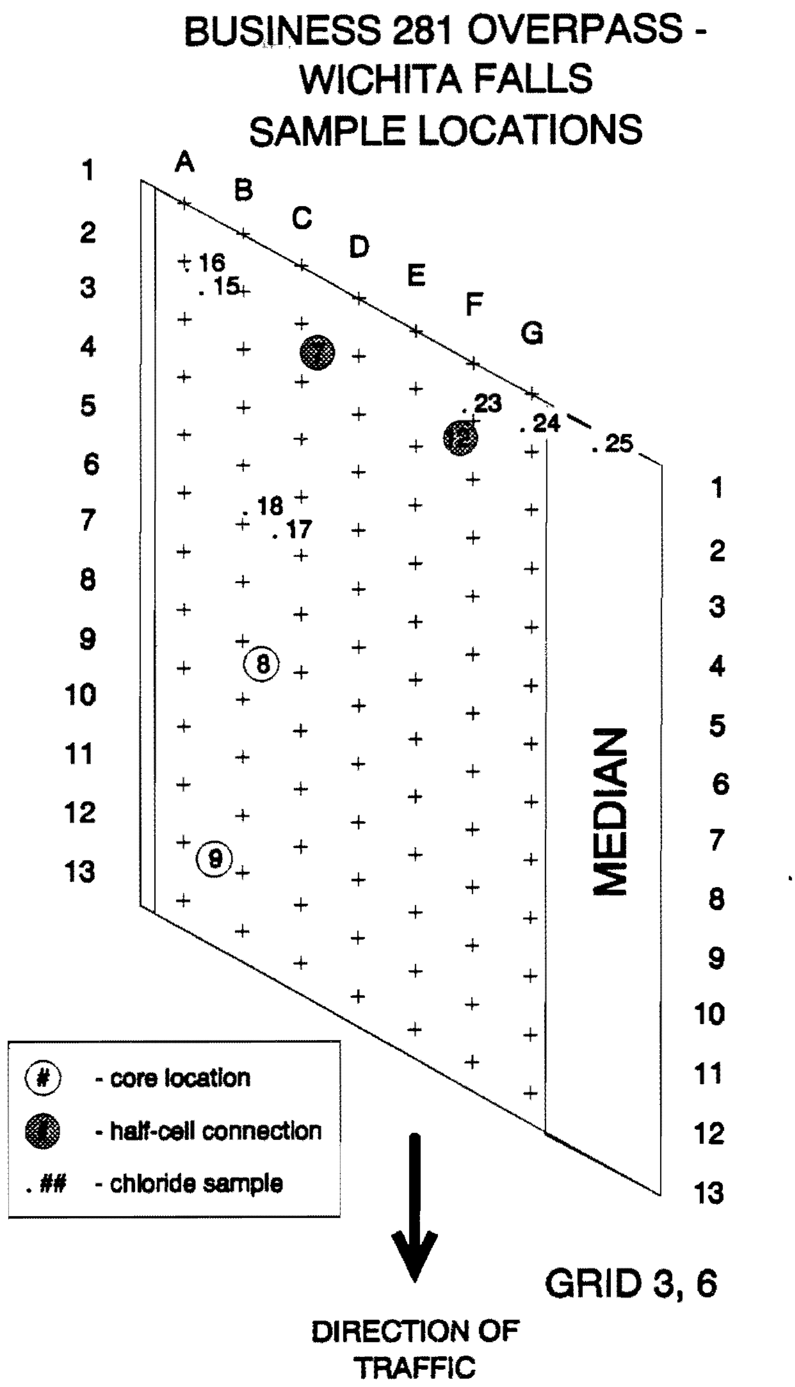

BUSINESS 281 OVERPASS -
WICHITA FALLS
SAMPLE LOCATIONS
A
B
C
D
E
F
G
1
2
3
4
5
6
7
8
9
10
11
12
13
16
15
23
24
25
18
17
7
12
8
9
MEDIAN
- core location
- half-cell connection
. ## - chloride sample
DIRECTION OF
TRAFFIC
GRID 3, 6

Figure A7.6

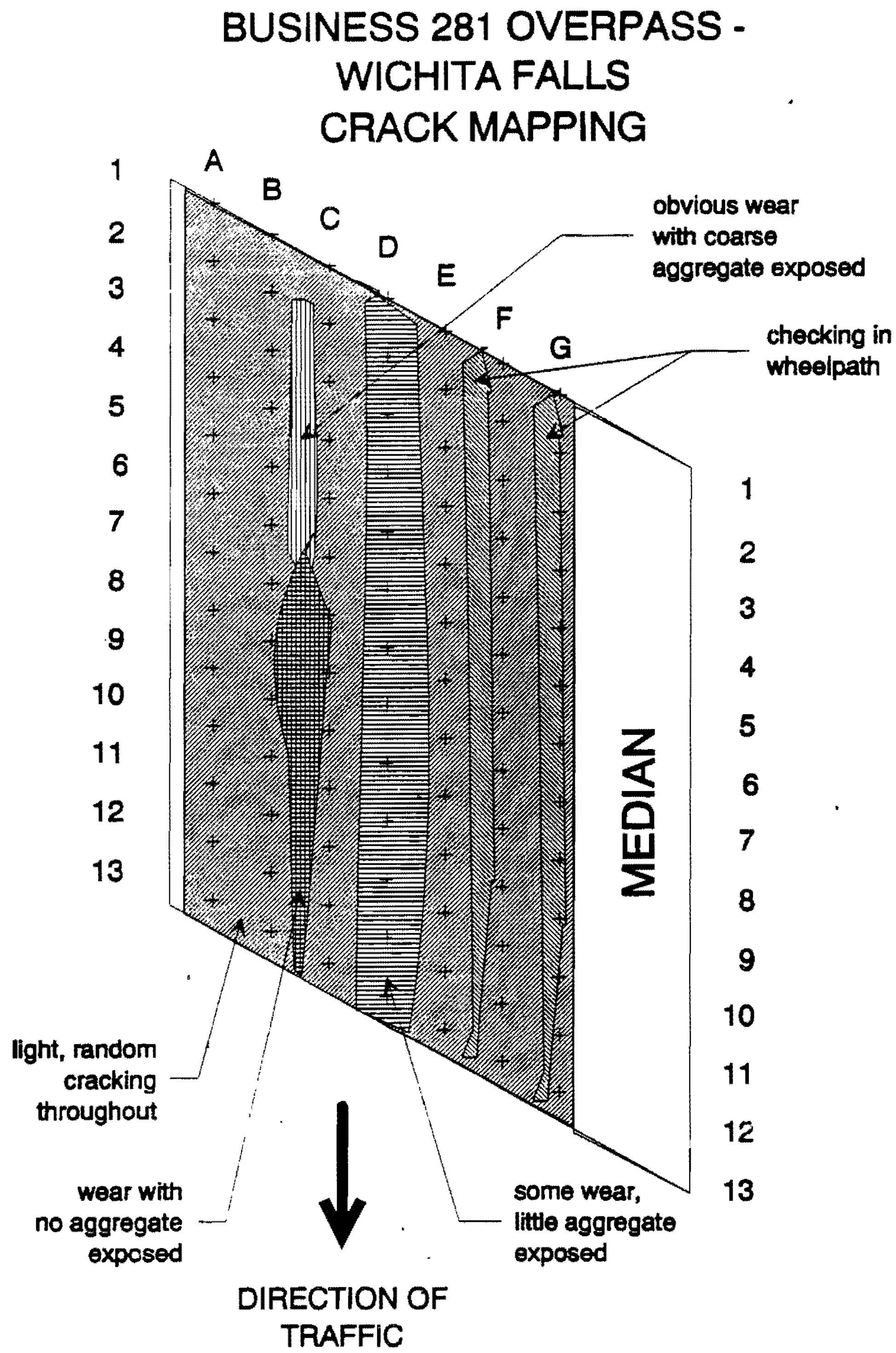

BUSINESS 281 OVERPASS -
WICHITA FALLS
CRACK MAPPING
A
B
C
D
E
F
G
1
2
3
4
5
6
7
8
9
10
11
12
13
obvious wear
with coarse
aggregate exposed
checking in
wheelpath
MEDIAN
1
2
3
4
5
6
7
8
9
10
11
12
13
light, random
cracking
throughout
wear with
no aggregate
exposed
some wear,
little aggregate
exposed
DIRECTION OF
TRAFFIC
GRID 3, 6

Figure A7.7

BUSINESS 281 OVERPASS - WICHITA FALLS
CHLORIDE CONTENTS (LB/CU. YD)

SAMPLE Number	SAMPLE DEPTH (IN) 0.0 - 0.5	0.5 - 1.0	1.0 - 1.5	1.5 - 2.0	REMARKS
Grid #1					
1	2.80	0.68	0.56	0.36	Centerline (CL) lane
2	3.60	2.40	1.40	0.64	wheelpath
3	3.60	2.40	1.20	0.36	wheelpath
4	3.20	1.44	1.32	1.32	CL breakdown lane
5	2.00	1.28	0.56	0.28	CL lane
6	3.60	1.32	0.32	0.00	wheelpath
Grid #2					
11	3.60	1.60	0.72	0.44	CL lane
12	3.20	1.40	0.52	0.44	wheelpath, worn surface
13	0.60	0.00	0.72	0.24	CL breakdown
14	1.20	0.36	0.24	0.40	at scupper
Grid #3					
15	0.84	0.68	0.40	0.20	wheelpath, worn surface
16	0.60	0.96	0.24	0.20	CL breakdown
17	1.44	0.84	0.48	0.00	CL travel lane
18	0.80	0.32	0.00	0.00	wheelpath, worn area
Grid #4					
19	0.76	0.24	0.00	0.20	center
20	1.08	0.80	0.28	0.20	wheelpath
21	0.68	0.28	0.24	0.28	wheelpath
Grid #5					
22	0.56	0.00	0.24	0.24	gutter
Grid #6					
23	1.60	1.04	0.68	0.28	CL lane
24	2.00	1.20	0.48	0.36	wheelpath
25	1.72	1.04	0.60	0.32	on median sidewalk
Abutment #1					
7	2.40	1.92	1.40	1.16	South hole in toe of wingwall
8	0.56	1.76	1.76	1.16	North hole in toe of wingwall
Bent 2					
9	0.56	0.44	0.56	0.56	South leg of bent 2
10	0.56	0.60	0.60	0.52	North leg of bent 2

BUSINESS 281 OVERPASS

Wash and Sweep Maintenance Strategy

Concrete Permeabilities (Coulombs)

	Finished Surface Intact		Surface Removed
Sample	**Cracked**	**Uncracked**	**Uncracked**
7-1-WBT-B6-C2A	4537		
7-1-WBT-B6-C2B			9989
7-1-WBT-C13-C3A	1857		
7-1-WBT-C13-C3B			5423
7-2-WBT-B9-C5A		4235	
7-2-WBT-B15-C6	2243		
7-4-WBP-F3-C10	3210		
7-3-WBT-B12-C9		2126	

Note: Sample 7-2-WBT-B9-C5B was tested, but was turned off due to excessive heat. The permeability was over 11,000 when it was removed at five hours.

APPENDIX A8
SH-67 AT THE BRAZOS RIVER, WICHITA FALLS DISTRICT

A8.1 BACKGROUND

The two-lane bridge carrying SH 67 over the Brazos River in Young County was built in 1985. The structure, located just west of Graham, Texas is approximately 840 feet long and carries an average daily traffic of 1750 vehicles, 10 percent of which is truck traffic. The system is exposed to deicing salts during the winter and to the mild chloride and sulfate content of the Brazos River. The structure consists of a concrete deck on precast, prestressed girders supported by concrete drilled shaft bents.

<Normal>The structure was used as a test bridge to compare various corrosion protection strategies and to compare various surface grooving techniques. The structure incorporates the use of calcium nitrate, linseed oil, methacrylate, and epoxy-coated reinforcement. The protection systems were originally monitored by periodic readings of corrosion potentials between the reinforcement mats and test bars placed in the deck during construction. Due to vandalism and erratic test results, the monitoring was discontinued. Reference 175 contains more information about the structure. No conclusions were drawn from the limited monitoring period covered by the report.

A8.2 TESTING

A8.2.1 Half-Cell Potential Testing

Half-cell potential measurements could only be made on three of the test spans. The other spans were all constructed using epoxy-coated reinforcement. Of the three spans tested, only two spans yielded usable results. Testing of the third span resulted in mainly positive readings that should not be regarded as correct in accordance with the test standard. The reason for the positive readings is not known, although insufficient connections between the bars or stray current interference is considered most possible.

Of the two spans tested, all for the readings indicated a 90 percent probability of no corrosion as defined by the ASTM standard. Although the potentials were all less negative than the -0.200 V CSE characterizing uncertain corrosion activities, some information can be drawn from the readings. The span protected with calcium nitrate had more negative readings than the methacrylate treated span. Although this cannot be interpreted as indicating that the span is not performing as well as the other spans, it does indicate that for the same exposure the calcium nitrate span is closer to uncertain performance than the other span. If this trend continues, the data indicates that the calcium nitrate span will experience active corrosion before the other span.

