

Linear Cracking in Bridge Decks

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<p>16. Abstract:</p> <p>Concrete cracking in bridge decks remains an important issue relative to deck durability. Cracks can allow increased penetration of chlorides, which can result in premature corrosion of the reinforcing steel and subsequent spalling of the concrete deck. Although it is understood that the service life of bridge decks is affected by concrete cracking, the degree to which cracking affects service life is unknown. Crack repairs may be expensive, and only a few state transportation agencies have developed effective decision-making tools to support engineering decisions about whether and how to repair cracks in bridges.</p> <p>To understand how various factors affect the formation of cracks and to comprehend how cracks influence the performance of bridge decks, a comprehensive literature review was performed of publications from the early 1970s to the present. With findings from more than 45 years of research, the influences of about 30 factors were included in the literature review.</p> <p>In this study, 37 highway bridges in Virginia were selected on the basis of environmental exposure, geographic location, traffic conditions, and construction era. Ten decks with ordinary portland cement (OPC) concrete with a water-cementitious material (w/c) ratio of 0.47 with uncoated reinforcement were built from 1968 through 1971, and 27 decks with concrete with a w/c ratio of 0.45 with epoxy-coated reinforcement were built from 1984 through 1991. Of the newer 27 decks, 11 had concrete with supplementary cementitious material (SCM) such as fly ash and slag. The study included field surveys, sampling, and extensive data collection with regard to the decks. In addition, a laboratory study of the collected samples was conducted to understand the material properties and to determine the chloride contents. Statistical methods were used to analyze the collected data and to form regression models for prediction of crack influence on chloride diffusion.</p> <p>The increase in chloride diffusion through cracks when compared to that of corresponding uncracked locations was statistically significant. No strong correlation was found between surface crack width and chloride diffusion; however, a significant correlation was found between crack depth and chloride diffusion.</p> <p>To understand the effects of cracks on the durability of the structures, service life was estimated using a probabilistic chloride diffusion model based on Fick's second law of diffusion. The estimated service life of the decks with concrete with SCM was around 100 years but only if no cracks were present. The presence of cracks affected the service life significantly. With higher crack frequencies, the service life plunged to the levels of decks built with OPC concrete, which was significantly lower to begin with. The service life of decks built with OPC concrete was not significantly affected by the presence of cracks, primarily because the high permeability of OPC concrete, with or without the presence of cracks, results in a shorter service life for OPC concrete decks. Time to corrosion initiation for corrosion-resistant reinforcing bars, ASTM A1035 (VDOT Class I reinforcement) and ASTM A955 (VDOT Class III reinforcement), was estimated, and the service lives were much longer compared to those of the decks in this study constructed with other types of reinforcement. Implementation guidance for quality assurance of newly built bridge decks with modern concrete mixtures and corrosion-resistant reinforcement and for maintenance of existing bridge decks was developed based on the study results.</p>					
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ABSTRACT

Concrete cracking in bridge decks remains an important issue relative to deck durability. Cracks can allow increased penetration of chlorides, which can result in premature corrosion of the reinforcing steel and subsequent spalling of the concrete deck. Although it is understood that the service life of bridge decks is affected by concrete cracking, the degree to which cracking affects service life is unknown. Crack repairs may be expensive, and only a few state transportation agencies have developed effective decision-making tools to support engineering decisions about whether and how to repair cracks in bridges.

To understand how various factors affect the formation of cracks and to comprehend how cracks influence the performance of bridge decks, a comprehensive literature review was performed of publications from the early 1970s to the present. With findings from more than 45 years of research, the influences of about 30 factors were included in the literature review.

