Impact of Deck Cracking on Durability

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ABSTRACT

Concrete bridge decks subjected to corrosive environment because of the application of de-icing chemical could deteriorate at a rapid rate. In an effort to minimize corrosion of the reinforcement and the corresponding delaminations and spalls, the Iowa Department of Transportation started using epoxy-coated rebars (ECR) in the top mat of reinforcing around 1976 and in both mats about 10 years later. The overall objective of this research was to determine the impact of deck cracking on durability and estimate the remaining functional service life of a bridge deck. This was accomplished by conducting a literature review, visually inspecting several bridge decks, collecting and sampling test cores from cracked and uncracked areas of bridge decks, determining the extent to which epoxy-coated rebars deteriorate at the site of cracks, and evaluating the impact of cracking on service life.

Overall, 81 bridges constructed with ECR were sampled. Fick's Second Law was applied in this study to estimate the time required to reach the corrosive threshold of chloride concentration at the rebar level, i.e., the time length of the corrosion initiation stage.

No signs of corrosion were observed on the rebars collected from uncracked locations. Rebars that had surface corrosion undercutting the epoxy coating were those collected from cores that were taken from cracked locations. In general, no delaminations or spalls were found on the decks where these bars were cored. The surface chloride concentration at 0.5 inches below the deck surface and the diffusion constant were found to be 14.0 lb/yd^3 and 0.05 in^2/yr, respectively. For a corrosion threshold range from 3.6 to 7.2 lb/yd^3, the predicted service life for Iowa bridge decks considering corrosion of ECR was over 50 years. This illustrates that ECR can significantly extend the service life when compared with bridges constructed with black rebars.

1 INTRODUCTION

1.1 Background

Corrosion is a natural phenomenon that occurs when the substance of a material reacts with the surroundings in a chemical or physical process. This would eventually result in an unwanted compound. Such a process is known as oxidation, i.e., metal reacts with oxygen and the unwanted compound is rust. Corrosion can take place without visible change in a material's weight and volume. However, a corrosive material can alter its inherent physical properties and, in many cases, such as in reinforced concrete structures, will result in structural failure. According to published literature, up to 20 percent of the annual iron production in the United States is used to replace the steel that is subjected to corrosion damage (*1*). A corrosive environment can speed deterioration of materials. Nevertheless, necessary precaution procedures can be taken to prevent or delay the corrosion of a material.

Concrete bridge components constructed with uncoated reinforcement and exposed to chloride salt solutions can suffer accelerated deterioration. For example, in bridge decks, these problems stem from the use of de-icing chemicals during the winter season. Because of concrete's permeability and its natural tendency to crack, these de-icing chemicals can infiltrate the concrete and come into direct contact with the reinforcing steel, resulting in corrosion. Steel can expand three to six times its original volume when it corrodes, which could result in delaminations and spallings of some areas of the concrete (*2*). The delaminations and spallings further increase the corrosion rate of the steel by allowing even more chloride to penetrate through the concrete. To repair these problems, many bridges decks may require replacement of the upper portion of concrete and in some cases the top mat reinforcement, i.e., performing class A type repair. In some instances, bridge deck repair requires replacement of the entire depth of the deck at some location, i.e., performing class B type repair after a few years of service.

In an effort to minimize corrosion of the reinforcement and the corresponding delaminations and spalls, the Iowa Department of Transportation (Iowa DOT) and many other transportation departments started using epoxy-coated rebars as the top mat reinforcing steel in bridge decks around 1976. Approximately 10 years later, ECR were used in the top and the bottom mats. Although the performance of epoxy-coated rebars in corrosive environments is thought to be superior to typical black steel rebars, the presence of cracks in bridge decks have caused some concern as to the condition of the reinforcement and epoxy coating in these areas.

In a study conducted by the Federal Highway Administration (FHWA) in 1996 (*3*), the performance of epoxy-coated rebars in bridge decks was evaluated in various states and in some parts of Canada. The study concluded that epoxy-coated rebars were performing well, except in some circumstances. For example, the study determined that defects in the epoxy coating at cracked locations and other areas with high chloride concentrations can result in corrosion of the reinforcement that could affect the performance of a concrete structure. There was also some evidence that exposure to high chloride concentrations tends to make the epoxy coatings more brittle and weakens the bond between the epoxy and steel (*3*).

A study was conducted in 1995 by the Structure Quality Management Steering Committee of the Iowa DOT to evaluate the condition of epoxy-coated rebars at cracked locations. The study revealed that corrosion of epoxy-coated rebars was occurring at some locations along these bars. Although the findings were valuable, the study only represented the conditions of very few bridge decks that were included in the study. The committee recommended further research to evaluate the performance of epoxy-coated rebars in Iowa's bridge decks.

1.2 Objectives

The objectives of this work were to determine the impact of deck cracking on durability and estimate the remaining functional service life of a bridge deck. In addition, the results from this research need to be presented in a manner that can be used as a guide for maintenance engineers to determine when to conduct preventative maintenance or overlay bridge decks. These objectives were accomplished by completing the following tasks:

1. Review related literature. This task consisted of reviewing previous studies related to the causes of cracking and the methods used to evaluate the performance of bridge decks.
2. Analyze Iowa DOT bridge decks inspection records. This process involved analyzing data for hundreds of bridges constructed with epoxy-coated rebars in Iowa. Inspection records and ratings were used to determine what bridge characteristics had the largest impact on deck conditions.
3. Select several bridge decks for evaluation. In this procedure, bridges were grouped according to age, structure type, and location within the state. From these groupings, bridges were selected so that the sample would be representative of Iowa's bridges.
4. Select bridge evaluation procedures. This task involved choosing and implementing evaluation techniques that would be economically feasible and provide the data necessary to assess the bridge and reinforcement conditions.
5. Conduct field and laboratory evaluation. The field and laboratory evaluation process consisted of several procedures and tasks conducted on the bridges during coring and in the laboratory during sample analysis.
7. Study the effect of using two-course placement construction and sealed bridge decks on chloride diffusion through decks.
8. Compile and analyze data. This task involved compiling the collected data to determine the diffusion constant for estimating chloride infiltration through a bridge deck and the condition of ECR.
9. Evaluate the impact of deck cracking on deck durability. This task investigated the effects of deck cracking on the durability and the performance of a bridge deck in the state of Iowa.

10. Evaluate the performance of ECR in Iowa bridge decks. This task compared the performance of ECR in Iowa bridge decks in comparison with that using plain steel reinforcement.

The results obtained from this research will help to determine the impact that deck cracking is having on service life. The information regarding current conditions of bridge decks constructed with epoxy-coated rebars will be an asset to engineers to determine when an overlay for a bridge deck is needed.

2 LITERATURE REVIEW

The chloride ion–induced corrosion damage of bridge decks has been known to highway agencies for several years. In the following sections, the corrosion mechanism, the corrosion process, and a model that can be used to determine the chloride ion diffusion in Iowa bridges are summarized. This information is necessary to develop a model that can be used to determine the service life of bridge decks.

2.1 Corrosion Process

The corrosion of reinforcement in bridge decks results from an electrochemical process (*4*). For this process to occur there has to be a current flow, which results from a potential difference between two nodes. In most cases, the top mat of steel in a bridge deck acts as the cathode and the bottom mat acts as the anode.

The amount of corrosion present and the rate at which the steel corrodes depend on various factors. Wet and dry cycles accelerate the corrosion process and, thus, make the environment an important factor (*4*). It has been found that the corrosion rate is highest during the spring season and lowest during the winter (*5, 6*). These rates can vary by a factor of about four or five times during the year (*5*).

The degree of interconnection between the rebars also has a direct impact on the corrosion rate. Decks without epoxy-coated rebars normally conduct current throughout the reinforcement quite well because the reinforcing steel is in direct contact with the surrounding concrete. This acts to significantly increase the corrosion of the reinforcement and reduce the life span of a bridge deck. The use of epoxy-coated rebars in bridge decks could decrease this type of continuity, but defects in the epoxy coating and careless material handling still allow contact between the reinforcing bars and concrete (*4*). Another significant factor that has an effect on the corrosion rate is the cathode-to-anode ratio of the steel found within the bridge deck (*5*).

The problems that result from corrosion of the reinforcement are the decrease in the structural capacity of the steel and the increase in the volume of the steel due to the corrosion products. Since the volume of the corrosion products is much larger than the volume of the original steel, significant stresses within the concrete can be induced, resulting in delaminations and spalls of the surrounding concrete (*2*).

The point at which delaminations and spalls start to occur is subject to some variability. According to Broomfield (*7*), “It has been shown that cracking is induced by less than 0.1 mm of steel section loss, but in some cases far less than 0.1 mm has been needed.” Broomfield (*7*) also stated that once these cracks occur, they allow for even more exposure of the steel to de-icing chemicals and the environment. This acts to further increase the corrosion of the reinforcement and can have a noticeable impact on the bridge deck and underlying structure.

2.2 Corrosion Mechanism

To investigate the performance of ECR in a bridge deck, one needs to understand the concept of the corrosion mechanism of reinforcement in the concrete. This knowledge provides insights and addresses the causes of the corrosion of reinforcement in concrete. Appendix A summarizes in some detail the corrosion mechanism of reinforcing rebars.

2.3 Condition Evaluation Methods for Bridge Decks

There have been many test methods and procedures devised to evaluate the condition and future performance of concrete bridge decks. Some of the tests mentioned by the FHWA (*8*) include

- visual inspection,
- delaminations survey,
- depth of cover measurements,
- determination of chloride content in concrete,
- electrical continuity tests,
- corrosion potential mapping,
- corrosion rate measurements,
- determination of cross-section loss on reinforcing steel,
- petrographic analysis, and
- rebound number and penetration resistance tests.

Although all of these procedures were not used directly in this work, knowledge of the various tests available was important in developing tests that would be beneficial and economical in the analysis used in this study. Furthermore, some of the tests discussed here may be incorporated into future work. Descriptions of the evaluation techniques follow.

2.3.1 Visual Inspection

The visual inspection of a bridge is a systematic procedure that includes locating and recording all defects found in the structure (*8*). Cracking, spalling, pop-outs, scaling, rust stains, and patches are the main concerns documented, and the location, type, and severity of these defects are noted on standard data sheets developed for the inspection. To aid in this process and other evaluation procedures, a grid system can be laid out on the surface of the deck or other structural members.

2.3.2 Delaminations Survey

There are several testing methods that can be used to determine where concrete is delaminated (*8*). The most common technique used is sounding, although other more expensive and elaborate methods may be more accurate and should be used if possible. In the sounding method, a steel hammer, rod, or chain is used to create sound vibrations within the concrete. If a sharp ringing sound is produced, the concrete is not delaminated.

If a dull, hollow sound is produced when a hammer strikes the concrete or when a chain is dragged across the surface of the concrete, the concrete is likely to have delaminations present. Areas of delaminations are then marked directly on the surface of the concrete or mapped and recorded for future investigation. After delaminated areas are marked on the bridge deck, the percentage of delaminated areas can be computed.

It should be noted that operator judgment and the presence of overlay can influence the results of the sounding method (*8*). The sounding method should not be used on bridge decks overlaid with asphalt. When the sounding method is used on bridge decks overlaid with cement concrete mixtures, it will detect debonding of the overlay, which will affect the validity of the results.

The Iowa DOT had not noticed any delaminations or spalls on bridges built with epoxy-coated rebars. Therefore, a delaminations survey was not recommended for the work presented herein.

2.3.3 Depth of Cover Measurements

The depth of cover can be obtained by using a nondestructive pachometer or a "covermeter," by drilling small diameter holes to expose reinforcing steel for direct measurement, or by measuring the cover depth in extracted cores (*8*). Covermeters determine the depth of cover by measuring variations in magnetic flux caused by the location of steel. For the covermeters to be accurate, the size of reinforcing steel has to be known so that the readings can be interpreted for depths.

Any of the above methods could be used in bridge deck evaluations, although using a covermeter may provide more depth measurements and, thus, give a better understanding of the true depth of cover over various parts of the bridge deck. Currently, there is no standard practice available for this technique (*8*).

2.3.4 Determination of Chloride Content in Concrete

The chloride concentration in the concrete at the reinforcing bar level is a major factor in the corrosion of reinforcing steel. Chloride ions can reach the reinforcing bar level by permeating through the concrete or by penetrating through cracks in the concrete. To initiate corrosion in the concrete, the concentration of chloride ions in the concrete must reach the corrosion threshold value for black bars of about 1.2 lb/yd3 (0.71 kg/m^3) (*9, 10*).

The chloride content of concrete can be evaluated using several different methods. The American Association of State Highway and Transportation Officials (AASHTO) T 260-94 gives three procedures for determining the chloride content in concrete (*8*). In procedure A, which is a very time consuming and complicated test, the chloride content is determined by potentiometric titration in a laboratory. Procedure B, utilizes an atomic absorption process in a laboratory to determine the chloride content in concrete. In procedure C, the chloride content is determined using a specific ion probe in the laboratory or the field. Procedure C was recently developed by the Strategic Highway Research Program (SHRP) and is simpler and easier to use than the other two procedures. It is supposed to give relatively accurate results.

Procedure C involves using an impact hammer with a stopping gage to drill out concrete powder at the desired depth and using a vacuum system with a collection unit to retrieve the powder. Three grams of the powder sample is then placed in 20.0 milliliters of digestion solution and shaken vigorously. Next, 80 milliliters of stabilizing solution is then added to the sample and shaken. Finally, a specific ion probe is inserted into the solution and voltage readings are taken. To determine the chloride content, the millivolt readings are mathematically converted into percent chloride by weight of concrete. Another alternative is to use an X-ray fluorescence spectrometer to analyze the chloride content in the samples.

2.3.5 Electrical Continuity, Corrosion Potential, and Corrosion Rate Tests

Electrical continuity testing must be performed on a bridge deck prior to performing corrosion potential mapping and corrosion rate measurements (*8*). The electrical continuity of the reinforcing steel must be known in order for corrosion potential and corrosion rate results to be valid. The corrosion potential and corrosion rate tests are useful for determining the state of corrosion and rate of corrosion of reinforcing steel on bridge decks without epoxy-coated rebars. These tests can also be run on decks with epoxy-coated rebars, but it is very arduous and is not recommended because there are no data interpretation guidelines developed for evaluating the voltage measurements.

The state of corrosion of reinforcing steel is found by making electrical connections to the reinforcement with a half-cell and taking voltage readings at various locations on the deck (*8*). The half-cell potential readings are then used to interpret the state of corrosion of the reinforcement. If enough readings are taken, a map of reinforcement corrosion can be developed for the deck that shows the reinforcement condition along the entire deck surface graphically.

The rate of corrosion of reinforcing steel is found by using a corrosion rate device (*8*). The corrosion rate device induces small currents or voltages into the reinforcing steel and measures the corresponding response. The voltage or current measurements are then mathematically converted into corrosion rates. The results of this test can be used to approximate the life of the bridge deck or to decide when it should be repaired.

2.3.6 Determination of Cross-Section Loss on Reinforcing Steel

The cross section loss in rebars is found by directly measuring the loss in diameter due to corrosion and comparing it with the original diameter (*8*). During this procedure, it is very important that corroded material is cleaned from the surface and that accurate measurements are taken. The results obtained from several of these measurements could give a direct indication of the deck's condition.

2.3.7 Petrographic Analysis

A petrographic analysis requires drilling concrete cores from the bridge deck and examining them with the unaided eye and with a microscope. This allows for detection of deterioration that could otherwise not be found. Information that can be obtained from a very thorough petrographic analysis is as follows (*8*):

- condition of material;
- causes of inferior quality;
- identification of distress or deterioration caused by chloride induced corrosion, carbonation, alkali-aggregate reactions, freeze-thaw cycles, etc.;
- probable future performance;
- compliance with project specifications;
- degree of cement hydration;
- estimation of water-cement ratio and unit weight;
- extent of paste carbonation;
- presence of fly ash and estimation of amount of fly ash;
- evidence of sulfate and other chemical attack;
- identification of potentially reactive aggregates;
- evidence of improper finishing;
- estimation of air content and how much of the air voids are entrained versus entrapped;
- evidence of early freezing; and
- assessment of the cause of cracking.

2.3.8 Rebound Number and Penetration Resistance Tests

The rebound number and the penetration resistance tests are conducted to determine the strength of the concrete at various locations on the deck (*8*). In the rebound number test, a device with a spring driven hammer is used. The distance the hammer rebounds after being dropped indicates the hardness of the concrete, which can be correlated to compressive strength. The penetration resistance test involves using a special gun to drive a small rod into the concrete. The further the rod penetrates the concrete, the smaller the strength of the concrete.

These tests were not run on the bridges evaluated in this work because the presence of cracks near the area being tested can influence the results. Since about half of the cores drilled came from cracked locations, the results could have been deceiving if these tests were run.

All the tests outlined above give important information on the condition of bridge decks, but some of the techniques only apply to bridges with certain characteristics and aren’t necessary or feasible for the work presented herein.

2.4 End of Functional Service Life of Bridge Deck

The estimate of bridge deck durability involves defining the time at which rehabilitation of a bridge deck is required. For a bridge deck the end of functional service life is reached when severe deterioration occurs. Although a deteriorated deck can still serve for traffic and it poses no immediate danger of collapse, the public insists that the traffic agency provide a smooth riding surface. Rehabilitation can range from patching deteriorated areas to overlaying an entire bridge deck with a new riding surface when the cracks, delaminations, spalls, and patching on the concrete deck exceed a reasonable limit.

Weyers et al. (*12*) conducted an intensive opinion survey of 60 bridge engineers to quantify the end of functional service life (*12*). The study concluded that, "based on recommended practices, it is likely that the end of functional service life for concrete bridge decks is reached when the percentage of the worst traffic lane surface area that is spalled, delaminated, and patched with asphalt ranges from 9.3% to 13.6%." Also Weyers et al. (*12*) also documented that "based on current local practices, it is likely that the end of functional service life for concrete decks is reached when the percentage of the whole deck surface area that is spalled, delaminated, and patched with asphalt ranges from 5.8% to 10.0%."

According to Iowa DOT practice, overlaying is performed when the whole deck surface that is spalled, delaminated, and patched with asphalt reaches about 8 to 10 percent (*13*).

2.5 Models for Estimating the Bridge Deck Service Life

Weyers et al. (*14*) summarized two methods of estimating the service life of a deteriorated bridge deck. The first approach, referred to as the diffusion–cracking-deterioration model, estimates the service life using the concepts of chloride diffusion period, and corrosion cracking and deterioration period. The other method is referred to as the diffusion-spalling model. This two-step procedure assumes that rehabilitation will take place only after spalling or delaminations have occurred on 9 to 14 percent of a deck surface, which was defined as the end of functional service life. Because of its simplicity, the latter was selected and was used in this work. The following section discusses the corrosion process of this model.

2.6 Corrosion Process Model

Corrosion of reinforcing steel in concrete can be modeled as a two-stage process. The first stage is known as the initiation or incubation period, in which chloride ions transport to the rebar level. In this stage the reinforcing steel experiences negligible corrosion. The time, T_1, required for the chloride concentration to reach the threshold value at the rebar level can be determined by the diffusion process of the chloride ion through concrete following Fick's Second Law (see section 2.6; *14*). In the second stage, known as the active and deterioration stage, corrosion of reinforcing steel occurs and propagates, resulting in a noticeable change in reinforcing rebar volume that could induce cracking and spalling of the surrounding concrete. The length of the second stage, $T_{2,}$ depends on how fast the corroded reinforcing rebars deteriorate resulting in an observable distress. Figure 2.1 illustrates an arithmetic plot of cumulative percent deterioration versus time based on the above model generated an S-shaped (ogive) curve (*15*). Although it is not an easy task to predict, once again, the length of the second stage, eventually a deck will reach a condition at which some types of maintenance activities must be taken.

The corrosion model discussed above was often used to assess corrosion of uncoated rebars (*14*). This concept was assumed herein to be applicable to estimate the service life of a bridge deck constructed using ECR. However, the corrosive threshold initiating corrosion of ECR and the length of the active and deterioration stage should be higher than those of uncoated steel bars. The determination of the length of these two stages is outlined in sections 4, 5, and 6 of this report.

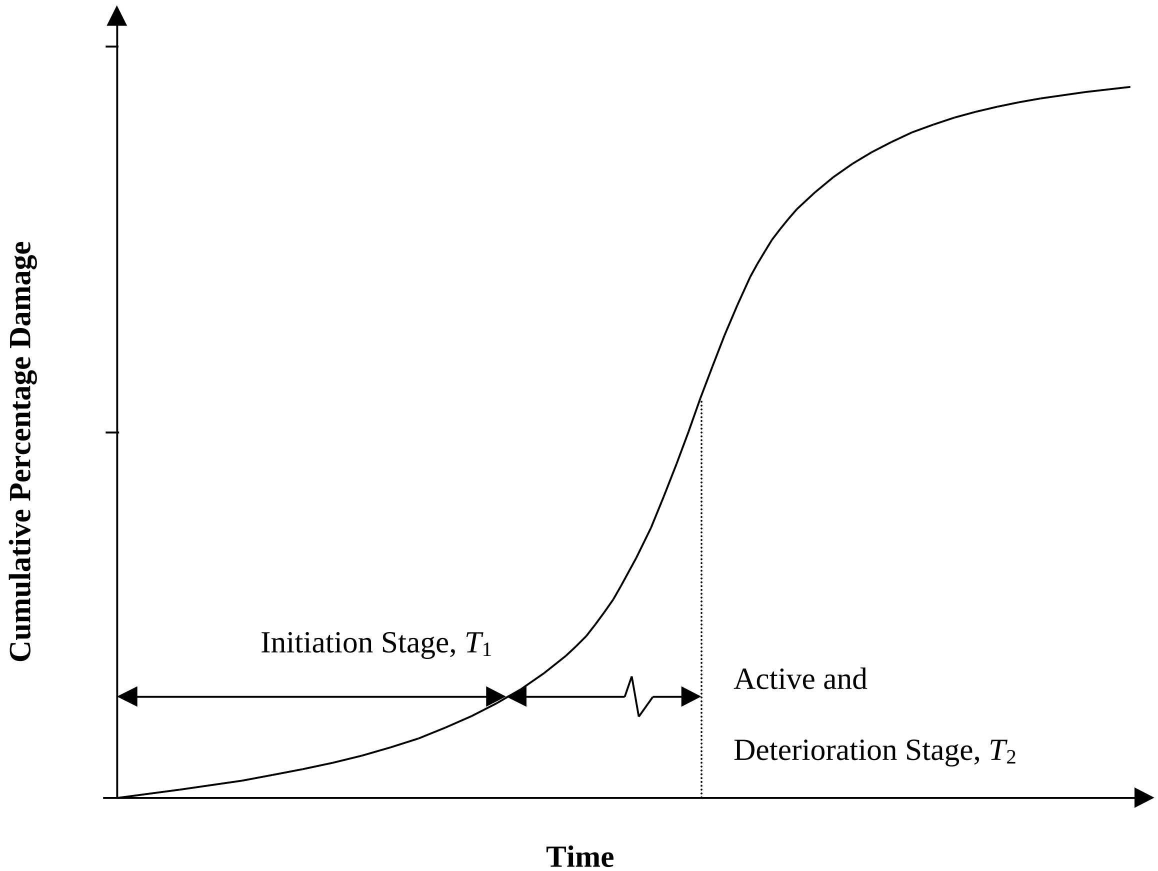

FIGURE 2.1 General Deterioration Curve versus Time

2.7 Corrosion Threshold

As discussed previously, chloride ions penetrate through concrete capillaries. As a result of chloride ion ingress, the chloride concentration may reach a corrosive threshold at the reinforcing bar level. This will initiate corrosion resulting in concrete distressed due to the change of reinforcing rebar volume. Ultimately, spalls and delaminations accelerate the deterioration of bridge deck and reduce its durability. The corrosion threshold at the steel bar level was determined to be 0.2 percent by weight of the cement content of concrete (*16, 17*). Cady and Weyers (*18*) estimated the corrosion threshold for unprotected reinforcement to be 1.2 lb/yd^3 (0.73 kg/m^3) of concrete based on 6.5 sacks of cement per cubic yard of concrete. However, it is believed that the use of ECR will delay the time required to initiate corrosion. As a result, the corrosive threshold should be higher than that for the bare steel bar. Sagues et al. (*19*) suggest a range of the corrosive threshold for ECR from 1.2 to 3.6 lb/yd^3. These limits will be investigated in this research using the chloride concentration–rebar rating relationships of ECR collected from bridges across the state of Iowa.

2.8 Fick's Second Law for Chloride Ions Ingression in Concrete

Fick's Second Law is the most common technique used to determine the length of the initiation stage, i.e., the time, T_1, it takes chloride ions to migrate through a bridge deck and reach the top reinforcing steel. Fick's Second Law assumes that the chloride ion diffuses in an isotropic medium (*20*). The fundamental second order differential equation of Fick's Second Law is as follows:

$$\frac{\partial C}{\partial t} = D_c \frac{\partial^2 C}{\partial x^2} , \qquad (2.1)$$

where C = chloride concentration with depth (in inches), t = time (in years), x = depth (in inches), and D_c = diffusion constant (in in^2/yr).

A closed form solution of the above differential equation for a semi-infinite deck, i.e., a small ratio of depth to length or width of a deck, can be expressed as follows (*21*):

$$C_{(x,t)} = C_o \left[1 - erf \left(\frac{x}{2\sqrt{D_c t}} \right) \right] , \qquad (2.2)$$

where $C_{(x,t)}$ = measured chloride concentration at desired depth, C_o = constant surface concentration measured at 0.5 inches below the deck surface (in lbs/yd^3; see section 2.8.1 for further discussion of C_o),

$$\operatorname{erf}(y) = \frac{2}{\sqrt{\pi}} \int_0^y e^{-s^2}\, ds \,, \qquad (2.3)$$

t = time (in years), and x = depth measured from the deck surface (in inches).

The erf(y) function is the integral of the Gaussian distribution function from 0 to y. Utilizing Matlab (*21*) program generated values of the integration of equation 2.3 and the results are given in Table 2.1.

2.8.1 Surface Chloride Content

As can be seen, the application of Fick's Second Law to assess the initial time to corrosion requires the determination of the surface chloride content, C_o, and the diffusion constant, D_c. Weyers et al. (*20*) investigated the chloride concentration in bridge decks and concluded that the chloride content measured at 0.5 inches from the deck surface reached a stable condition after it had been in service for four to six years. For this reason Weyers et al. recommended using a chloride concentration measured at 0.5 inches from the deck surface as the surface chloride concentration, C_o, in equation 2.2.

One should realize that the steel bars will not commence corrosion when the chloride content ingress reaches the rebar level, but rather it takes some time to initiate the corrosion and break the passive protection layer formed by the concrete alkalinity. Thus, it is reasonable to assume that corrosion begins when the chloride ion penetrates to another 0.5 inches below the top layer of the reinforcing bar and reaches the corrosion threshold. Consequently, the depths of 0.5 inches below the deck surface and 0.5 inches below the top layer of reinforcing bar are canceled out (*14*).

2.8.2 Chloride Diffusion Constant

The transport of chloride ions in concrete is assumed to be a diffusion process in one dimension, downward in the case of bridge decks. In reality, the ingress of chloride ions in concrete can be attributed to the means of concrete capillaries and cracking. Apparently, the concrete quality affects the phenomenon of the diffusion process in terms of time needed for chloride content to reach a certain level. The omnipresent cracking that increases the rate of chloride diffusion is affected by many factors, such as traffic volume, water-cement ratio, temperature fluctuation, and the curing and construction process. For example, Herald (*23*) observed the strong correlation between the diffusion constant and the water-cement ratio in controlled experimental specimens. Moreover, Brown (*21*) concluded that temperature has a significant impact on the diffusion process of chloride in hardened cement paste. Thus, the diffusion constant is characterized by the construction practice from state to state. The following sections briefly summarize some factors that influence the diffusion of chloride in concrete decks.

TABLE 2.1 Error Function Values *y* for the Argument of *y*

y	erf(*y*)	*y*	erf(*y*)	*y*	erf(*y*)	*y*	erf(*y*)	*y*	erf(*y*)
0.02	0.022565	0.62	0.619411	1.22	0.915534	1.82	0.989943	2.42	0.999379
0.04	0.045111	0.64	0.634586	1.24	0.920505	1.84	0.990736	2.44	0.999441
0.06	0.067622	0.66	0.649377	1.26	0.925236	1.86	0.991472	2.46	0.999497
0.08	0.090078	0.68	0.663782	1.28	0.929734	1.88	0.992156	2.48	0.999547
0.10	0.112463	0.70	0.677801	1.30	0.934008	1.90	0.992790	2.50	0.999593
0.12	0.134758	0.72	0.691433	1.32	0.938065	1.92	0.993378	2.52	0.999635
0.14	0.156947	0.74	0.704678	1.34	0.941914	1.94	0.993923	2.54	0.999672
0.16	0.179012	0.76	0.717537	1.36	0.945561	1.96	0.994426	2.56	0.999706
0.18	0.200936	0.78	0.730010	1.38	0.949016	1.98	0.994892	2.58	0.999736
0.20	0.222703	0.80	0.742101	1.40	0.952285	2.00	0.995322	2.60	0.999764
0.22	0.244296	0.82	0.753811	1.42	0.955376	2.02	0.995719	2.62	0.999789
0.24	0.265700	0.84	0.765143	1.44	0.958297	2.04	0.996086	2.64	0.999811
0.26	0.286900	0.86	0.776100	1.46	0.961054	2.06	0.996423	2.66	0.999831
0.28	0.307880	0.88	0.786687	1.48	0.963654	2.08	0.996734	2.68	0.999849
0.30	0.328627	0.90	0.796908	1.50	0.966105	2.10	0.997021	2.70	0.999866
0.32	0.349126	0.92	0.806768	1.52	0.968413	2.12	0.997284	2.72	0.999880
0.34	0.369365	0.94	0.816271	1.54	0.970586	2.14	0.997525	2.74	0.999893
0.36	0.389330	0.96	0.825424	1.56	0.972628	2.16	0.997747	2.76	0.999905
0.38	0.409009	0.98	0.834232	1.58	0.974547	2.18	0.997951	2.78	0.999916
0.40	0.428392	1.00	0.842701	1.60	0.976348	2.20	0.998137	2.80	0.999925
0.42	0.447468	1.02	0.850838	1.62	0.978038	2.22	0.998308	2.82	0.999933
0.44	0.466225	1.04	0.858650	1.64	0.979622	2.24	0.998464	2.84	0.999941
0.46	0.484655	1.06	0.866144	1.66	0.981105	2.26	0.998607	2.86	0.999948
0.48	0.502750	1.08	0.873326	1.68	0.982493	2.28	0.998738	2.88	0.999954
0.50	0.520500	1.10	0.880205	1.70	0.983790	2.30	0.998857	2.90	0.999959
0.52	0.537899	1.12	0.886788	1.72	0.985003	2.32	0.998966	2.92	0.999964
0.54	0.554939	1.14	0.893082	1.74	0.986135	2.34	0.999065	2.94	0.999968
0.56	0.571616	1.16	0.899096	1.76	0.987190	2.36	0.999155	2.96	0.999972
0.58	0.587923	1.18	0.904837	1.78	0.988174	2.38	0.999237	2.98	0.999975
0.60	0.603856	1.20	0.910314	1.80	0.989091	2.40	0.999311	3.00	0.999978

2.8.2.1 Permeability Although concrete is a dense material, it contains pores. Ultimately pores form a network of paths, allowing salt, water and oxygen ingress into concrete, which initiates the corrosion of steel bar. Conventional concrete without special treatment is permeable. The permeability of concrete is the physical property of concrete to resist the migration of water or ions through concrete. Thus, low permeability concrete provides sufficient resistance for the penetration of chloride ions dissolved in water and other chemical attacks.

Generally the permeability of concrete is the function of pore size, water-cement ratio, type of cement, length of adequate moisture curing periods, degree of consolidation, and the relative proportion of paste to aggregate (*24*). Data reveal that type I cement (low C_3A), quartz fine and coarse aggregates and silica fume show the excellent ability to resist concrete deterioration (*25*). The low permeability of concrete is attainable if proper care is practiced (e.g., low water-cement ratio, adequate moisture curing, and good quality of consolidation). Studies have shown the correlation of water-cement ratio and degree of consolidation on the rate of transport of chloride ions through concrete (*26*). Concrete with a water-cement ratio of 0.4 had significantly lower permeability than that of a water-cement ratio of 0.6 and 0.7 (*24*). Seven days of moist curing can also reduce concrete permeability compared to one day of moist curing. Appropriate consolidation is equally important to produce good quality concrete that resists the penetration of chloride ions since proper consolidation practices can reduce the amount of pores and segregation.