For the span protected with methacrylate, the more negative potentials tended to occur in the gutter area where delamination of the methacrylate was observed. The span protected with calcium nitrate showed the more negative regions to be in the center of the bridge and away from the gutter.

A8.2.2 Chloride Content Testing

Thirty-eight locations were sampled and analyzed for chloride content. Samples were taken from cracked and uncracked locations as well as from centerline and wheelpath areas. Chloride contents were low over the bridge. The span treated with calcium nitrate and one of the spans treated with linseed oil had higher chloride than the other spans. Figure A8.1 represents a graphical comparison of the chloride contents of the samples taken at 1-1/2 to 2 inches below the surface. It must be noted, however, that the linseed oil section showing the high chloride contents was observed to have different surface characteristics than the other spans. The surface of the span showed exposed coarse aggregate throughout and had numerous popouts as reflected in the the crack mapping diagrams.

The surface grooving techniques showed no influence on chloride content. Similarly, the centerline or wheelpath exposure showed no influence on chloride content.

A8.2.3 Permeability Testing

Fifteen specimens were tested to determine the permeability of the concrete in the bridge. Ten of the specimens were tested with the surface treatments left intact and five were tested with the upper two inches of the core removed. The permeabilities were generally low with

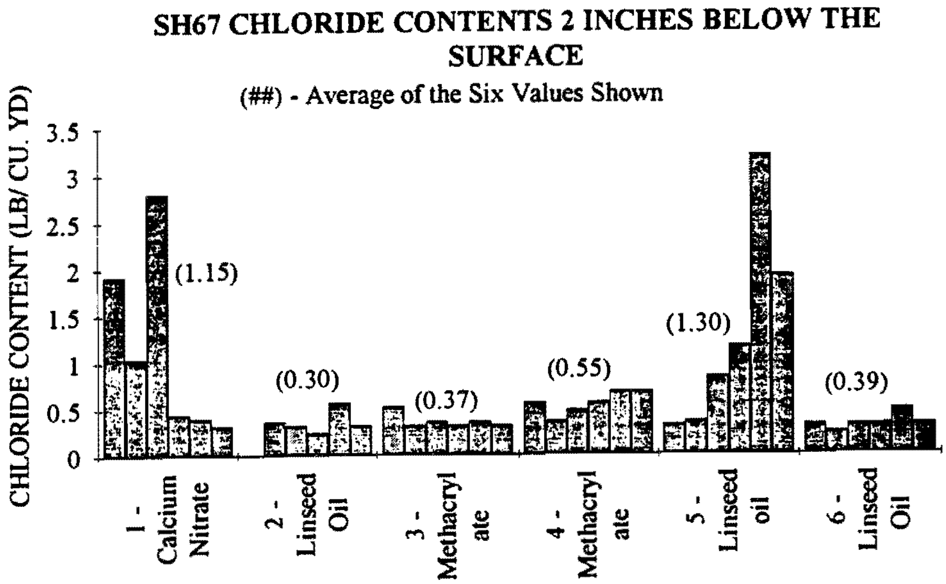


Figure A8.1 Chloride Contents from 1-1/2 to 2 Inches Below the Surface

permeabilities of less than 4000 coulombs found in all specimens except those from the linseed oil span having the poor surface noted during the crack mapping. This span will be discussed later. No large differences were noted between the treated surface specimens and their companion specimens tested with the surface removed. Any differences in permeability were within approximately 500 coulombs. In the span treated with calcium nitrate, the specimen with the surface removed had a lower permeability than the specimens tested with the surface intact.

The specimens from the linseed oil span with the exposed coarse aggregate on the surface had highly variable permeabilities ranging from 1976 to 11638 coulombs. The concrete from this span was visually inspected and found to have large entrapped air voids. Also, two samples from this portion of the bridge were soaked in a penetrating methacrylate to determine the extent of the pore and crack systems. The penetration was found to be highly variable and extensive in areas. Lastly, it must be noted that the sample with the permeability of 11638 coulombs was only 1-3/4 inches long because of insufficient overall core length. The high variability of this span's permeability can be attributed to the highly variable quality of the concrete observed.

A8.2.4 Crack Mapping

Because of rain, only four of the six spans investigated could be inspected for cracking and other surface defects. The four spans that were investigated showed quite different surface features. The methacrylate treated spans showed only small areas of cracking and chipping. The methacrylate spans both exhibited delaminations of the methacrylate in the gutter region of the structure. The two linseed oil spans showed more surface damage than the other spans. As previously discussed, one of the linseed oil spans exhibited widespread exposure of the coarse aggregate and numerous popouts. The other linseed oil span had some areas of exposed coarse aggregate and wear. This span also exhibited some delaminations in the gutter region.

Also, all of the spans showed a four- to eight-foot band of exposed coarse aggregate and wear immediately after the joint connecting the span to the previous. The reason for this deterioration is not known.

A8.2.5 Strength Testing

A sample core taken from this structure exhibited a strength of 7076 psi.

A8.2.6 Delamination Testing

All of the spans visually examined for cracking were sounded for delaminations. No widespread delaminations were noted. The only delaminations found were in the gutter region of the bridge and are described in the crack mapping section above.

A8.2.7 Petrographic analysis

One core from the bridge was examined to determine the characteristics of the air void system. The results are as follows:

Note: Recommended values from the American Concrete Institute Manual of Concrete Practice (216) are included in parenthesis after the results.

•Air Content	5.0 %	(>5.0% ± 1.5%)
•Voids per Inch	6.7	(>7.5)
•Specific Surface	537 in^2/in^3	(>600 in^2/in^3)
•Spacing Factor	0.009	(<0.008)

Although the air content meets current standards, the other air system parameters are below currently accepted values for effective freeze-thaw protection.

The sample submitted for analysis was from the span treated with methacrylate and was taken from the region of wear and exposed aggregate at the beginning of the span.

A8.3 CONCLUSIONS

This bridge is working well in its test capacity. No evidence of any active corrosion was found, but some indicators of the relative performance of the systems such as permeabilities and chloride content are becoming apparent. A number of differences between this structure and the others described herein were found. The decrease in permeability associated with surface treatments was not seen on this bridge as the untreated specimens tested within 500 coulombs of the treated counterparts. With the exception of the linseed oil span with the observed surface problems and the calcium nitrate span, the chloride contents were all comparable.

As the above differences from previously observed performance may be due to the longer exposure time of this bridge, it may serve as an indicator of sealer wear and loss of effectiveness with time. Monitoring of this span should continue to better explain the observed phenomena.