In this study, 37 highway bridges in Virginia were selected on the basis of environmental exposure, geographic location, traffic conditions, and construction era. Ten decks with ordinary portland cement (OPC) concrete with a water–cementitious material (w/c) ratio of 0.47 with uncoated reinforcement were built from 1968 through 1971, and 27 decks with concrete with a w/c ratio of 0.45 with epoxy-coated reinforcement were built from 1984 through 1991. Of the newer 27 decks, 11 had concrete with supplementary cementitious material (SCM) such as fly ash and slag. The study included field surveys, sampling, and extensive data collection with regard to the decks. In addition, a laboratory study of the collected samples was conducted to understand the material properties and to determine the chloride contents. Statistical methods were used to analyze the collected data and to form regression models for prediction of crack influence on chloride diffusion.

The increase in chloride diffusion through cracks when compared to that of corresponding uncracked locations was statistically significant. No strong correlation was found between surface crack width and chloride diffusion; however, a significant correlation was found between crack depth and chloride diffusion.

To understand the effects of cracks on the durability of the structures, service life was estimated using a probabilistic chloride diffusion model based on Fick's second law of diffusion. The estimated service life of the decks with concrete with SCM was around 100 years but only if no cracks were present. The presence of cracks affected the service life significantly. With higher crack frequencies, the service life plunged to the levels of decks built with OPC concrete, which was significantly lower to begin with. The service life of decks built with OPC concrete was not significantly affected by the presence of cracks, primarily because the high permeability of OPC concrete, with or without the presence of cracks, results in a shorter service life for OPC decks. Time to corrosion initiation for corrosion-resistant reinforcing bars, ASTM A1035 (VDOT Class I reinforcement) and ASTM A955 (VDOT Class III reinforcement), was estimated, and the service lives were much longer compared to those of the decks in this study constructed with other types of reinforcement. Implementation guidance for quality assurance of newly built bridge decks with modern concrete mixtures and corrosion-resistant reinforcement and for maintenance of existing bridge decks was developed based on the study results.

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INTRODUCTION

Cracking of concrete remains an inevitable problem. Effects can range from an aesthetically unpleasing appearance to costly maintenance issues by allowing increased corrosive chemical species to penetrate to the level of the reinforcement, resulting in premature corrosion damage. Cracks can be indicative of the use of improper construction materials, higher structural stiffness, drying environmental conditions, poor construction practices, or, as most often noted, a combination of several such factors.

The following is a list of factors that have either impeding or imminent influence on the cracking of concrete.

Concrete material properties:

- water content
- slump
- cement paste content
- pozzolans
- aggregate type
- air content
- compressive/tensile strength
- concrete temperature.

Deck design parameters:

- reinforcement details
- concrete top cover
- deck thickness.

Bridge design factors:

- superstructure stiffness
- span length
- composite action
- continuity/end condition
- girder material
- skew
- stay-in-place (SIP) forms.

Construction practices:

- surface finishing
- curing.

Environmental conditions:

- air temperature
- evaporation rate.

PURPOSE AND SCOPE

The purpose of this study was to determine the influence of cracks on the chloride-induced corrosion deterioration of bridge decks in Virginia. This was determined by evaluating the chloride diffusion with cracked specimens and corresponding uncracked specimens together with the influence of surface crack width and crack depth. The bridge design factors influencing bridge deck cracking and the effect of the cracks on the service life of the structures were also studied.

Data collected from field surveys and laboratory testing of concrete cores from two previous studies (Keller, 2004; Sansone, 2006) were used in this study. The study included 37 bridges, with at least 1 from each of the nine districts of the Virginia Department of Transportation (VDOT). The selected bridges ranged in age from 10 to 30 years; contained a controlled selection of conventional and high-performance concrete; and had not received overlays at the time of inspection. Several parameters concerning the characterization of traffic volume and road classifications were gathered from the Highway and Traffic Records Information System (HTRIS) to supplement the study. The influence of active cracks was outside the scope of the study as these cracks require individual studies as to their cause and thus different sealing methods.