Moreover, as a rule of thumb, a low water-cement ratio mix design leads to higher compressive strength concrete and could provide better resistance to cracking resulting from the distress by steel corrosion and could extend the life of the structure.

2.8.2.2 Environmental Factors The published literature recognizes those corrosive environmental factors such as temperature, humidity, and applications of salt that have significant impacts on deterioration of concrete bridge decks. However, the interaction of these three variables is too complex to exclusively incorporate them into the deterioration model (*27*). Nevertheless, Thompson et al. (*27*) document that the presence of any chloride concentration, temperature, and humidity can induce noticeable impacts on corrosion of steel in concrete. This fact serves to explain why corrosion of steel in Florida, a humid and marine climate, is considerably more severe than in some other states.

2.8.2.3 Cracking on Bridge Decks Concrete cracks have many causes and have been studied to a large extent (*28, 29*). Several investigators [*30, 31, 32*] have pointed out that that a few bridge decks with epoxy-coated reinforcing bars have developed an excessive amount of deep cracks during the early stages of curing. This is attributed to the higher volume of cement contents and the lower water-cement ratio of the concrete.

Cracking can adversely affect structure durability and hence shorten its service life since it could facilitate a direct path for corrosive chemicals to attack the steel reinforcement embedded in concrete. In some cases, deck cracking appears along the top reinforcing steel because of the inadequate cover depth or the steel bar depicts a weakened plane. This phenomenon increases the potential for corrosion of reinforcement and hence reduces the durability of the structure.

Correlation between crack width and concrete deterioration was documented by Krauss and Ernest (*33*). Concrete with cracks, particularly when the crack is wide and extended to the depth of unprotected steel bars, shows a rapid rate of deterioration of steel. Many factors can contribute to the width of the crack: the origin of the crack, amount of cover depth, stress in the steel, concrete creep, reinforcement ratio, arrangement of reinforcement, bar diameter, and stress profile in the deck (*33*).

2.9 Surface Chloride, C_o, and Diffusion Constant, D_c, for Some States

Weyers et al. (*34*) conducted an analysis of the diffusion constant and the surface chloride constant in several states. This database consists of over 2,700 powdered samples from 321 bridges in 16 states. Table 2.2 presents ranges for C_o based on the severity of climatic exposure conditions. Table 2.3 shows the calculated mean values of the diffusion constants, D_c, for bridges in several states (*34*). Weyers et al. (*34*) also reported that bridge decks in the state of Iowa have a diffusion constant D_c = 0.05 in^2/yr and a mean surface chloride content C_o = 9.0 lb/yd^3.

TABLE 2.2 Corrosion Environment: Chloride Content Categories, C_o

	Low (lb/yd^3)	**Moderate (lb/yd^3)**	**High (lb/yd^3)**	**Severe (lb/yd^3)**
C_o	$0 < C_o < 4$	$4 \le C_o < 8$	$8 \le C_o < 10$	$10 \le C_o < 15$
Mean	3.0	6	9.0	12.4
States	Kansas, California	Minnesota, Florida	Delaware, Iowa, West Virginia, Indiana	Wisconsin, New York

TABLE 2.3 Mean Diffusion Constants, D_c

	Mean
California	0.25
Delaware	0.05
Florida	0.33
Indiana	0.09
Iowa	**0.05**
Kansas	0.12
Minnesota	0.05
New York	0.13
West Virginia	0.07
Wisconsin	0.11

2.10 Epoxy-coated Rebar Condition Rating

The surface condition of ECR extracted from the bridge decks reflects directly on ECR effectiveness. Thus, visual inspection of the ECR surface provides the assessment to evaluate ECR performance. The rating scale shown in Table 2.4 is adopted from a Pennsylvania Department of Transportation study (*35*). One can use these rating scales to develop relationships between ECR rating and age. This process was adopted herein to predict the performance of ECR in the state of Iowa bridge decks.

TABLE 2.4 Rebar Rating Description

Rating	Description
5	No evidence of corrosion
4	A number of small, countable corrosion
3	Corrosion area less than 20 percent of total ECR surface area
2	Corrosion area between 20 to 60 percent of total ECR surface area
1	Corrosion area greater than 60 percent of total ECR surface area

2.11 Rebar Cover Depth

To utilize Fick's law for determining the length of the initiation stage, one needs to calculate the time required for the chloride ions to reach the rebar level. A sufficient cover depth can effectively provide corrosion protection for the reinforcement. As reinforcing steel cover depth increases, the corrosion protection increases and hence the initiating time, T_1 (see Figure 2.1), increases. Studies have shown that the chloride concentration decreases significantly along with increasing depth from the deck surface (*36*).

A cover depth is defined as the clear distance from the surface of deck to the top of first layer of steel bars. However, to calculate a realistic time T_1 for chloride ion to reach the rebar level, one must make full use of the end of functional service life through the realization the rehabilitation will take place only after spalling or deterioration has occurred. Weyers (*11*) recommended use the average of 9 to 14 percent, i.e., 11.5 percent, damage in the worst traffic lane as an indication of the end of a bridge deck functional service life. In this case, Weyers *(11)* recommends not to use the mean value of the cover depth in equation 2.2 to determine the time T_1. Rather a more realistic value for the cover depth that accounts for the possibility that some bars could be located at a depth less than the mean value. This can be calculated as

$$x = \bar{x} + \alpha\sigma \quad , \tag{2.4}$$

where $\bar{x}$ = mean reinforcing steel cover depth (in inches) and α = values corresponding to a given cumulative percentage. This can be selected as the percent damage of the worst traffic lane as suggested by Weyers (*11*), and σ = standard deviation of the cover depth.

Statistical analysis of the measured reinforcing cover depth taken from several bridge decks illustrates a normal distribution (this is verified later herein, as summaries in section 6.3). Therefore, one can use a standard normal cumulative probability table to establish α. Table 2.5 lists the α values associated with the cumulative percentage for a concrete cover depth that is less than the calculated mean concrete cover depth.

TABLE 2.5 Standard Normal Cumulative Probabilities (*38*)

Cumulative Percentage	α	Cumulative Percentage	α	Cumulative Percentage	α	Cumulative Percentage	α
0.5	-2.576	13.0	-1.126	25.5	-0.659	38.0	-0.305
1.0	-2.326	13.5	-1.103	26.0	-0.643	38.5	-0.292
1.5	-2.170	14.0	-1.080	26.5	-0.628	39.0	-0.279
2.0	-2.054	14.5	-1.058	27.0	-0.613	39.5	-0.266
2.5	-1.960	15.0	-1.036	27.5	-0.598	40.0	-0.253
3.0	-1.881	15.5	-1.015	28.0	-0.583	40.5	-0.240
3.5	-1.812	16.0	-0.994	28.5	-0.568	41.0	-0.228
4.0	-1.751	16.5	-0.974	29.0	-0.553	41.5	-0.215
4.5	-1.695	17.0	-0.954	29.5	-0.539	42.0	-0.202
5.0	-1.645	17.5	-0.935	30.0	-0.524	42.5	-0.189
5.5	-1.598	18.0	-0.915	30.5	-0.510	43.0	-0.176
6.0	-1.555	18.5	-0.896	31.0	-0.496	43.5	-0.164
6.5	-1.514	19.0	-0.878	31.5	-0.482	44.0	-0.151
7.0	-1.476	19.5	-0.860	32.0	-0.468	44.5	-0.138
7.5	-1.44	20.0	-0.842	32.5	-0.454	45.0	-0.126
8.0	-1.405	20.5	-0.824	33.0	-0.440	45.5	-0.113
8.5	-1.372	21.0	-0.806	33.5	-0.426	46.0	-0.100
9.0	-1.341	21.5	-0.789	34.0	-0.412	46.5	-0.088
9.5	-1.311	22.0	-0.772	34.5	-0.399	47.0	-0.075
10.0	-1.282	22.5	-0.755	35.0	-0.385	47.5	-0.063
10.5	-1.254	23.0	-0.739	35.5	-0.372	48.0	-0.005
11.0	-1.227	23.5	-0.722	36.0	-0.358	48.5	-0.038
11.5	-1.200	24.0	-0.706	36.5	-0.345	49.0	-0.025
12.0	-1.175	24.5	-0.690	37.0	-0.332	49.5	-0.013
12.5	-1.15	25.0	-0.674	37.5	-0.319	50.0	-0.000

3 BRIDGE SELECTION

Review of the Iowa DOT's bridge records indicated that there were 711 bridge decks in Iowa that were constructed with epoxy-coated rebars in either the top mat or both the top and bottom mats. These bridges were built between 1978 and 1995 and are located all across the state of Iowa. Inspection records for these bridges were obtained from the Bridge Maintenance office and were utilized in selecting the bridges to be included in this work.

In deciding which bridges to select for evaluation in this work, the following data were obtained from the Iowa DOT for each of the 711 bridges. The effects of many characteristics, such as bridge span, average daily traffic (ADT), bridge type, and geographic location, on the deck condition rating of each bridge were analyzed. Although the deck condition ratings given by Iowa DOT inspectors were rated according to surface characteristics of the decks, they were the best sources of information available describing deck conditions.

In phase I of this project, the bridges were categorized into various groups to examine the impact of certain characteristics on deck condition ratings. The mean deck condition ratings were then calculated for the groups, and hypothesis tests (*t*-tests) were used to determine whether differences in deck condition ratings were impacted by certain bridge characteristics.

Many factors that could have an impact on the rating of bridge decks were examined. The analyses showed that the age, the geographic location, the type of structure (concrete or steel), and the average daily traffic volume have the most significant impact on the deck condition rating (see Table 3.1 and Figure 3.1). For this reason, the selection of bridges was grouped in two-year intervals starting from 1987 and taking into account the geographic location (northern or southern Iowa) and the type of structure (concrete or steel), as shown in Figure 3.2. The average daily traffic was not included in the grouping process because this would have restricted the sample size of each group so much that many of the groups would be too small to be represented in the sampling process.

Because the long-term durability of bridge decks with epoxy-coated rebars was the most important part of this project, more percentage of older bridges were selected over newer bridges. About 50 percent of the bridges sampled were built from 1978 to 1983, about 30 percent were built from 1984 to 1989, and about 20 percent were built from 1990 to 1995. Within each period, bridges were selected from their respective group randomly. The number of bridges selected from each group depended on the total number of bridges within the group. Thus, large groups had a proportionately larger amount of bridges selected than small groups in the same time period.

As an initial rough estimate, it was assumed that 80 bridges could be sampled throughout the course of the project. However, as the testing procedures and evaluation processes became more apparent, the target number of bridges that would be evaluated for the entire duration of the project was changed to 40. In selecting 40 of the 80 bridges previously chosen, the inspection records of bridges selected in each group were further

TABLE 3.1 Summary of Main Deck Condition Rating Factors

	Mean Deck Condition Rating
Traffic Volume	
ADT < 5750	7.73
ADT > 5750	7.49
Structure Type	
Concrete	7.69
Steel	7.52
Geographic Location	
Southern Iowa	7.71
Northern Iowa	7.64

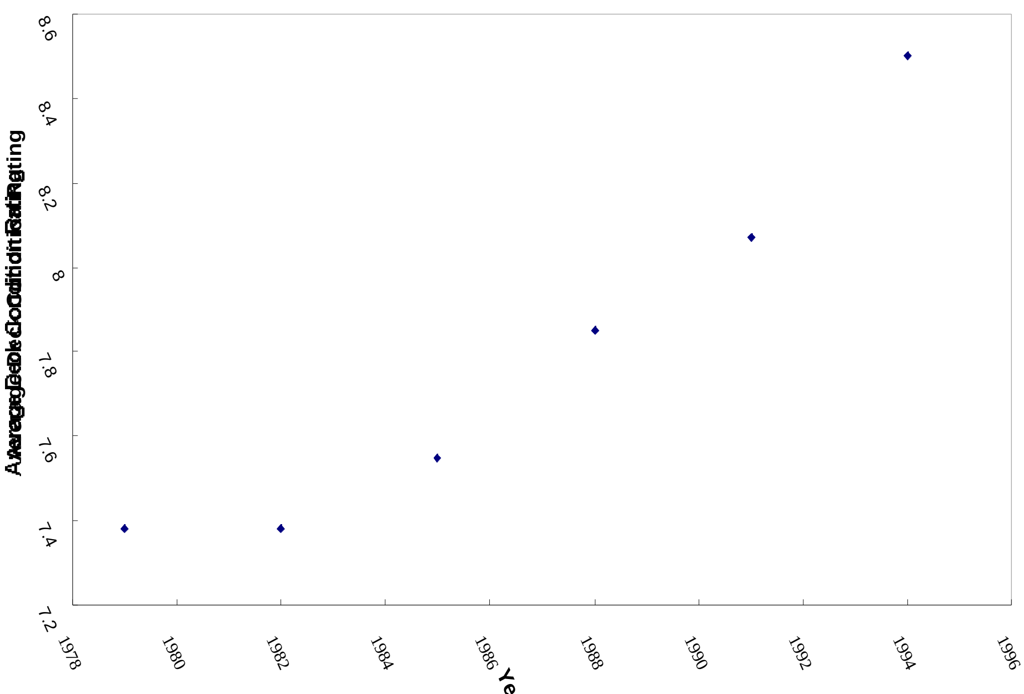

FIGURE 3.1 Average Deck Condition Rating versus Year Built

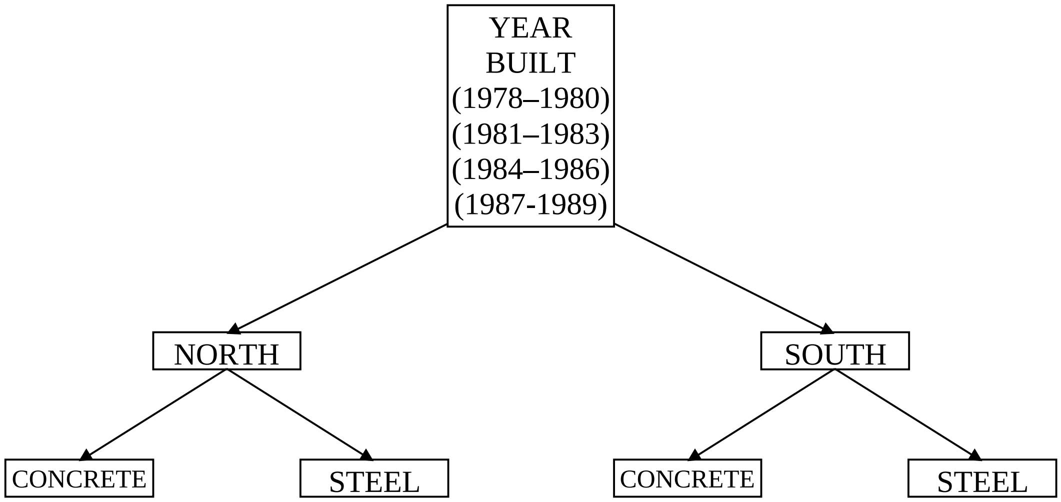

FIGURE 3.2 Bridge Grouping Used in Phase I

examined. The bridges with the most deck cracking were chosen from each group. This allowed the selection of bridges based on cracking severity without having to compare inspection records for all 711 bridges.

After the results of phase I were examined and presented to the Project Advisory Committee, it was concluded that it is necessary to build a broader database regarding the condition of ECR. This would allow one to develop a more reliable relationship that can be used to interpret the condition of ECR and its age.

For this purpose, the selection of bridges utilized in phase I was not followed. Rather, additional bridges were selected so that the number of bridges with common age would be at least five per each one-year interval. This grouping resulted in 37 and 43 bridges that were selected in phase I and phase II, respectively. Table 3.2 summarizes the number of bridge selected according to their geographical locations. More detailed information regarding the selected of bridges is summarized in appendix B.

TABLE 3.2 Summary of Bridge Selection

	North	**South**	**Subtotal**
Phase I	23	14	37
Phase II	24	19	43
Total			80

One can notice that the selection contained more bridges located in northern Iowa than in southern Iowa. This is because there were more bridges constructed with ECR in northern Iowa than southern Iowa. The locations of bridges being evaluated and the divided line for north and south are shown in Figure 3.3. As can be seen in the figure, bridges from all across Iowa were selected. The figure illustrates that a larger proportion of bridges was selected from eastern Iowa. This is due to the fact that a significantly larger number of bridges were built between 1978 to 1993 in that part of state. This can be attributed to the construction of Interstate Highway 380 during this time period.

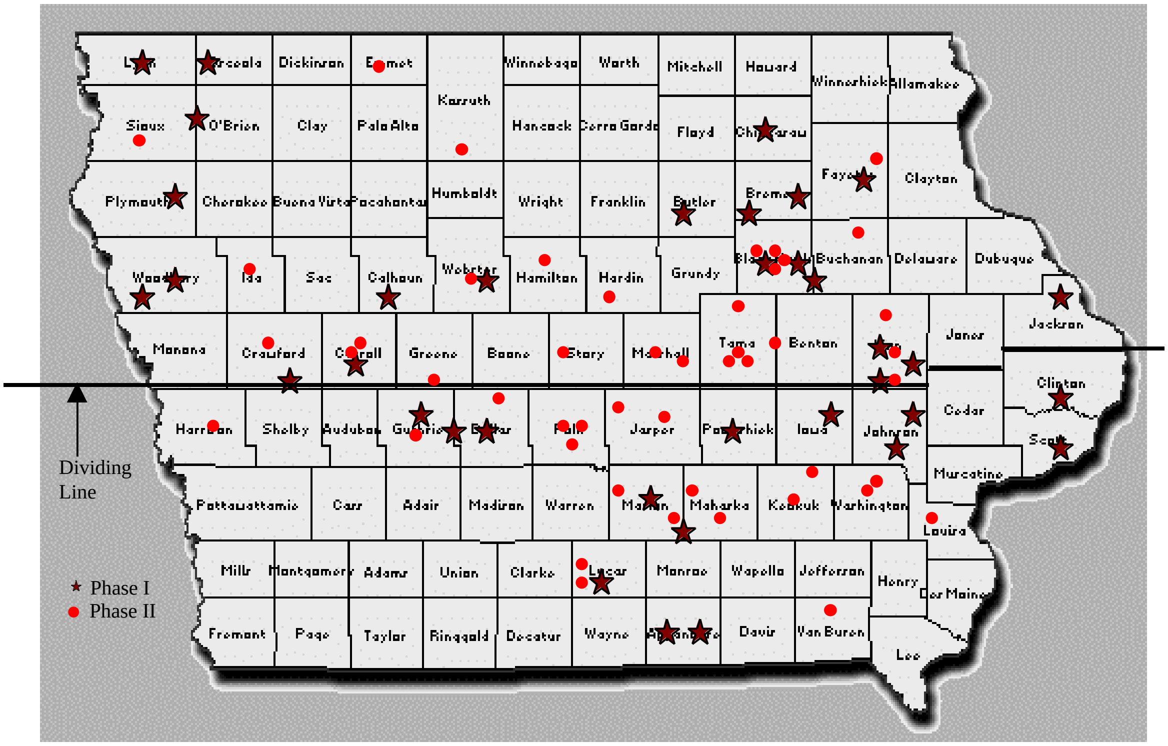

FIGURE 3.3 Locations of the Selected Bridges

3.1 Additional Bridges

The Project Advisory Committee suggested to investigate a bridge located in Lyon County during the study of phase I. This bridge was built in 1976, and it was one of the first bridges in Iowa built with ECR in the deck. Furthermore, three Tama County bridges that were built in 1968 with black reinforcing steel were investigated during the study of phase II. Sealer was first applied to the deck at one of these bridges in 1984 and thereafter at each five-year intervals, i.e., 1989, 1994, and 1999 (*39*). This bridge was designated as Tama 1. The sealer was not applied to other two bridges. Hence, it is of special interest to know the effectiveness of sealer resisting the diffusion of chloride ions and the condition of rebars in these bridge decks. Furthermore, the Project Advisory Committee recommended to include an evaluation of three bridge decks that were constructed using black rebars in two-course placements. In this method, approximately three-fourths of deck thickness was cast and was allowed to cure and deflect (*40*). The remaining concrete of the deck slab thickness was added using Iowa low slump overlay mix design concrete. The effect of this construction method on the permeability of the chloride ion through the bridge decks was investigated. For detail information of these additional bridges, the reader is referred to appendix B.

3.2 Bridge ID Designation

The identification of Iowa bridges consists of the combination of numbers and letters. Each portion of the identification number has a unique representation (*41*). For instance, a bridge ID designated as 0475.4S002 can be explained as follows: The first two numbers, 04, represent the county ID number. The three digits following the county number, 75.4, represent the milepost at which the bridge is located. The single letter indicates the type of bridge, in this case, S, a single two-lane bridge. The last three numbers, 002, represent the highway where the bridge is sited. Table 3.3 summarizes the characteristics represented by the letters. Table 3.4 lists counties with their designated numbers (*41*).

TABLE 3.3 Characteristic of Bridge Designation (*41*)

	Characteristic
A	Bridges located in a highway ramp
L	Bridges located in a four-lane or wider divided highway at which the bridge oriented to the left side of highway when one faces the increasing miles
O	Bridges overhead a highway
R	Bridges located in a four-lane or wider divided highway at which the bridge oriented to the left side of highway when one faces the decreasing miles
S	Bridges located in a two-lane undivided highway

TABLE 3.4 County Identity (*41*)

County	ID	County	ID	County	ID	County	ID
Adair	01	Davis	26	Jefferson	51	Pacahontas	76
Adams	02	Decatur	27	Johnson	52	Polk	77
Allamakee	03	Delware	28	Jones	53	Pottawatt	78
Appanoose	04	Des Moines	29	Keokuk	54	Poweshiek	79
Audubon	05	Dickinson	30	Kossuth	55	Ringgold	80
Benton	06	Dubuque	31	Lee	56	Sac	81
Black Hawk	07	Emmet	32	Linn	57	Scott	82
Boone	08	Fayette	33	Louisa	58	Shelby	83
Bremer	09	Floyd	34	Lucas	59	Sioux	84
Buchanan	10	Franklin	35	Lyon	60	Story	85
Buena Vista	11	Fremont	36	Madison	61	Tama	86
Bulter	12	Greene	37	Mahaska	62	Taylor	87
Calboun	13	Grundy	38	Marion	63	Union	88
Carroll	14	Guthrie	39	Marshall	64	Van Buren	89
Cass	15	Hamilton	40	Mills	65	Wapello	90
Cedar	16	Hancock	41	Mitchell	66	Warren	91
Cerro Gordo	17	Hardin	42	Monona	67	Washington	92
Cherokee	18	Harrison	43	Monroe	68	Wayne	93
Chickasaw	19	Henry	44	Montgomery	69	Webster	94
Clarke	20	Howard	45	Muscatine	70	Winnebago	95
Clarke	21	Humboldt	46	O’Brien	71	Winneshiek	96
Clay	22	Ida	47	Osceola	72	Woodbury	97
Clayton	23	Iowa	48	Page	73	Worth	98
Clinton	24	Jackson	49	Palo Alto	74	Wright	99
Crawford	25	Jasper	50	Plymouth	75		

4 FIELD AND LABORATORY EVALUATIONS

4.1 Field Evaluations

The field evaluation for the selected bridges involved conducting the visual inspection of bridge decks for spallings and delaminations as well as collecting four cores from each bridge deck

4.1.1 Coring Location

Four cores were taken from each bridge deck. Two cores were taken directly at crack locations, while the other two cores were taken from locations of the deck that showed no signs of cracking. One of the cores taken from the "cracked" and one from the "uncracked" locations were taken near the gutter line, while the other two were taken near the center line of the deck. To simplify traffic control and to allow traffic to flow smoothly over the bridge during the coring process, all cores on each deck were taken from only one side of the bridge. The side of the bridge selected for coring was arbitrarily chosen.

4.1.2 Coring Collection

Reinforcing bars in each bridge deck were located using a pachometer. As often as possible, cores were taken at locations where longitudinal and transverse top mat rebars intersected. Photographs of the core locations were taken and sketches of the general cracking pattern were made prior to drilling the cores from each bridge deck. The bridge identification number and a core letter were recorded on each core. Also the orientation of each core within the deck was recorded. Photos of the cores were then taken, and any visible spots of rust or other types of deterioration were documented. The cores were allowed to air dry after coring and were not stored in sealed containers prior to examining them in the lab.

Prior to coring, a pachometer, as shown in Figure 4.1, was used first to locate reinforcing bars in the concrete. The coring drill bit was then centered at the intersection of transverse and longitudinal reinforcing bars (see Figure 4.2). The diameter of the extracted core was four inches and the length varied depending on the breaking depth of a core. The height of a core ranged approximately from 3 to 6 inches. Figures 4.3, 4.4, and 4.6 illustrate the process used in collecting the cores and patching the holes in a cored bridge deck.

4.1.3 Powder Sample Collection in the Field

In phase I, while the cores were being drilled, concrete powder samples at five locations across each bridge deck were collected. Two samples were drilled with a three-eighths–inch drill bit at each location. One sample at each location contained concrete powder drilled from a depth of 0.5 to 1.5 inches. The other sample contained concrete powder drilled from a depth of 2.5 to 3.5 inches. The five locations where powder samples were drilled are shown in Figure 4.7.

FIGURE 4.1 Pachometer Used to Locate Reinforcing Steel in a Bridge Deck

FIGURE 4.2 The Setup of the Coring Process

FIGURE 4.3 Breaking the Core

FIGURE 4.4 Extracting the Core

FIGURE 4.5 Extracted Cores

FIGURE 4.6 Patching the Cored Hole after a Core Being Extracted

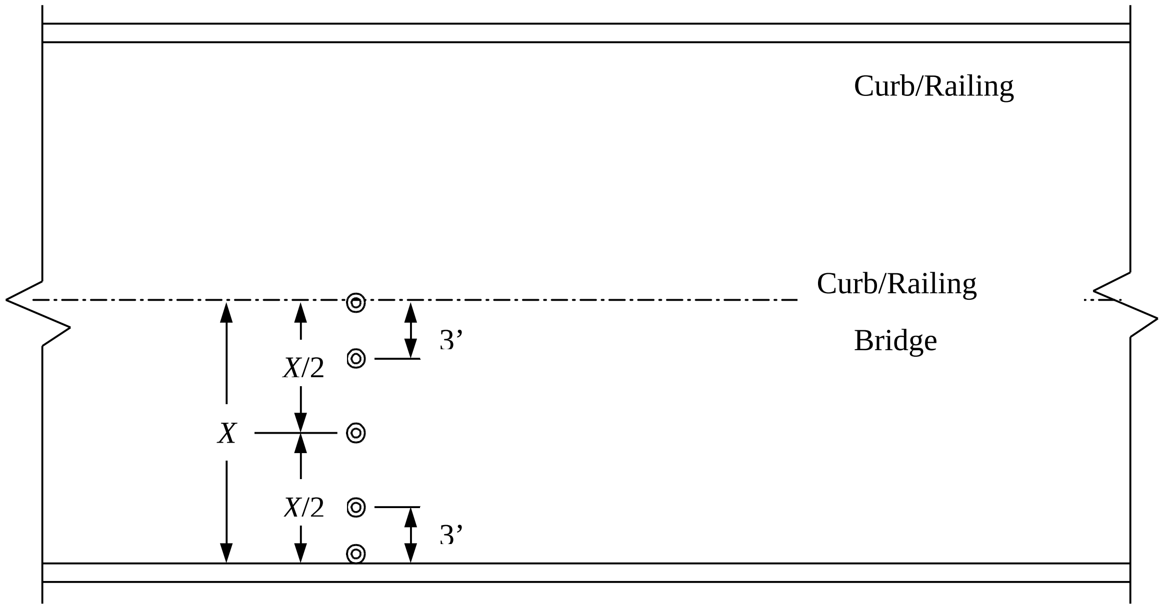

FIGURE 4.7 Field Concrete Powder Sample Locations

4.2 Laboratory Evaluations

The lab evaluation included the following: general physical properties of cores, measurement of crack depth and length, collection of powder sample, rebar rating, epoxy coating hardness, epoxy coating bond, and analysis of chloride content.

4.2.1 General Physical Properties

This task consisted of visual inspections and various measurements of cores. Measurements included the concrete cover depth over reinforcing bars, the diameter of reinforcing bars, the length of extracted cores, the orientation of rebars embedded in a core, and the orientation of cracks. The inspection of the extracted cores also included recording the number of rebars embedded in a core and the number of pieces per core if a core was broken.

4.2.2 Cracked Dimension

The width and the depth of cracks that penetrated in the cores collected from cracked locations were measured. The procedure to accomplish this is outlined as follows:

1. sketch crack orientation related to traffic direction on the attached data sheet;
2. locate on desired depth 0.5 inches below the surface;
3. use hand micrometer to measure the widths along the core at each side and document two readings;

4. average the readings to obtained the surface crack width; and
5. use ruler to measure cracked lengths along the core and record the reading.

4.2.3 Collection of Powder Samples from the Cores

Collection of powder samples is shown in Figure 4.8. At least 20 grams of powder were collected for chloride content analysis. Four powder samples were collected from each core using three-eighths–inch drill bit. The location of these samples were at 0.5 inches below the surface, midway between the first sample and rebar level, rebar level, and one inch below the rebar level.

Figure 4.8 Collection of Powder Samples

The procedure used in collecting the sample was as follows:

1. mark down the location at the desired depth as described above;
2. drill and collect powder from the marked locations in a pan;
3. place the drilled powder in the zip-lock plastic bag;
4. record the bridge ID, core letter, and the exact depth at the bag;
5. clean the pan and the bit thoroughly with a brush to avoid contamination between powder samples; and
6. repeat the same procedure for each location.

Powder samples from cracked cores were drilled from the uncracked quadrant to avoid splitting the cores into half. Drilling penetrated through the crack so that the sample contained powders collected from the cracked surface. However, the percentage of the powder sample collected from near the face of a crack was not determined.

4.2.4 Rebar Condition

After the powder samples were collected from the cores, the cores were broken to extract rebars for future investigation. A hammer was used to break out the core. This was done in a deliberate manner to avoid damaging the epoxy coating film on the rebars. The evaluation of rebars condition involved describing and classifying the condition of rebars in a core. A rebar was rated on the scale from one to five as described in section 2.10.

4.2.5 Epoxy Coating Hardness

The epoxy-coating hardness was conducted to determine the correlation between the epoxy-coating hardness and other characteristics, such as chloride content, bridge age, and corrosion. The coating hardness of each rebar was tested using the pencil hardness test, as shown in Figure 4.9, described in NACE TM-0174, section 6.1.5. The procedure is outlined as follows:

1. Strip the wood from the lead of each test pencil for about 0.25 inches (6.35 millimeters), using care to prevent nicking of the lead.
2. Flatten the tip of the exposed lead by pressing against number 400 carbide abrasive paper and rotating with a gentle motion.
3. With the pencil held in the writing position or at an approximate 45 degree angle, push the lead forward against the coating.
4. Remove the lead marks with soap and water or an art gum eraser. Any marring of the coating surface when viewed at an oblique angle in strong light indicates that the pencil lead is harder than the film.
5. Express the hardness of the coating as the next softer grade of pencil to the pencil grade used in the test. Grades of pencil hardness from soft to hard are 6B, 5B, 4B, 3B, 2B, B, HB, F, H, 2H, 3H, 4H, 5H, 6H, 7H, 8H, and 9H.
6. After each pencil hardness test, the pencil should be turned to produce a new edge. Three or four tests can be made without redressing the lead.

4.2.6 Epoxy Coating Bond

To determine the coating bond in between steel and coating film, the dry knife adhesion test was performed for each rebar as shown in Figure 4.10. The recommended standard procedure is described in NACE TM-0185, section 5.3.2.1, as follows:

> The recommended method for determining adhesion is to cut the coating to the base metal using a Number 22 hobby knife blade. The point of the blade shall be drawn across the film (using multiple cuts if necessary to cut a single V-shaped groove. Using the sharp side of the blade as a wedge, the coating film should be pried up within the groove. The exposed base should be observed under a 10 to 15X microscope to determine adhesion performance. An average of three attempts shall be used to rate the sample.

FIGURE 4.9 Coating Hardness Test

FIGURE 4.10 Epoxy Coating Bond Test

The epoxy coating was rated following the recommendation given in the Pennsylvania Department of Transportation study (*35*). Table 4.1 summarizes the description of each bond rating value recommended by Sohanghpurwala et al. (*35*).