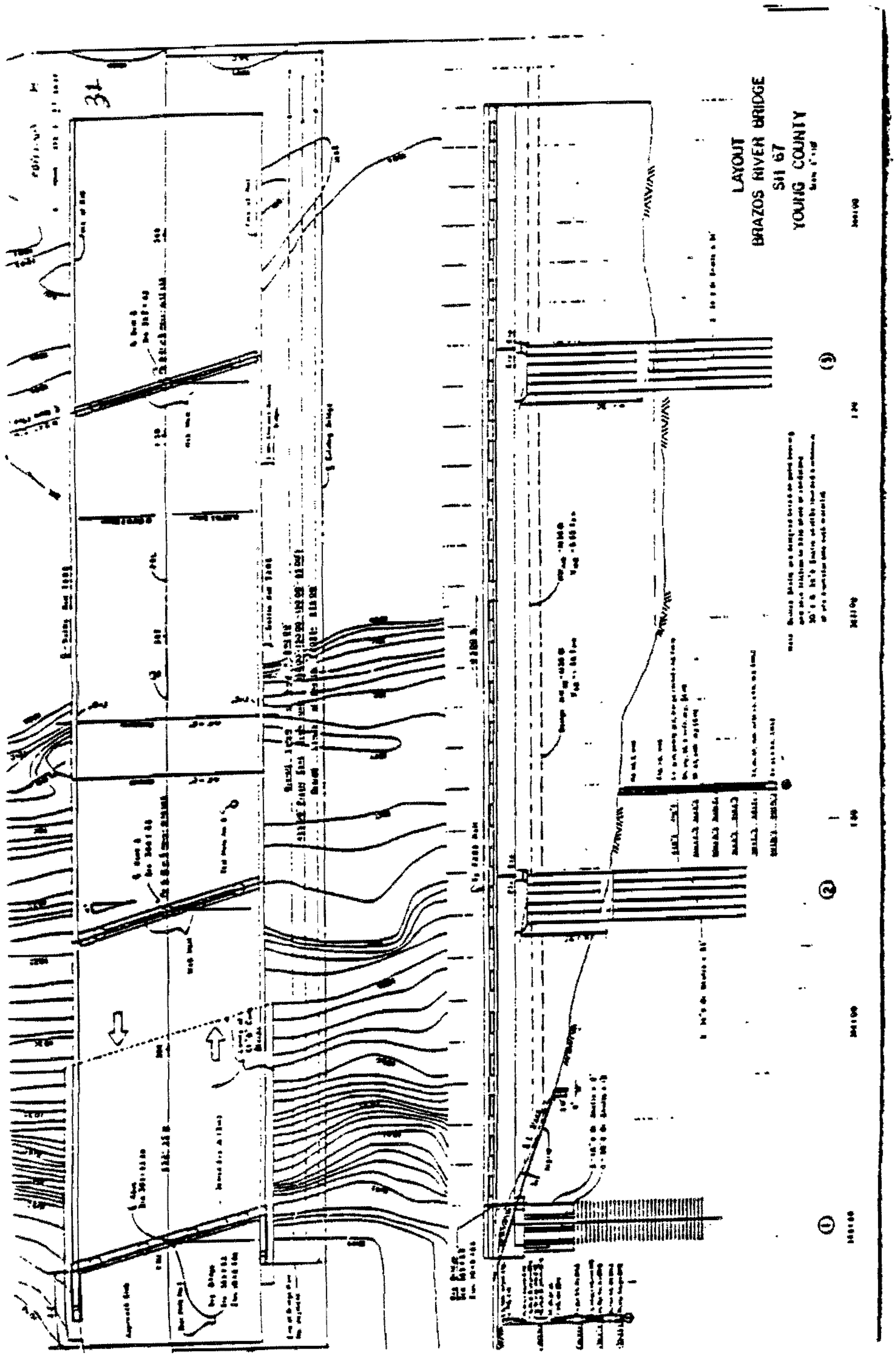


Figure A8.2

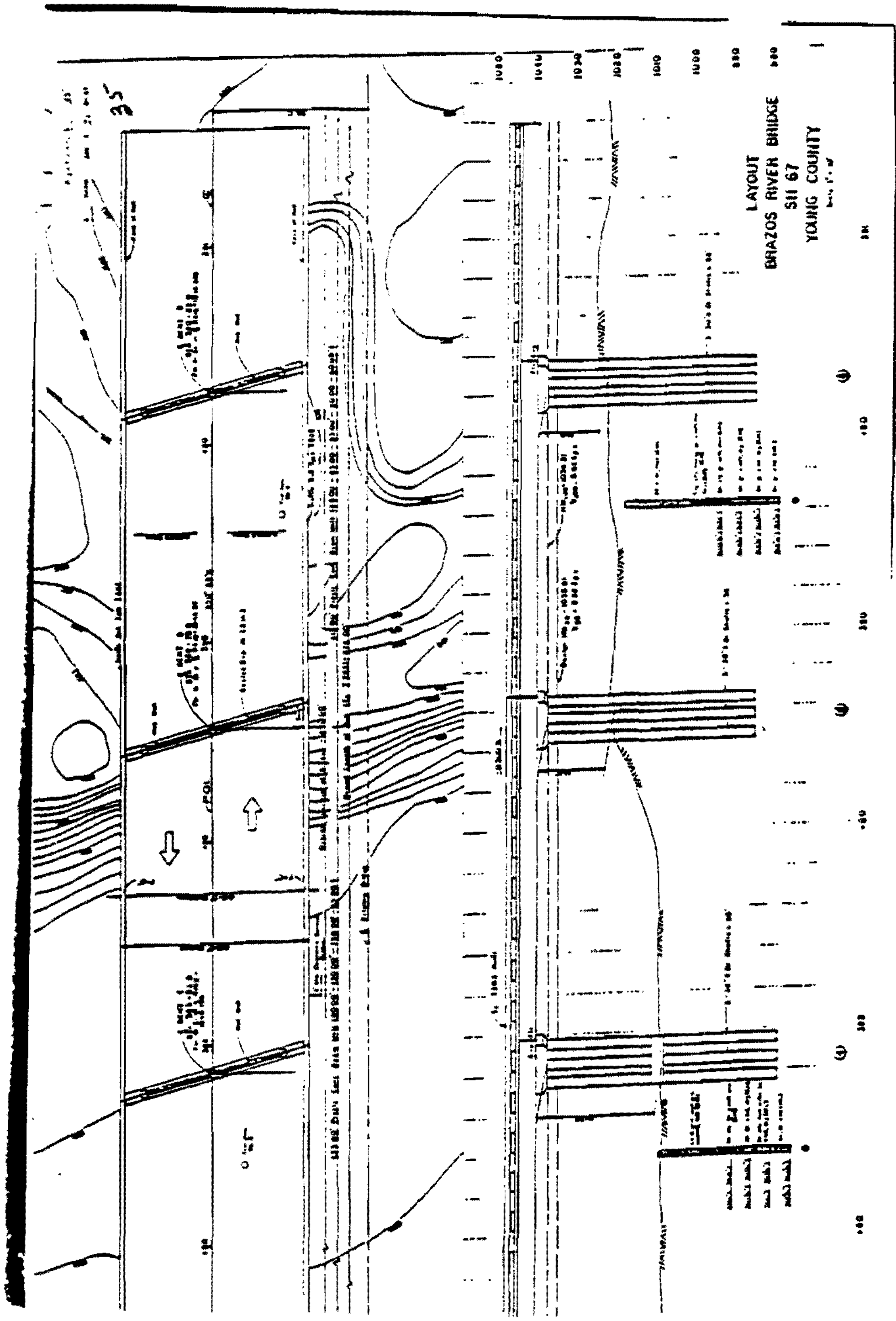


Figure A8.3

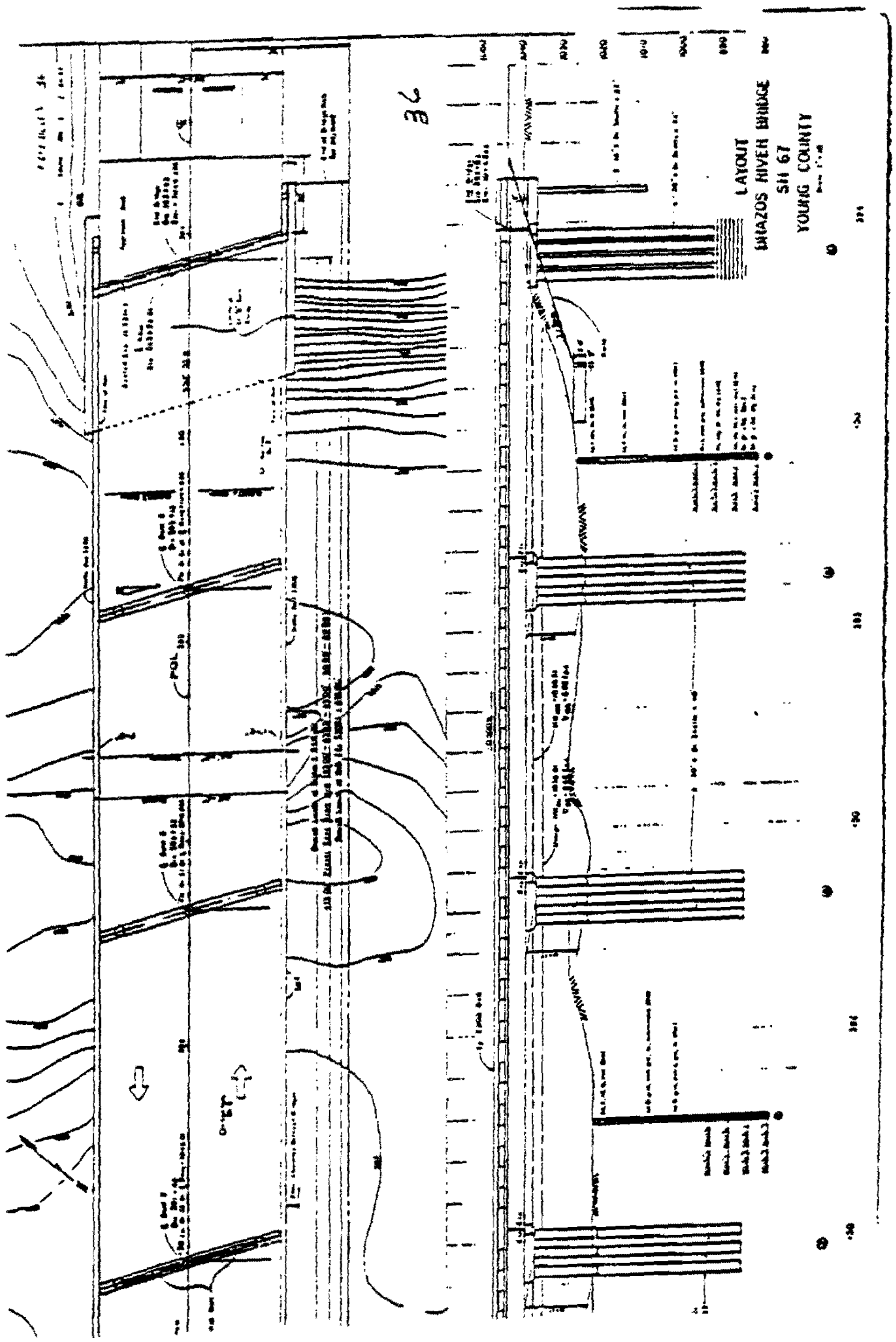
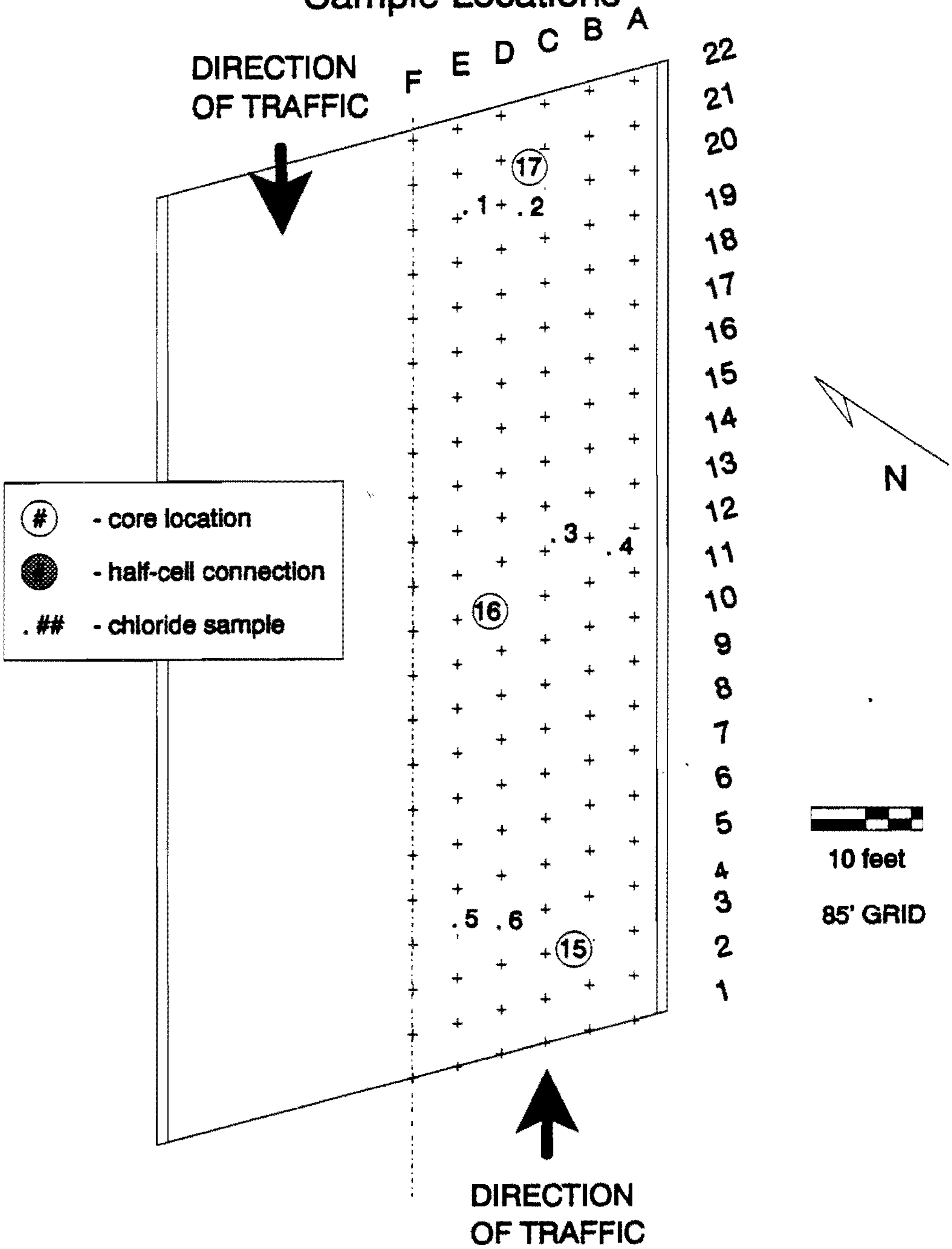


Figure A8.4

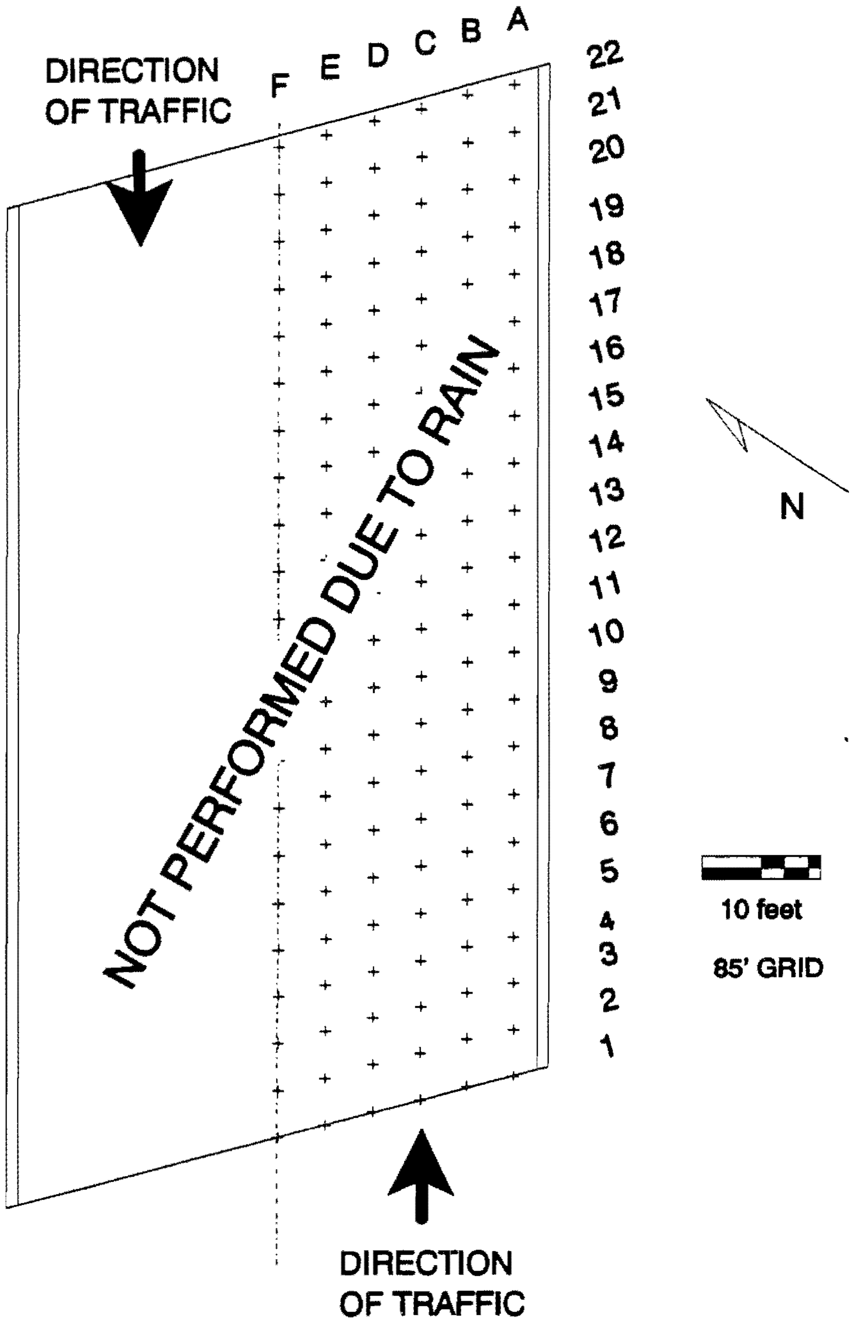
SH67 at the Brazos River
Wichita Falls
Sample Locations