METHODS

Three tasks were conducted to achieve the study objectives:

1. *A literature review was conducted.* To understand how various factors affect the formation of cracks and to comprehend how cracks influence the performance of bridge decks, a literature review on concrete cracking was performed of publications from the early 1970s to the present. With findings from more than 45 years of research, the influences of about 30 factors on the formation of cracks were included in the literature review.
2. *Studies of 37 highway bridge decks were conducted.* Field surveying, sampling, and extensive data collection from the decks of 37 highway bridges selected on the basis of environmental exposure, geographic location, traffic conditions, and construction era were conducted. Of the 37 decks, 10 decks were built with ordinary portland cement (OPC) concrete with a water–cementitious material (w/c) ratio of 0.47 with uncoated reinforcement in 1968 through 1971 and 27 decks were built with concrete with a w/c ratio of 0.45 with epoxy-coated reinforcement (ECR) in 1984 through 1991. Of the newer 27 decks, 11 were built with concrete with supplementary cementitious material (SCM) such as fly ash and slag. In addition, a laboratory study of the collected concrete samples was conducted to determine the material properties and the chloride content.
3. *Statistical methods were used to analyze the collected data and to form regression models for prediction of crack influence on chloride diffusion.*

Literature Review

The literature search included peer-reviewed research conference papers and technical reports as identified from TRID, RiP, WorldCat, National Technical Reports Library, Civil Engineering Abstracts, Compendex, Web of Science, Mechanical and Transportation Engineering Abstracts, Proquest Dissertations and Theses, Strategic Highway Research Program (SHRP) reports, and ASCE Library.

Selection and Study of Bridge Decks

Bridge Selection

General information on the 37 selected bridges is presented in Table 1. As shown in Table 1, at least 1 bridge was selected from each of VDOT's nine districts. Of the 37 bridges, 10 bridge decks were built with uncoated reinforcement with a specified maximum w/c ratio of 0.47 for the construction period 1968 through 1971 (hereinafter 1968-71). The remaining 27 bridges were built with ECR with a specified maximum w/c ratio of 0.45 for the construction period 1984 through 1991 (hereinafter 1984-91); 16 of these decks contained OPC concrete alone. For the decks built in 1984-91, concrete mixtures for 7 of the decks contained ground granulated blast furnace slag and those for 4 of the decks contained fly ash; however, the accurate quantities are not known. Concrete mixtures for the decks built in 1968-71 did not contain any SCM. The

remaining bridge was not cored, and thus the types of reinforcing bar (hereinafter “rebar”) and concrete were not verified.