TABLE 4.1 Coating Bond Rating Description

Rating	Description
3	Well adhered coating that cannot be peeled or lifted from the substrate steel
2	Coating that can be pried from the substrate steel in small pieces but cannot be peeled off easily
1	Coating that can be peeled from the substrate steel easily, without residue

4.2.7 Chloride Content Analysis

Powder samples collected from cores were then sent to the Material Analysis and Research Laboratory for analyzing chloride concentration. The chloride concentration was tested by using the Phillips PW 2404 X-ray fluorescence (XRF) spectrometer which is a nondestructive analytical device used to determine and identify the concentration of elements contained in a solid, powdered, and liquid sample (*42*).

5 CHLORIDE CONTENT AND DIFFUSION CONSTANT IN IOWA BRIDGES

The chloride content data collected from the cores extracted from uncracked locations were used to determine the surface chloride content and diffusion constant in Iowa bridges. As previously mentioned, these parameters are required to use Fick's Second Law to assess the diffusion process of chloride ions through uncracked bridge decks. This section summarizes the procedure used to estimate these two constants. For this purpose, the chloride content data in all samples collected from cores taken at uncracked locations were used. Appendix C summarizes the results of chloride concentration for cracked cores.

5.1 Chloride Distribution along Bridge Decks

Prior to examining the chloride content in Iowa bridge decks, it was expected that the chloride content would be higher in the samples taken near the bridge gutters than those collected from other areas of the decks. However, the results showed the exact opposite. Figures 5.1 and 5.2 show the chloride contents of powder samples analyzed from the first 20 bridges in this study (referred to here after as the *P*-powder samples). Figure 5.1 presents the powder samples taken from 0.5 to 1.5 inch depths, while Figure 5.2 presents those taken from 2.5 to 3.5 inch depths.

As shown on these graphs, the concrete powder samples taken adjacent to the gutters from a depth of 0.5 to 1.5 inches contained lower chloride content than those taken from the other locations at the same depth. At locations 2 through 5, there was little difference in chloride content from a depth of 0.5 to 1.5 inches. The concrete powder samples taken from a depth of 2.5 to 3.5 inches showed less variation across the bridge decks, although the samples taken adjacent to the gutters still contained significantly less chloride than those taken from other locations.

In order to verify the above results, a different sampling scheme was used in collecting samples from the bridges that remained to be sampled in phase I. For these bridges, powder samples were taken from the locations shown in Figure 5.3. These are referred to as *X*-powder samples hereafter. Although the depths of the powder samples remained the same as the first set of bridges, the five locations were different. Location 1 remained adjacent to the bridge gutter. Locations 2, 3, and 4 were 1, 2, and 3 feet away from the bridge gutter, respectively, and location 5 remained at the center line of the bridge deck.

The chloride concentration results from these *X*-powder samples are displayed in Figures 5.4 and 5.5. These plots also show that the chloride content in the concrete near the gutter is less than in other areas of the deck. The samples taken at the gutter from a depth of 0.5 to 1.5 inches still lower chloride content than those taken at 3 feet from the gutter and at the center line of the bridges at the same depth. Figures 5.4 and 5.5 also show a general increase in the chloride content from the curb/railing to 3 feet from the gutter, with the largest increase between 2 and 3 feet from the gutter.

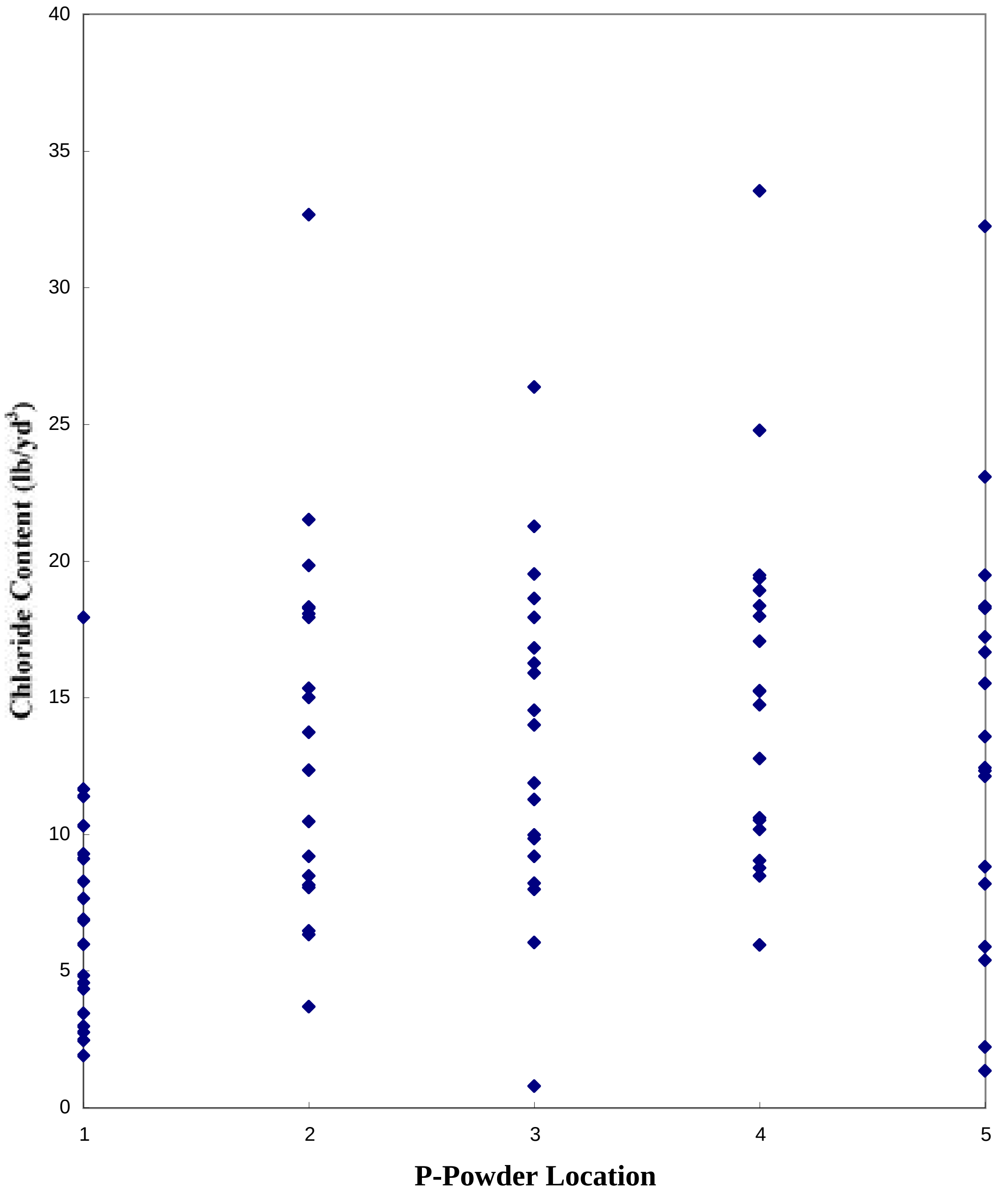

Figure 5.1 Chloride Content vs. Location (0.5 in. to 1.5 in. Depth)

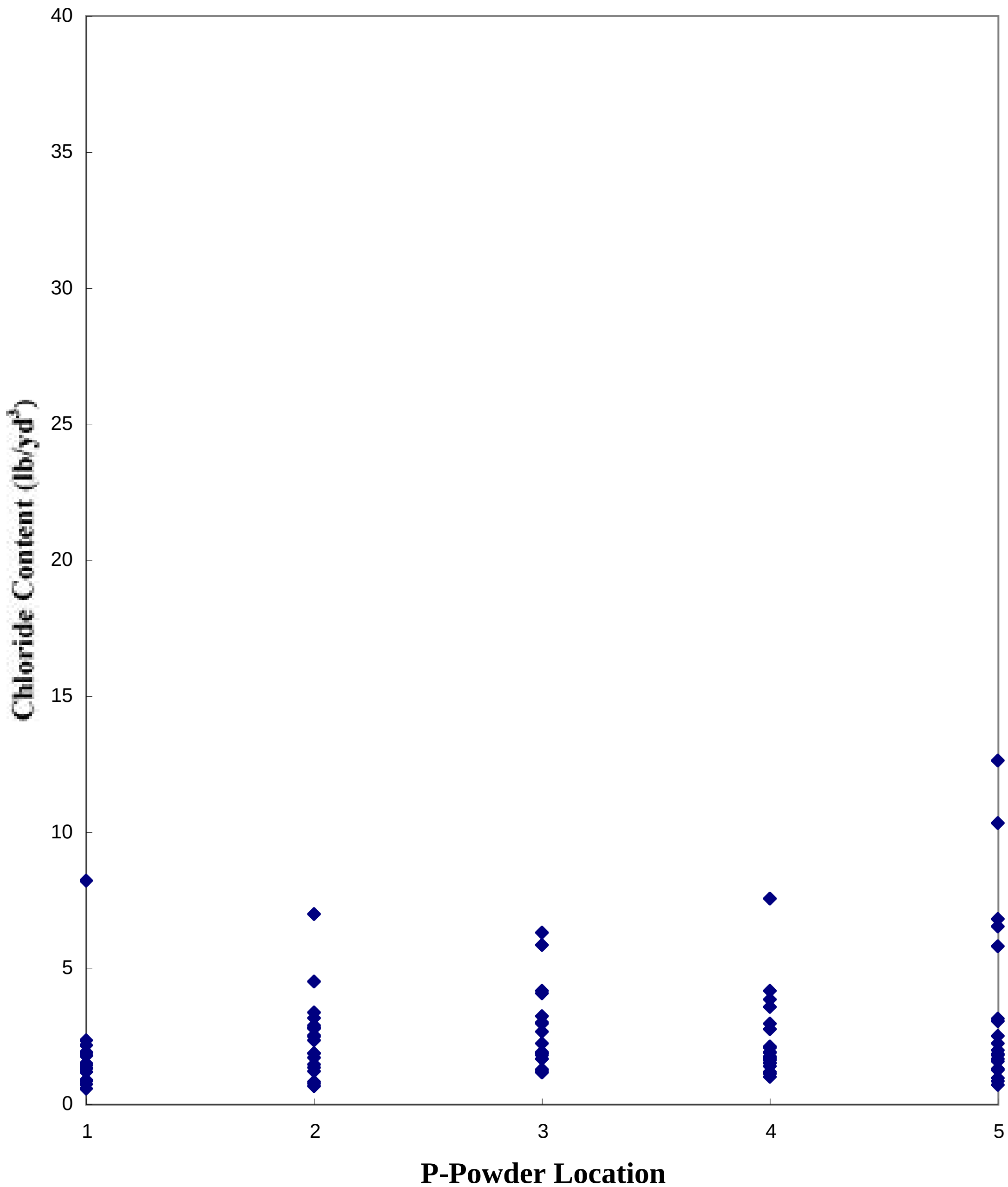

Figure 5.2 Chloride Content vs. Location
(2.5 in. to 3.5 in. Depth)

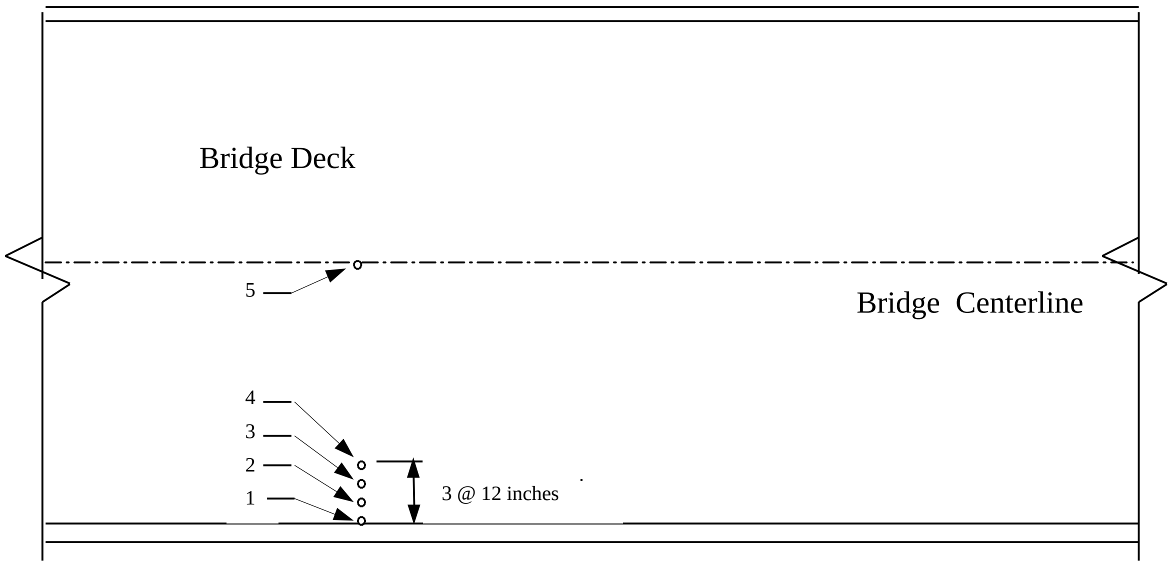

FIGURE 5.3 Field Concrete Powder Sample Locations (*X*-powders)

The reason that the chloride content in the samples taken from uncracked locations near the gutters was less than those taken from other areas could be because of dilution of the de-icing mix caused by water running toward the gutter. The areas away from the gutter are probably subjected to higher concentrations of chlorides for longer periods of time. High water flow in the gutter could flush out chlorides on the surface of the concrete and slow the chloride infiltration. One also needs to realize that the powder samples in this study represent only uncracked areas of bridge decks. The Project Advisory Committee agreed not to pursue a similar investigation in phase II since the interest was to study the performance of the ECR rather than to try to predict the overall distribution of chloride over a bridge deck.

5.2 Chloride Content

5.2.1 Chloride Content versus Age

The chloride content results from the *P*-powder and *X*-powder samples were also used to investigate the relationship between chloride content and bridge age. The graph of chloride content versus bridge age is shown in Figure 5.6 for 0.5 to 1.5 inch depths and in Figure 5.7 for 2.5 to 3.5 inch depths. Figures 5.6 shows that chloride content in most of the 0.5- to 1.5-inch–deep samples is significantly higher than the corrosion threshold for bare steel, i.e., greater than 1.2 lb/yd^3 (0.81 kg/m^3). The data recorded in Figure 5.7 illustrate that a large number of samples have a chloride concentration greater than the corrosion threshold in the samples taken from depths between 2.5 to 3.5 inches. Figures 5.6 and 5.7 also show that relatively high chloride contents were noticed in some of the

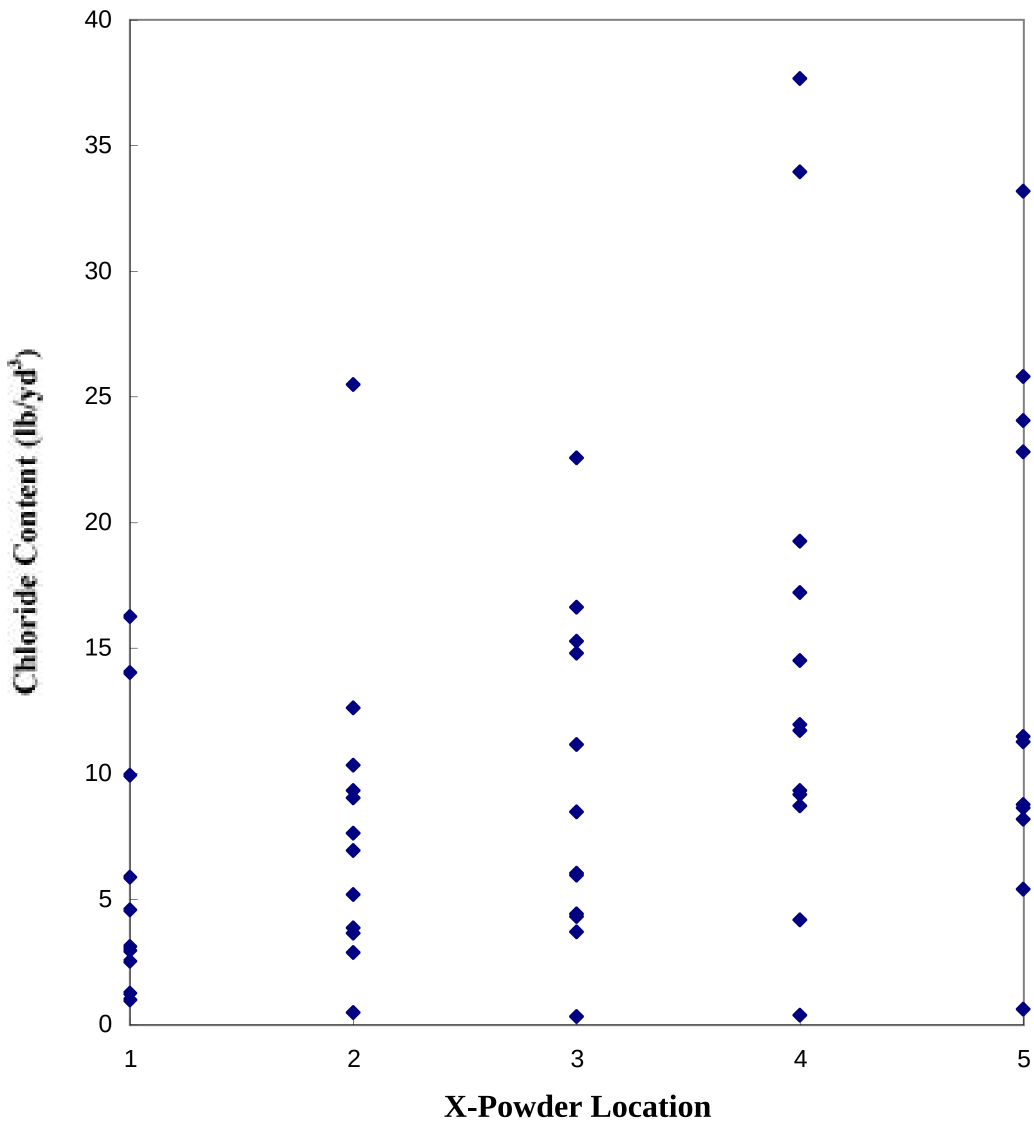

Figure 5.4 Chloride Content vs. Location (0.5 in. to 1.5 in. Depth)

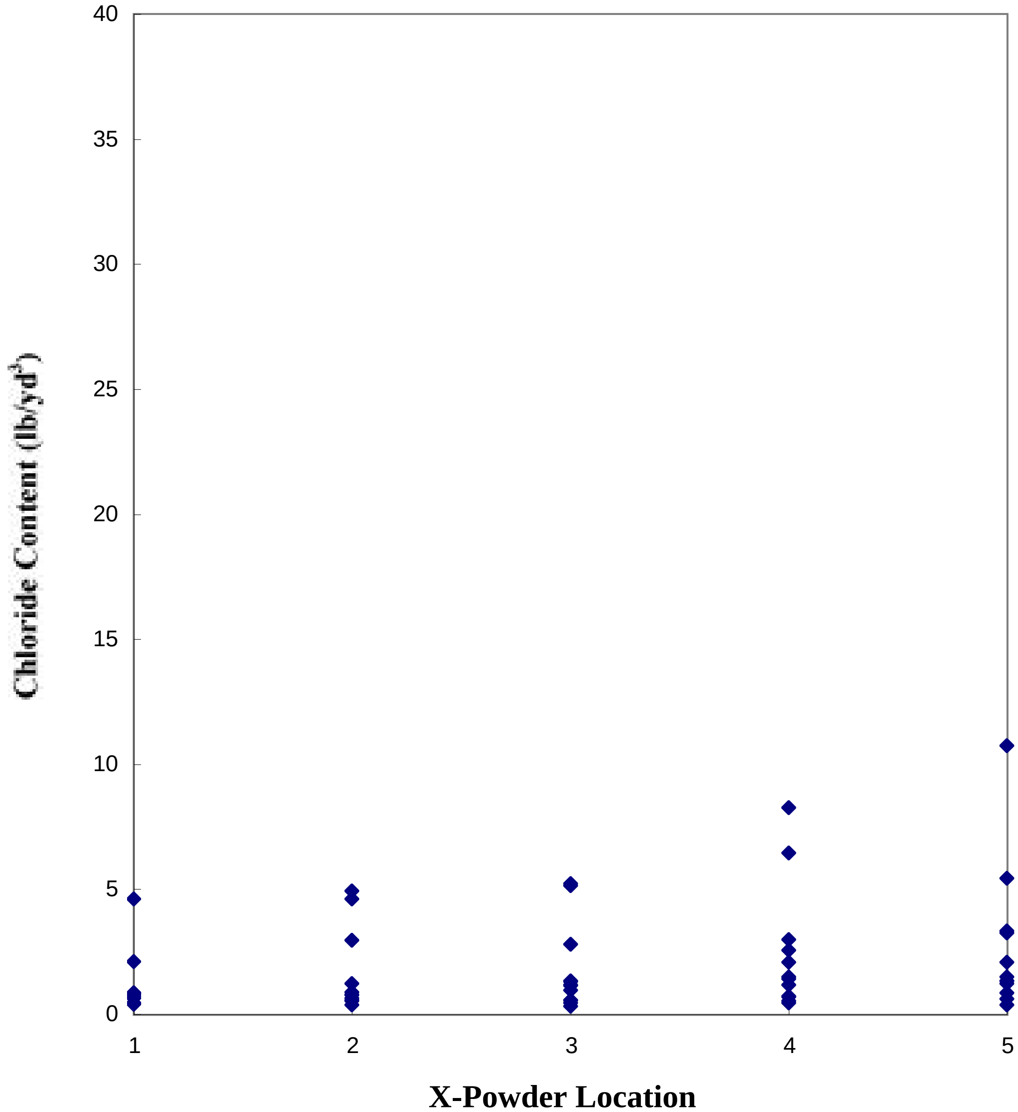

Figure 5.5 Chloride Content vs. Location
(2.5 in. to 3.5 in. Depth)

newer bridges constructed from 1990 to 1993. Although these samples contained less chloride than the older bridges in the study, the amount of chloride was still significant when compared with the threshold value. This could be a result of large numbers of de-icing applications on these bridges, or it could be due to poor permeability properties of the concrete on these bridges. However, this principal investigator did not pursue to obtain such information so that a conclusive reason for that high chloride content could be reached.

5.2.2 Determination of Surface Chloride Constant and Diffusion Constant

To determine the chloride content at the given depth, one needs to establish the surface chloride concentration, C_o, at 0.5 inches below the deck surface, and the diffusion constant, D_c. Chloride contents at three different depths along the extracted core were measured and used to calculate these two constants. Once these values are obtained, one can then use equation 2.2 to estimate the time required for chloride to reach the corrosive threshold at the rebar level.

The chloride diffusion constant D_c can be calculated directly if the chloride concentration at 0.5 inches from the surface is known. However, in situations where the constant C_o is not available, an iterative process needs to be utilized to determine the two constants C_o and D_c.

Two alternatives to determine common C_o and D_c for Iowa bridge decks were investigated. In the first alternative (approach I), the chloride data collected from all cores in phase I and II were utilized to obtained a relationship that best fits equation 2.2. This was accomplished by specifying approximate ranges of C_o and D_c for each core samples and an iterative solution was carried out using the Matlab software (*23*). The solution continued until the sum of squared errors between the predicted and measured was within a specified tolerance. The programming code used to carry out this solution is listed in appendix C. Appendix D summarizes the results of this analysis and chloride concentration at different depths for each core extracted from bridge decks included in this investigation.

However, when reviewing the chloride data, it was noticed that some data appeared to be unrealistic. For instance, the chloride analysis showed that, in some cases, higher percentage of chloride existed at deeper locations than at shallower locations. This could have resulted from some errors that could have occurred during sample collection. Therefore, it was decided to exclude such data prior to determining general values for C_o and D_c. In addition, it was decided to eliminate the chloride data that yielded an estimated C_o that is less than 8 lb/yd^3 or larger than 20 lb/yd^3 and a D_c that is less than 0.02 as calculated by approach I listed above. This approach is referred to as approach II. The results of these two approaches are listed in Table 5.1.

For illustrative purpose, based on equation 2.2, the results of chloride diffusion in three bridge decks are shown in Figures 5.8, 5.9, and 5.10 for bridges with ID numbers 0668.7S021A, 8609.2S030B, and 0781.5L218A, respectively. Shown in each figure are three relationships that were obtained using C_o and D_c from approaches I, using C_o and D_c from approach II, and using C_o and D_c for each individual bridge. Also included in these figures is the chloride concentration measured in each bridge.

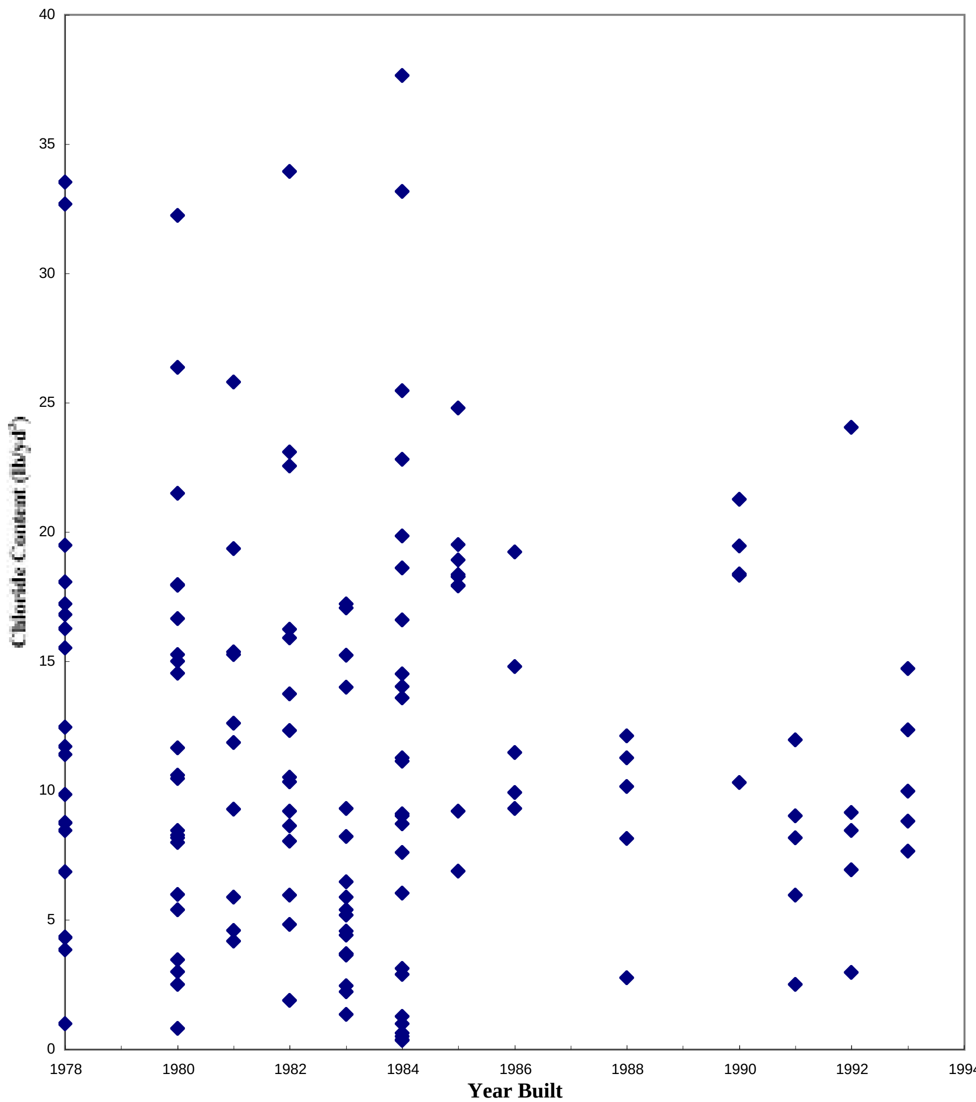

Figure 5.6 Chloride Content vs. Year Built
(0.5 in. to 1.5 in. Depth)

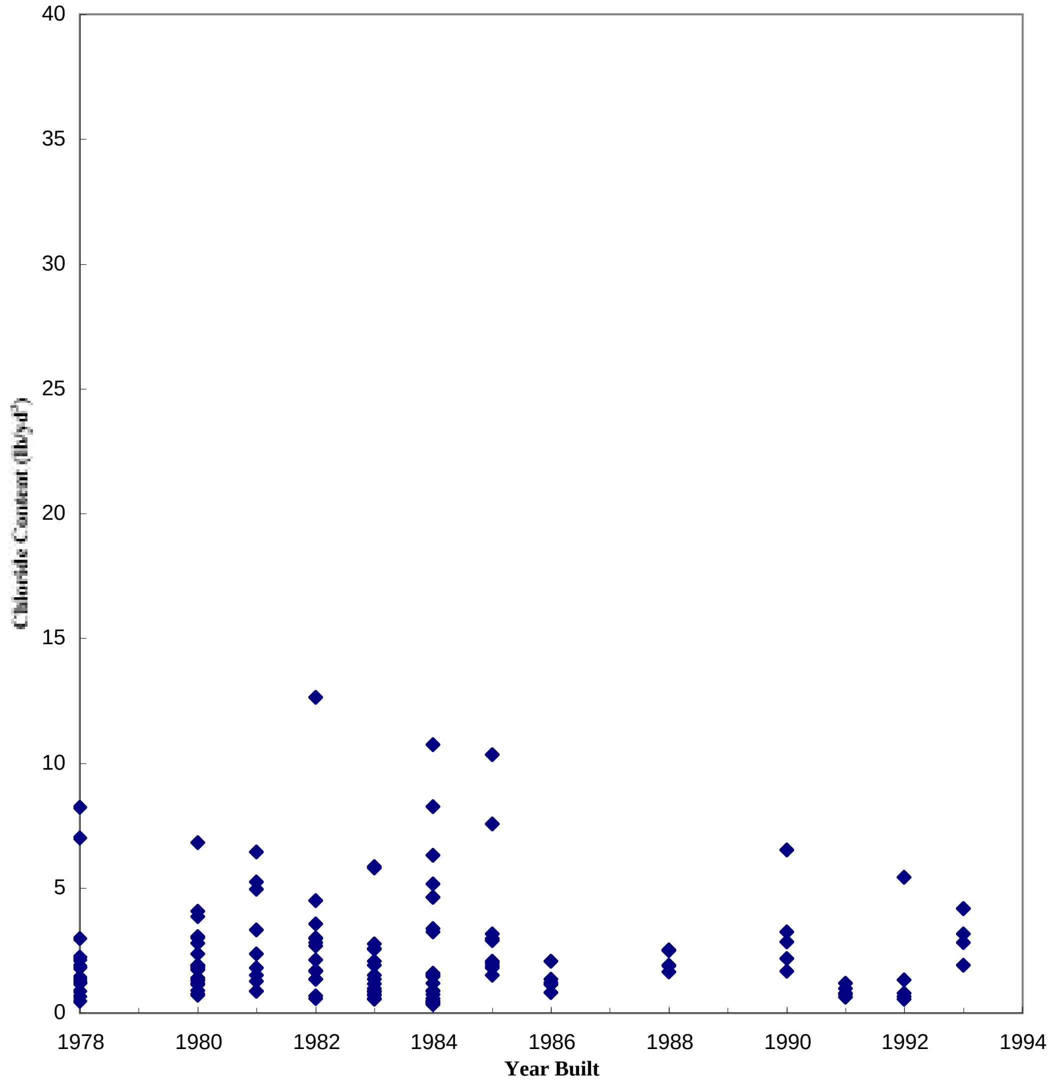

Figure 5.7 Chloride Content vs. Year Built
(2.5 in. to 3.5 in. Depth)

As can be seen, the chloride concentration decreases to almost zero sharply at about depth of 4 inches regardless of what approach is used in developing the chloride concentration and depth relationship. In addition, although the three relations showed significant difference at surface chloride content between the measured and predicted chloride contents, these differences were insignificant at the rebar level. The figures reveal that approach II yields closer results to the measured values than those of approach I.

Therefore, the results of approach II were recommended as a general surface chloride concentration and diffusion constant for the bridge decks in the state of Iowa. The surface chloride content, C_o, and the diffusion constant, D_c, associated with this general relationship are of 14 lb/yd^3 and 0.05 in^2/yr, respectively.