Span 4 - Linseed Oil

Figure A8.5

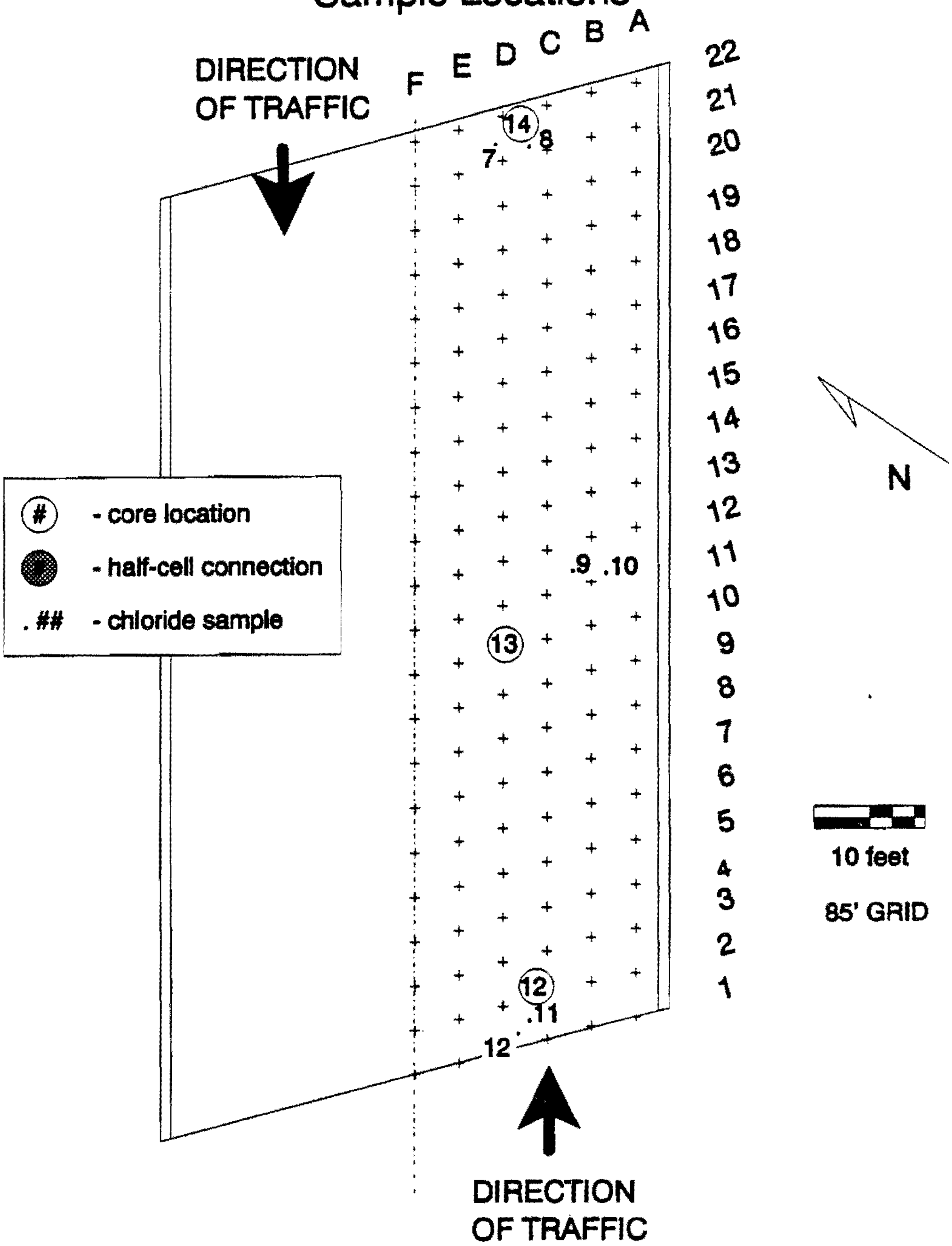
SH67 at the Brazos River
Wichita Falls



Span 4 - Control

Figure A8.6

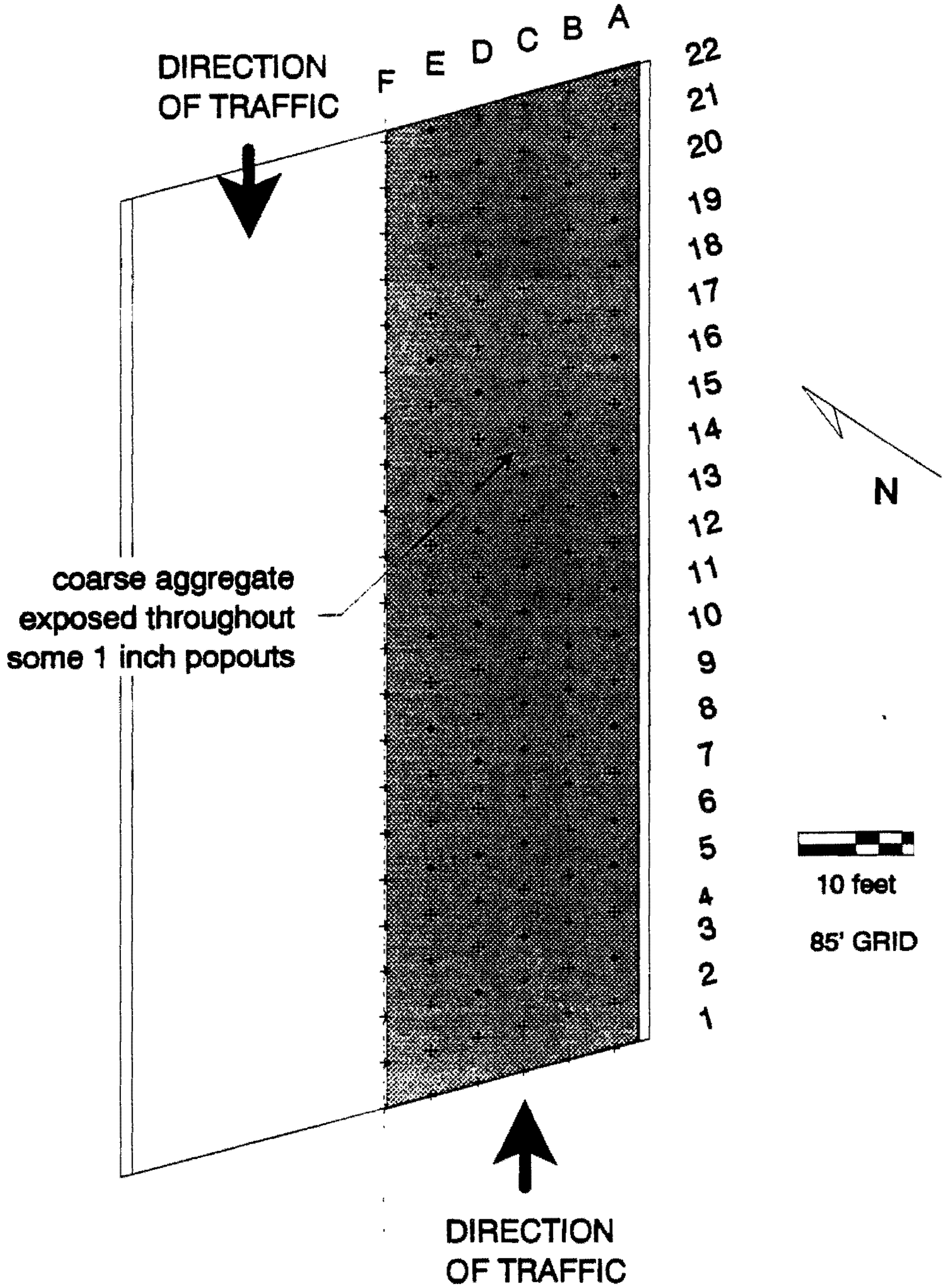
SH67 at the Brazos River
Wichita Falls
Sample Locations