Table 1. General Information on the 37 Study Bridges

VDOT District	State Structure No.	Federal Structure No.	Year Built/ Replaced	w/c Ratio	SCM	Rebar Type	AADT as of 2003
1-Bristol	1132	16305	1988	0.45	None	ECR	5,426
1-Bristol	1133	16307	1988	0.45	None	ECR	5,067
1-Bristol	1152	18526	1987	0.45	Fly ash	ECR	6,632
1-Bristol	1804	22469	1969	0.47	None	Bare bar	2,106
1-Bristol	2815	22374	1986	0.45	Fly ash	ECR	24,531
1-Bristol	2819	22356	1986	0.45	Fly ash	ECR	--
1-Bristol	2820	22380	1986	0.45	None	ECR	25,181
1-Bristol	6051	17624	1990	0.45	None	ECR	981
1-Bristol	6101	19192	1969	0.47	None	Bare bar	1,865
2-Salem	1020	8364	1986	0.45	None	ECR	4,597
2-Salem	1021	7775	1988	0.45	Fly ash	ECR	12,612
2-Salem	2007	12144	1970	0.47	None	Bare bar	24,552
3-Lynchburg	1000	23033	1991	0.45	Slag	ECR	4,315
3-Lynchburg	1003	5711	1988	0.45	None	ECR	1,207
3-Lynchburg	1017	4191	1990	0.45	Slag	ECR	4,862
3-Lynchburg	1021	12419	1971	0.47	None	Bare bar	404
4-Richmond	1007	5142	1990	0.45	None	ECR	14,910
4-Richmond	1062	3577	1969	0.47	None	Bare bar	1000
4-Richmond	2049	5972	1968	0.47	None	Bare bar	13,457
4-Richmond	2901	23233	1991	0.45	None	ECR	16,812
5-Hampton Roads	1800	20240	1970	0.47	None	Bare bar	1,451
5-Hampton Roads	2547	21945	1984	0.45	None	ECR	23,910
5-Hampton Roads	2812	23098	1991	0.45	Slag	ECR	3,241
6-Fredericksburg	1032	18067	1971	0.47	None	Bare bar	3,361
7-Culpeper	1920	12919	1991	0.45	None	ECR	3,922
8-Staunton	1002	1683	1988	0.45	Slag	ECR	5,246
8-Staunton	1019	15217	1984	0.45	None	ECR	7,875
8-Staunton	1042	15361	1990	0.45	Slag	ECR	1,894
8-Staunton	1133	1046	1987	0.45	None	ECR	3,687
9-NoVA	1002	158	1987	0.45	Slag	ECR	29,833
9-NoVA	1014	11079	1987	0.45	None	ECR	5,662
9-NoVA	1031	11033	1990	0.45	None	ECR	9,503
9-NoVA	1098	114	1988	0.45	None	ECR	26,736
9-NoVA	1139	11061	1987	0.45	None	ECR	9,503
9-NoVA	2801	19948	1970	0.47	None	Bare bar	7,311
9-NoVA	6042	6742	1969	0.47	None	Bare bar	12,824
9-NoVA	6058	23121	1991	0.45	Slag	ECR	--

VDOT = Virginia Department of Transportation; w/c ratio = water–cementitious material ratio; SCM = supplementary cementitious material; AADT = annual average daily traffic; ECR = epoxy-coated reinforcement; NoVA = Northern Virginia District.

The decks built in 1968-71 with uncoated reinforcement were from a pool of 129 bridge decks selected for a previous study (Newlon, 1974). At the time of the selection of bridges for this study from the 129 bridges that had been in service for more than 30 years, only the bridge decks that had not been subjected to a major rehabilitation such as overlaying were selected for this study. Including any overlaid decks would have introduced a bias, since they would have performed differently than the decks in this study. About 50% of the 129 bridge decks were overlaid by the time of this study. Thus, the selected 10 bridge decks comprised a representative sample of the other 50% decks that had not needed overlaying.

Virginia is divided into six environmental zones, which are based on climate differences, which are related to deicer salt usage. Figure 1 shows the environmental zone map and the description of zones and corresponding average roadway chloride exposure for a 3-year period of deicing salt usage. At least one of the selected bridge decks was located in each environmental zone. The salt usage information was used to compare deck chloride exposure and chloride diffusion parameters in this study.

Virginia is also divided into three basic geographic zones, based on precipitation, wind, altitude, and proximity to the ocean: Appalachian, Piedmont, and Coastal. The Appalachian zone includes Bristol, Salem, and Staunton; the Piedmont zone includes Lynchburg, Culpeper, and Northern Virginia; and the Coastal zone includes Richmond, Hampton Roads, and Fredericksburg.