Table 5.1 Summary of C_o and D_c

	Mean	Standard Deviation
Nonfiltered, approach I:		
C_o (lb/yd$^{3)}$	18.0	8.920
D_c (in^2/yr)	0.061	0.054
Filtered, approach II:		
C_o (lb/yd^3)	14.0	3.62
D_c (in^2/yr)	0.050	0.038

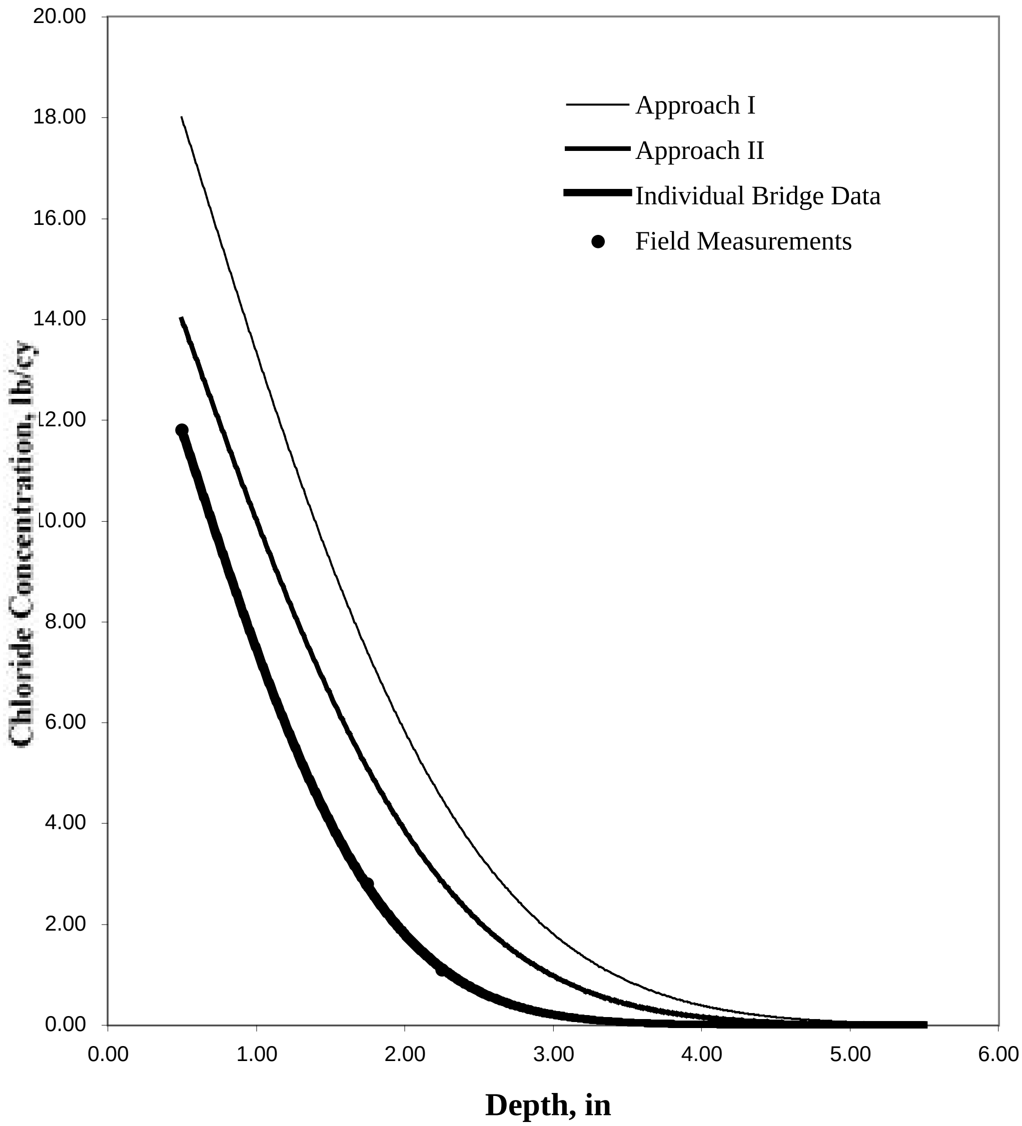

Figure 5.8 Chloride Concentration vs. Depth, 0668.7S021A

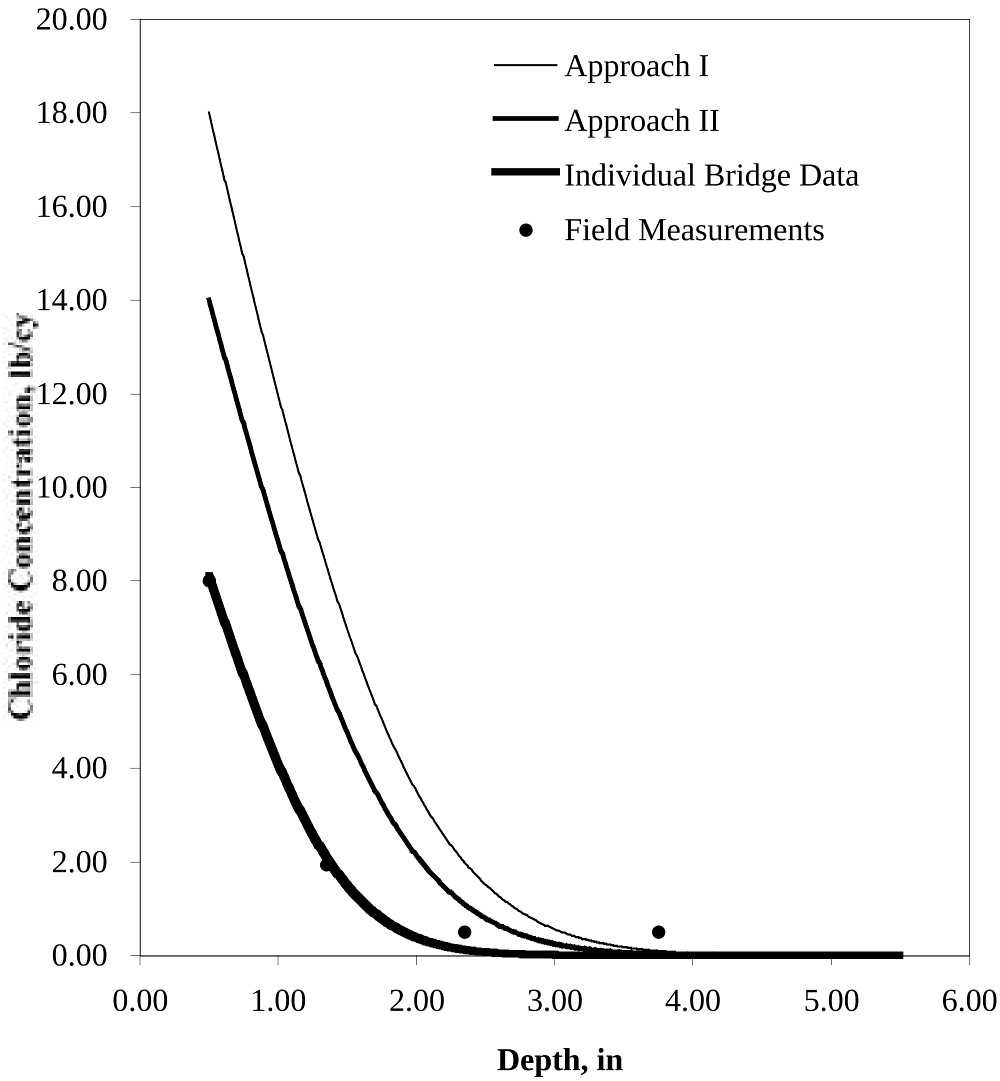

Figure 5.9 Chloride Concentration vs. Depth , 8609.2S030 B

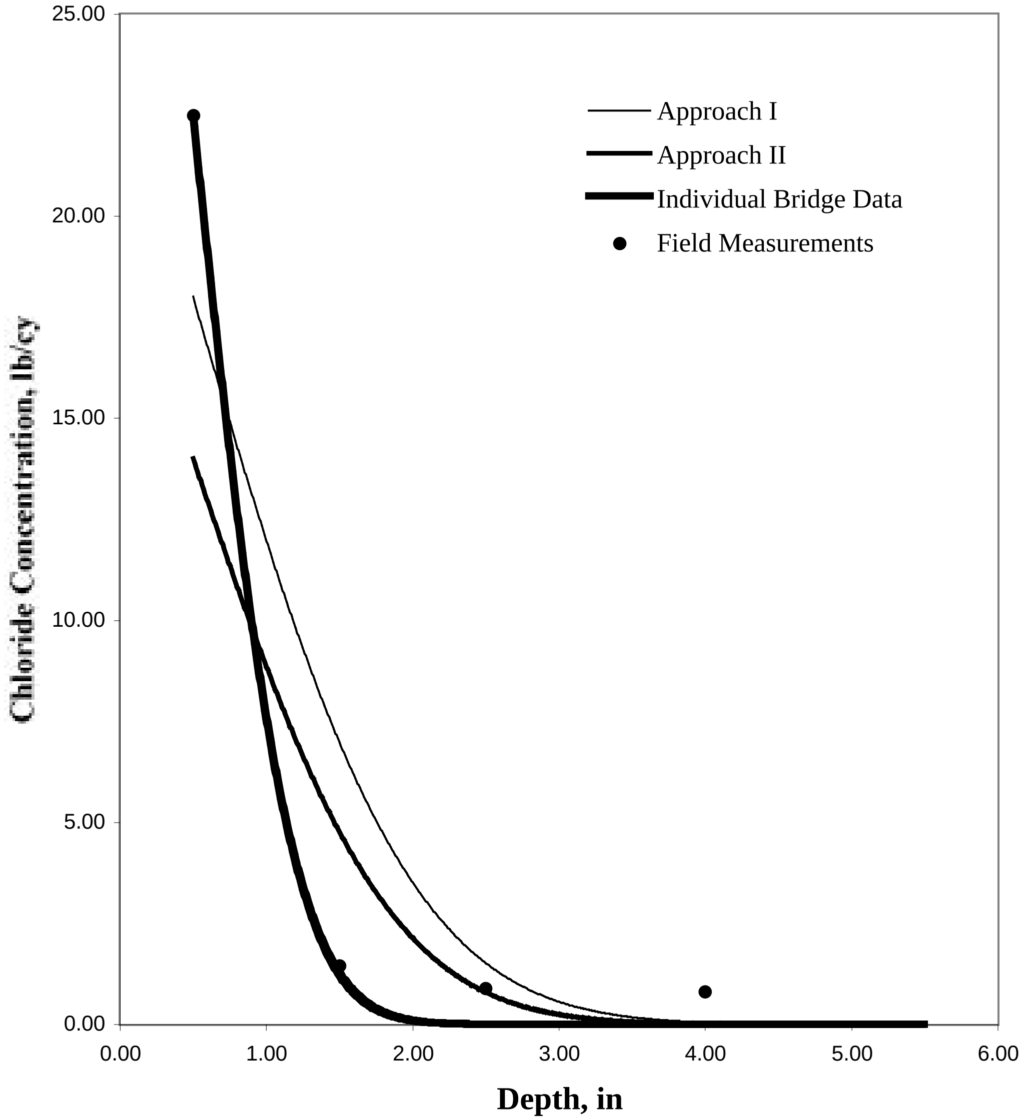

Figure 5.10 Chloride Concentration vs. Depth, 0781.5L218 A

6 PERFORMANCE OF EPOXY-COATED REBARS IN IOWA BRIDGES

6.1 Epoxy Discoloration

Most if not all of the collected rebar samples exhibited some level of epoxy discoloration. In most of the samples, the epoxy coating was discolored more on the top region of the rebars than on the bottom and side regions. There were a few exceptions to this trend though. Most discolored rebar samples came from the older bridges in this study, although some of the rebar samples from the newer bridges showed significant levels of discoloration as well. Figure 6.1 shows five rebar with some discoloration.

From analyzing some of the rebar samples on the scanning electron microscope, it was observed that the severely discolored epoxy coating areas have small micro-cracks on the surface of the epoxy as shown in Figure 6.2. The exact cause of these microcracks is not known. Exposure to weather prior to construction or from aging and exposure to chloride could have caused these microcracks and discoloration. The impact that these microcracks have on the epoxy coating's ability to protect against reinforcement corrosion is not known. Because these cracks appear to be only on the surface of the epoxy coating, their significance may be minimal.

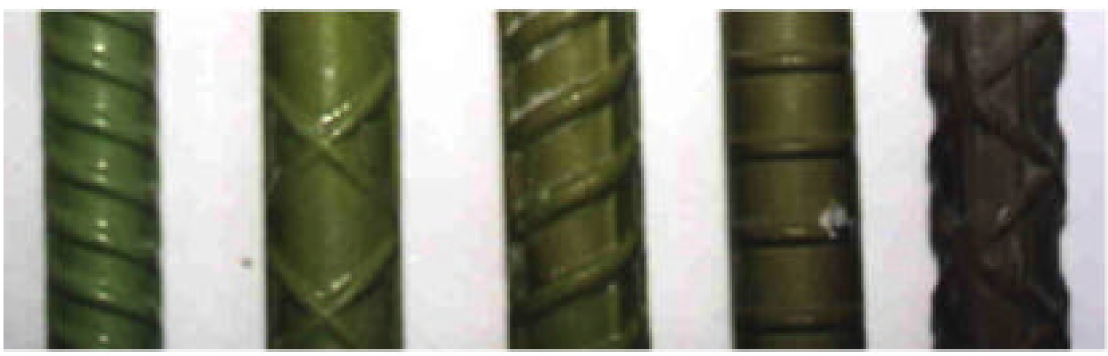

FIGURE 6.1 Rebar Samples Showing Some Discoloration

6.2 Epoxy Coating Hardness

The pencil hardness ratings (see section 4.2.5) of the epoxy coatings gave results ranging from four to seven, with most rebar samples exhibiting a rating of six. There was a slight trend showing an increase in hardness with bridge age. Also, the rebar samples with ratings of 7 (the hardest samples seen in the study) all came from bridges built from 1980 to 1983. An insignificant difference in hardness between the most discolored areas and the least discolored areas of the epoxy coatings was noticed. Therefore, no relationship between epoxy hardness and rebar rating or epoxy hardness and chloride content was observed from the data.

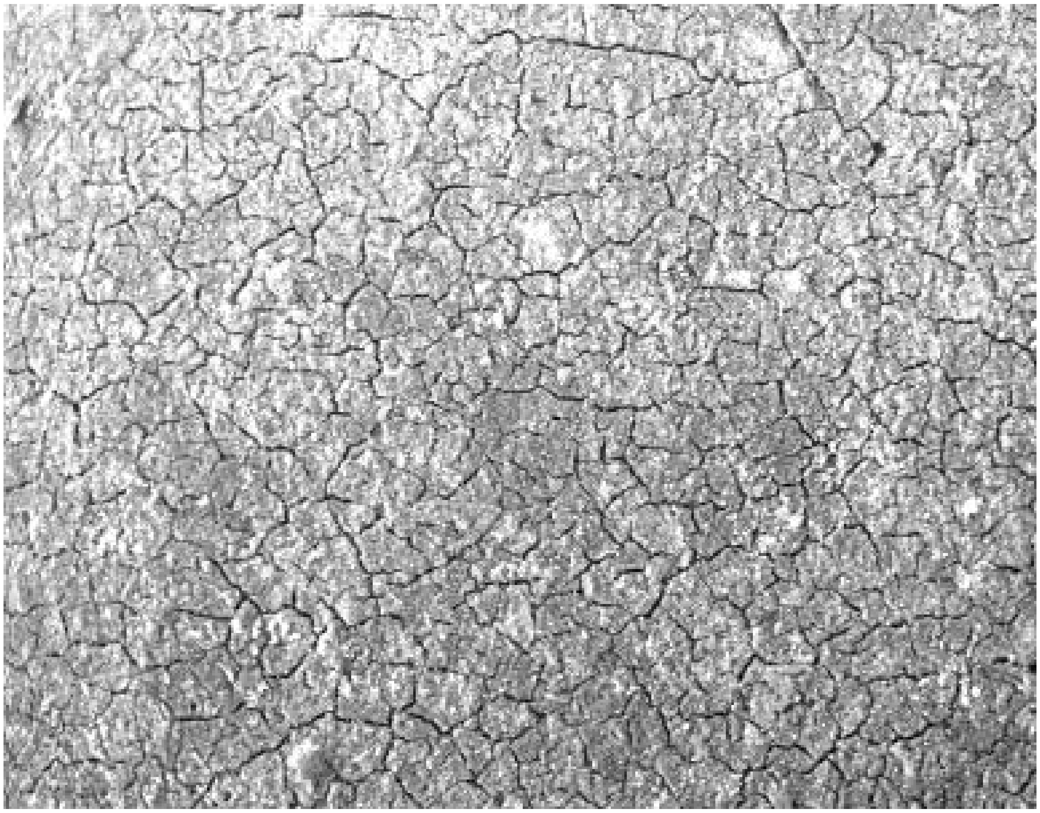

FIGURE 6.2 Microcracking in Discolored Epoxy Surface

6.3 General Observations of Epoxy-coated Rebars

Evaluation of the rebar samples collected from the bridge decks that were included in this study indicated a range of deterioration. The most corroded rebar samples were those collected from cracked locations. Although most of this corrosion was only on the surface of the steel, a couple samples were observed to have spots where corrosion product had built up slightly underneath the epoxy coating. The largest area of these spots observed was about two centimeters squared and bulged out to a depth of about one to two millimeters. One of the most significant findings from this study was that all of the rebar samples that were evaluated as having a rebar rating of one, two, or three came from cores that were taken from cracked locations. On all of the rebar samples rated one, two, or three, it appeared that surface corrosion was undercutting the epoxy coating. Figure 6.3 shows some of the rebar samples evaluated in this study that were given a rebar rating of one, two, or three.

FIGURE 6.3 Rebars Samples with Significant Corrosion

Of all the rebars taken from uncracked areas of the bridge decks, none had a rebar rating lower than four. The rebar ratings of four and five represented relatively good rebar conditions, although the defects in the epoxy coatings of rebars rated four or five could lead to corrosion problems in the future. Thus, it appeared that the presence of cracks in the deck surface has an impact on the condition of the rebars below these cracks.

6.4 Epoxy-coated Rebar Rating versus Age Relationship

As previously mentioned, several ECR samples from the top mat reinforcing steel in Iowa bridge decks were collected and were rated on the scale one to five as discussed in section 2.10. The result of rebar rating is summarized in appendix E.

To investigate the effects of deck cracking on rebar condition and hence on the durability on bridge decks, the collected rebar samples were grouped into two groups. The first group represented those bars retrieved from cores taken at cracked locations, while the second group included rebars samples obtained from cores drilled at uncracked locations. Only the first layer of top mat reinforcing steel was examined to develop the relationship between ECR rating and age since corrosion always commences at the outermost layer near the deck surface.

Examining the collected samples revealed that the rebar samples retrieved from cracked locations were more corroded than those obtained from uncracked locations. All the rebar samples collected from uncracked locations were evaluated as having rating of five or four, which indicated no corrosion appeared on the rebar surface. In contrast, five, 10.7, and 2.9 percent of the rebar samples obtained from cracked locations were rated three, two, and one, respectively. This indicated that there was some degree of corrosion and distress appearing on some of these rebars samples. The distribution of rebar rating for the first layer of reinforcement is summarized in Table 6.1.

TABLE 6.1 Distribution of Rebar Rating

Rebar Rating	Percent of Samples Taken from Uncracked Areas	Percent of Samples Taken from Cracked Areas
5	92.9	76.4
4	7.1	5.0
3	0.0	5.0
2	0.0	10.7
1	0.0	2.9

In general, the data collected in this investigation indicate that ECR performed well when no visible cracks were present in a bridge deck. In fact, no visible corrosion was observed on rebar segments collected at uncracked locations. The corrosion observed on the ECR at cracked locations can be attributed to the presence of high chloride content at the rebar level. This was not surprising since presence of cracks in a bridge deck expedite the diffusion process through cracked concrete.

Bars in each group were further subgrouped according to bridge age. According to the Federal Highway Administration (*43*), bridges are inspected every two years. Thus, it was reasonable to subgroup the bridges according to age in two-year intervals. Since there is a range of possible values of rebar samples that can be rated at a specific rating condition, one would naturally be interested in some central value such as the average. However, since different numbers of rebars in each time interval can be associated with different rating conditions and probabilities, one needs to use a weighted average (*44*), i.e., the expected value of the rating within each interval, rather than just use a straight average value. The following describes how the weighted average for the rebar rating within an interval was calculated.

Let *N(j)* be the number of rebar samples collected from bridge decks in the two-year interval, *j*. Further, let *n(k, j)* be the number of rebar samples rated at condition, *k*, (where *k* = 1, 2, 3, 4, or 5) within the particular interval. Using these assumptions, one can then calculate the probability *P(k, j)* as

$$P(k,j) = \frac{n(k,j)}{N(j)} \; . \tag{6.1}$$

The expected rating value *E (r, j)* for the bridges within the *j* interval can then be calculated using the following relation:

$$E(r,j) = 1 \times P(1,j) + 2 \times P(2,j) + 3 \times P(3,j) + 4 \times P(4,j) + 5 \times P(5,j) \tag{6.2}$$

Having calculated the expected rating value *E(r, j)*, one can then utilize a second-order polynomial model to develop a rebar condition-age relationship. The second-order polynomial model used herein was expressed by the following formula (*45*):

$$r(t) = \beta_0 + \beta_1 t + \beta_2 t^2 + \varepsilon \; , \tag{6.3}$$

where *r(t)* = rebar rating at time *t*, *t* = bridge deck age (in years), β_i = constants, *i = 1, 2, 3, …,* and ε = an error term.

For a new bridge deck, i.e., *t* = 0, the recorded rebar rating should be always five. Therefore, the intercept of the regression line integer β_0 was specified to be five.

Calculation of the constants in the relationship in equation 6.3 was accomplished using the calculated expected rating in conjunction with the Minitab software package for a second order polynomial model (*38*). It is worth noting that in equation 6.3 the error term represented the degree of uncertainty between predicted and measured values. The regression analysis yielded the following two relationships:

1. ECR condition-age relationship for rebars collected from cracked locations

$$r(t) = 5.0 + 0.0038t - 0.00311t^2 \; . \tag{6.4}$$

2. ECR condition-age relationship for rebars collected from uncracked locations

$$r(t) = 5.0 + 0.0135t - 0.00134\, t^2 \; . \tag{6.5}$$

A graphical presentation of these two relations is shown in Figure 6.4. As can be seen from Figure 6.4, the point (cracked locations) at age 18 (combining bridges constructed in 1978 and 1979) seemed to be lower than the expected values for rebars extracted from cracked locations. The bridge IDs 3988.5S025 and 5722.7O380 constructed in 1980 had exceptionally low rebar weighted averages of 1.5 and 2.0 respectively. Examining the source of these particular data points revealed that the crack width was wide and extended to the rebar level. Thus, as time went by, moisture and chloride ions directly attacked the coating films, causing the deterioration of ECR. The accuracy of the regression model was checked to ensure its appropriateness of application when a model was selected for the analysis. The coefficient of determination, R^2, associated with the regression analysis in equations 6.4 and 6.5 were found to be (from the output of Minitab regression analysis) 0.81 and 0.76, respectively.

Furthermore, residual plots, i.e., the relationships between the residual error and the normal score, were obtained to check the constancy of variance (*38*). The residual plots are shown in Figures 6.5 and 6.6. These figures illustrated that the second order polynomial regression model on raw data of rebar rating appear to be reasonably acceptable. Neglecting the two "outliers" points designated *a* and *b* in Figures 6.5 and 6.6, respectively, results in a fairly linear normal probability plots of the residuals, indicating a generally bell-shaped distribution of residuals (*45*). This indicates that the relationships in equations 6.4 and 6.5 are acceptable. These two points were those of the data obtained from bridges with age of 18 years old. Reviewing this data revealed that two of the five bridges with this age were rated low. These low conditions resulted in a low overall weighted average. In the author's opinion, one needs to collect more data for this particular age group to have more reliable results.

6.5 Effect of Deck Cracking on Epoxy-coated Rebar Rating

The relationships in equations 6.4 and 6.5 can be employed to estimate the effect of deck cracking on ECR conditions in bridge decks in the state of Iowa. Using these, one can estimate the time it takes an ECR located at cracked and uncracked locations to reach such a specific rating condition. For example, equations 6.4 and 6.5 yield approximately 32 and 53 years for an ECR to reach condition two for rebars located at cracked and uncracked locations, respectively.

One may notice that the relationships developed above do not directly account for the condition of the ECR prior to being placed in the deck. In other words, these relationships do not include terms that account for the degree of severity of existing chips in the coating, cracks in the coating film, thickness of the epoxy coating, or holidays.

Direct inclusion of all of these factors in one relationship representing the performance of ECR in bridge decks would be a formidable task. However, the influence of these effects on the performance of ECR could have been reflected in the collected data.

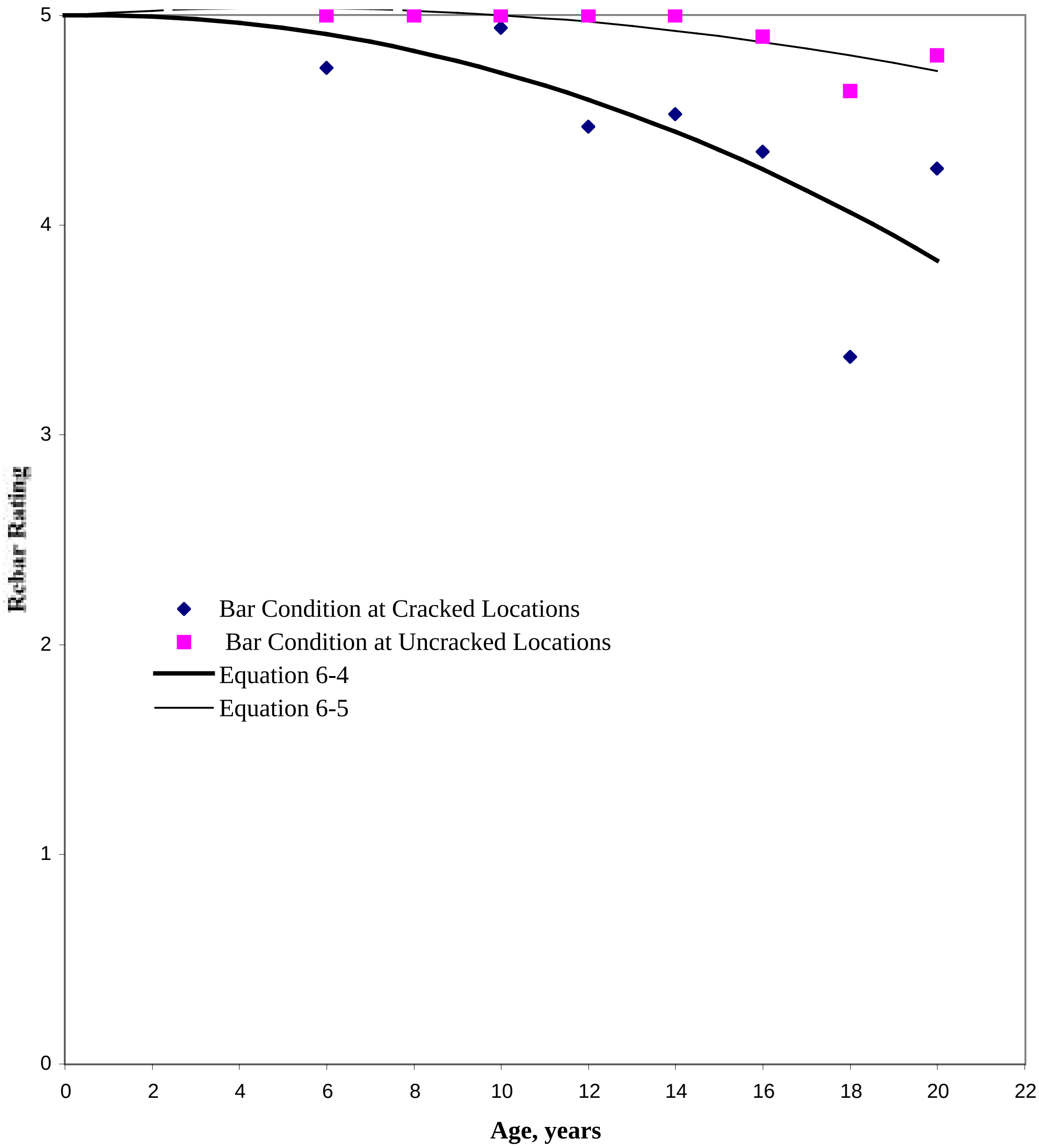

Figure 6.4 Rebar Rating vs. Age (Equations 6-4 and 6-5)

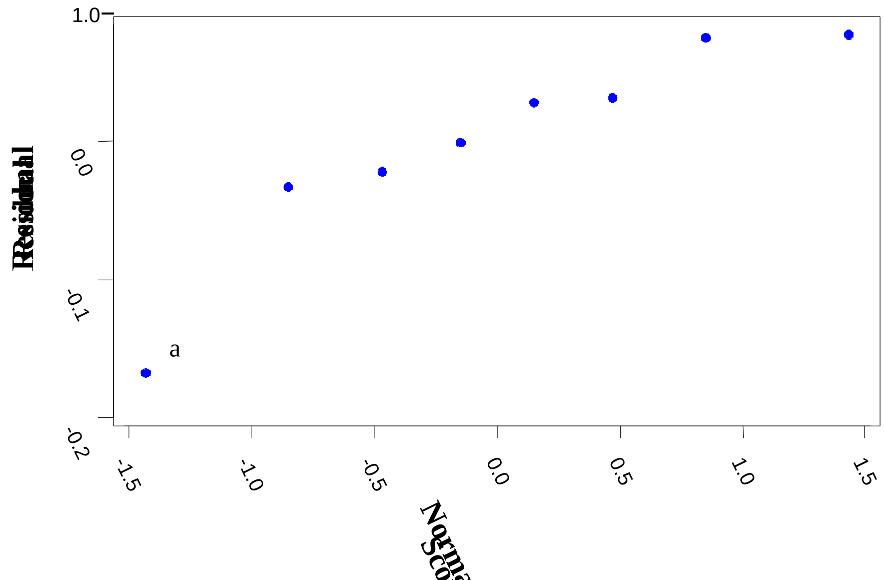

FIGURE 6.5 Normal Probability Plot of the Residuals for Equation 6.4

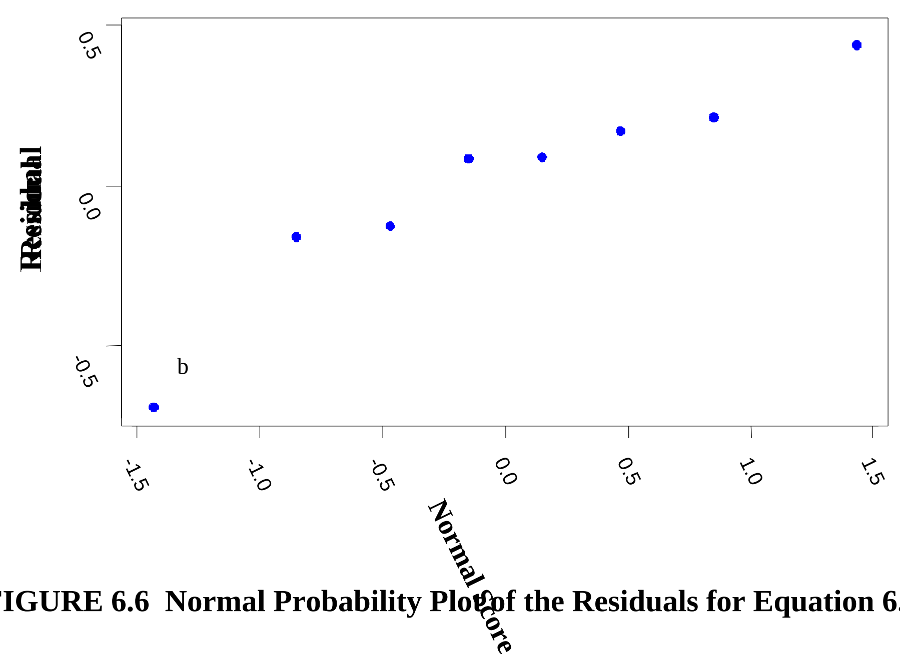

FIGURE 6.6 Normal Probability Plot of the Residuals for Equation 6.5

6.6 Adhesion of Coating to the Steel

The dry-knife adhesion test was performed on the collected rebar samples. The approach described in section 6.1 was utilized to determine the deterioration of the adhesion of the coating of the ECR in the state of Iowa. The result is summarized in appendix F.

The test revealed that coating adhesion decreases as time increases. Table 6.2 summarizes the distribution of the adhesion rating on rebar samples. Figure 6.7 illustrates how the adhesion decreased as time increased. In addition, the figure illustrates that there were significant effects of cracking on the adhesion of the ECR collected from cracked locations and these ECR were less bonded than those of uncracked locations. This reveals that the moisture and the high chloride concentration at cracked locations have some effects on the bond between the epoxy-coated film and the reinforcing bars.