Span 5 - Linseed Oil

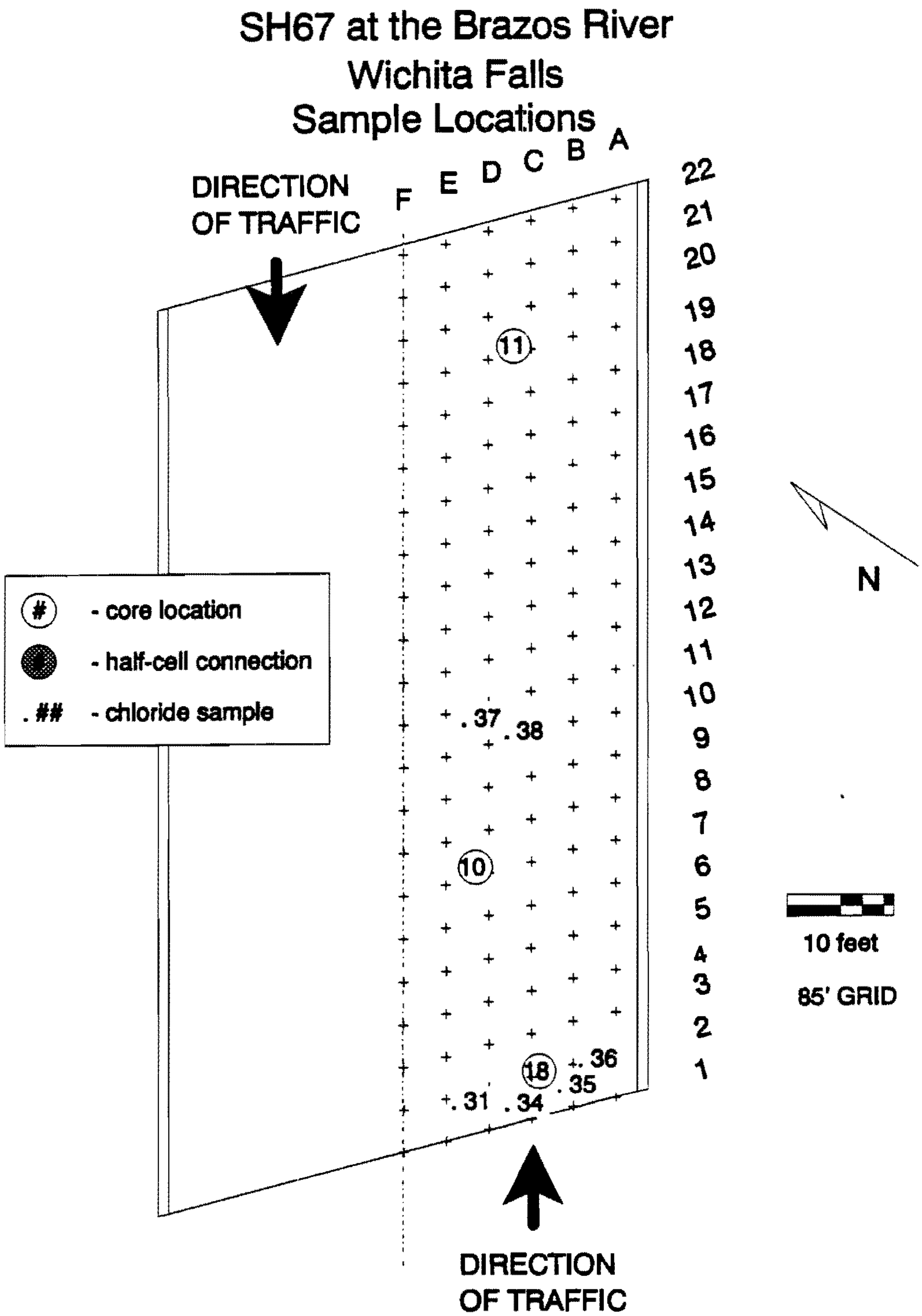
Figure A8.7

SH67 at the Brazos River
Wichita Falls



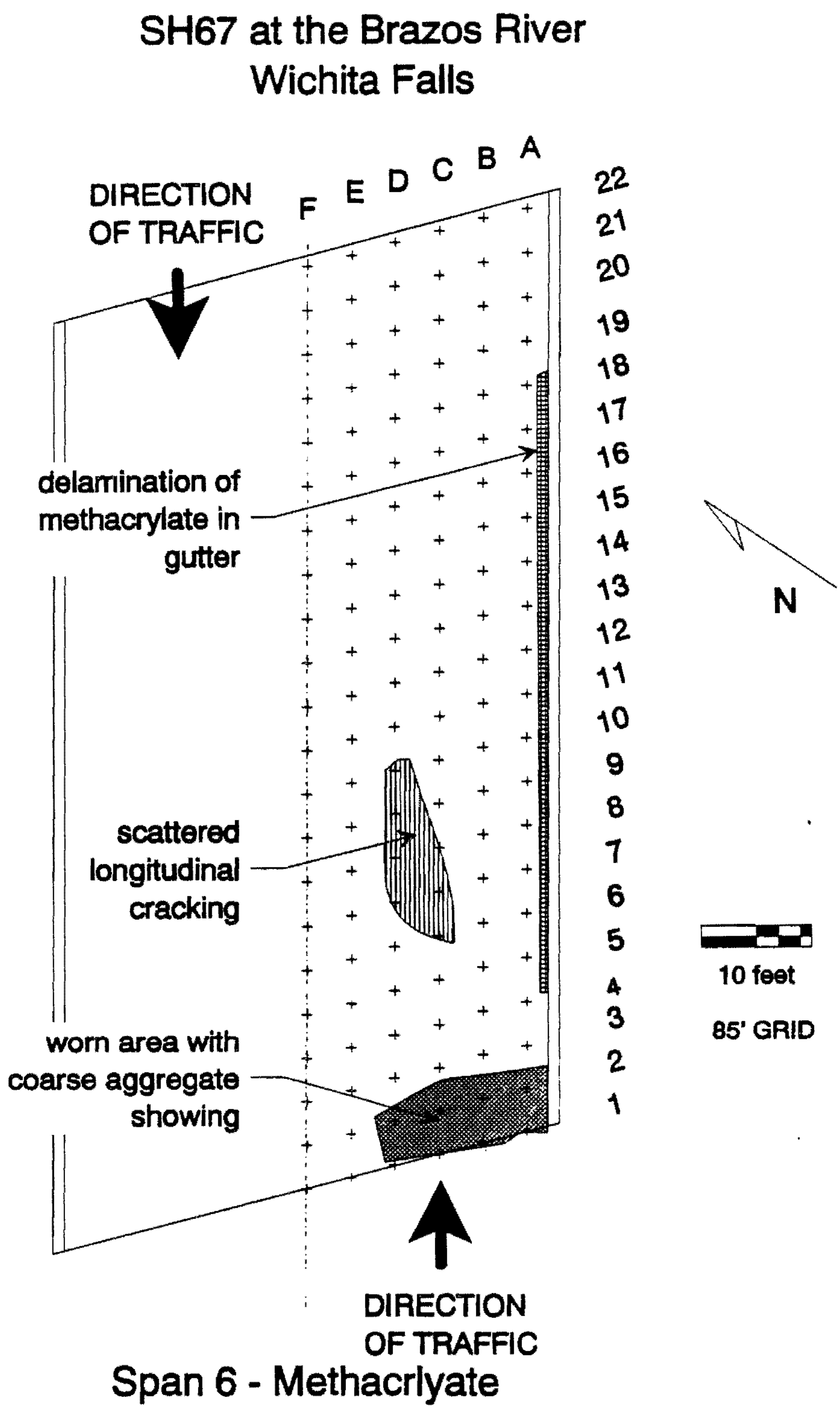
Span 5 - Linseed Oil

Figure A8.8



Span 6 - Methacrlyate

Figure A8.9



SH67 at the Brazos River
Wichita Falls
DIRECTION OF TRAFFIC
F
E
D
C
B
A
22
21
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
delamination of methacrylate in gutter
scattered longitudinal cracking
worn area with coarse aggregate showing
N
10 feet
85' GRID
DIRECTION OF TRAFFIC
Span 6 - Methacrlyate

Figure A8.10

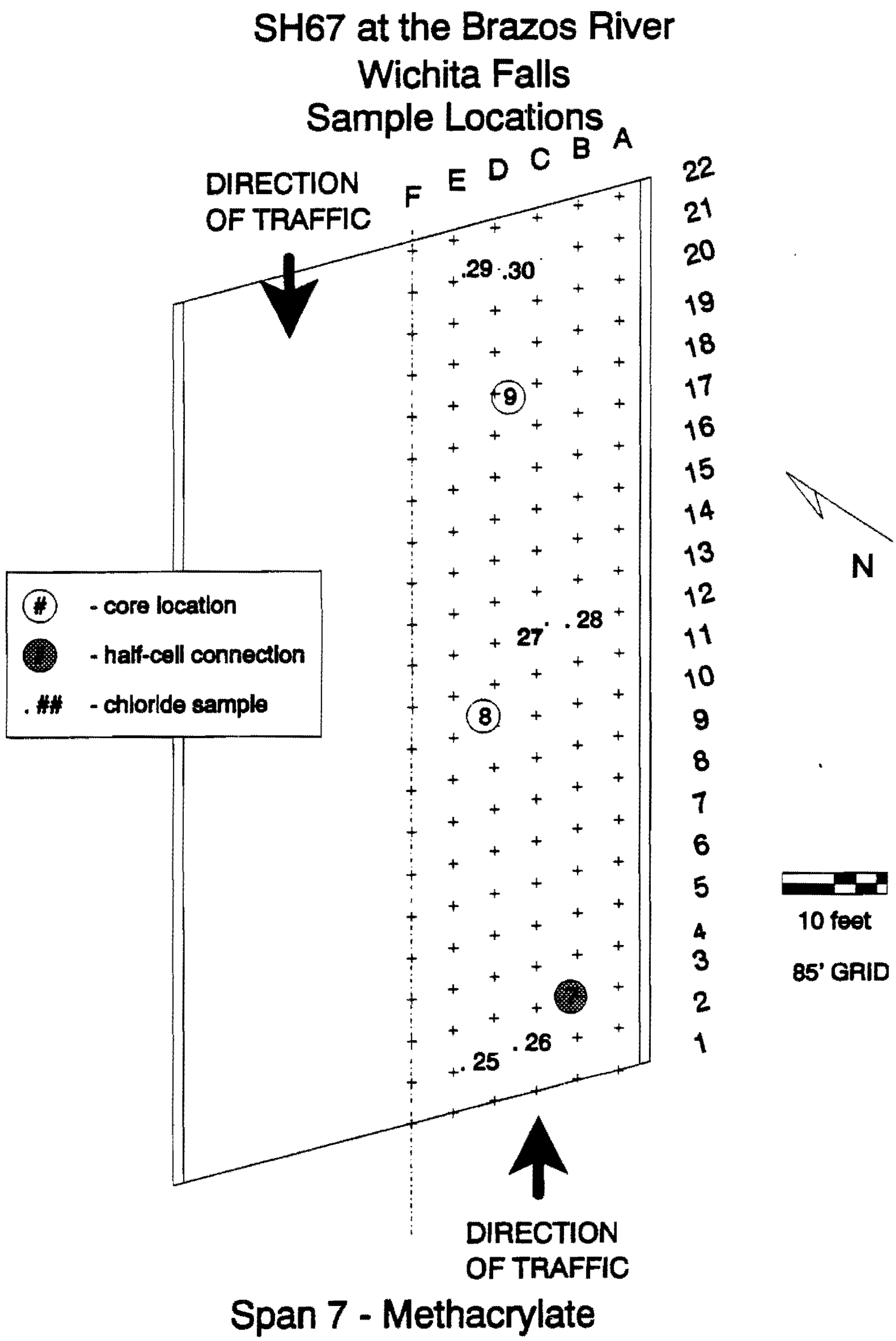
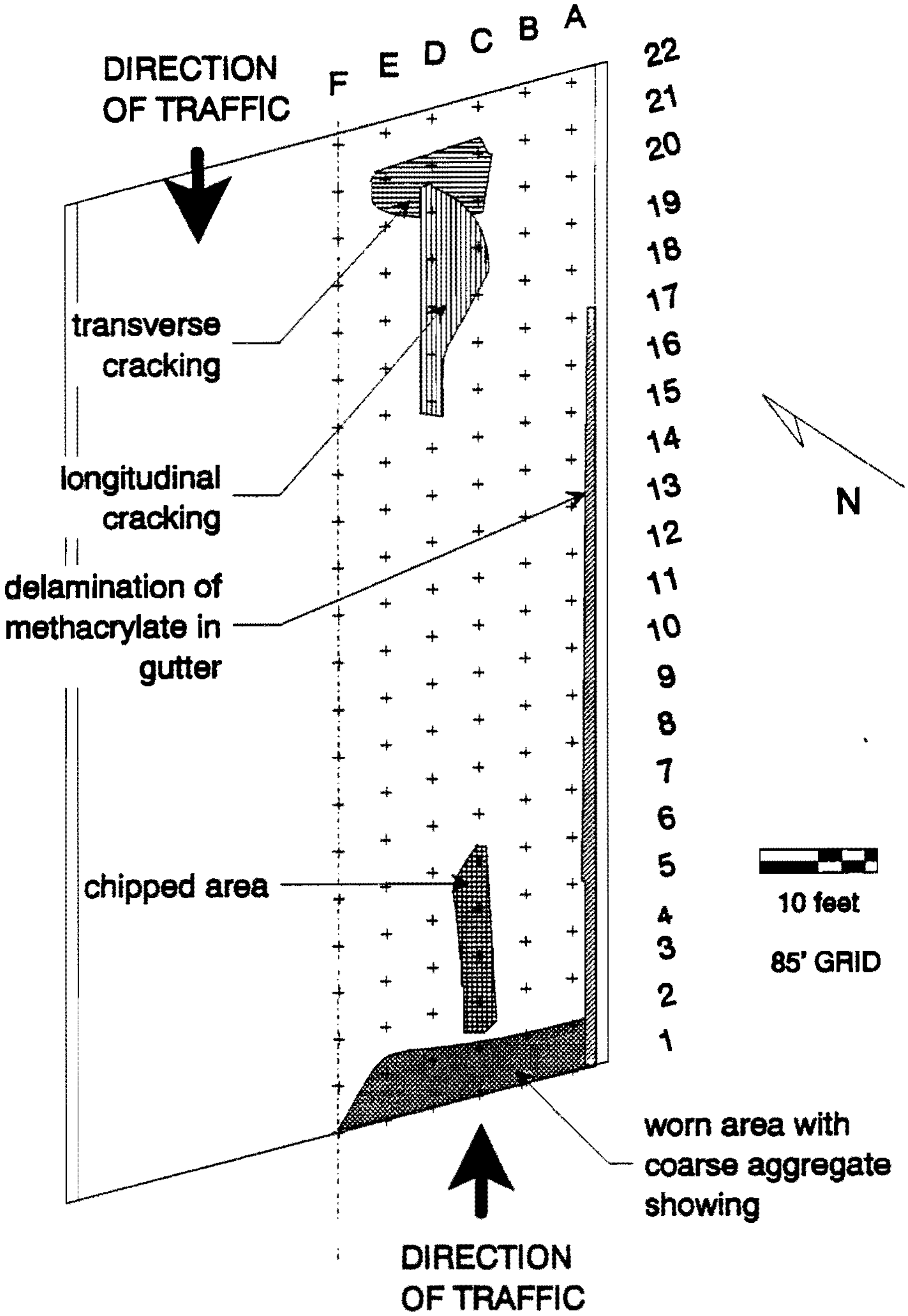

SH67 at the Brazos River
Wichita Falls
Sample Locations
A B C D E F
DIRECTION OF TRAFFIC
22 21 20 19 18 17 16 15 14 13 12 11 10 9 8 7 6 5 4 3 2 1
29 30 27 28 25 26
9
8
N
10 feet
85' GRID
- core location
- half-cell connection
. ## - chloride sample
DIRECTION OF TRAFFIC
Span 7 - Methacrylate