TABLE 6.2 Distribution of Coating Adhesion on Rebars

Adhesion Rating	Percent of Samples Taken from Uncracked Areas	Percent of Samples Taken from Cracked Areas
3	48	43
2	47	40
1	5	17

6.7 Comparison between the Performance of Black Steel and Epoxy-coated Rebars in Iowa Bridges

As previously mentioned, in section 2.3, the end of functional service life of a concrete bridge based on corrosion damage that influences riding quality is 9 to 14 percent of the worst traffic, i.e., of the right lane (*14*). Following this definition and using the diffusion-spalling model discussed in section 2.4, one can estimate the service life of a bridge deck. To accomplish such a purpose, one needs the mean value and standard deviation of the cover depth as well as the rate of chloride diffusion and the chloride content at 0.5 inches from the top surface of the deck. Estimation of these elements was discussed in detail in section 5. The following sections summarize the measurements of the cover depth for Iowa bridges and examples of calculating the service life for a deck using black steels or ECR in Iowa bridges.

TABLE 6.3 Means and Standard Deviations

	Mean (inches)	Standard Deviation (inches)
Phase 1	2.70	0.456
Phase 2	2.77	0.433
Overall	2.74	0.444

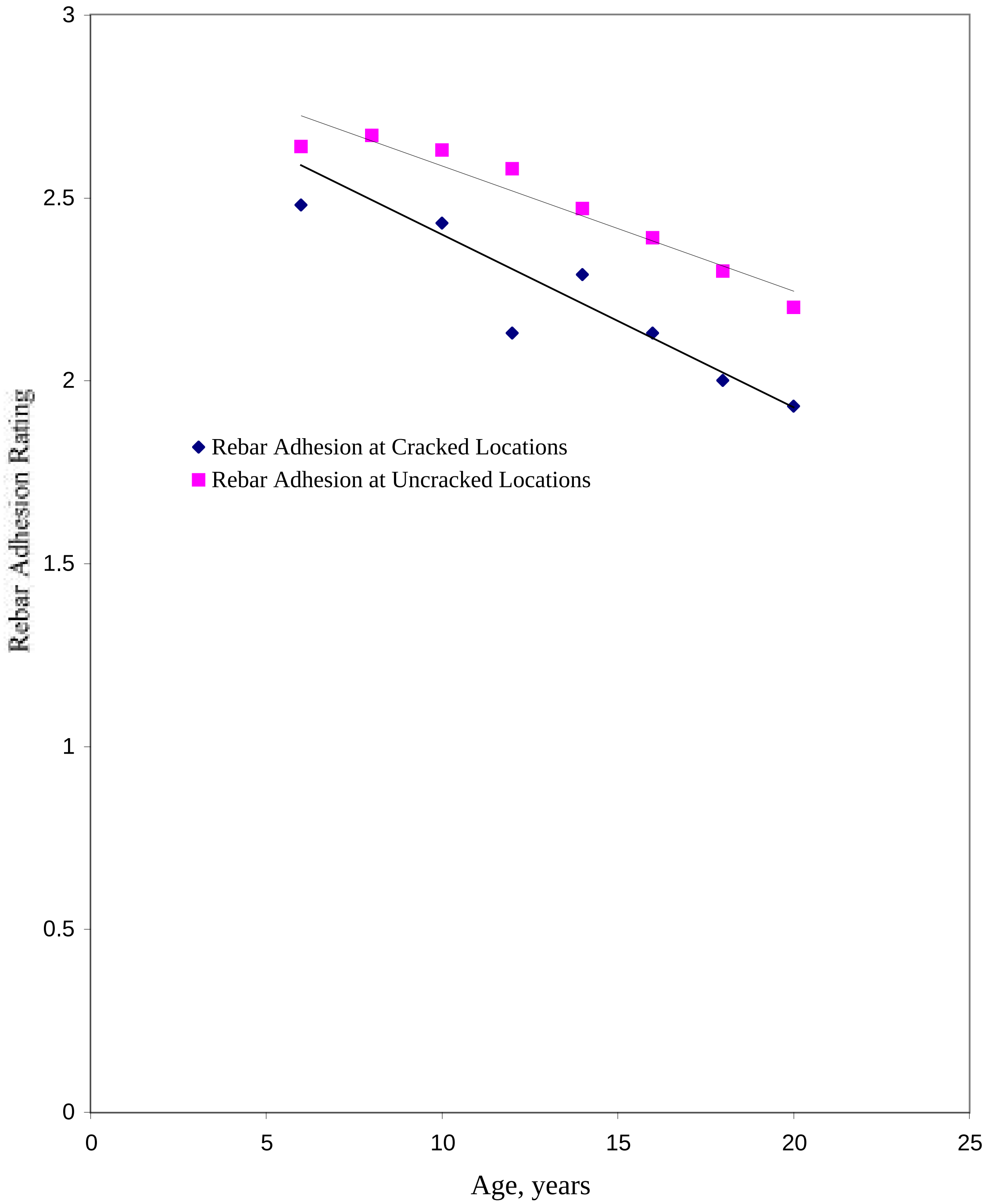

Figure 6.7 Rebar Adhesion Rating vs. Age

6.7.1 Mean and Standard Deviation for Cover Depth in Iowa Bridges

The cover of the top rebars for all sampled cores was measured. Table 6.3 summarizes the average reinforcing cover depth through the project. Figure 6.8 shows the histogram plot as well as the reasonably bell-shaped normal distribution curve for the measured values. As expected, the cover depth appeared to be the normally distributed. To further verify the normal distribution of cover depth, a normal probability plot (*45*) was developed using the Minitab software (*38*), and the results are summarized in Figure 6.9. The figure illustrates a linear relation between the cumulative probability and the measured depth. This verifies the normal distribution of the cover depth.

6.7.2 Corrosion Threshold for Epoxy-coated Rebars

The corrosive threshold for ECR was defined by Sagues et al. (*19*) to be about 1.2 to 3.6 lb/yd^3; and for black steel bar it is 1.2 lb/yd^3. However, the data collected herein reveal that an average chloride concentration of 7.5 lb/yd^3 existed in locations where rebar samples having rating of three, i.e., the condition representing zero to 20 percent of corrosion on the ECR surface. This is the condition at which corrosion becomes noticeable on ECR. Using this finding, a corrosive threshold for ECR ranges from 3.6 to 7.5 lb/yd^3 can then be assumed. The example in section 6.7.4 illustrates how to calculate the service life of a bridge deck in the state of Iowa that is reinforced with epoxy-coated rebars.

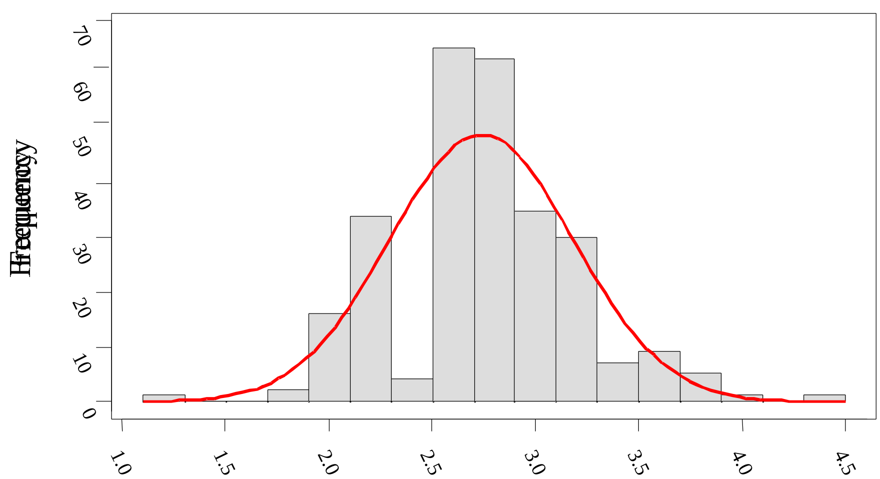

FIGURE 6.8 Histogram of Cover Depth with Normal Distribution Curve for Iowa Bridge Decks

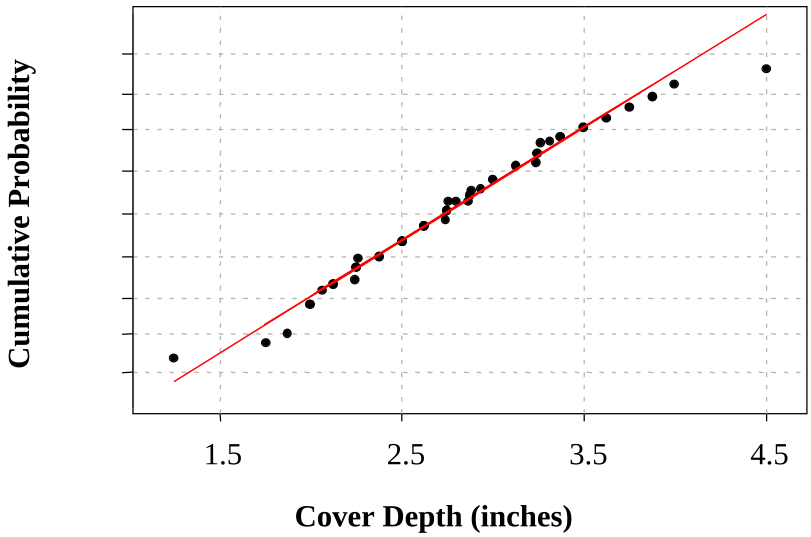

FIGURE 6.9 Normal Probability Plot

6.7.3 Time between Corrosion Initiation and Spalling

As previously mentioned, Fick's Second Law can be used to calculate the time in which the chloride concentration at the rebar level reaches the corrosive threshold for either black or epoxy coated rebars. Assuming an additional time needed for spalling to take place in bridge decks, one can then estimate the service life of a bridge deck. For example, Larson et al. (*15*) documents that it take three to five years for spalling to occur in bridges reinforced with black bars after the chloride concentration reaches the threshold value. However, searching the literature did not reveal any date regarding the time required for spalling to occur in bridge decks with ECR. But because the main objective of using a thin coating on the reinforcing rebars is to prevent corrosion, one may safely assume a time longer than two to five years for the ECR to corrode to a condition that may result in spalling.

In general spalling will occur after enough corrosion has built up, causing significant increase in the rebar volume. This was assumed herein to take place a few years after approximately 60 percent or more of the rebar surface was corroded, i.e., when the rebar rating reaches condition one. In addition, a period of approximately 5 to 8 years was assumed for the time from corrosion build up to spalling. This time period is slightly longer than that associated with black bars.

6.7.4 Illustrative Example to Calculate Service Life of a Bridge Deck

The following example uses the diffusion-spalling model (see section 2.4) and illustrates how to incorporate the above assumptions to estimate the functional service life of a bridge deck in the state of Iowa:

> Given an Iowa Bridge deck with C_o = 14.0 lb/yd^3 and D_c = 0.05 in^2/yr: End of functional life = 11.5 percent, which is the average of 9 to 14 percent damage in the worst traffic lane (*14*). Average concrete cover depth $\bar{x}$ = 2.74 inches, associated with standard deviation σ = 0.444 inches. The corrosive chloride threshold ranged from 3.6 to 7.5 lb/yd^3 for ECR. Assume that 11.5 percent of the rebar is contaminated by the chloride ion. The alpha value (Table 2.5) for calculating the rebar cover depth is $\alpha = -1.2$. Calculate the time required to reach the corrosive threshold and the time to rehabilitation.
>
> Use equation 2.6 to calculate the cover depth, x:
>
> $x = \bar{x} + \alpha\sigma = 2.74 + (-1.2)(0.444) = 2.21$ inches ,

Use equation 2.7 to calculate the time, t, required so that the chloride content at the rebar level reaches a threshold value of 3.6 lb/yd^3:

$$C_{x,t} = C_o\left(1 - \mathrm{erf}\left(\frac{x}{2\sqrt{D_c t}}\right)\right)$$

For the threshold of 3.6 lb/yd^3,

$$3.6 = 14\left(1 - \mathrm{erf}\left(\frac{2.21}{2\sqrt{0.05t}}\right)\right)$$

Solving for t, one estimates a time, t, of 38 years for the chloride to reach the rebar level. The corresponding rebar rating at that time following equation 6.4 is 3.6. In addition, equation 6.4 will yield an additional 22 years for the rebar to reach condition 1. Assuming five years from corrosion build up to spalling, one can then estimate the time for spalling of the above described bridge deck to be 65 years.

In comparison to black steel bar, the corrosive threshold is 1.2 lb/yd^3. Thus, the time to reach the threshold is calculated as follows:

t = 17 years,

$$1.2 = 14.0\left(1 - \mathrm{erf}\left(\frac{2.21}{2\sqrt{0.05t}}\right)\right)$$

The average time for spalling ranged between two and five years = 3.5 = (15) years for black steel. Thus, time required to rehabilitation for unprotected steel = 17 + 3.5 = 20.5 years. Therefore, the example above illustrates the significantly increase in the service life of a bridge constructed with ECR.

The above estimated time for spalling to occur can even be longer if one uses the fact that the data collected in this work reveal that an average chloride concentration of 7.5 lb/yd^3 existed in location where bar samples has condition rating of 3 (see section 6.7.2).

7 INVESTIGATION OF THE SELECTED BRIDGE DECKS WITH BLACK REBARS

During the progress of this research, the Research Advisory Committee requested the inclusion of few bridge decks constructed with black bars. Especially, the committee requested the inclusion of three bridge decks that were constructed using what is referred to as a "two-course placements" construction approach. Three bridges in Tama County where sealer was applied to one of these bridge decks were also evaluated. To address the requests, the approaches outlined in previous chapters were used. The chloride concentration at different depths was measured, and the associated diffusion constants were computed. Appendix G summarizes the findings of the measurements. The determination of the diffusion constant and the rebar rating are summarized in the following sections. However, one should carefully interpret the results summarized herein since very small samples were included in the investigation.

7.1 Two-Course Placements Bridges

Three bridge decks constructed in 1976 and 1977 using two-course placements were evaluated. In this method, approximately three-fourths of deck thickness is cast and is allowed to deflect and cure. The remaining deck slab thickness was added later using Iowa low slump overlay mix design concrete. The effect of this construction procedure on the permeability of the chloride ion ingress was investigated.

About 2.5 inches thickness of the low slump dense overlay concrete was observed from the extracted cores. The mean cover depth and standard deviation were found to be 3.70 inches and 0.313, respectively. The mean cover depth is considerably greater than the eighty bridges included in this study.

Table 7.1 summarizes C_o and D_c for the two-course placements bridge. The table shows that a two-course placements concrete deck has a lower diffusion constant, which coupled with larger cover depth will significantly delay the accumulation of chloride ions at the rebar level. The rebar ratings for two-course placements bridge decks are summarized in Table 7.2. These results illustrate that bridge decks constructed with two course placements are in good condition.

TABLE 7.1 C_o and D_c for Two-Course Placements Bridges

	C_o (lb/yd^3)	D_c (in^2/yr)
2401.1S039	10.2	0.0085
3966.4S044	11.2	0.0395
4039.6R020	12.8	0.0050
Average	11.4	0.0176

TABLE 7.2 Rebar Rating for Two-Course Placements Bridge Decks

	Rebar Obtained from Cracked Locations	Rebar Obtained from Uncracked Locations
2401.1S039	3.0	3.4
3966.4S044	2.5	3.3
4039.6R020	3.0	3.0
Weighted Average	3.0	3.7

7.2 Tama County Bridges

The Three Tama County bridges included in this study were built in 1968 with black reinforcing steel. The first two bridges referred herein as Tama 1 and Tama 2 are of steel girder–type structure with a total length about 505 feet, whereas the third bridge, Tama 3, is a concrete slab–type structure with a span length of 39 feet. According to Tama County Engineer Office (*39*), sealer had been first applied only to the bridge, designated as Tama 1, in 1984 and thereafter at every five years interval, i.e., in 1989, 1994, and 1999. The purpose of the application of sealer was to enhance the performance of bridge decks and thus to provide protection against deterioration of the reinforcing steel in the concrete deck.

Tables 7.3 and Table 7.4 summarize the average diffusion constant and the weighted average rebar rating, respectively, for each bridge. As can be seen, Tama 1 had the lowest surface chloride concentration. This can be attributed to the effectiveness of the sealer, which prevented from more chloride ions to penetrate the deck surface. On other hand, a higher diffusion was estimated for this particular bridge than that of Tama 2. This could have been caused by higher chloride concentration that existed in the bridge prior to the application of sealer.

If the entrapped chloride concentration was high prior to sealing the bridge deck, the consecutive application of sealer will not provide the full protection against the ingress of chloride ions through the deck. Consequently, a sufficient chloride accumulation at rebar level could initiate corrosion of reinforcement. Therefore, a sealed surface will not prevent corrosion of rebars but rather only slow down the accumulation of chloride ions.

TABLE 7.3 Summary of C_x, C_o, and D_c for Tama County Bridges

	C_x @ Rebar Level (lb/yd^3)	C_o (lb/yd^3)	D_c (in^2/yr)
Tama 1	0.97	11.6	0.010
Tama 2	0.46	16.0	0.004
Tama 3	1.70	12.7	0.032

TABLE 7.4 Weighted Average of Rebar Rating for Tama County Bridge

	Rebar Obtained from Cracked Locations	Rebar Obtained from Uncracked Locations
Tama 1	3.0	4.0
Tama 2	1.5	3.5
Tama 3	4.0	4.0

Table 7.4 reveals that Tama 1, with the application of sealer, had a better rebar rating in both uncracked and cracked locations when compared with Tama 2. One can notice that Tama 1 and Tama 3 have only one scale difference of the rebar rating between cracked and uncracked locations, whereas Tama 2 has two scales difference of the rebar rating.

Nevertheless, Tama 3 has an excellent rebar rating even after 30 years of service without the application of sealer on the deck surface. During the coring it was observed that Tama 1 and Tama 2, i.e., the two steel girder bridges, had many transverse cracks on the deck surface, while the Tama 3 bridge had few cracks. Moreover, it was noticed that cracks on the extracted cores from cracked locations on Tama 3 did not extend to the rebar level. The presence of the cracks in Tama 1 and Tama 2 can be related to the large flexibility that is associated with the long span and the small dimensions of the steel girder used to construct these two bridges. Those findings can explain why the rebar rating in Tama 3 performed exceptionally well when compared with those of Tama 1 and Tama 2.

8 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

8.1 Summary

The use of epoxy-coated rebars was first used in the state of Iowa in 1976 as the reinforcing steel in the top mat of bridge decks. Although it was long believed that ECR has a superior performance over black bars, concerns of the effect of deck cracking on the durability of these decks still represent a concern to DOT engineers. The objective of this work was to address this concern and to estimate the time to conduct preventative maintenance or to overlay a bridge deck.

Published literature was searched to review related work. Cause of cracking, corrosion mechanism, corrosion process, and the performance of ECR in bridge decks on other states were reviewed. The end of a bridge deck service functional life and the corrosion process were defined. In addition, the corrosion threshold was introduced and used in conjunction with Fick's Second Law to estimate the length of the corrosion initiation stage of the black rebar and ECR.

Eighty-one bridges constructed with ECR in either top mat or both mats were selected for collecting core samples. Geographical location and age were considered when selecting these bridges. Two core samples from cracked locations and two cores from uncracked locations in a bridge deck were obtained. Powder samples from different depths through these cores were gathered and analyzed for chloride concentration, using an X-ray fluorescence spectrometer. Rebar samples in these cores were rated on a scale from one to five, with five being a rebar in perfect condition. The epoxy coating hardness and adhesion were also documented.

The chloride analysis results were used to determine the surface chloride concentration and diffusion constants required the utilization of Fick's Second Law. A chloride concentration–depth relationship was developed and calibrated using measured chloride concentration in different bridge decks. Data related to rebar rating were used in a statistical model to relate the condition of ECR to the age of a bridge deck taking into account the effects of deck cracking. These developed relations were then applied to estimate the service life of a bridge decks and the time when preventative maintenance will be needed.

8.2 Conclusions

The following conclusions can be drawn regarding the performance of epoxy-coated rebars in Iowa bridge decks:

- The average reinforcing steel cover depth was found to be 2.75 inches.
- Adequate concrete cover depth can significantly prolong the initiation of reinforcing steel corrosion.
- No delaminations or spallings had been found in bridge decks constructed with ECR in which the oldest bridge deck is 20 years. No maintenance had been yet performed for those constructed with ECR in Iowa.

- The chloride content, C_o, at 0.5 inch below the deck surface and the diffusion constant, D_c, were found to be 14.0 lb/yd^3 and 0.050 in^2/yr, respectively.
- The corrosive threshold range from 3.6 lb/yd^3 to 7.5 lb/yd^3 can be used to estimate the service life of a bridge deck. Using ECR in bridge decks can significantly prolong the service life of bridge decks.
- Most of the corrosion was found on ECR extracted from cracked locations in bridge decks.
- All of the rebars extracted from uncracked locations showed no evidence of corrosion.
- The developed relationships (equations 6.4 and 6.5) between rebar condition rating and age illustrated that cracking in a bridge deck had significant impact on the deck durability.
- Sealers can slow down the accumulation of chloride ions in bridge decks and could effectively provide protection against corrosion.
- The rebar adhesion was found to decrease as time increases.
- The moisture and high chloride concentration are among the factors that can weaken the coating adhesion.

8.3 Recommendations

The followings are recommended for future work:

- The overall condition of a bridge deck could not be exactly assessed using four cores taken from each bridge deck. Thus continued research involving detailed analyses of bridges with epoxy-coated rebars is needed.
- The effect of coating defects, such as coating holiday due to manufacturing process and coating chip resulting from construction practice, need to be investigated since the coating defect is a critical factor in the performance of ECR.
- The density of the cracking on a deck in terms of cracking length per area needs to be defined and considered in estimating the durability of a bridge deck.
- The effect of bridge deck flexibility on the performance of a bridge deck needs to be investigated.

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APPENDIX A: CORROSION MECHANISM

A.1 Why Metal Corrodes

Energy is required to derive metals from ores. Ores are the natural oxides, sulfides, and other reaction products of metals with the environment. Usually, the desired compounds or substances must be separated from large quantities of unwanted deposits by a chemical process to make the material useful. To be released from ores, metals absorb heats as the required energy to escape its original state. The energy is then stored in the metal and later released when corrosion takes place. This is the reversed process as metals return to its beginning stable state, the ore. The amount of energy needed to separate the desired metals from minerals is varied from one to another. Table A.1 lists some metals in the order of diminishing amount of energy required converting them from their ores (*4*).

Corrosion of iron is a naturally renewable cycle from mineral to iron and vice versa. The product of corrosion of iron is rust, which has the same chemical compounds as the ore, known as hematite (*5*), which is used for producing metallic iron. Hematite is an oxide of iron (Fe_2O_3).

Figures A.1 and A.2 illustrate the conversion cycles following the typical paths of refining and corrosion process (*5*).

TABLE A.1 Required Energy for Some Metals to Be Separated from Minerals

	Energy Required
Potassium	Most
Magnesium	↓
Beryllium	
Aluminum	
Zinc	
Chromium	
Iron	
Nickel	
Tin	
Copper	
Silver	
Platinum	↓
Gold	Least

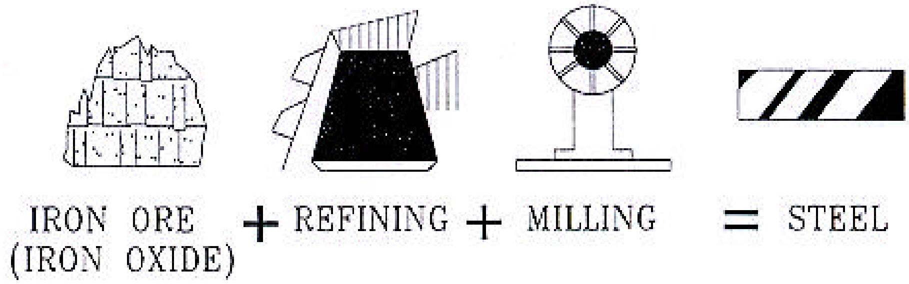

FIGURE A.1 Conversion of Iron Ore to Steel (*5*)

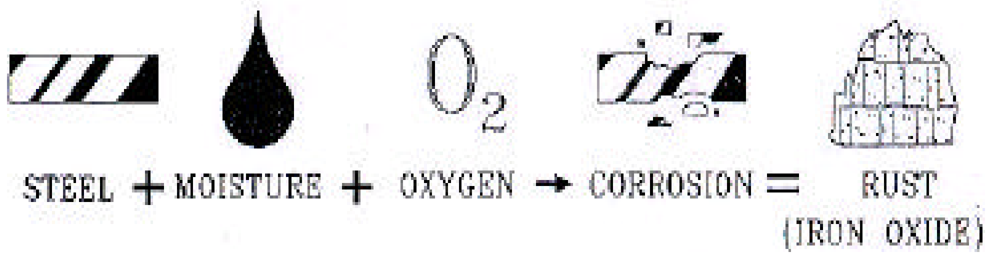

FIGURE A.2 Conversion of Steel to Rust (*5*)

A.2 Electrochemistry of Iron Corrosion

Electrochemistry deals with the relationships between transfer of electricity and chemical reactions. The understanding of the electrochemical process provides an insight into the cause of corrosion. Corrosion is defined as the conversion of a metal into other forms of metal compound by chemical reaction involved with metal and elements surrounding its environment. The most common elements existing in the environment that react with metal are water, oxygen, acids, and salts. These elements are called reactants.

When corrosion is taking place, the metal loses electrons and forms cations, which are ions with a positive charge. Oxidation is an ion loses electrons by a substance reacting with it. For example, the surface of the iron serves as an anode at which the iron undergoes oxidation. The following is the chemical reaction equation of iron that undergoes oxidation:

$$Fe \rightarrow Fe^{+2} + 2e^{-} \quad , \qquad (A.1)$$

where Fe is the chemical formula for iron, Fe^{+2} is iron losing two electrons, known as the ferrous ion, and $2e^{-}$ are two lost electrons.

At the presence of oxygen and water molecules contained in the atmosphere, for example, oxygen is transformed from a neutral molecular to an anion, which has become more negatively charged by gaining electrons. This process is called reduction. The gain of electrons comes from loses of electrons in two substances that react with each other. Oxidation and reduction are coupled together as electrons transferred between them. The following chemical equation illustrates the cathodic reaction:

$$O_2 + 2H_2O + 4e^- \rightarrow 4OH^- \ , \qquad \text{(A.2)}$$

where O_2 is an oxygen atom, H_2O is water, and OH^- is hydroxyl.

Reaction of equation A.1 forms ferrous ions, whereas reaction of equation A.2 forms hydroxyl ions. Both ions react and produce ferrous hydroxide [$Fe(OH)_2$]:

$$2Fe^{+2} + 4OH^- \rightarrow 2Fe(OH)_2 \ . \qquad \text{(A.3)}$$

In the course of corrosion, ferrous hydroxide is further oxidized to Fe^{+3}, forming ferric hydroxide [$Fe(OH)_3$]:

$$2Fe(OH)_2 + 0.5O_2 + H_2O \rightarrow 2Fe(OH)_3 \ . \qquad \text{(A.4)}$$

As an effect of dehydration through the exposure to the environment, ferric hydroxide becomes ferric oxide (Fe_2O_3), known as rust. Combining with equations A.2, A.3, and A.4 and the effect of dehydration, the general chemical equation of corrosion of iron can be explained as follows:

$$Fe^{+2} + O_2 + 4H_2O + 2xH_2O \rightarrow Fe_2O_3 \quad xH_2O + 8H^+ \ , \qquad \text{(A.5)}$$

where Fe_2O_3 is the rust and H^+ is the hydrogen atom losing one electron.

It is observed that anodic and cathodic reactions are coupled mutually when corrosion is taking place. One can possibly reduce corrosion by eliminating one of either anodic or cathodic reaction. This idea, for example, by eliminating cathodic reaction, can be achieved by insulation of air from contacting the aqueous solution or by removing the dissolved air. Iron cannot corrode in the water unless oxygen is present. Prevention of rusting is achievable if cathodic reaction can be eliminated by means of coating. Thus the use of epoxy-coated reinforcement is believed to be an effective means of preventing steel from the corrosion.

APPENDIX B: INFORMATION RELATED TO BRIDGES INCLUDED IN THIS STUDY

TABLE B.1 Selected Bridges Constructed with ECR

ECR	BRIDGE ID	ADT	TYPE	COUNTY	MAIT. DIV.	C/S	REG	BUILT	LOCATION	CROSSED
TOP	6011.6S009	NA	NA	LYON	Sioux City Area	Conc.	N	1976	NA	NA
TOP	0476.4S002	2520	282	Appanoose	Chariton/Ottumwa	Conc.	S	1978	4.7 MI.E.OF JCT.5	BR DOGGETT CREEK
TOP	4801.5S220	2000	502	Iowa	Cedar Rapids Area	Conc.	S	1978	1.5 MI. N. JCT. US #6	IOWA RIVER
TOP	6360.4S005	3530	423	Marion	Chariton/Ottumwa	Steel	S	1978	AT E.JCT.92	OVER IA 92 (E JCT)
TOP	9700.8S982	2180	502	Woodbury	Sioux City Area	Conc.	N	1978	0.8 MI.E.OF SIOUX CITY	OVER ABANDON RR
TOP	9708.3S982	760	282	Woodbury	Sioux City Area	Conc.	N	1978	8.3 MI.E.OF SIOUX CITY	SMALL STREAM
TOP	0668.7S021	1730	502	Benton	Ames Area	Conc.	N	1979	7.9 MI.N.OF JCT.30	BRANCH SALT CREEK
TOP	1410.2S071	3310	201	Carroll	Sioux City Area	Conc.	N	1979	1.3 MI.S.OF CARROLL	SMALL STREAM
TOP	5098.3S065	7100	402	Jasper	Des Moines Area	Steel	S	1979	AT JCT.117	INDIAN CREEK
TOP	5752.0R030	17300	502	Linn	Cedar Rapids Area	Conc.	N	1979	0.8 MI.W.OF JCT.380	OVER CR&IC RR
TOP	6345.2S092	1570	201	Marion	Chariton/Ottumwa	Conc.	S	1979	2.1 MI.E.OF WARREN CO.	COAL CREEK
TOP	1390.7S175	2050	502	Calhoun	Sioux City Area	Conc.	N	1980	2.7 MI.W.OF LAKE CITY	PRAIRIE CREEK
TOP	3988.5S025	860	502	Guthrie	Altantic/Creston Area	Conc.	S	1980	2.4 MI. N. GUTHRIE CENTER	BRUSHY CREEK
TOP	5721.6R380	69300	502	Linn	Cedar Rapids Area	Conc.	N	1980	2.3 MI.S.OF JCT.100	OVER H AVE N.E.
TOP	5722.4R380	69300	502	Linn	Cedar Rapids Area	Conc.	N	1980	1.5 MI.S.OF JCT.100	COLDSTREAM AVE N.E.
TOP	5722.7O380	65200	423	Linn	Cedar Rapids Area	Steel	N	1980	1.1 MI.S.OF JCT.100	I-380
TOP	2579.9S044	2670	201	Dallas	Altantic/Creston Area	Conc.	S	1981	1.3 MI.E.OF GUTHRIE CO.	MOSQUITO CREEK
TOP	3236.8S004	3470	502	Emmet	Storm Lake	Conc.	N	1981	1.1 MI.S.OF JCT.9	WEST FK DES MOINES RV
TOP	3975.9S044	3150	201	Guthrie	Altantic/Creston Area	Conc.	S	1981	0.8 MI.E.OF PANORA	BAYS BRANCH CREEK
TOP	7526.9S003	3020	502	Plymouth	Storm/Lake Area	Conc.	N	1981	0.8 MI.W.OF JCT.75	FLOYD RIVER
TOP	8224.1R061	23300	423	Scott	Davenport Area	Steel	S	1981	1.1 MI. N.OF JCT.80	OVER MT JOY RD F-55

ECR	BRIDGE ID	ADT	TYPE	COUNTY	MAIT. DIV.	C/S	REG	BUILT	LOCATION	CROSSED
TOP	0643.5L380	10300	423	Benton	Cedar Rapids Area	Steel	N	1982	AT JCT.150	OVER IA 150
TOP	1479.8S030	5300	502	Carroll	SiouxCity Area	Conc.	N	1982	1.0 MI.W.OF JCT.71	MIDDLE RACCOON RIVER
TOP	5738.1L380	15200	502	Linn	Cedar Rapids Area	Conc.	N	1982	1.3 MI. S. BENTON CO.	EAST BLUE CREEK
TOP	6219.3S137	5900	502	Mahaska	Chariton/Ottumwa	Conc.	S	1982	5.2 MI.S.OF JCT.92	SMALL STREAM
TOP	9259.9S218	5400	201	Washington	Fairfield/ Washington	Conc.	S	1982	2.8 MI.N.OF HENRY CO.	DRAINAGE DITCH
TOP	0475.2S002	3182	502	Appanoose	Chariton/Ottumwa	Conc.	S	1983	3.1 MI.E.OF CENTERVILLE	CHARITON RIVER
TOP	0727.5A020	10000	502	Black Hawk	Waterloo Area	Conc.	N	1983	AT JCT.63	RAMP OVER US 63
TOP	0777.9L218	10100	201	Black Hawk	Waterloo Area	Conc.	N	1983	0.8 MI.S.OF JCT.380-20	SINK CREEK
TOP	5293.7L218	4160	502	Johnson	Cedar Rapids Area	Conc.	S	1983	2.3 MI.N. OF JCT.1	OVER MELROSE AVE
TOP	6348.5S005	3530	502	Marion	Chariton/Ottumwa	Conc.	S	1983	2.2 MI.N.OF MONROE CO.	SOUTH CEDAR CREEK
TOP	0757.1L380	10000	502	Black Hawk	Waterloo Area	Conc.	N	1984	1.7 MI.N.OF BUCHANAN CO.	SPRING CREEK
TOP	0761.5O380	440	423	Black Hawk	Waterloo Area	Steel	N	1984	3.5 MI.S.OF E.JCT.20	I-380
TOP	1253.3S014	2780	502	Butler	Waterloo Area	Conc.	N	1984	4.9 MI.S. OF JCT.3	W FK CEDAR RIVER
TOP	1910.0S346	1450	502	Chickasaw	Waterloo Area	Conc.	N	1984	2.1 MI.W. OF JCT.63	WAPSIPINICON RIVER
TOP	2336.2O061	130	502	Clinton	Davenport Area	Conc.	S	1984	1.6 MI.N.OF SCOTT	MUSKRAT RD OVER US 61
TOP	1411.6S071	4090	201	Carroll	Sioux City Area	Conc.	N	1985	IN CARROLL	MIDDLE RACCOON RIVER
TOP	4227.3S065	980	201	Hardin	Ames Area	Conc.	N	1985	1.3 MI.N. OF STORY CO.	MINERVA CREEK
TOP	5298.6S001	4160	201	Johnson	Cedar Rapids Area	Conc.	S	1985	0.1 MI.N.OF SOLON	MILL CREEK
TOP	5587.2S169	2310	502	Kossuth	Forest City Area	Conc.	N	1985	0.2 MI. N. OF HUMBOLDT CO	DRAINAGE DITCH
TOP	7993.4S063	2280	201	Poweshiek	Des Moines Area	Conc.	S	1985	3.6 MI.S.OF JCT.80	NORTH ENGLISH RIVER
BOTH	1052.2S150	5700	201	Buchanan	Cedar Rapids Area	Conc.	N	1986	IN HAZELTON	HAZELTON CREEK
BOTH	4926.7S052	2160	502	Jackson	Davenport Area	Conc.	N	1986	2.5 MI. N.OF BELLEVUE	SPRUCE CREEK
BOTH	5419.0S149	2940	502	Keokuk	Fairfield/ Washington	Conc.	S	1986	2.0 MI.S.OF W.JCT.92	NORTH SKUNK RIVER
BOTH	6488.8S030	9400	502	Marshall	Ames Area	Conc.	N	1986	3.1 MI.W.OF JCT.146	OVER C&NW RR
BOTH	7702.4S160	14700	502	Polk	Des Moines Area	Conc.	S	1986	AT JCT.I 35	OVER I-35

ECR	BRIDGE ID	ADT	TYPE	COUNTY	MAIT. DIV.	C/S	REG	BUILT	LOCATION	CROSSED
BOTH	5435.5S149	1850	502	Keokuk	Fairfield/ Washington	Conc.	S	1987	1.3 MI.N.OF SOUTH ENGLISH	SO.FORK ENGLISH RIVER
BOTH	5713.7L013	6300	201	Linn	Cedar Rapids Area	Conc.	N	1987	6.0 MI.N.OF N.JCT.151	EAST INDIAN CREEK
BOTH	6403.6L014	6700	423	Marshall	Ames Area	Steel	N	1987	IN MARSHALLTOWN	IOWA RIVER
BOTH	8609.2S030	4230	502	Tama	Ames Area	Conc.	N	1987	4.8 MI.E.OF TAMA	OTTER CREEK
BOTH	9245.7S022	1930	201	Washington	Fairfield/ Washington	Conc.	S	1987	0.7 MI.W.OF JOHNSON CO.	IOWA RIVER OVERFLOW
BOTH	2468.5S141	1240	502	Crawford	Sioux City Area	Conc.	N	1988	1.3 MI.N.OF SOUTH ENGLISH	SO.FORK ENGLISH RIVER
BOTH	2504.7S169	3060	502	Dallas	Des Moines Area	Conc.	S	1988	1.0 MI. N. OF JCT. 141	BEAVER CREEK
BOTH	3372.6S018	2380	502	Fayette	Waterloo Area	Conc.	N	1988	IN CLERMONT	TURKEY RIVER
BOTH	4323.4S030	4510	502	Harrison	Council Bluffs	Conc.	S	1988	0.4 MI.E. OF JCT.44	SIX MILE CREEK
BOTH	4751.0S020	1990	502	Ida*	Sioux City Area	Conc.	N	1988	4.1 MI.E. OF JCT. 59	MAPLE RIVER
BOTH	5803.0S070	2160	502	Louisa	Fairfield/ Washington	Conc.	S	1989	0.9 MI N OF JCT IOWA #92	IOWA RIVER
BOTH	7239.2S009	1000	201	Osceola	Storm Lake	Conc.	N	1989	3 MI W. OF #60	OTTER CREEK
BOTH	8433.0S075	4750	502	Sioux	Storm Lake	Conc.	N	1989	0.1 MI N. IOWA #10	W FORK FLOYD RIVER
BOTH	8600.5S008	2440	502	Tama	Ames Area	Conc.	N	1989	IN TRAER	COON CREEK
BOTH	8920.5S016	970	502	Van Buren	Fairfield/ Washington	Conc.	S	1989	1.7 MI W. W OF JCT IA. #1	LITTLE LICK CREEK
BOTH	0937.1S003	2110	201	Bremer	Waterloo Area	Conc.	N	1990	4.4 MI. W. FAYETTE CO.	WAPSIPINICON OVERFL.
BOTH	6206.4S102	1600	502	Mahaska	Chariton/Ottumwa	Conc.	S	1990	4.6 MI. E. MARION CO.	SOUTH SKUNK RIVER
BOTH	6303.1S156	1170	502	Marion	Chariton/Ottumwa	Conc.	S	1990	3.1 MI. E. OF IOWA #5	CEDAR CREEK
BOTH	9424.1L020	3330	302	Webster	Ames Area	Steel	N	1990	2.6 MI.E. OF JCT. US #169	DES MOINES RIVER
BOTH	9424.1R020	3330	302	Webster	Ames Area	Steel	N	1990	2.6 MI.E.OF JCT. US #169	DES MOINES RIVER
BOTH	0781.1R218	19300	502	Black Hawk	Waterloo Area	Conc.	N	1991	IN WATERLOO	5TH,4TH & W.PARK
BOTH	0781.5L218	14700	402	Black Hawk	Waterloo Area	Steel	N	1991		OVER NB US 63
BOTH	5926.7S065	1750	502	Lucas	Chariton/Ottumwa	Conc.	S	1991	1.6 MI. N. OF JCT. 306	HAMILTON CREEK
BOTH	5930.9S065	1750	502	Lucas	Chariton/Ottumwa	Conc.	S	1991	1.1 MI. S. OF JCT. US #34	WHITE BREAST CREEK
BOTH	8554.2L030	9500	201	Story	Ames Area	Conc.	N	1991	2.8 MI. E. OF JCT. I-35	GRANT CREEK

ECR	BRIDGE ID	ADT	TYPE	COUNTY	MAIT. DIV.	C/S	REG	BUILT	LOCATION	CROSSED
BOTH	3364.6S150	1740	201	Fayette	Waterloo Area	Conc.	N	1992	IN MAYNARD	LITTLE VOLGA RIVER
BOTH	5001.9S224	2990	502	Jasper	Des Moines Area	Conc.	S	1992	IN KELLOGG	NORTH SKUNK RIVER
BOTH	5704.2S001	4410	502	Linn	Cedar Rapids Area	Conc.	N	1992	2.2 MI N OF JOHNSON	CEDAR RIVER
BOTH	5931.7S065	1750	502	Lucas	Chariton/Ottumwa	Conc.	S	1992	0.3 MI. S. OF JCT. US #34	WHITE BREAST CREEK
BOTH	8441.3S018	4020	502	Sioux	Storm Lake	Conc.	N	1992	0.4 MI W OF O'BRIEN CO.	FLOYD RIVER
BOTH	0709.3R058	8300	402	Black Hawk	Waterloo Area	Steel	N	1993	IN CEDAR FALLS	S. MAIN ST.
BOTH	0996.0L218	7300	502	Bremer	Waterloo Area	Conc.	N	1993	1.1 MI N OF BLACKHAWK CO	CEDAR RIVER
BOTH	3712.3S004	1230	502	Greene	Ames Area	Conc.	N	1993	0.5 MI N OF GUTHRIE CO.	GREENBIAR CREEK
BOTH	7707.2S415	3010	502	Polk	Des Moines Area	Conc.	S	1993	1.8 MI.N.OF IOWA #160	ROCK CREEK
BOTH	7783.1L065	4290	502	Polk	Des Moines Area	Conc.	S	1993	1.0 MI. S. OF JCT. I-80	US#6

Notes: Shaded areas represent the bridges sampled in phase I. Types: 201, continuous concrete slab; 282, continuous concrete culvert no fill on the top; 302, steel stringer multiple beam or girder; 402, continuous steel stringer multiple beam or girder; 423, steel continuous welded I girder with diaphragms; 502, prestressed concrete multiple beam.

TABLE B.2 Selection of Bridges with Two-Course Placement Deck

BRIDGE ID	ADT	FHWA	COUNTY	Maint. Div.	C/S	REG	BUILT	LOCATION	CROSSED
2401.1S039	N.A	021521	Crawford	Sioux City Area	Conc.	N	1977	1.1 MI. N. OF JCT. #59	BUFFALO CREEK
3966.4S044	N.A.	026191	Guthrin	Atlantic/Creston Area	Conc.	S	1977	IN GUTHRIE CENTER	RACCOON RIVER
4039.6R020	N.A.	603680	Hamilton	Ames Area	Conc.	N	1976	0.8 MI. W. OF JCT. 17	CO RD R27

TABLE B.3 Selection of Tama County Bridges

BRIDGE ID	ADT	FHWA	COUNTY	Maint. Div.	C/S	REG	BUILT	LOCATION	CROSSED
TAMA 1	N.A.	316580	Tama	Tama County	Steel	N	1968	0821317	Iowa River Overflow
TAMA 2	N.A.	316610	Tama	Tama County	Steel	N	1968	0821320	Iowa River
TAMA 3	N.A.	316600	Tama	Tama County	Conc.	N	1968	0821320	Iowa River Overflow

APPENDIX C: CHLORIDE CONCENTRATION OF CRACKED CORES AT DIFFERENT DEPTHS

TABLE C.1 Chloride Concentration of Cracked Cores

Bridge ID	Core	Year	Age	Cracked	Depth (in)	Cl.(%)	Clx.(lb/yd^3)
0475.2S002	A	1983	15	Y	1.10	0.350	14.17
0475.2S002	A	1983	15	Y	2.30	0.191	7.73
0475.2S002	A	1983	15	Y	3.40	0.188	7.61
0475.2S002	B	1983	15	Y	1.30	0.237	9.59
0475.2S002	B	1983	15	Y	2.60	0.122	4.94
0475.2S002	B	1983	15	Y	3.90	0.048	1.94
0476.4S002	A	1978	20	Y	1.40	0.453	18.34
0476.4S002	A	1978	20	Y	2.80	0.244	9.88
0476.4S002	A	1978	20	Y	4.20	0.116	4.70
0643.5R380	A	1982	16	Y	1.20	0.174	7.04
0643.5R380	A	1982	16	Y	2.40	0.072	2.91
0643.5R380	A	1982	16	Y	3.60	0.044	1.78
0643.5R380	B	1982	16	Y	1.30	0.274	11.09
0643.5R380	B	1982	16	Y	2.70	0.183	7.41
0643.5R380	B	1982	16	Y	4.00	0.138	5.59
0727.5R020	A	1983	15	Y	1.10	0.240	9.71
0727.5R020	A	1983	15	Y	2.20	0.272	11.01
0727.5R020	A	1983	15	Y	3.50	0.242	9.80
0727.5R020	B	1983	15	Y	1.10	0.120	4.86
0727.5R020	B	1983	15	Y	2.20	0.065	2.63
0727.5R020	B	1983	15	Y	3.30	0.071	2.87
0757.1L380	A	1984	14	Y	1.00	0.280	11.33
0757.1L380	A	1984	14	Y	2.00	0.226	9.15
0757.1L380	A	1984	14	Y	3.00	0.115	4.66
0757.1L380	B	1984	14	Y	1.00	0.153	6.19
0757.1L380	B	1984	14	Y	2.00	0.026	1.05
0757.1L380	B	1984	14	Y	3.00	0.023	0.93
0761.5O380	B	1984	14	Y	1.10	0.314	12.71
0761.5O380	B	1984	14	Y	2.20	0.239	9.67
0761.5O380	B	1984	14	Y	3.40	0.193	7.81
0937.1S003	A	1990	8	Y	0.90	0.544	22.02
0937.1S003	A	1990	8	Y	1.80	0.309	12.51
0937.1S003	A	1990	8	Y	2.70	0.097	3.93
0937.1S003	B	1990	8	Y	1.00	0.371	15.03
0937.1S003	B	1990	8	Y	2.00	0.146	5.92
0937.1S003	B	1990	8	Y	3.00	0.059	2.41

Bridge ID	Core	Year	Age	Cracked	Depth (in)	Cl.(%)	Clx.(lb/yd^3)
1253.3S014	A	1984	14	Y	1.40	0.440	17.82
1253.3S014	A	1984	14	Y	2.80	0.242	9.80
1253.3S014	A	1984	14	Y	4.20	0.164	6.65
1390.7S175	A	1980	18	Y	1.30	0.291	11.78
1390.7S175	A	1980	18	Y	2.60	0.255	10.31
1390.7S175	A	1980	18	Y	3.90	0.200	8.10
1390.7S175	D	1980	18	Y	1.00	0.256	10.37
1390.7S175	D	1980	18	Y	2.00	0.134	5.42
1390.7S175	D	1980	18	Y	3.00	0.063	2.55
1479.8S030	A	1982	16	Y	1.00	0.346	14.02
1479.8S030	A	1982	16	Y	2.00	0.221	8.94
1479.8S030	A	1982	16	Y	3.00	0.051	2.08
1479.8S030	D	1982	16	Y	1.30	0.448	18.13
1479.8S030	D	1982	16	Y	2.60	0.249	10.08
1479.8S030	D	1982	16	Y	3.90	0.095	3.85
1910.0S346	A	1984	14	Y	1.10	0.644	26.07
1910.0S346	A	1984	14	Y	2.20	0.350	14.17
1910.0S346	A	1984	14	Y	3.30	0.272	11.01
1910.0S346	B	1984	14	Y	1.30	0.475	19.23
1910.0S346	B	1984	14	Y	2.60	0.287	11.62
1910.0S346	B	1984	14	Y	3.90	0.220	8.91
2336.2O061	A	1984	14	Y	1.20	0.014	0.57
2336.2O061	A	1984	14	Y	2.50	0.015	0.61
2336.2O061	A	1984	14	Y	3.70	0.014	0.57
2336.2O061	B	1984	14	Y	1.10	0.021	0.85
2336.2O061	B	1984	14	Y	2.20	0.015	0.61
2336.2O061	B	1984	14	Y	3.30	0.013	0.53
2468.5S141	C	1988	10	Y	1.00	0.346	14.00
2468.5S141	C	1988	10	Y	2.00	0.135	5.47
2468.5S141	C	1988	10	Y	3.00	0.104	4.22
2579.9S004	A	1981	17	Y	1.00	0.482	19.51
2579.9S004	A	1981	17	Y	2.00	0.288	11.65
2579.9S004	A	1981	17	Y	3.00	0.258	10.44
2579.9S004	B	1981	17	Y	1.20	0.358	14.48
2579.9S004	B	1981	17	Y	2.40	0.270	10.93
2579.9S004	B	1981	17	Y	3.50	0.119	4.82
3364.6S150	A	1992	6	Y	1.50	0.315	12.75
3364.6S150	A	1992	6	Y	3.00	0.187	7.57
3364.6S150	A	1992	6	Y	4.60	0.063	2.55
3364.6S150	B	1992	6	Y	1.60	0.335	13.56
3364.6S150	B	1992	6	Y	3.20	0.335	13.56
3364.6S150	B	1992	6	Y	4.80	0.195	7.89

Bridge ID	Core	Year	Age	Cracked	Depth (in)	Cl.(%)	Clx.(lb/yd^3)
3975.9S044	A	1981	17	Y	1.10	0.214	8.66
3975.9S044	A	1981	17	Y	2.20	0.191	7.73
3975.9S044	A	1981	17	Y	3.30	0.108	4.37
3975.9S044	B	1981	17	Y	1.30	0.312	12.62
3975.9S044	B	1981	17	Y	2.60	0.171	6.90
3975.9S044	B	1981	17	Y	3.90	0.160	6.48
3988.5S025	D	1980	18	Y	1.30	0.799	32.34
3988.5S025	D	1980	18	Y	2.60	0.496	20.10
3988.5S025	D	1980	18	Y	3.50	0.480	19.43
4801.5S220	A	1978	20	Y	1.10	0.493	19.96
4801.5S220	A	1978	20	Y	2.20	0.134	5.44
4801.5S220	A	1978	20	Y	3.30	0.020	0.81
4801.5S220	B	1978	20	Y	1.50	0.385	15.59
4801.5S220	B	1978	20	Y	3.00	0.222	8.98
4801.5S220	B	1978	20	Y	4.50	0.057	2.31
4926.7S052	A	1986	12	Y	1.00	0.357	14.45
4926.7S052	A	1986	12	Y	2.00	0.245	9.92
4926.7S052	A	1986	12	Y	3.00	0.242	9.80
4926.7S052	B	1986	12	Y	1.30	0.320	12.95
4926.7S052	B	1986	12	Y	3.00	0.166	6.72
4926.7S052	B	1986	12	Y	4.60	0.084	3.40
5293.7L218	B	1983	15	Y	1.40	0.242	9.78
5293.7L218	B	1983	15	Y	2.80	0.063	2.53
5293.7L218	B	1983	15	Y	4.20	0.027	1.08
5298.6S001	A	1985	13	Y	0.90	0.371	15.03
5298.6S001	A	1985	13	Y	1.80	0.275	11.14
5298.6S001	A	1985	13	Y	2.70	0.234	9.47
5298.6S001	B	1985	13	Y	1.50	0.376	15.23
5298.6S001	B	1985	13	Y	3.00	0.218	8.82
5298.6S001	B	1985	13	Y	4.50	0.094	3.81
5721.6R380	A	1980	18	Y	1.10	0.291	11.79
5721.6R380	A	1980	18	Y	2.20	0.151	6.13
5721.6R380	A	1980	18	Y	3.30	0.119	4.84
5721.6R380	C	1980	18	Y	1.10	0.355	14.36
5721.6R380	C	1980	18	Y	2.20	0.245	9.90
5721.6R380	C	1980	18	Y	3.30	0.188	7.60
5722.4R380	A	1980	18	Y	1.30	0.158	6.38
5722.4R380	A	1980	18	Y	2.60	0.098	3.96
5722.4R380	A	1980	18	Y	3.90	0.082	3.31
5722.4R380	B	1980	18	Y	1.30	0.304	12.31
5722.4R380	B	1980	18	Y	2.60	0.232	9.37
5722.4R380	B	1980	18	Y	3.90	0.188	7.60

Bridge ID	Core	Year	Age	Cracked	Depth (in)	Cl.(%)	Clx.(lb/yd^3)
5722.7O380	A	1980	18	Y	1.00	0.291	11.80
5722.7O380	A	1980	18	Y	2.00	0.175	7.08
5722.7O380	A	1980	18	Y	3.00	0.087	3.51
5722.7O380	B	1980	18	Y	1.10	0.484	19.57
5722.7O380	B	1980	18	Y	2.20	0.305	12.34
5722.7O380	B	1980	18	Y	3.30	0.207	8.36
5738.1L380	A	1982	16	Y	0.90	0.193	7.81
5738.1L380	A	1982	16	Y	1.80	0.055	2.23
5738.1L380	A	1982	16	Y	2.70	0.024	0.98
5738.1L380	B	1982	16	Y	1.20	0.199	8.07
5738.1L380	B	1982	16	Y	2.40	0.141	5.70
5738.1L380	B	1982	16	Y	3.60	0.122	4.92
5930.9S065	A	1991	7	Y	1.50	0.207	8.38
5930.9S065	A	1991	7	Y	3.00	0.120	4.86
5930.9S065	A	1991	7	Y	4.50	0.057	2.31
5930.9S065	B	1991	7	Y	1.60	0.163	6.60
5930.9S065	B	1991	7	Y	3.20	0.037	1.50
5930.9S065	B	1991	7	Y	4.80	0.021	0.85
6011.6S009	B	1976	22	Y	1.00	0.280	11.33
6011.6S009	B	1976	22	Y	2.00	0.197	7.97
6011.6S009	B	1976	22	Y	3.10	0.152	6.15
6011.6S009	C	1976	22	Y	0.90	0.334	13.52
6011.6S009	C	1976	22	Y	1.80	0.270	10.93
6011.6S009	C	1976	22	Y	2.70	0.114	4.61
6348.5S005	B	1983	15	Y	1.10	0.206	8.36
6348.5S005	B	1983	15	Y	2.20	0.093	3.77
6348.5S005	B	1983	15	Y	3.30	0.068	2.75
6348.5S005	C	1983	15	Y	1.00	0.220	8.90
6348.5S005	C	1983	15	Y	2.00	0.123	4.99
6348.5S005	C	1983	15	Y	3.00	0.117	4.73
6360.4S005	A	1978	20	Y	1.40	0.199	8.05
6360.4S005	A	1978	20	Y	2.80	0.064	2.60
6360.4S005	A	1978	20	Y	4.20	0.034	1.38
6360.4S005	C	1978	20	Y	1.20	0.311	12.57
6360.4S005	C	1978	20	Y	2.40	0.275	11.14
6360.4S005	C	1978	20	Y	3.60	0.217	8.79
7526.9S003	B	1981	17	Y	1.30	0.485	19.63
7526.9S003	B	1981	17	Y	2.60	0.430	17.41
7526.9S003	B	1981	17	Y	3.90	0.354	14.33
7526.9S003	C	1981	17	Y	0.90	0.263	10.65
7526.9S003	C	1981	17	Y	1.80	0.241	9.76
7526.9S003	C	1981	17	Y	2.80	0.092	3.72

Bridge ID	Core	Year	Age	Cracked	Depth (in)	Cl.(%)	Clx.(lb/yd^3)
7993.4S063	A	1985	13	Y	1.40	0.351	14.21
7993.4S063	A	1985	13	Y	2.80	0.145	5.85
7993.4S063	A	1985	13	Y	4.20	0.091	3.67
7993.4S063	D	1985	13	Y	1.10	0.421	17.05
7993.4S063	D	1985	13	Y	2.20	0.198	8.00
7993.4S063	D	1985	13	Y	3.30	0.055	2.23
8224.1R061	A	1981	17	Y	1.00	0.331	13.40
8224.1R061	A	1981	17	Y	2.00	0.197	7.97
8224.1R061	A	1981	17	Y	3.00	0.157	6.36
8224.1R061	B	1981	17	Y	1.30	0.232	9.39
8224.1R061	B	1981	17	Y	2.70	0.140	5.67
8224.1R061	B	1981	17	Y	4.00	0.098	3.97
8441.3S018	B	1992	6	Y	1.40	0.436	17.65
8441.3S018	B	1992	6	Y	2.80	0.161	6.52
8441.3S018	B	1992	6	Y	4.20	0.090	3.64
8441.3S018	C	1992	6	Y	0.90	0.401	16.23
8441.3S018	C	1992	6	Y	1.80	0.172	6.96
8441.3S018	C	1992	6	Y	2.70	0.086	3.48
9424.1L020	A	1990	8	Y	1.40	0.223	9.03
9424.1L020	A	1990	8	Y	2.80	0.163	6.60
9424.1L020	A	1990	8	Y	4.20	0.152	6.17
9424.1L020	C	1990	8	Y	1.40	0.453	18.33
9424.1L020	C	1990	8	Y	2.80	0.394	15.96
9424.1L020	C	1990	8	Y	4.20	0.357	14.45
9700.8S982	A	1978	20	Y	1.20	0.515	20.85
9700.8S982	A	1978	20	Y	2.50	0.347	14.05
9700.8S982	A	1978	20	Y	3.80	0.333	13.48
9700.8S982	B	1978	20	Y	1.10	0.406	16.43
9700.8S982	B	1978	20	Y	2.20	0.149	6.03
9700.8S982	B	1978	20	Y	3.30	0.061	2.47

APPENDIX D: MATLAB PROGRAMMING CODES FOR CALCULATING SURFACE CHLORIDE CONTENT AND DIFFUSION CONSTANT

```
%The following is the source code utilized in Matlab to compute Dc
%values for each core at which Co was a known value through field
%measurement. Three chloride concentration measurements were taken
%along the core.

%File name diffcons_ph2_3.m
format short
clear
close

%open data file
fid1 = fopen('xph2_n_3.dat','r');         %depth
fid2 = fopen('Cxph2_n_3.dat','r');        %chloride concentration
fid3 = fopen('ageph2_n_3.dat', 'r');
    %age
fid4 = fopen('Coph2_n_3.dat', 'r');       %Co measurements

%Read data file as input data
x=fscanf(fid1,'%g')’                      %depth
Cx=fscanf(fid2,'%g');                     %chloride concentration
t=fscanf(fid3,'%g');                      %age
Co=fscanf(fid4,'%g');                     %Co measurements

%Calculate best value for D
N=61;
SSE=[];
A=[];

D=linspace(0.01,0.2,N);
for j=1:10
 for i=2*j-1:j*2
  for k=1 : N
   Z=Co(j)*(1-erf((x(i)-0.5)/(2*sqrt(D(k)*t(i)))));
   ERR(k,1,i)=(Z-Cx(i))^2;
  end
 end
 SSE(:,:,j)=ERR(:,:,i-1)+ERR(:,:,i);
 w(j)=min(min(SSE(:,:,j)));
 [e(j),f(j)]=find(SSE(:,:,j) == min(min(SSE(:,:,j))));
 D(e(j));
 A(j,1)=D(e(j));
 A(j,2)=w(j);
End

%Output data
A
m=mean(A)
s=std(A)
t=cputime
status=fclose('all');
```

```
%The following is the source code utilized in Matlab to compute Dc
%values for each core at which Co was a known value through field
%measurement. Three chloride concentration measurements were taken
%along the core.

%File name diffcons_ph1_n.m

clear
close
format short

%Open data files
fid1 = fopen('xph1_n.dat','r');    %depth
fid2 = fopen('Cxph1_n.dat','r');   %chloride concentration
fid3 = fopen('ageph1_n.dat', 'r'); %age

%Read data files as input data for calculation
x=fscanf(fid1,'%g');               %depth
Cx=fscanf(fid2,'%g');              %chloride concentration
t=fscanf(fid3,'%g');               %age

%Compute Co and D
N=61;
SSE=[];
A=[];
Co=linspace(5,35,N);
D=linspace(0.01,0.2,N);
for j=1:49
 for i=j+2*(j-1):j*3
  for k=1 : N
   for n=1 : N
   Z=Co(k)*(1-erf((x(i)-0.5)/(2*sqrt(D(n)*t(i)))));
   ERR(k,n,i)=(Z-Cx(i))^2;
   end
  end
 end
 SSE(:,:,j)=ERR(:,:,i-2)+ERR(:,:,i-1)+ERR(:,:,i);
 w(j)=min(min(SSE(:,:,j)));
 [e(j),f(j)]=find(SSE(:,:,j) == min(min(SSE(:,:,j))));
 Co(e(j));
 D(f(j));
 A(j,1)=Co(e(j));
 A(j,2)=D(f(j));
 A(j,3)=w(j);
End

%Output data
A
m=mean(A)
s=std(A)
t=cputime
status=fclose('all');
```

```
%The following is the source code utilized in Matlab to compute Dc
%values for each core at which Co was a known value through field
%measurement. Four chloride concentration measurements were taken
%along the core.

%File name diffcons_ph2_4.m
format short
clear
close

%Open data files
fid1 = fopen('xph2_n_4.dat','r');     %depth
fid2 = fopen('Cxph2_n_4.dat','r');    %chloride concentration
fid3 = fopen('ageph2_n_4.dat', 'r'); %age
fid4 = fopen('Coph2_n_4.dat', 'r');  %Co measurements

%Read data files as input data
x=fscanf(fid1,'%g');                 %depth
Cx=fscanf(fid2,'%g');                %chloride concentration
t=fscanf(fid3,'%g');                 %age
Co=fscanf(fid4,'%g');                %Co measurement


%Calculate D best values
N=61;
SSE=[];
A=[];
D=linspace(0.01,0.2,N);
for j=1:26
 for i=j+2*(j-1):j*3
  for k=1 : N
   Z=Co(j)*(1-erf((x(i)-0.5)/(2*sqrt(D(k)*t(i)))));
   ERR(k,1,i)=(Z-Cx(i))^2;
  end
 end
 SSE(:,:,j)=ERR(:,:,i-2)+ERR(:,:,i-1)+ERR(:,:,i);
 w(j)=min(min(SSE(:,:,j)));
 [e(j),f(j)]=find(SSE(:,:,j) == min(min(SSE(:,:,j))));
 D(e(j));
 A(j,1)=D(e(j));
 A(j,2)=w(j);
end

A
m=mean(A)
s=std(A)
t=cputime
status=fclose('all');
```

APPENDIX E: COMPUTED DIFFUSION CONSTANT AND SURFACE CHLORIDE CONCENTRATION FOR BRIDGE DECKS INCLUDED IN THE STUDY

TABLE E.1 Computed Diffusion Constant and Surface Chloride Concentration

Bridge ID	Core	Year	Age	Crack	Depth(in)	Cl.(%)	Clx.(lb/yd^3)	D_c(in^2/yr)	C_o(lb/cy^3)
0475.2S002	C	1983	15	N	1.20	0.197	7.97	0.035	16.0
0475.2S002	C	1983	15	N	2.40	0.021	0.85	0.035	16.0
0475.2S002	C	1983	15	N	3.60	0.019	0.77	0.035	16.0
0475.2S002	D	1983	15	N	0.90	0.483	19.55	0.032	28.5
0475.2S002	D	1983	15	N	1.90	0.101	4.09	0.032	28.5
0475.2S002	D	1983	15	N	2.80	0.017	0.69	0.032	28.5
0476.4S002	C	1978	20	N	1.00	0.165	6.68	0.029	10.5
0476.4S002	C	1978	20	N	2.50	0.015	0.61	0.029	10.5
0476.4S002	C	1978	20	N	4.10	0.014	0.57	0.029	10.5
0643.5R380	C	1982	16	N	1.30	0.205	8.30	0.035	18.5
0643.5R380	C	1982	16	N	2.60	0.021	0.85	0.035	18.5
0643.5R380	C	1982	16	N	3.90	0.013	0.53	0.035	18.5
0643.5R380	D	1982	16	N	1.20	0.224	9.07	0.026	20.5
0643.5R380	D	1982	16	N	2.40	0.019	0.77	0.026	20.5
0643.5R380	D	1982	16	N	3.60	0.014	0.57	0.026	20.5
0668.7S021	A	1979	19	N	0.50	0.291	11.78	0.029	11.8
0668.7S021	A	1979	19	N	1.75	0.069	2.79	0.029	11.8
0668.7S021	A	1979	19	N	2.25	0.027	1.09	0.029	11.8
0668.7S021	B	1979	19	N	0.50	0.351	14.21	0.013	14.2
0668.7S021	B	1979	19	N	1.55	0.036	1.46	0.013	14.2
0668.7S021	B	1979	19	N	2.75	0.017	0.69	0.013	14.2
0727.5R020	C	1983	15	N	1.20	0.053	2.15	0.029	5.0
0727.5R020	C	1983	15	N	2.40	0.013	0.53	0.029	5.0
0727.5R020	C	1983	15	N	3.70	0.014	0.57	0.029	5.0
0727.5R020	D	1983	15	N	1.20	0.019	0.77	0.010	5.0
0727.5R020	D	1983	15	N	2.40	0.017	0.69	0.010	5.0
0727.5R020	D	1983	15	N	3.60	0.017	0.69	0.010	5.0
0757.1L380	D	1984	14	N	1.20	0.206	8.34	0.039	16.5
0757.1L380	D	1984	14	N	2.30	0.029	1.17	0.039	16.5
0757.1L380	D	1984	14	N	3.50	0.024	0.97	0.039	16.5
0761.5O380	C	1984	14	N	1.20	0.069	2.79	0.054	5.0
0761.5O380	C	1984	14	N	2.50	0.013	0.53	0.054	5.0
0761.5O380	C	1984	14	N	3.80	0.012	0.49	0.054	5.0
0761.5O380	D	1984	14	N	1.30	0.116	4.70	0.039	10.5
0761.5O380	D	1984	14	N	2.60	0.011	0.45	0.039	10.5
0761.5O380	D	1984	14	N	3.90	0.011	0.45	0.039	10.5
0777.9L218	A	1983	15	N	0.50	0.542	21.94	0.026	21.9
0777.9L218	A	1983	15	N	1.50	0.144	5.83	0.026	21.9
0777.9L218	A	1983	15	N	2.75	0.015	0.61	0.026	21.9
0777.9L218	B	1983	15	N	0.50	0.645	26.11	0.048	26.1
0777.9L218	B	1983	15	N	1.75	0.193	7.81	0.048	26.1
0777.9L218	B	1983	15	N	3.00	0.016	0.65	0.048	26.1