Figure A8.11

SH67 at the Brazos River
Wichita Falls



Span 7 - Methacrylate

Figure A8.11

SH 67 AT THE BRAZOS RIVER - HALF-CELL POTENTIALS
SPAN 7 - METHACRYLATE

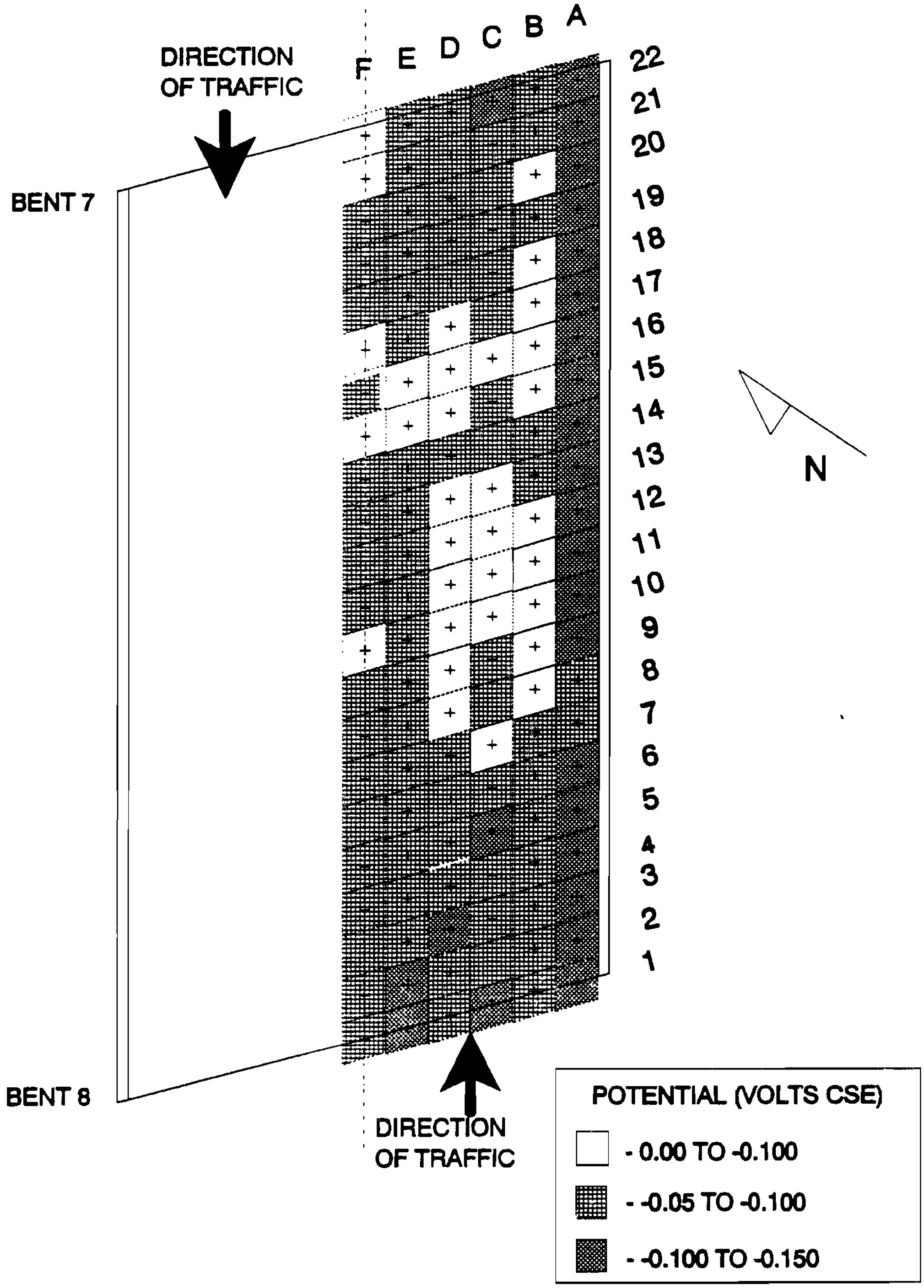
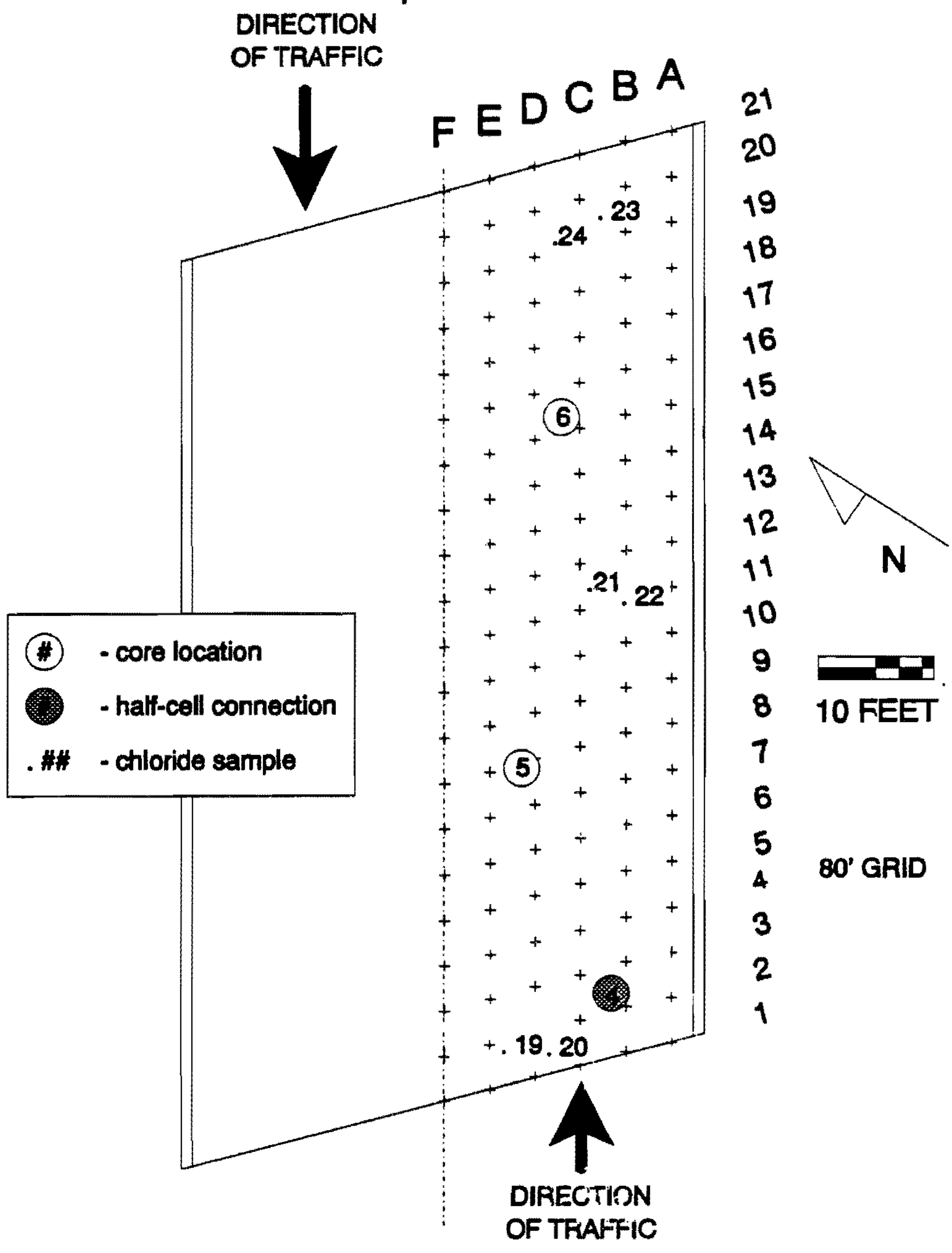


Figure A8.12

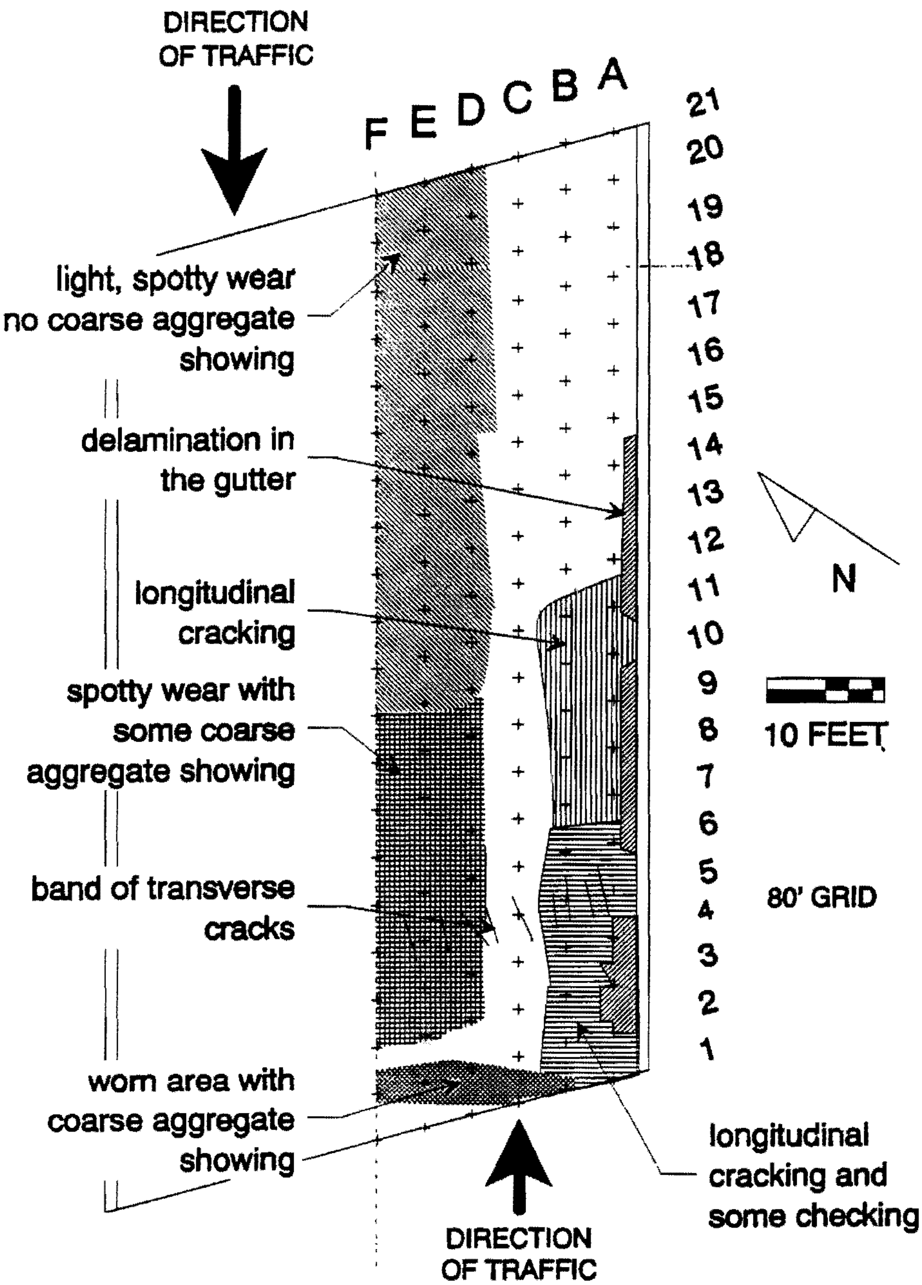
SH67 at the Brazos River
Wichita Falls
Sample Locations



Span 8 - Linseed Oil

Figure A8.13

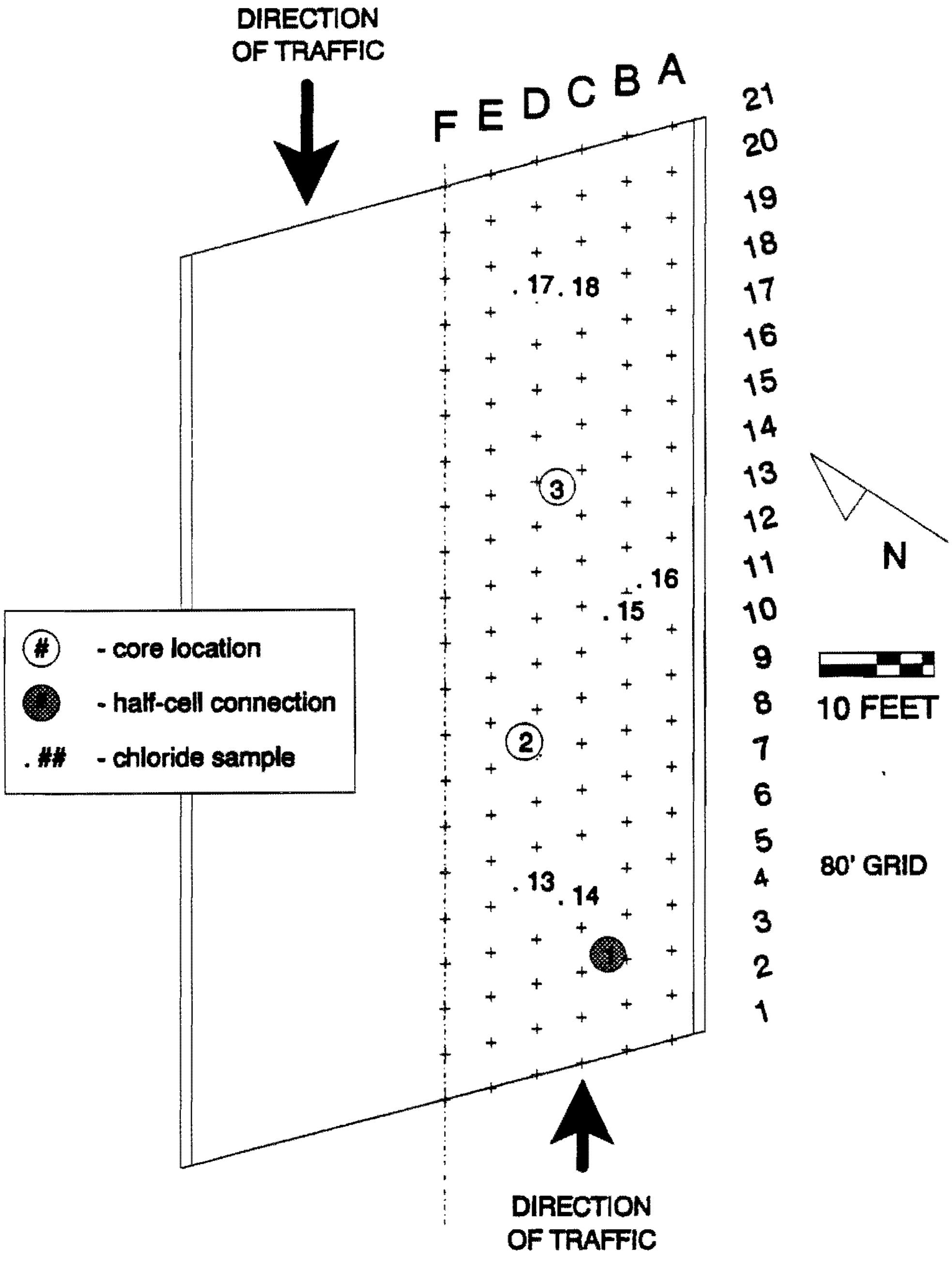
SH67 at the Brazos River
Wichita Falls



Span 8 - Linseed Oil

Figure A8.14

SH67 at the Brazos River
Wichita Falls
Sample Locations



Span 9 - Calcium Nitrate

Figure A8.15

SH67 at the Brazos River
Wichita Falls

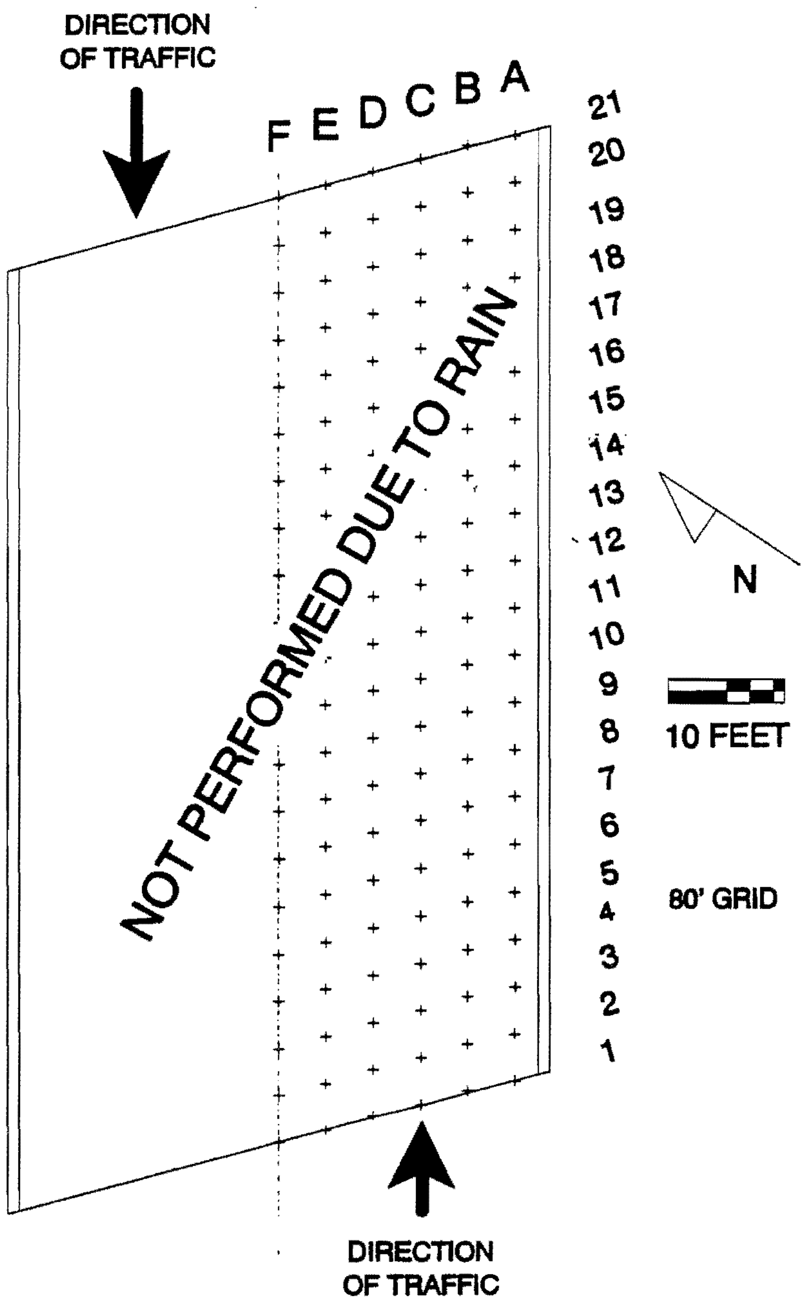
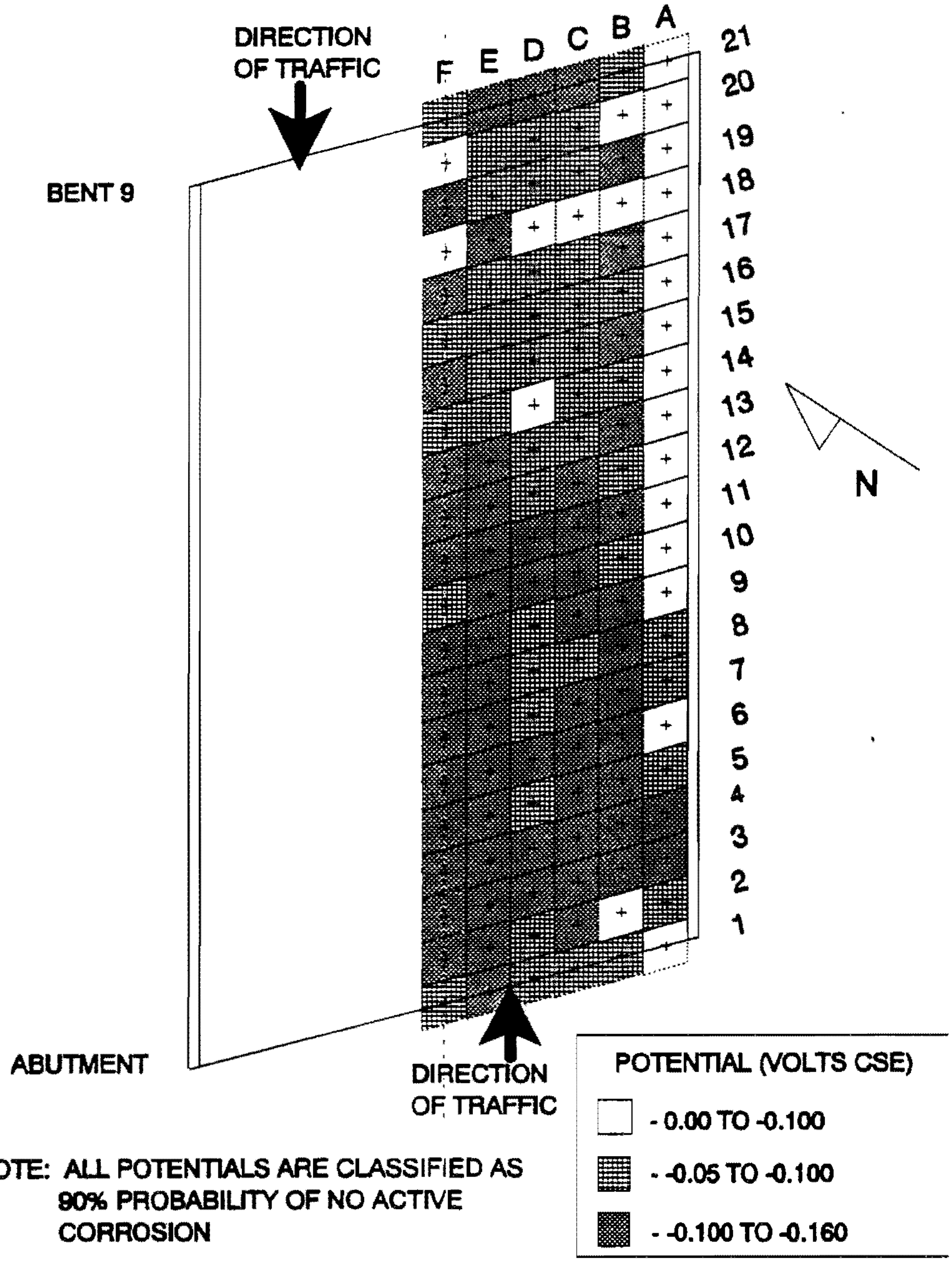


Figure A8.16

SH 67AT THE BRAZOS RIVER - HALF-CELL POTENTIALS
SPAN 9 - CALCIUM NITRATE



NOTE: ALL POTENTIALS ARE CLASSIFIED AS 90% PROBABILITY OF NO ACTIVE CORROSION

Figure A8.17

SH67 AT THE BRAZOS RIVER - WICHITA FALLS CHLORIDE CONTENTS (LB/CU. YD)

SAMPLE		SAMPLE DEPTH (IN)				REMARKS
system		0.0 - 0.5	0.5 - 1.0	1.0 - 1.5	1.5 - 2.0	
1 - Calcium Nitrate						
	13	2.80	3.20	2.40	1.92	CL on crack
	14	2.80	2.40	1.72	1.04	wheelpath on a crack
	15	3.60	3.60	3.60	2.80	breakdown lane, sound conc.
	16	2.00	1.32	0.96	0.44	breakdown lane, good conc.
	17	1.76	1.20	0.80	0.40	CL lane,in a "bald" spot of shallow grooves
	18	3.60	2.40	1.32	0.32	wheelpath
2 - Linseed Oil						
	19	1.40	0.72	0.56	0.00	CL, in area showing coarse aggregate
	20	0.96	0.48	0.24	0.36	wheelpath, same conc. as 19
	21	1.76	0.68	0.32	0.32	breakdown lane
	22	1.60	0.64	0.24	0.24	CL breakdown
	23	2.00	0.68	0.28	0.56	CL lane, good conc.
	24	1.44	0.60	0.28	0.32	wheelpath, good conc.
3 - Methacrylate						
	25	1.60	1.20	0.56	0.52	CL, good conc.
	26	1.40	0.76	0.32	0.32	wheelpath
	27	1.20	0.48	0.32	0.36	breakdown
	28	1.08	0.80	0.56	0.32	breakdown
	29	2.00	3.20	0.96	0.36	CL lane, good conc.
	30	2.40	1.20	0.36	0.32	wheelpath, good conc.
4 - Methacrylate						
	31	1.88	0.88	3.60	0.56	center of lane
	34	2.80	1.04	0.36	0.36	wheelpath
	35	1.28	0.72	0.56	0.48	CL breakdown with aggregate showing
	36	0.84	0.92	0.68	0.56	same as 35
	37	2.80	1.60	0.76	0.68	
	38	2.00	1.32	0.96	0.68	
5 - Linseed oil						
	7	1.88	1.40	0.36	0.32	CL in non-saw groove area
	8	2.00	1.92	0.56	0.36	wheelpath in non-saw groove area
	9	1.32	2.40	1.44	0.84	breakdown lane
	10	0.64	0.92	1.92	1.16	breakdown lane, sound conc.
	11	0.52	3.20	3.60	3.20	CL lane in good conc., more fine-less coarse
	12	1.32	2.80	2.00	1.92	wheelpath, same conc. as 11
6 - Linseed Oil						
(Control)	1	3.60	2.40	0.84	0.32	CL lane
	2	4.00	1.88	0.56	0.24	wheelpath
	3	1.08	0.56	0.36	0.32	
	4	0.84	0.52	0.72	0.32	breakdown
	5	3.20	0.72	0.68	0.48	CL lane
	6	3.20	2.40	0.88	0.32	wheelpath

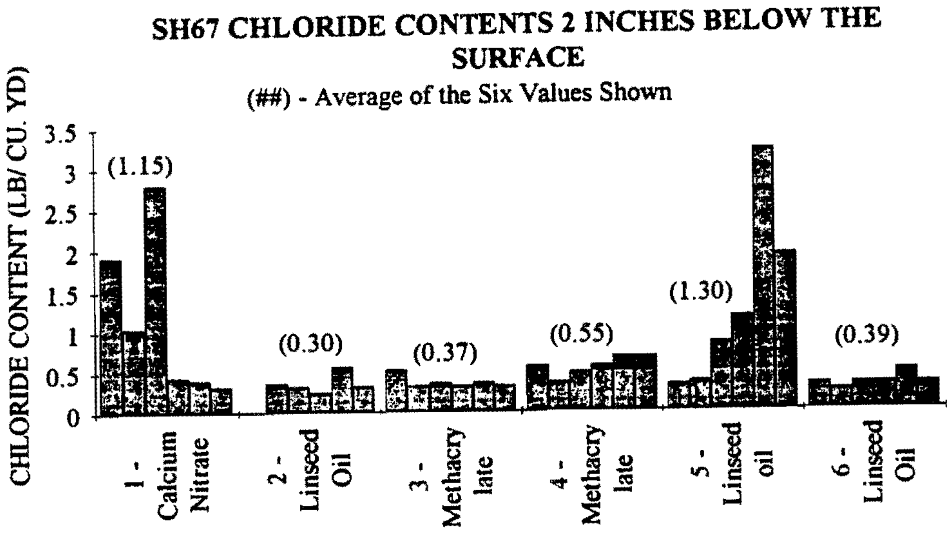


Figure A8.18

Table A8.2

SH 67 AT THE BRAZOS RIVER

Multiple Test Systems

Concrete Permeabilities (Coulombs)

		Treated Surface Intact		Surface Removed
Sample	**Surface Treatment**	**Cracked**	**Uncracked**	**Uncracked**
8-5-NB-22-C14	Linseed Oil		1976	
8-4-NB-7D-C10	Methacrylate		2283	
8-1-NB-D15-C3B	Calcium Nitrate*			2312
8-3-NB-10D-C8	Methacrylate	2584		
8-6-NB-21C-C17	None (Control)			2777
8-3-NB-18D-C9B	Methacrylate			3090
8-6-NB-11D-C16	None (Control)		3300	
8-2-NB-D8-C5	Linseed Oil	3402		
8-3-NB-18D-C9A	Methacrylate		3662	
8-1-NB-D15-C3A	Calcium Nitrate*	3939		
8-1-NB-8D-C2	Calcium Nitrate*		4169	
8-5-NB-2D-C12A	Linseed Oil		4406	
8-5-NB-9-C13A	Linseed Oil		5100	
8-5-NB-9-C13B	Linseed Oil			8027
8-5-NB-9-C12B	Linseed Oil			11638

* Although not a surface treatment, calcium nitrate is listed for convenience.