Bridge ID	Core	Year	Age	Crack	Depth(in)	Cl.(%)	Clx.(lb/cy^3)	D_c(in^2/yr)	C_0(lb/cy^3)
0781.5L218	A	1991	7	N	0.50	0.556	22.51	0.020	22.5
0781.5L218	A	1991	7	N	1.50	0.036	1.46	0.020	22.5
0781.5L218	A	1991	7	N	2.50	0.022	0.89	0.020	22.5
0781.5L218	A	1991	7	N	4.00	0.020	0.81	0.020	22.5
0781.5L218	B	1991	7	N	0.50	0.395	15.99	0.023	16.0
0781.5L218	B	1991	7	N	1.50	0.029	1.17	0.023	16.0
0781.5L218	B	1991	7	N	2.50	0.019	0.77	0.023	16.0
0781.5L218	B	1991	7	N	4.00	0.017	0.69	0.023	16.0
0996.0L218	B	1993	5	N	1.00	0.282	11.42	0.200	16.0
0996.0L218	B	1993	5	N	2.00	0.097	3.93	0.200	16.0
0996.0L218	B	1993	5	N	3.00	0.091	3.70	0.200	16.0
0996.0L218	C	1993	5	N	1.10	0.167	6.76	0.200	11.0
0996.0L218	C	1993	5	N	2.20	0.097	3.93	0.200	11.0
0996.0L218	C	1993	5	N	3.30	0.063	2.54	0.200	11.0
1253.3S014	C	1984	14	N	0.90	0.230	9.29	0.061	12.0
1253.3S014	C	1984	14	N	1.80	0.077	3.10	0.061	12.0
1253.3S014	C	1984	14	N	2.70	0.052	2.09	0.061	12.0
1253.3S014	D	1984	14	N	1.30	0.236	9.56	0.115	14.5
1253.3S014	D	1984	14	N	2.60	0.082	3.32	0.115	14.5
1253.3S014	D	1984	14	N	3.90	0.029	1.16	0.115	14.5
1390.7S175	C	1980	18	N	0.80	0.152	6.16	0.026	8.0
1390.7S175	C	1980	18	N	1.60	0.044	1.79	0.026	8.0
1390.7S175	C	1980	18	N	3.10	0.046	1.85	0.026	8.0
1410.2S071	A	1979	19	N	0.50	0.166	6.72	0.016	6.7
1410.2S071	A	1979	19	N	1.50	0.030	1.21	0.016	6.7
1410.2S071	A	1979	19	N	2.50	0.024	0.97	0.016	6.7
1410.2S071	A	1979	19	N	3.50	0.038	1.54	0.016	6.7
1410.2S071	B	1979	19	N	0.50	0.313	12.67	0.092	12.7
1410.2S071	B	1979	19	N	1.50	0.150	6.07	0.092	12.7
1410.2S071	B	1979	19	N	2.50	0.083	3.36	0.092	12.7
1410.2S071	B	1979	19	N	3.50	0.082	3.32	0.092	12.7
1411.6S071	A	1985	13	N	0.50	0.465	18.82	0.077	18.8
1411.6S071	A	1985	13	N	1.75	0.155	6.27	0.077	18.8
1411.6S071	A	1985	13	N	3.00	0.068	2.75	0.077	18.8
1411.6S071	B	1985	13	N	0.50	0.521	21.09	0.070	21.1
1411.6S071	B	1985	13	N	2.50	0.064	2.59	0.070	21.1
1411.6S071	B	1985	13	N	3.72	0.043	1.74	0.070	21.1
1479.8S030	B	1982	16	N	1.00	0.304	12.31	0.029	20.5
1479.8S030	B	1982	16	N	2.00	0.051	2.08	0.029	20.5
1479.8S030	B	1982	16	N	3.00	0.034	1.36	0.029	20.5
1479.8S030	C	1982	16	N	1.00	0.321	12.98	0.029	21.5
1479.8S030	C	1982	16	N	2.00	0.065	2.63	0.029	21.5
1479.8S030	C	1982	16	N	3.10	0.014	0.58	0.029	21.5

Bridge ID	Core	Year	Age	Crack	Depth(in)	Cl.(%)	Clx.(lb/cy^3)	D_c(in^2/yr)	C_0(lb/cy^3)
1910.0S346	C	1984	14	N	1.30	0.723	29.27	0.200	35.0
1910.0S346	C	1984	14	N	2.60	0.349	14.13	0.200	35.0
1910.0S346	C	1984	14	N	3.90	0.048	1.94	0.200	35.0
1910.0S346	D	1984	14	N	1.20	0.547	22.14	0.105	32.5
1910.0S346	D	1984	14	N	2.40	0.223	9.03	0.105	32.5
1910.0S346	D	1984	14	N	3.70	0.032	1.30	0.105	32.5
2336.2O061	C	1984	14	N	1.30	0.013	0.53	0.010	5.0
2336.2O061	C	1984	14	N	2.70	0.012	0.49	0.010	5.0
2336.2O061	C	1984	14	N	4.00	0.014	0.57	0.010	5.0
2336.2O061	D	1984	14	N	1.30	0.014	0.57	0.010	5.0
2336.2O061	D	1984	14	N	2.60	0.010	0.40	0.010	5.0
2336.2O061	D	1984	14	N	3.90	0.012	0.49	0.010	5.0
2468.5S141	B	1988	10	N	0.80	0.308	12.46	0.026	18.5
2468.5S141	B	1988	10	N	1.60	0.051	2.06	0.026	18.5
2468.5S141	B	1988	10	N	2.40	0.027	1.10	0.026	18.5
2468.5S141	D	1988	10	N	1.00	0.368	14.91	0.029	29.0
2468.5S141	D	1988	10	N	2.00	0.031	1.25	0.029	29.0
2468.5S141	D	1988	10	N	3.00	0.025	1.01	0.029	29.0
2504.7S169	A	1988	10	N	0.50	0.658	26.64	0.070	26.6
2504.7S169	A	1988	10	N	1.50	0.275	11.13	0.070	26.6
2504.7S169	A	1988	10	N	2.63	0.033	1.34	0.070	26.6
2504.7S169	A	1988	10	N	4.00	0.014	0.57	0.070	26.6
2504.7S169	B	1988	10	N	0.50	0.771	31.21	0.070	31.2
2504.7S169	B	1988	10	N	1.50	0.330	13.36	0.070	31.2
2504.7S169	B	1988	10	N	2.50	0.033	1.34	0.070	31.2
2504.7S169	B	1988	10	N	4.00	0.023	0.93	0.070	31.2
3236.8S004	A	1981	17	N	0.50	0.505	20.44	0.051	20.4
3236.8S004	A	1981	17	N	1.50	0.255	10.32	0.051	20.4
3236.8S004	A	1981	17	N	2.25	0.051	2.06	0.051	20.4
3236.8S004	A	1981	17	N	3.50	0.037	1.50	0.051	20.4
3236.8S004	B	1981	17	N	0.50	0.389	15.75	0.016	15.7
3236.8S004	B	1981	17	N	1.75	0.033	1.34	0.016	15.7
3236.8S004	B	1981	17	N	3.13	0.034	1.38	0.016	15.7
3236.8S004	B	1981	17	N	4.25	0.038	1.54	0.016	15.7
3975.9S044	C	1981	17	N	1.30	0.124	5.01	0.042	10.0
3975.9S044	C	1981	17	N	2.60	0.019	0.75	0.042	10.0
3975.9S044	C	1981	17	N	3.90	0.009	0.38	0.042	10.0
3975.9S044	D	1981	17	N	1.70	0.141	5.71	0.137	10.0
3975.9S044	D	1981	17	N	3.50	0.052	2.10	0.137	10.0
3975.9S044	D	1981	17	N	4.10	0.012	0.48	0.137	10.0
3988.5S025	A	1980	18	N	0.90	0.695	28.15	0.032	35.0
3988.5S025	A	1980	18	N	1.80	0.122	4.94	0.032	35.0
3988.5S025	A	1980	18	N	2.70	0.103	4.18	0.032	35.0

Bridge ID	Core	Year	Age	Crack	Depth(in)	Cl.(%)	Clx.(lb/cy^3)	D_c(in^2/yr)	C_o(lb/cy^3)
4323.4S030	A	1988	10	N	0.50	0.122	4.94	0.020	4.9
4323.4S030	A	1988	10	N	1.50	0.013	0.53	0.020	4.9
4323.4S030	A	1988	10	N	2.63	0.014	0.57	0.020	4.9
4323.4S030	A	1988	10	N	3.75	0.015	0.61	0.020	4.9
4323.4S030	B	1988	10	N	0.50	0.118	4.78	0.026	4.8
4323.4S030	B	1988	10	N	1.50	0.018	0.73	0.026	4.8
4323.4S030	B	1988	10	N	2.50	0.022	0.89	0.026	4.8
4323.4S030	B	1988	10	N	3.75	0.018	0.73	0.026	4.8
4801.5S220	C	1978	20	N	1.30	0.112	4.53	0.035	9.0
4801.5S220	C	1978	20	N	2.70	0.013	0.54	0.035	9.0
4801.5S220	C	1978	20	N	4.00	0.017	0.68	0.035	9.0
4801.5S220	D	1978	20	N	1.00	0.343	13.88	0.016	26.0
4801.5S220	D	1978	20	N	2.00	0.036	1.46	0.016	26.0
4801.5S220	D	1978	20	N	3.00	0.014	0.56	0.016	26.0
4926.7S052	C	1986	12	N	1.00	0.269	10.89	0.039	18.0
4926.7S052	C	1986	12	N	2.00	0.048	1.94	0.039	18.0
4926.7S052	C	1986	12	N	2.90	0.013	0.53	0.039	18.0
4926.7S052	D	1986	12	N	1.20	0.164	6.64	0.039	14.0
4926.7S052	D	1986	12	N	2.50	0.013	0.53	0.039	14.0
4926.7S052	D	1986	12	N	3.70	0.014	0.57	0.039	14.0
5298.6S001	C	1985	13	N	1.10	0.474	19.20	0.111	26.5
5298.6S001	C	1985	13	N	2.20	0.215	8.70	0.111	26.5
5298.6S001	C	1985	13	N	3.30	0.053	2.13	0.111	26.5
5298.6S001	D	1985	13	N	1.40	0.345	13.97	0.172	20.5
5298.6S001	D	1985	13	N	2.80	0.124	5.00	0.172	20.5
5298.6S001	D	1985	13	N	4.20	0.063	2.55	0.172	20.5
5419.0S149	A	1986	12	N	0.50	0.391	15.83	0.010	15.8
5419.0S149	A	1986	12	N	1.00	0.086	3.48	0.010	15.8
5419.0S149	A	1986	12	N	2.50	0.012	0.49	0.010	15.8
5419.0S149	A	1986	12	N	4.00	0.019	0.77	0.010	15.8
5419.0S149	B	1986	12	N	0.50	0.432	17.49	0.035	17.5
5419.0S149	B	1986	12	N	1.60	0.098	3.97	0.035	17.5
5419.0S149	B	1986	12	N	2.60	0.019	0.77	0.035	17.5
5419.0S149	B	1986	12	N	4.00	0.020	0.81	0.035	17.5
5704.2S001	A	1992	6	N	0.50	0.586	23.72	0.070	23.7
5704.2S001	A	1992	6	N	1.75	0.101	4.09	0.070	23.7
5704.2S001	A	1992	6	N	2.86	0.020	0.81	0.070	23.7
5704.2S001	B	1992	6	N	0.50	0.592	23.96	0.105	24.0
5704.2S001	B	1992	6	N	1.50	0.226	9.15	0.105	24.0
5704.2S001	B	1992	6	N	2.60	0.024	0.97	0.105	24.0
5704.2S001	B	1992	6	N	4.00	0.026	1.05	0.105	24.0
5713.7L013	A	1987	11	N	0.50	0.369	14.94	0.064	14.9
5713.7L013	A	1987	11	N	1.50	0.139	5.63	0.064	14.9
5713.7L013	A	1987	11	N	3.00	0.034	1.38	0.064	14.9
5713.7L013	B	1987	11	N	0.50	0.531	21.49	0.146	21.5
5713.7L013	B	1987	11	N	2.00	0.226	9.15	0.146	21.5
5713.7L013	B	1987	11	N	3.50	0.033	1.34	0.146	21.5
5713.7L013	B	1987	11	N	5.00	0.014	0.57	0.146	21.5

Bridge ID	Core	Year	Age	Crack	Depth(in)	Cl.(%)	Clx.(lb/cy^3)	D_c(in^2/yr)	C_0(lb/cy^3)
5721.6R380	B	1980	18	N	1.10	0.578	23.38	0.039	35.0
5721.6R380	B	1980	18	N	2.20	0.071	2.87	0.039	35.0
5721.6R380	B	1980	18	N	3.20	0.058	2.35	0.039	35.0
5722.7O380	C	1980	18	N	0.90	0.579	23.46	0.039	32.0
5722.7O380	C	1980	18	N	1.80	0.208	8.40	0.039	32.0
5722.7O380	C	1980	18	N	2.90	0.041	1.65	0.039	32.0
5722.7O380	D	1980	18	N	0.80	0.703	28.45	0.035	35.0
5722.7O380	D	1980	18	N	1.60	0.282	11.40	0.035	35.0
5722.7O380	D	1980	18	N	2.50	0.037	1.51	0.035	35.0
5738.1L380	C	1982	16	N	1.10	0.340	13.76	0.035	24.0
5738.1L380	C	1982	16	N	2.20	0.062	2.50	0.035	24.0
5738.1L380	C	1982	16	N	3.30	0.029	1.19	0.035	24.0
6011.6S009	A	1976	22	N	1.00	0.188	7.61	0.029	11.5
6011.6S009	A	1976	22	N	2.00	0.045	1.82	0.029	11.5
6011.6S009	A	1976	22	N	3.00	0.032	1.30	0.029	11.5
6011.6S009	D	1976	22	N	0.90	0.313	12.67	0.013	21.0
6011.6S009	D	1976	22	N	1.80	0.044	1.78	0.013	21.0
6011.6S009	D	1976	22	N	2.70	0.033	1.34	0.013	21.0
6219.3S137	A	1982	16	N	0.50	0.252	10.20	0.010	10.2
6219.3S137	A	1982	16	N	1.25	0.027	1.09	0.010	10.2
6219.3S137	A	1982	16	N	2.00	0.019	0.77	0.010	10.2
6219.3S137	A	1982	16	N	4.25	0.019	0.77	0.010	10.2
6219.3S137	B	1982	16	N	0.50	0.198	8.01	0.020	8.0
6219.3S137	B	1982	16	N	1.35	0.049	1.98	0.020	8.0
6219.3S137	B	1982	16	N	2.25	0.021	0.85	0.020	8.0
6219.3S137	B	1982	16	N	4.25	0.020	0.81	0.020	8.0
6348.5S005	A	1983	15	N	1.00	0.358	14.48	0.020	28.0
6348.5S005	A	1983	15	N	2.00	0.033	1.35	0.020	28.0
6348.5S005	A	1983	15	N	3.00	0.025	1.03	0.020	28.0
6360.4S005	B	1978	20	N	1.10	0.119	4.80	0.042	7.5
6360.4S005	B	1978	20	N	2.20	0.025	1.01	0.042	7.5
6360.4S005	B	1978	20	N	3.30	0.028	1.15	0.042	7.5
7526.9S003	D	1981	17	N	1.00	0.093	3.76	0.058	5.0
7526.9S003	D	1981	17	N	2.00	0.024	0.97	0.058	5.0
7526.9S003	D	1981	17	N	3.00	0.023	0.93	0.058	5.0
7707.2S415	A	1993	5	N	0.50	0.802	32.46	0.058	32.5
7707.2S415	A	1993	5	N	1.45	0.162	6.56	0.058	32.5
7707.2S415	A	1993	5	N	2.35	0.028	1.13	0.058	32.5
7707.2S415	A	1993	5	N	4.00	0.031	1.25	0.058	32.5
7707.2S415	B	1993	5	N	0.50	1.003	40.60	0.086	40.6
7707.2S415	B	1993	5	N	1.50	0.267	10.81	0.086	40.6
7707.2S415	B	1993	5	N	2.75	0.068	2.75	0.086	40.6
7707.2S415	B	1993	5	N	4.00	0.031	1.25	0.086	40.6

Bridge ID	Core	Year	Age	Crack	Depth(in)	Cl.(%)	Clx.(lb/cy^3)	D_c(in^2/yr)	C_0(lb/cy^3)
7783.1L065	A	1993	5	N	0.50	0.339	13.72	0.200	13.7
7783.1L065	A	1993	5	N	1.65	0.027	1.09	0.200	13.7
7783.1L065	A	1993	5	N	2.75	0.028	1.13	0.200	13.7
7783.1L065	A	1993	5	N	4.00	0.027	1.09	0.200	13.7
7783.1L065	B	1993	5	N	0.50	0.254	10.28	0.058	1.1
7783.1L065	B	1993	5	N	1.75	0.025	1.01	0.058	1.1
7783.1L065	B	1993	5	N	3.00	0.034	1.38	0.058	1.1
7993.4S063	B	1985	13	N	1.00	0.540	21.86	0.083	29.5
7993.4S063	B	1985	13	N	2.00	0.205	8.30	0.083	29.5
7993.4S063	B	1985	13	N	3.00	0.086	3.47	0.083	29.5
8224.1R061	D	1981	17	N	1.20	0.346	14.01	0.051	23.5
8224.1R061	D	1981	17	N	2.60	0.061	2.47	0.051	23.5
8224.1R061	D	1981	17	N	4.10	0.036	1.46	0.051	23.5
8433.0S075	A	1989	9	N	0.50	0.359	14.53	0.058	14.5
8433.0S075	A	1989	9	N	1.50	0.109	4.41	0.058	14.5
8433.0S075	A	1989	9	N	2.50	0.029	1.17	0.058	14.5
8433.0S075	A	1989	9	N	4.00	0.027	1.09	0.058	14.5
8433.0S075	B	1989	9	N	1.50	0.440	17.81	0.064	17.8
8433.0S075	B	1989	9	N	1.50	0.150	6.07	0.064	17.8
8433.0S075	B	1989	9	N	2.50	0.033	1.34	0.064	17.8
8433.0S075	B	1989	9	N	4.00	0.025	1.01	0.064	17.8
8441.3S018	A	1992	6	N	1.00	0.423	17.12	0.181	23.5
8441.3S018	A	1992	6	N	2.00	0.190	7.69	0.181	23.5
8441.3S018	A	1992	6	N	3.00	0.041	1.66	0.181	23.5
8441.3S018	D	1992	6	N	1.10	0.271	10.97	0.121	17.5
8441.3S018	D	1992	6	N	2.20	0.055	2.23	0.121	17.5
8441.3S018	D	1992	6	N	3.30	0.047	1.90	0.121	17.5
8609.2S030	A	1987	11	N	0.50	0.500	20.24	0.035	20.2
8609.2S030	A	1987	11	N	1.50	0.132	5.34	0.035	20.2
8609.2S030	A	1987	11	N	2.25	0.012	0.49	0.035	20.2
8609.2S030	B	1987	11	N	0.50	0.200	8.10	0.026	8.1
8609.2S030	B	1987	11	N	1.35	0.048	1.94	0.026	8.1
8609.2S030	B	1987	11	N	2.35	0.012	0.49	0.026	8.1
8609.2S030	B	1987	11	N	3.75	0.013	0.53	0.026	8.1
9259.9S218	A	1982	16	N	0.50	0.442	17.89	0.051	17.9
9259.9S218	A	1982	16	N	1.38	0.207	8.38	0.051	17.9
9259.9S218	A	1982	16	N	3.00	0.038	1.54	0.051	17.9
9259.9S218	A	1982	16	N	4.00	0.023	0.93	0.051	17.9
9259.9S218	B	1982	16	N	0.50	0.364	14.73	0.064	14.7
9259.9S218	B	1982	16	N	1.50	0.141	5.71	0.064	14.7
9259.9S218	B	1982	16	N	3.00	0.071	2.87	0.064	14.7
9259.9S218	B	1982	16	N	4.00	0.031	1.25	0.064	14.7

Bridge ID	Core	Year	Age	Crack	Depth(in)	Cl.(%)	Clx.(lb/cy^3)	D_c(in^2/yr)	C_0(lb/cy^3)
9424.1L020	D	1990	8	N	1.30	0.140	5.68	0.168	9.0
9424.1L020	D	1990	8	N	2.60	0.035	1.41	0.168	9.0
9424.1L020	D	1990	8	N	4.00	0.030	1.21	0.168	9.0
9424.1R020	A	1990	8	N	0.50	0.464	18.78	0.035	18.8
9424.1R020	A	1990	8	N	1.50	0.084	3.40	0.035	18.8
9424.1R020	A	1990	8	N	2.50	0.020	0.81	0.035	18.8
9424.1R020	A	1990	8	N	3.50	0.024	0.97	0.035	18.8
9424.1R020	B	1990	8	N	0.50	0.493	19.96	0.016	20.0
9424.1R020	B	1990	8	N	1.50	0.023	0.93	0.016	20.0
9424.1R020	B	1990	8	N	3.00	0.026	1.05	0.016	20.0
9424.1R020	B	1990	8	N	4.00	0.027	1.09	0.016	20.0
9700.8S982	D	1978	20	N	1.20	0.429	17.37	0.035	31.0
9700.8S982	D	1978	20	N	2.40	0.081	3.28	0.035	31.0
9700.8S982	D	1978	20	N	3.60	0.039	1.58	0.035	31.0
9708.3S982	B	1978	20	N	1.20	0.208	8.42	0.200	10.0
9708.3S982	B	1978	20	N	2.40	0.099	4.01	0.200	10.0
9708.3S982	B	1978	20	N	3.60	0.090	3.64	0.200	10.0
9708.3S982	C	1978	20	N	1.10	0.112	4.53	0.200	5.0
9708.3S982	C	1978	20	N	2.20	0.053	2.15	0.200	5.0
9708.3S982	C	1978	20	N	3.30	0.060	2.43	0.200	5.0

Note: Shaded areas represent bridges sampled in phase I.

APPENDIX F: THE RESULTS OF REBAR AND ADHESION RATING

TABLE F.1 Results of Rebar and Adhesion Rating

BRIDGE I.D.	Core	Crack	Year	Bar#	L/T	Cover,in	Adhesion	Rebar Rating
0475.2S002	A	Y	1983	1	T	3	1	5
0475.2S002	A	Y	1983	2	L	3.75	2	5
0475.2S002	B	Y	1983	1	L	3	3	5
0475.2S002	B	Y	1983	2	T	3.875	1	5
0475.2S002	B	Y	1983	3	T	3.875	2	5
0475.2S002	C	N	1983	1	T	2.625	2	5
0475.2S002	C	N	1983	2	L	3.375	3	4
0475.2S002	D	N	1983	1	T	2.5	NA	5
0476.4S002	A	Y	1978	1	X	3.875	NA	5
0476.4S002	C	N	1978	1	T	3.625	3	5
0643.5R380	A	Y	1982	1	L	3	2	4
0643.5R380	B	Y	1982	1	L	3.5	3	4
0643.5R380	C	N	1982	1	T	2.5	2	5
0643.5R380	D	N	1982	1	T	2.25	3	5
0643.5R380	D	N	1982	2	L	3.125	3	5
0668.7S021	A	N	1979	1	T	2.5	3	5
0668.7S021	A	N	1979	2	L	3.25	3	5
0668.7S021	B	N	1979	1	T	2.5	1	5
0668.7S021	C	Y	1979	1	T	3.25	1	3
0668.7S021	C	Y	1979	2	L	4.125	3	4
0668.7S021	D	Y	1979	1	T	3.25	2	2
0709.3R058	A	N	1993	1	T	3.25	3	5
0709.3R058	A	N	1993	2	L	4.25	3	5
0709.3R058	B	N	1993	1	T	2.875	2	5
0709.3R058	C	Y	1993	1	T	3	3	5
0709.3R058	D	Y	1993	1	T	3.125	3	5
0709.3R058	D	Y	1993	2	T	3.125	3	5
0727.5R020	A	Y	1983	1	T	2.26	1	2
0727.5R020	A	Y	1983	2	L	3	2	4
0727.5R020	B	Y	1983	1	L	3.125	2	5
0727.5R020	C	N	1983	1	T	2.5	2	5
0727.5R020	C	N	1983	2	L	3.375	3	4
0727.5R020	D	N	1983	1	T	2.5	2	5
0727.5R020	D	N	1983	2	L	3.25	2	5

BRIDGE I.D.	Core	Crack	Year	Bar#	L/T	Cover in.	Adhesion	Rebar R.
0757.1L380	A	Y	1984	1	T	3	2	5
0757.1L380	A	Y	1984	2	L	3.75	NA	5
0757.1L380	B	Y	1984	1	T	2.625	2	5
0757.1L380	C	N	1984	1	T	2.25	2	5
0757.1L380	C	N	1984	2	L	3.125	2	5
0757.1L380	D	N	1984	1	T	2.25	2	5
0757.1L380	D	N	1984	2	L	3.125	2	5
0761.5O380	A	Y	1984	1	L	3.5	2	5
0761.5O380	A	Y	1984	1	T	2.625	1	2
0761.5O380	B	Y	1984	1	L	3	2	5
0761.5O380	B	Y	1984	1	T	2	2	3
0761.5O380	C	N	1984	1	T	2.625	3	5
0761.5O380	C	N	1984	2	L	3.5	1	5
0761.5O380	D	N	1984	1	L	3.5	3	5
0761.5O380	D	N	1984	1	T	2.5	3	5
0777.9L218	A	N	1983	1	L	3.125	3	5
0777.9L218	B	N	1983	1	L	3.25	3	5
0777.9L218	B	N	1983	2	T	4.125	2	5
0777.9L218	C	Y	1983	1	L	3.25	3	3
0777.9L218	D	Y	1983	1	L	2.75	2	4
0777.9L218	D	Y	1983	2	T	4.25	3	5
0781.1R218	A	N	1991	1	T	3	3	5
0781.1R218	A	N	1991	2	L	3.75	3	5
0781.1R218	B	N	1991	1	T	3	3	5
0781.1R218	B	N	1991	2	L	3.75	3	5
0781.1R218	C	Y	1991	1	T	2.5	3	5
0781.1R218	C	Y	1991	2	L	3.5	3	5
0781.1R218	D	Y	1991	1	T	2.5	3	5
0781.1R218	D	Y	1991	2	L	3.375	3	5
0781.5L218	A	N	1991	1	T	2.75	3	5
0781.5L218	A	N	1991	2	L	3.625	3	5
0781.5L218	B	N	1991	1	T	2.75	3	5
0781.5L218	B	N	1991	2	L	3.6	3	5
0781.5L218	C	Y	1991	1	T	2.375	3	5
0781.5L218	C	Y	1991	2	L	3.125	3	5
0781.5L218	D	Y	1991	1	T	2.625	2	5
0781.5L218	D	Y	1991	2	L	3.4375	1	5
0937.1S003	A	Y	1990	1	L	2	3	5
0937.1S003	A	Y	1990	2	T	3.5	2	5
0937.1S003	B	Y	1990	1	L	2.25	2	5

BRIDGE I.D.	Core	Crack	Year	Bar#	L/T	Cover in.	Adhesion	Rebar R.
0996.0L218	A	Y	1993	1	L	3	1	5
0996.0L218	B	N	1993	1	T	2.75	2	5
0996.0L218	B	N	1993	2	L	3.25	2	5
0996.0L218	C	N	1993	1	T	3	NA	NA
0996.0L218	C	N	1993	2	L	3.75	2	5
0996.0L218	D	Y	1993	1	T	3	1	5
0996.0L218	D	Y	1993	2	L	3.5	2	4
1052.2S150	A	N	1986	1	L	4.5	3	5
1052.2S150	B	N	1986	1	L	3.25	3	5
1052.2S150	B	N	1986	2	T	4	3	5
1052.2S150	C	Y	1986	1	L	2.125	3	5
1052.2S150	C	Y	1986	2	T	3.5	3	5
1052.2S150	D	Y	1986	1	L	3.125	2	5
1052.2S150	D	Y	1986	2	T	3.875	3	5
1253.3S014	A	Y	1984	1	T	2.75	2	4
1253.3S014	A	Y	1984	2	L	3.5	2	5
1253.3S014	C	N	1984	1	T	2.5	2	5
1253.3S014	C	N	1984	2	L	3.25	2	4
1253.3S014	D	N	1984	1	T	2.75	2	5
1253.3S014	D	N	1984	2	L	3.5	2	5
1390.7S175	A	Y	1980	1	T	2.865	3	2
1390.7S175	A	Y	1980	2	L	3.5	2	5
1390.7S175	B	N	1980	2	L	3.125	2	4
1390.7S175	B	N	1980	NA	NA	NA	NA	NA
1390.7S175	C	N	1980	1	T	2	2	5
1390.7S175	C	N	1980	2	L	2.75	2	5
1390.7S175	D	Y	1980	1	T	2.625	2	5
1390.7S175	D	Y	1980	2	L	3.75	2	5
1410.2S071	A	N	1979	NA	NA	NA	NA	NA
1410.2S071	B	N	1979	NA	NA	NA	NA	NA
1410.2S071	C	Y	1979	1	L	2.5	1	2
1410.2S071	D	Y	1979	1	T	4.125	2	5
1411.6S071	A	N	1985	1	L	3.3125	NA	0
1411.6S071	B	N	1985	1	T	4.125	3	5
1411.6S071	C	Y	1985	1	L	3	3	5
1411.6S071	D	Y	1985	1	L	3	3	5

BRIDGE I.D.	Core	Crack	Year	Bar#	L/T	Cover in.	Adhesion	Rebar R.
1479.8S030	A	Y	1982	1	T	2.75	2	5
1479.8S030	A	Y	1982	2	L	3.5	2	5
1479.8S030	B	N	1982	1	T	2.5	2	5
1479.8S030	B	N	1982	2	L	3.5	2	5
1479.8S030	C	N	1982	1	T	2.75	2	4
1479.8S030	C	N	1982	2	L	3.5	2	5
1479.8S030	D	Y	1982	1	T	4	NA	5
1910.0S346	A	Y	1984	1	T	2.875	2	5
1910.0S346	B	Y	1984	1	L	3.5	2	4
1910.0S346	C	N	1984	1	T	2.625	2	5
1910.0S346	C	N	1984	2	L	3.5	2	5
1910.0S346	D	N	1984	1	T	2.5	2	5
1910.0S346	D	N	1984	2	L	3.25	2	5
2336.2O061	A	Y	1984	1	L	3.25	2	5
2336.2O061	A	Y	1984	2	X	6.25	NA	NA
2336.2O061	B	Y	1984	1	T	2.875	2	5
2336.2O061	C	N	1984	1	T	2.875	3	5
2336.2O061	C	N	1984	2	L	3.75	2	5
2336.2O061	D	N	1984	1	T	2.5	3	5
2336.2O061	D	N	1984	2	L	3.25	1	5
2468.5S141	A	Y	1988	1	T	2	2	5
2468.5S141	B	N	1988	1	T	2.125	NA	5
2468.5S141	B	N	1988	2	L	2.875	NA	5
2468.5S141	C	Y	1988	1	T	3	2	5
2468.5S141	C	Y	1988	2	L	3.75	2	4
2468.5S141	C	Y	1988	3	L	3.75	2	5
2468.5S141	D	N	1988	1	T	2.75	2	5
2468.5S141	D	N	1988	2	L	3.75	1	4
2504.7S169	A	N	1988	1	T	2.75	2	5
2504.7S169	A	N	1988	2	L	3.5	3	5
2504.7S169	B	N	1988	1	T	2.625	2	5
2504.7S169	B	N	1988	2	L	3.375	3	5
2504.7S169	C	Y	1988	1	T	3	3	5
2504.7S169	D	Y	1988	1	T	3.125	3	5
2504.7S169	D	Y	1988	2	L	4.125	3	4
2579.9S044	A	Y	1981	1	L	2.5	2	5
2579.9S044	A	Y	1981	2	T	4	2	5
2579.9S044	B	Y	1981	X	X	3.25	NA	NA

BRIDGE I.D.	Core	Crack	Year	Bar#	L/T	Cover in.	Adhesion	Rebar R.
3236.8S004	A	N	1981	1	T	2.5	3	5
3236.8S004	A	N	1981	2	L	3.25	3	5
3236.8S004	B	N	1981	1	T	3.25	2	5
3236.8S004	C	Y	1981	1	T	3.25	3	4
3236.8S004	C	Y	1981	2	L	4	2	5
3236.8S004	D	Y	1981	1	T	3.375	3	5
3236.8S004	D	Y	1981	2	L	4.128	3	5
3364.6S150	A	Y	1992	1	L	2.75	1	4
3364.6S150	A	Y	1992	2	T	4.125	1	5
3364.6S150	B	Y	1992	1	T	4.625	1	5
3372.6S018	A	Y	1988	1	T	2.5	2	5
3372.6S018	A	Y	1988	2	L	3.25	3	5
3372.6S018	B	Y	1988	1	L	2.25	3	5
3372.6S018	B	Y	1988	2	T	3	3	5
3372.6S018	C	N	1988	1	T	2.375	3	5
3372.6S018	C	N	1988	2	L	3.25	3	5
3372.6S018	C	N	1988	3	L	5.125	3	5
3372.6S018	D	N	1988	1	T	2.25	1	5
3372.6S018	D	N	1988	2	L	5	3	4
3712.3S004	A	N	1993	1	L	3.25	3	5
3712.3S004	A	N	1993	2	L	3.125	3	5
3712.3S004	B	N	1993	1	T	2.25	3	5
3712.3S004	B	N	1993	2	L	3	3	5
3712.3S004	C	Y	1993	1	T	2.75	3	5
3712.3S004	D	Y	1993	1	T	3.5	3	5
3712.3S004	D	Y	1993	2	L	4.5	3	5
3975.9S044	A	Y	1981	1	L	2.625	1	5
3975.9S044	B	Y	1981	1	L	3.5	2	5
3975.9S044	C	N	1981	1	L	3.25	2	4
3975.9S044	D	N	1981	1	L	3.75	2	5
3975.9S044	D	N	1981	2	T	4.5	2	5
3988.5S025	A	N	1980	1	T	2.125	2	4
3988.5S025	A	N	1980	2	X	5.125	NA	NA
3988.5S025	B	N	1980	1	T	2.25	2	4
3988.5S025	B	N	1980	2	L	3	2	5
3988.5S025	C	Y	1980	1	T	2.76	NA	1
3988.5S025	C	Y	1980	2	L	3.5	2	5
3988.5S025	D	Y	1980	1	L	3.625	2	2
4227.3S065	A	N	1985	1	L	2	2	5
4227.3S065	B	N	1985	1	L	2	3	5
4227.3S065	C	Y	1985	1	L	2	2	5
4227.3S065	D	Y	1985	1	L	2	3	5

BRIDGE I.D.	Core	Crack	Year	Bar#	L/T	Cover in.	Adhesion	Rebar R.
4323.4S030	A	N	1988	1	T	2.75	3	5
4323.4S030	A	N	1988	2	L	3.5	3	5
4323.4S030	B	N	1988	1	T	2.625	3	5
4323.4S030	B	N	1988	2	L	3.25	3	5
4323.4S030	B	N	1988	3	L	3.25	2	4
4323.4S030	C	Y	1988	1	T	2.75	3	5
4323.4S030	C	Y	1988	2	L	3.5	3	5
4323.4S030	D	Y	1988	1	T	2.75	2	4
4323.4S030	D	Y	1988	2	L	3.375	2	5
4323.4S030	D	Y	1988	3	L	3.875	2	5
4751.0S020	A	N	1988	1	T	2.75	3	5
4751.0S020	A	N	1988	2	L	3.5	3	5
4751.0S020	B	N	1988	1	T	3.625	3	5
4751.0S020	B	N	1988	2	L	4.5	2	5
4751.0S020	C	Y	1988	1	T	3.375	2	5
4751.0S020	D	Y	1988	1	T	2.875	3	5
4751.0S020	D	Y	1988	2	L	3.625	3	5
4751.0S020	D	Y	1988	3	L	5.625	2	5
4801.5S220	A	Y	1978	1	T	2.875	2	5
4801.5S220	B	Y	1978	1	L	4	2	5
4801.5S220	B	Y	1978	2	T	3	3	5
4801.5S220	C	N	1978	1	T	2.875	2	5
4801.5S220	C	N	1978	2	L	3.75	2	NA
4801.5S220	D	N	1978	1	T	2.75	2	5
4926.7S052	A	Y	1986	1	T	2.625	2	5
4926.7S052	A	Y	1986	2	L	3.5	1	3
4926.7S052	A	Y	1986	3	L	3.5	1	4
4926.7S052	B	Y	1986	1	L	4.5	1	5
4926.7S052	C	N	1986	1	L	2.5	2	5
4926.7S052	C	N	1986	2	X	5	NA	NA
4926.7S052	D	N	1986	1	T	2.625	2	5
4926.7S052	D	N	1986	2	L	3.5	2	5
5001.9S224	A	N	1992	1	T	3.25	3	5
5001.9S224	A	N	1992	2	L	4.125	3	5
5001.9S224	B	N	1992	1	T	3	3	5
5001.9S224	B	N	1992	2	L	3.75	3	5
5001.9S224	C	Y	1992	1	L	3.75	3	5
5001.9S224	D	Y	1992	1	T	3.5	2	3

BRIDGE I.D.	Core	Crack	Year	Bar#	L/T	Cover in.	Adhesion	Rebar R.
5098.3S065	A	N	1979	1	T	2.5	3	4
5098.3S065	A	N	1979	2	L	3.25	3	5
5098.3S065	B	N	1979	1	T	2.75	3	5
5098.3S065	B	N	1979	2	L	3.625	3	5
5098.3S065	C	Y	1979	1	T	2.5	1	5
5098.3S065	C	Y	1979	2	L	3.375	3	5
5098.3S065	D	Y	1979	1	T	2.5	2	5
5098.3S065	D	Y	1979	2	L	3.375	3	5
5293.7L218	A	Y	1983	1	T	2.5	3	1
5293.7L218	A	Y	1983	2	L	3.375	2	5
5293.7L218	B	Y	1983	1	T	2.875	2	5
5293.7L218	B	Y	1983	2	L	3.75	2	5
5293.7L218	C	N	1983	1	T	3.75	2	5
5293.7L218	C	N	1983	2	L	3	2	5
5293.7L218	D	N	1983	1	T	2	2	5
5293.7L218	D	N	1983	2	L	2.875	2	5
5298.6S001	A	Y	1985	1	L	2.25	2	5
5298.6S001	B	Y	1985	1	T	4	2	5
5298.6S001	C	N	1985	1	T	3.5	2	5
5298.6S001	D	N	1985	1	T	4.375	2	5
5298.6S001	D	N	1985	2	T	4.375	3	5
5419.0S149	A	N	1986	1	T	2.625	3	5
5419.0S149	A	N	1986	2	L	3.375	3	5
5419.0S149	B	N	1986	1	T	2.875	3	5
5419.0S149	B	N	1986	2	L	3.625	3	5
5419.0S149	C	Y	1986	1	T	3.25	1	2
5419.0S149	C	Y	1986	2	L	4.25	1	4
5419.0S149	D	Y	1986	1	T	3	1	3
5419.0S149	D	Y	1986	2	L	3.75	3	5
5435.5S149	A	N	1987	1	T	2.625	2	5
5435.5S149	A	N	1987	2	L	3.5	2	5
5435.5S149	B	N	1987	1	T	2.75	3	5
5435.5S149	B	N	1987	2	L	3.5	2	5
5435.5S149	C	Y	1987	1	T	2.75	1	3
5435.5S149	C	Y	1987	2	L	3.625	2	5
5435.5S149	D	Y	1987	1	T	2.25	3	5
5435.5S149	D	Y	1987	2	L	3.125	2	4

BRIDGE I.D.	Core	Crack	Year	Bar#	L/T	Cover in.	Adhesion	Rebar R.
5587.2S169	A	N	1985	1	T	3.125	2	5
5587.2S169	A	N	1985	2	L	3.875	3	5
5587.2S169	B	N	1985	1	T	2.75	3	5
5587.2S169	B	N	1985	2	L	3.5	3	5
5587.2S169	C	Y	1985	1	T	2.75	2	5
5587.2S169	C	Y	1985	2	L	3.625	3	5
5587.2S169	D	Y	1985	1	T	3.25	3	5
5587.2S169	D	Y	1985	2	L	4.25	2	
5587.2S169	D	Y	1985	3	L	4.25	3	5
5704.2S001	A	N	1992	1	T	3.125	2	5
5704.2S001	A	N	1992	2	L	3.875	2	5
5704.2S001	A	N	1992	3	L	3.875	3	5
5704.2S001	B	N	1992	1	T	2.875	3	5
5704.2S001	B	N	1992	2	L	3.75	3	5
5704.2S001	C	Y	1992	1	T	2.75	3	5
5704.2S001	D	Y	1992	1	T	3	3	5
5704.2S001	D	Y	1992	2	L	3.75	3	5
5713.7L013	A	N	1987	1	L	3.25	3	5
5713.7L013	B	N	1987	1	T	4.25	3	5
5713.7L013	B	N	1987	2	L	3	3	5
5713.7L013	C	Y	1987	1	T	NA	3	5
5713.7L013	C	Y	1987	2	L	NA	3	5
5713.7L013	D	Y	1987	1	T	4.25	3	5
5713.7L013	D	Y	1987	2	L	3.5	3	5
5721.6R380	A	Y	1980	1	T	3	2	5
5721.6R380	A	Y	1980	2	L	3.75	2	3
5721.6R380	B	N	1980	1	T	2.75	2	5
5721.6R380	B	N	1980	2	L	3.5	2	5
5721.6R380	C	Y	1980	1	T	3	2	5
5722.4R380	A	Y	1980	1	T	2.75	2	5
5722.4R380	A	Y	1980	2	L	3.5	2	5
5722.4R380	B	Y	1980	1	T	3.26	3	2
5722.4R380	B	Y	1980	2	L	4.125	2	3
5722.4R380	B	Y	1980	3	L	4.375	3	2
5722.7O380	A	Y	1980	1	T	2.125	3	2
5722.7O380	A	Y	1980	2	L	2.625	2	5
5722.7O380	A	Y	1980	3	X	4.875	N.A.	3
5722.7O380	B	Y	1980	1	T	2.24	3	2
5722.7O380	B	Y	1980	2	L	3	2	5
5722.7O380	C	N	1980	1	T	2.375	2	4
5722.7O380	C	N	1980	2	L	3.25	2	5
5722.7O380	D	N	1980	1	T	2	2	4
5722.7O380	D	N	1980	2	L	3	2	4

BRIDGE I.D.	Core	Crack	Year	Bar#	L/T	Cover in.	Adhesion	Rebar R.
5738.1L380	A	Y	1982	1	T	2.875	2	5
5738.1L380	A	Y	1982	2	L	3.5	2	5
5738.1L380	B	Y	1982	1	T	2.25	2	NA
5738.1L380	B	Y	1982	2	L	3.125	3	5
5738.1L380	C	N	1982	1	T	2.625	2	4
5738.1L380	D	N	1982	1	T	2.25	2	5
5738.1L380	D	N	1982	2	X	5.375	N.A.	NA
5752.0R030	A	N	1979	1	T	2.5	3	5
5752.0R030	B	N	1979	1	T	2.25	1	5
5752.0R030	B	N	1979	2	L	3.125	3	5
5752.0R030	C	Y	1979	1	T	2.125	2	5
5752.0R030	C	Y	1979	2	L	3	3	5
5752.0R030	D	Y	1979	1	T	2.5	1	5
5752.0R030	D	Y	1979	2	L	3.375	3	5
5803.0S070	A	N	1989	1	T	2.875	3	5
5803.0S070	A	N	1989	2	L	3.625	3	5
5803.0S070	B	N	1989	1	T	2.875	3	5
5803.0S070	B	N	1989	2	L	3.75	3	5
5803.0S070	C	Y	1989	1	L	3.25	1	5
5803.0S070	C	Y	1989	2	L	3.5	2	5
5803.0S070	D	Y	1989	1	T	2.875	3	5
5803.0S070	D	Y	1989	2	L	3.625	3	5
5926.7S065	A	N	1991	1	T	2.875	2	5
5926.7S065	A	N	1991	2	L	3.625	2	5
5926.7S065	B	N	1991	1	T	2.625	2	5
5926.7S065	B	N	1991	2	L	3.375	2	5
5926.7S065	C	Y	1991	1	T	2.875	3	5
5926.7S065	C	Y	1991	2	L	3.75	2	5
5926.7S065	D	Y	1991	1	T	3	3	5
5926.7S065	D	Y	1991	2	L	4	2	5
5930.9S065	A	Y	1991	1	T	4.125	1	5
5930.9S065	B	Y	1991	1	T	4.125	2	5
5930.9S065	C	N	1991	1	T	2.5	3	5
5930.9S065	C	N	1991	2	L	3.375	2	5
5930.9S065	D	N	1991	1	T	2.25	2	5
5930.9S065	D	N	1991	2	L	3	3	5
5930.9S065	D	N	1991	3	L	3	2	5
5930.9S065	D	N	1991	4	L	5.75	1	5

BRIDGE I.D.	Core	Crack	Year	Bar#	L/T	Cover in.	Adhesion	Rebar R.
5931.7S065	A	N	1992	1	T	2	3	5
5931.7S065	A	N	1992	2	L	2.75	3	5
5931.7S065	B	N	1992	1	T	1.25	3	5
5931.7S065	B	N	1992	2	L	2	3	5
5931.7S065	C	Y	1992	1	T	2.5	3	5
5931.7S065	C	Y	1992	2	L	3.5	3	5
5931.7S065	D	Y	1992	1	T	3.25	3	5
5931.7S065	D	Y	1992	2	L	4	3	5
5931.7S065	D	Y	1992	3	L	4	3	5
6011.6S009	A	N	1976	1	T	1.875	2	5
6011.6S009	A	N	1976	2	L	2.625	2	5
6011.6S009	B	Y	1976	1	T	2	1	2
6011.6S009	B	Y	1976	2	L	2.75	2	5
6011.6S009	C	Y	1976	1	T	1.75	1	1
6011.6S009	C	Y	1976	2	L	2.625	2	4
6011.6S009	D	N	1976	1	L	2.25	2	5
6011.6S009	D	N	1976	2	X	4.5	NA	NA
6206.4S102	A	N	1990	1	T	2.75	3	5
6206.4S102	A	N	1990	2	L	3.5	2	5
6206.4S102	B	N	1990	1	T	2.125	3	5
6206.4S102	B	N	1990	2	L	3.0625	3	5
6206.4S102	C	Y	1990	1	T	2.5	3	5
6206.4S102	C	Y	1990	2	L	3.25	3	5
6206.4S102	D	Y	1990	1	T	2.5	2	5
6206.4S102	D	Y	1990	2	L	3.25	3	5
6219.3S137	A	N	1982	1	T	2.25	3	5
6219.3S137	A	N	1982	2	L	3	2	5
6219.3S137	A	N	1982	3	L	3	3	5
6219.3S137	B	N	1982	1	T	2.5	3	5
6219.3S137	B	N	1982	2	L	3.375	3	5
6219.3S137	C	Y	1982	1	T	2.5	2	5
6219.3S137	C	Y	1982	2	L	3.875	3	5
6219.3S137	C	Y	1982	3	L	3.25	3	5
6219.3S137	D	Y	1982	1	T	2.5	2	5
6219.3S137	D	Y	1982	2	L	3.375	3	5
6219.3S137	D	Y	1982	3	L	3.375	3	5

BRIDGE I.D.	Core	Crack	Year	Bar#	L/T	Cover in.	Adhesion	Rebar R.
6303.1S156	A	N	1990	1	T	3.25	3	5
6303.1S156	A	N	1990	2	L	4	3	5
6303.1S156	A	N	1990	3	L	4.25	3	5
6303.1S156	B	N	1990	1	T	2.5	3	5
6303.1S156	B	N	1990	2	L	3.25	3	5
6303.1S156	C	Y	1990	1	T	2.25	2	5
6303.1S156	C	Y	1990	2	L	3	3	5
6303.1S156	D	Y	1990	1	T	3.875	3	5
6303.1S156	D	Y	1990	2	L	4.5	3	5
6345.2S092	A	N	1979	1	T	4.375	2	5
6345.2S092	B	N	1979	1	T	4	1	5
6345.2S092	B	N	1979	2	L	2.75	2	5
6345.2S092	C	Y	1979	1	L	2.875	3	5
6345.2S092	D	Y	1979	1	L	3	2	5
6348.5S005	A	N	1983	1	T	2.75	NA	NA
6348.5S005	A	N	1983	2	L	3.625	2	5
6348.5S005	B	Y	1983	1	T	3	2	5
6348.5S005	C	Y	1983	1	T	2.75	2	5
6348.5S005	C	Y	1983	2	L	3.625	2	5
6348.5S005	D	N	1983	1	T	2.25	2	5
6348.5S005	D	N	1983	2	L	3	2	5
6360.4S005	A	Y	1978	1	?	2.875	2	5
6360.4S005	A	Y	1978	2	?	3.5	2	4
6360.4S005	B	N	1978	1	L	3.75	2	4
6360.4S005	B	N	1978	2	T	2.75	2	5
6360.4S005	C	Y	1978	1	L	3	2	2
6360.4S005	C	Y	1978	2	X	5.5	NA	NA
6360.4S005	D	N	1978	1	T	2.25	2	4
6360.4S005	D	N	1978	2	L	3.25	2	5
6403.6L014	A	N	1987	1	T	3.125	2	5
6403.6L014	B	N	1987	1	T	2.5	1	5
6403.6L014	C	Y	1987	1	T	2.75	3	5
6403.6L014	D	Y	1987	1	T	2	1	2
6488.8S030	A	N	1986	1	T	2.75	3	5
6488.8S030	B	N	1986	1	T	NA	3	5
6488.8S030	C	Y	1986	1	T	3	2	5
6488.8S030	D	Y	1986	1	T	2.75	3	5
7239.2S009	A	N	1989	1	L	3.375	2	5
7239.2S009	A	N	1989	2	T	4.25	3	5
7239.2S009	B	Y	1989	1	L	2.75	3	5

BRIDGE I.D.	Core	Crack	Year	Bar#	L/T	Cover in.	Adhesion	Rebar R.
7526.9S003	A	N	1981	1	T	2.75	2	5
7526.9S003	B	Y	1981	1	T	2.885	1	2
7526.9S003	B	Y	1981	2	L	3.75	1	1
7526.9S003	C	Y	1981	1	T	2.74	1	1
7526.9S003	D	N	1981	1	T	2.5	2	5
7702.4S160	A	N	1986	1	T	2.0625	2	5
7702.4S160	A	N	1986	2	L	2.875	2	5
7702.4S160	B	N	1986	1	T	2	3	5
7702.4S160	B	N	1986	2	L	2.625	3	5
7702.4S160	C	Y	1986	1	L	2.75	3	5
7702.4S160	D	Y	1986	1	T	2	1	5
7702.4S160	D	Y	1986	2	L	3	3	5
7707.2S415	A	N	1993	1	T	2.5	3	5
7707.2S415	A	N	1993	2	L	3.375	2	5
7707.2S415	A	N	1993	3	L	3.625	2	5
7707.2S415	B	N	1993	1	T	3	3	5
7707.2S415	B	N	1993	2	L	3.75	3	4
7707.2S415	C	Y	1993	1	T	2.5	3	5
7707.2S415	C	Y	1993	2	L	3.5	3	5
7783.1L065	A	N	1993	1	T	3	2	5
7783.1L065	A	N	1993	2	L	3.75	3	5
7783.1L065	B	N	1993	1	T	3.25	3	5
7783.1L065	B	N	1993	2	L	4	3	5
7783.1L065	C	Y	1993	1	T	3	3	5
7783.1L065	D	Y	1993	1	T	2.25	3	5
7783.1L065	D	Y	1993	2	L	3.25	3	5
7993.4S063	A	Y	1985	1	L	2.5	3	3
7993.4S063	A	Y	1985	2	T	4	2	5
7993.4S063	B	N	1985	1	L	2.75	3	5
7993.4S063	B	N	1985	2	T	4	2	5
7993.4S063	D	Y	1985	1	L	3	3	5
8224.1R061	A	Y	1981	1	T	2.125	1	4
8224.1R061	A	Y	1981	2	L	3	2	5
8224.1R061	B	Y	1981	1	T	3.24	1	2
8224.1R061	B	Y	1981	2	L	4.125	2	5
8224.1R061	C	N	1981	1	T	2.5	1	5
8224.1R061	C	N	1981	2	L	3.5	1	5
8224.1R061	D	N	1981	1	T	2.5	2	5
8224.1R061	D	N	1981	2	L	3.5	2	4

BRIDGE I.D.	Core	Crack	Year	Bar#	L/T	Cover in.	Adhesion	Rebar R.
8433.0S075	A	N	1989	1	T	2.5	3	5
8433.0S075	A	N	1989	2	L	3.375	3	5
8433.0S075	B	N	1989	1	T	2.625	3	5
8433.0S075	B	N	1989	2	L	3.375	3	5
8433.0S075	C	Y	1989	1	T	2.25	3	5
8433.0S075	C	Y	1989	2	L	3.125	3	5
8433.0S075	D	Y	1989	1	T	2.75	3	5
8433.0S075	D	Y	1989	2	L	3.5	3	5
8441.3S018	A	N	1992	1	T	2.5	2	5
8441.3S018	B	Y	1992	1	T	3	2	5
8441.3S018	B	Y	1992	2	L	3.875	2	5
8441.3S018	C	Y	1992	1	T	2.5	2	5
8441.3S018	D	N	1992	1	T	2.25	2	5
8441.3S018	D	N	1992	2	L	3	2	5
8441.3S018	D	N	1992	3	L	3	2	4
8554.2L030	A	N	1991	1	L	3	3	5
8554.2L030	A	N	1991	2	T	4.25	3	5
8554.2L030	B	N	1991	1	L	2.875	2	5
8554.2L030	B	N	1991	2	T	4.125	3	5
8554.2L030	C	Y	1991	1	L	3	3	5
8554.2L030	C	Y	1991	2	T	4.375	3	5
8554.2L030	D	Y	1991	1	L	3	2	5
8554.2L030	D	Y	1991	2	T	4.125	2	5
8600.5S008	A	N	1989	1	T	2.8	3	5
8600.5S008	A	N	1989	2	L	3.75	3	5
8600.5S008	B	N	1989	1	T	2.9375	3	5
8600.5S008	C	Y	1989	1	T	2.25	2	5
8600.5S008	C	Y	1989	2	L	3.5	3	5
8600.5S008	D	Y	1989	2	L	3.75	2	5
8609.2S030	A	N	1987	1	T	3	3	5
8609.2S030	B	N	1987	1	T	2.75	2	5
8609.2S030	C	Y	1987	1	T	2.625	3	5
8609.2S030	C	Y	1987	2	L	3.375	3	5
8609.2S030	C	Y	1987	3	L	3.375	3	5
8609.2S030	D	Y	1987	1	T	2.5	2	5
8609.2S030	D	Y	1987	2	L	3.25	2	5
8920.5S016	A	N	1989	1	T	2.625	3	5
8920.5S016	A	N	1989	2	L	3.375	3	5
8920.5S016	B	N	1989	1	T	2.75	3	5
8920.5S016	B	N	1989	2	L	3.5	3	5
8920.5S016	C	Y	1989	1	L	3.25	3	5
8920.5S016	D	Y	1989	1	L	3	3	5

BRIDGE I.D.	Core	Crack	Year	Bar#	L/T	Cover in.	Adhesion	Rebar R.
9245.7S022	A	N	1987	1	L	2.375	3	5
9245.7S022	A	N	1987	2	T	3.5	3	5
9245.7S022	B	N	1987	1	L	3.375	3	5
9245.7S022	B	N	1987	2	T	4.125	3	5
9245.7S022	C	Y	1987	1	T	4	3	5
9245.7S022	D	Y	1987	1	L	2.5	3	5
9245.7S022	D	Y	1987	2	T	3.875	2	5
9259.9S218	A	N	1982	1	L	2.125	3	5
9259.9S218	B	N	1982	1	L	2.5	3	5
9259.9S218	B	N	1982	2	T	3.75	3	5
9259.9S218	C	Y	1982	1	T	3.5	2	5
9259.9S218	D	Y	1982	1	L	2.625	3	4
9424.1L020	A	Y	1990	1	L	4.25	2	5
9424.1L020	B	N	1990	1	T	3.25	2	5
9424.1L020	B	N	1990	2&3	L	4.125	3	5
9424.1L020	C	Y	1990	1	L	3.75	2	5
9424.1L020	C	Y	1990	3	T	5.875	NA	NA
9424.1L020	D	N	1990	1	T	3.5	2	5
9424.1R020	A	N	1990	1	T	2.625	3	5
9424.1R020	A	N	1990	2	L	3.5	3	5
9424.1R020	B	N	1990	1	T	3.125	3	5
9424.1R020	B	N	1990	2	L	4	3	5
9424.1R020	C	Y	1990	1	T	2.625	3	5
9424.1R020	C	Y	1990	2	L	3.625	3	5
9424.1R020	D	Y	1990	1	T	2.875	3	5
9424.1R020	D	Y	1990	1	T	2.875	3	5
9424.1R020	D	Y	1990	2	L	3.75	3	5
9424.1R020	D	Y	1990	2	L	3.75	3	5
9700.8S982	A	Y	1978	1	L	3.375	3	5
9700.8S982	A	Y	1978		X	6	NA	NA
9700.8S982	B	Y	1978	1	L	3	2	5
9700.8S982	B	Y	1978	2	X	5.625	NA	NA
9700.8S982	C	N	1978	1	T	2.25	2	5
9700.8S982	C	N	1978	2	L	3	1	4
9700.8S982	C	N	1978	3	L	3	2	5
9700.8S982	D	N	1978	1	T	2.25	2	5
9700.8S982	D	N	1978	2	L	3	2	5
9700.8S982	D	N	1978	3	L	3.125	3	5
9708.3S982	A	Y	1978	1	T	4	1	1
9708.3S982	B	N	1978	1	L	3.375	1	5
9708.3S982	C	N	1978	1	T	3	2	5

Notes: Shaded areas represent bridges sampled in phase I. T, transverse; L, longitudinal; X, diagonal; NA, data not available; Y, cracked locations; N, uncracked locations. 1, 2, 3: bar numbering from top.

APPENDIX G: COMPUTED DIFFUSION CONSTANT AND SURFACE CHLORIDE CONCENTRATION FOR TAMA COUNTY BRIDGES AND TWO-COURSE PLACEMENTS DECKS

TABLE G.1 Computed Diffusion Constant and Surface Chloride Concentration

Bridge ID	Core	Year	Age	Crack	Depth(in)	Cl.(%)	Clx.(lb/cy^3)	D_c(lin^2/yr)	C_o.(lb/cy^3)
2401.1S039	A	1977	21	N	0.5	0.2830	11.456	0.011	11.456
2401.1S039	A	1977	21	N	1.5	0.0400	1.619	0.011	11.456
2401.1S039	A	1977	21	N	2.625	0.0100	0.405	0.011	11.456
2401.1S039	A	1977	21	N	3.75	0.0100	0.405	0.011	11.456
2401.1S039	B	1977	21	N	0.5	0.2200	8.905	0.006	8.905
2401.1S039	B	1977	21	N	1.5	0.0100	0.405	0.006	8.905
2401.1S039	B	1977	21	N	2.5	0.0100	0.405	0.006	8.905
2401.1S039	B	1977	21	N	3.75	0.0100	0.405	0.006	8.905
3966.4S044	A	1977	21	N	0.5	0.3260	13.196	0.036	13.196
3966.4S044	A	1977	21	N	2	0.0620	2.510	0.036	13.196
3966.4S044	A	1977	21	N	3.5	0.0550	2.226	0.036	13.196
3966.4S044	A	1977	21	N	4.75	0.0140	0.567	0.036	13.196
3966.4S044	B	1977	21	N	0.5	0.2260	9.148	0.043	9.148
3966.4S044	B	1977	21	N	2	0.0520	2.105	0.043	9.148
3966.4S044	B	1977	21	N	3.5	0.0290	1.174	0.043	9.148
3966.4S044	B	1977	21	N	4.75	0.0220	0.891	0.043	9.148
4039.6R020	A	1978	20	N	0.5	0.3370	13.641	0.001	13.641
4039.6R020	A	1978	20	N	1.75	0.0300	1.214	0.001	13.641
4039.6R020	A	1978	20	N	2.75	0.0190	0.769	0.001	13.641
4039.6R020	A	1978	20	N	4.15	0.0170	0.688	0.001	13.641
4039.6R020	B	1978	20	N	0.5	0.2960	11.982	0.009	11.982
4039.6R020	B	1978	20	N	1.75	0.0120	0.486	0.009	11.982
4039.6R020	B	1978	20	N	3	0.0150	0.607	0.009	11.982
4039.6R020	B	1978	20	N	4	0.0320	1.295	0.009	11.982
TAMA1	A	1968	30	N	0.5	0.3850	15.584	0.003	15.584
TAMA1	A	1968	30	N	1.06	0.0610	2.469	0.003	15.584
TAMA1	A	1968	30	N	1.6	0.0180	0.729	0.003	15.584
TAMA1	A	1968	30	N	4	0.0140	0.567	0.003	15.584
TAMA1	B	1968	30	N	0.5	0.1860	7.529	0.017	7.529
TAMA1	B	1968	30	N	1.5	0.0550	2.226	0.017	7.529
TAMA1	B	1968	30	N	2	0.0300	1.214	0.017	7.529
TAMA1	B	1968	30	N	4	0.0090	0.364	0.017	7.529
TAMA2	A	1968	30	N	0.5	0.4460	18.054	0.004	18.054
TAMA2	A	1968	30	N	1.25	0.0500	2.024	0.004	18.054
TAMA2	A	1968	30	N	1.9	0.0130	0.526	0.004	18.054
TAMA2	A	1968	30	N	3.5	0.0110	0.445	0.004	18.054
TAMA2	B	1968	30	N	0.5	0.3470	14.046	0.004	14.046
TAMA2	B	1968	30	N	1.5	0.0110	0.445	0.004	14.046
TAMA2	B	1968	30	N	2.5	0.0100	0.405	0.004	14.046
TAMA2	B	1968	30	N	4	0.0170	0.688	0.004	14.046

Bridge ID	Core	Year	Age	Crack	Depth(in)	Cl.(%)	Clx.(lb/cy^3)	D_c(lin^2/yr)	C_o.(lb/cy^3)
TAMA3	A	1968	30	N	0.5	0.1860	7.529	0.035	7.529
TAMA3	A	1968	30	N	1.25	0.1150	4.655	0.035	7.529
TAMA3	A	1968	30	N	2.44	0.0290	1.174	0.035	7.529
TAMA3	A	1968	30	N	4	0.0130	0.526	0.035	7.529
TAMA3	B	1968	30	N	0.5	0.4430	17.932	0.028	17.932
TAMA3	B	1968	30	N	1.33	0.2580	10.444	0.028	17.932
TAMA3	B	1968	30	N	2.25	0.0550	2.226	0.028	17.932
TAMA3	B	1968	30	N	3.25	0.0130	0.526	0.028	17.932