APPENDIX B

TESTING EQUIPMENT CHECKLIST

APPENDIX B
TESTING EQUIPMENT CHECK LIST

PROJECT 1300- SITE INVESTIGATION MATERIALS CHECKLIST

Site Data/Info:

__ background data
  __weather
  __de-icing agent used
  __# of de-icing applications
  __# of freeze/thaw cycles
  __ADT and % trucks
  __design loading
  __sealer materials
  __year and month built (time to Cl)
__ maps
__ grid layout
__ bridge data forms
  __cores
  __Cl samples
__ traffic control plan

Half-Cell Testing:
__ spray paint and marking plate
__ measuring tapes 30', 100'
__ brooms
__ chain drag equipment
__ Half-cell apparatus
  __supply of distilled water
  __2 surfactant bottles
  __1 spare surfactant electrode
  __1 standard or calibration cell
__ charger and transfer cables
__ Sincorder
__ copper sulfate supply
__ test leads and lead reel
__ extra lead wire
__ spare plugs and connectors
__ water supply
__ dish soap
__ small bucket or spray bottle
__ pachometer
__ generator
__ hammer-drill
__ welding apparatus
  __torch
  __hose clamps
  __solder
__ file
__ small hand chisel
__ hammer
__ tie wires
__ quick patch material
__ thermometer

__ back-up "cane" voltmeter
__ data book

Chloride Sampling:
__ portable generator with extra gas
__ electric hammer-drill w/ 3/4" masonry bits
__ drill for Tapcons
__ paint brush
__ air pump
__ small spoons
__ sample bottles
__ sample labels
__ generator
__ extension cord
__ allen wrenches
__ Cl testing briefcase
    __powder trapping fixtures
    __clamp
__ Tapcons

Crack Mapping:
__ measuring tapes
__ calipers
__ optical comparaters
__ grid maps

Core Sampling:
__ pachometer
__ coring rig(if needed)
__ portable coring equipment
__ 4" core bit
__ extractors
__ sample bags
__ sample labels
__ boxes to hold all samples
__ quick patch material
    __trowel
    __buckets
    __mixing equipment
__ sealer to patch surface treatment

Worker Attire:
__ steel-toed boots
__ long pants and T-shirts
__ reflective vests
__ hard hats
__ sunscreen

Computer/Data Reduction:
__ computer
__ printer
__ charger
__ batteries
__ computer diskettes
__ paper
__ ink cartridges

Miscellaneous:
__ toolbox
__ camera
  __flash
  __batteries
__ slide film
__ photo measuring stick and rulers
__ pads, pens, pencils
__ binders
__ extension cords
__ cooler
__ potable water supply
__ suntan lotion
__ insect repellant
__ hats
__ 3x5 cards and magic markers
__ safety glasses
__ ear plugs
__ flashlight/lantern
__ batteries
__ duct tape
__ 5-minute epoxy
__ laundry detergent
__ towels

APPENDIX C

SURVEY OF CURRENT PRACTICE

TRANSMITTAL

Dear Sir:

In response to the continuing problem of bridge deterioration due to reinforcing steel corrosion, the Texas Department of Transportation has sponsored research project 1300 to investigate the effectiveness of the many reinforcing corrosion protection systems available for use on bridge decks, sub-structures, and super-structures. The ultimate goal of the project is to determine the best systems for use in future construction and rehabilitation. the program is being conducted under the supervision of Ramon Carrasquillo and David Fowler, both of the Center for Transportation Research at the University of Texas at Austin.

The first major portion involves the determination and evaluation of corrosion protection systems in present and past use throughout Texas, in order to determine the direction and focus of further investigation. To accomplish this, the enclosed survey was developed, and is being sent to all of the TxDOT districts. We are asking you to complete the survey and to route it through the appropriate personnel within the design, construction, and maintenance divisions of your district to ensure that no information is missed. The survey consists of four portions, which should be addressed as follows:

- System Use Sheet

  To determine all of the different corrosion protection systems in past and present use, and as an aid to you in outlining the systems to be covered in detail in the questionnaire section, we have included a general one page corrosion protection system check-off sheet. Please mark the boxes corresponding to the systems and their category of use.

- General Survey of Practice Questionnaire

  The intent of this portion of the survey is to develop a performance database of the many corrosion protection systems which have been used and are currently in use in Texas. *Please **duplicate the questionnaire sheets as many times as needed** to describe all of the protection systems (epoxy reinforcing, the Texas Protection System, linseed oil, etc.) which have been used in your district for the protection of bridge decks, superstructure components, and substructure components.* Examples of corrosion protection systems can be found on the attached system summary sheet, and a sample questionnaire for one of the systems is also attached. Please note, however, that specific behaviors are as important as overall impressions, so please describe any seemingly "abnormal" experiences, making sure to indicate the location of the bridge of note.

- Performance Data Investigation

  A second part of this research project is the collection of bridge condition data, including the close inspection and monitoring of existing or future bridges. This portion of the survey will help us to determine the location and availability of performance data and will also help us to determine possible inspection sites. Please answer the two questions as thoroughly as you can, as we would much rather have to sift through too much data than not enough. Also, if you or anyone else in your district know of any installation that could be of particular interest, please let us know.

- Repair Record

  In order to quantitatively track the performances of the various bridge corrosion protection systems, we need some "hard" data describing their field performances. Hopefully, by tracking major repairs or deck replacements, some overall trends will become apparent, especially when the data from your district is combined with that of al of the other districts'. In the upper portion of the attached table, please note the number of repairs or replacements that were performed by your district within the last three years, giving the reason for repair and the type of deck which had to be replaced/repaired. Also, in the lower portion of the chart, please indicate the number of new decks installed in the last three years, according to the protection system. Please see the example chart.

When completed, the surveys should be returned to:

Project 1300 Survey of Current Practice
Construction Materials Research Group
Bldg. 18B, Mail Code 79100
University of Texas at Austin
10100 Burnet Road
Austin, TX 78758
FAX: (512) 471-4555

As a last note, please feel free to give us any information that you think might be beneficial. The most difficult part of this project will be the collection of enough pertinent data to base sound decisions on. As mentioned above, more data are better than less. Any information you have is useful, especially regarding specific strengths or weaknesses of the various protection systems, or past installation performances.

Lastly, we are looking for new or recently installed structures for use with long-term half-cell corrosion potential monitoring. The bridges must have conventional steel reinforcing to allow for these readings. If you know of any installations with conventional "black" bar reinforcing in the deck, superstructure, or substructure, please attach a brief description to the completed survey.

Please feel free to call us with any questions or comments. We would also like to thank you in advance for all of your assistance.

Sincerely

David W. Fowler

Ramon Carrasquillo

SYSTEM USE SHEET

District: ______________

What de-icing agents are used in your district?: ____________________________

Please check the boxes corresponding to the period and area of use of the following reinforcement protection systems within your district.

PROTECTION SYSTEM	PERIOD OF USE					AREA OF USE		
	experimental use only	never	past	present	future	deck	sub-structure	super-structure
none	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
epoxy coated rebar	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
cathodic protection	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
waterproof membranes	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
polymer impregnation	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
calcium nitrate	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
three inch cover	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
Texas Protection Sys.	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
overlays:								
dense concrete	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
latex concrete	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
polymer concrete	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
sealers:								
methacrylate	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
linseed oil	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
silane	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
siloxane	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
sodium silicate	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
other:								
___________	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
___________	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
___________	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
___________	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]

completed by:
date:
address:

title:
phone:
FAX:

PROJECT 1300

GENERAL SURVEY-OF-PRACTICE QUESTIONNAIRE

PROTECTION SYSTEM ______________________________

GENERAL

year of first installation of the protection system ____________
year of most recent installation of the protection system ____________
number of installations in your district ____________
estimated service life (years) ____________
will the protection system be used in future construction ____________
areas of primary use (deck, substructure, superstructure) ____________

Comments on uses:

PERFORMANCE

performance evaluation

- good → poor / too early to tell ____________
- performance consistency (highly variable Æ consistent) ____________
- comments:

(describe performance, including any unusual behavior, noting installation location)

Performance - continued

CONSTRUCTABILITY EVALUATION

- ease (easy/average/difficult) __________
- construction consistency (highly variable Æ consistent) __________
- comments - specific drawbacks/merits to the system

(please note installation location in the responses)

- comments - contractor difficulties with the procedures or materials

PROJECT 1300

PROBLEMS AND MAINTENANCE

what types of problems have been experienced - e.g. delamination, cracking

please include installation if possible, and please include as detailed a description as possible - e.g. "spalling at gutters and joints" rather than "spalling"

forecast repairs (removal, patching, reapplication, etc.)

Problems and Maintenance - continued

what maintenance/reapplication is required and how is it performed

what performance data has been collected regarding this protection system

PROJECT 1300

ESTIMATE COST INFORMATION ($/FT^2)

installation:	materials	____________
	labor	____________
	surface preparation	____________
annual maintenance:		____________

ESTIMATE COST INFORMATION

how much time is required for

lane closure/traffic control	____________
preparation	____________
actual installation	____________
curing/polymerization	____________

COMMENTS

completed by: title:
date: phone:
address: FAX:

completed by: title:
date: phone:
address: FAX:

completed by: title:
date: phone:
address: FAX:

PROJECT 1300

PERFORMANCE DATA INVESTIGATION

An important part of this study entails collecting performance evaluations of different reinforcement corrosion protection systems as an aid in determining the best systems to use for future construction.

What kinds of inspections are performed on brides in your district, and how is the information recorded? This could include special system-specific inspections, or more general annual bridge safety/condition surveys. Please describe the type and scope of the inspections.

As another part of this research project, a number of bridges with reinforcing corrosion protection systems will be closely inspected and monitored. All past and future installations are possibilities. The locations and histories of any test installations of corrosion protection systems which were "experimental" at the time of construction would also be extremely useful. Do you know of, or could you suggest, any bridges of this type? If so, please list them below.

corrosion protection system	brief description	location	age	contact person	phone

completed by:
date:
address:

title:
phone:
FAX:

PROJECT 1300

How many bridge deck rehabilitations/repairs were perfromed in the last three years in your district? What problems necessitated the repairs, and what type of repairs were done? Please fill in the number fo structures repaired due to a specific problem in the column in the column describing the type of work done.

REASON FOR REPAIR	MATERIALS REPAIRED OR REPLACED												
	OVERLAYS (note 1)					SEALED CONCRETE (note 1)					admixed	three	epoxy
	plain concrete	asphalt (note 2)	latex modified	low slump	polymer concrete	linseed oil	silane	siloxane	meth-acrylates	other	calcium nitrate	inch cover	coated rebar
delamination													
spalling													
rutting/wear													
skid resistance													
ride quality													
rust staining													
substructure deterioration													
superstructure deterioration													
______													
______													
______													
______													
______													

	MATERIALS USED FOR REPAIR OR REPLACEMENT												
	OVERLAYS (note 1)					SEALED CONCRETE (note 1)					admixed	three	epoxy
	plain concrete	asphalt (note 2)	latex modified	low slump	polymer concrete	linseed oil	silane	siloxane	meth-acrylates	other	calcium nitrate	inch cover	coated rebar
Number of installations													

note 1 - include patching in either the overlay or sealed concrete category, according to the patching material used
note 2 - this includes the Texas Protection System