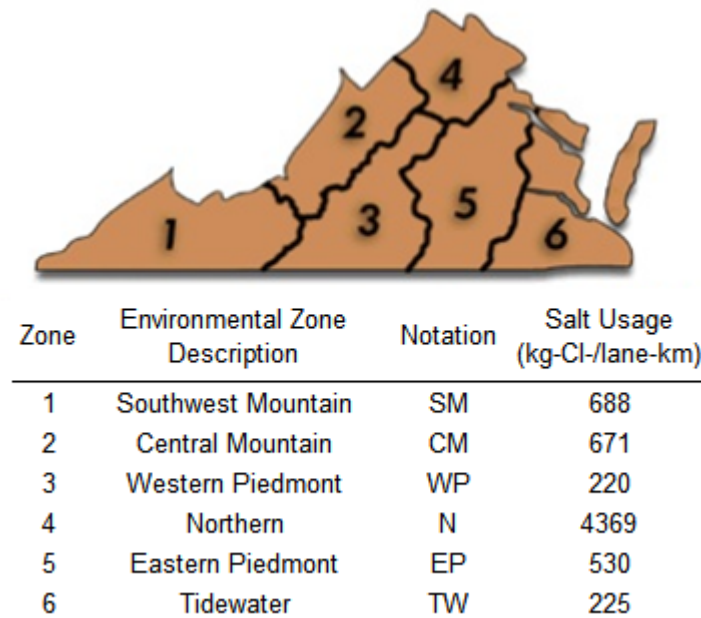


Figure 1. The Six Environmental Zones of Virginia

Field Survey

Visual and Damage Survey

An initial visual assessment of the bridge decks was performed to locate problem areas. Then, the deck area to be surveyed was mapped out and locations for corrosion testing and core sampling were laid out with the use of spray paint. The damage survey consisted of identifying

and mapping the cracks, spalls, patches, and delaminations. Chain dragging supplemented with a 3-lb hammer was employed to identify and map approximately the underlying delaminations (ASTM International [ASTM], 1997c). Crack lengths and crack widths were measured at 1-ft (305-mm) intervals using a crack comparator. The comparator, a clear plastic card, has printed graduations from 0.010 to 0.080 in (0.20 to 2.0 mm). Survey results were recorded so that the collected information could be put in computerized form for further analysis.

Core Sampling

Concrete cores with and without reinforcements were collected for material testing and petrography in the laboratory. Initially, the researchers planned to sample 15 cores, as indicated in Table 2; however, because of constraints in traffic control, survey deck area, and mechanical failures during coring, fewer cores were secured from some of the decks.

Three companion cores were taken where one was on a crack and the other was over an uncracked surface over the adjacent rebar at the same transverse distance, labeled CR1 through CR3 and C7 through C9, respectively. Another core was taken without reinforcement for petrography and material assessments. This allowed a comparison of the diffusion parameters of cracked and corresponding uncracked surfaces. A total of 116 cores were extracted from the decks built in 1968-71 with a w/c ratio of 0.47 and bare steel reinforcement, and 300 cores were extracted from decks built in 1984-91 with a w/c ratio of 0.45 and ECR.

One bridge was not included in the data collection and coring but was surveyed for cracks. The bridge, No. 12111 (State No. 1009), from the Salem District was not included because it was a strong slab structure.

Table 2. Core Sample Descriptions and Labels

Core Description	Contain Rebar?	Surface Crack?	No. From Each Bridge	Labels
Petrographic cores	No	No	3	P1-P3
Cores with surface cracks	Yes	Yes	3	CR1-CR3
Cores through sound areas	Yes	No	3	C7-C9
Additional cores	Yes	No	6	C1-C6

Data Collection

In addition to the information relative to the deterioration state of the bridge decks, information on the quality of concrete and the corrosion state was gathered. The information included the following:

- Forty concrete cover depths were measured over the top reinforcement using a pachometer at 4-ft (1.22-m) intervals along the left and right wheel paths of the chosen survey area, normally in the right traffic lane.
- Concrete resistance was measured using a Nilsson four-point resistivity meter. Five measurements were taken at each location, and average resistivity values were calculated. Seven to nine locations were selected for each bridge deck, four to six at three-electrode linear polarization locations and three at petrographic core locations.

- Half-cell potentials were measured using a copper-copper sulfate standard half-cell at the same locations at which the cover depths were measured.
- Three-electrode linear polarization corrosion current densities, typically referred to as corrosion rates, were measured at six uncracked locations.

Laboratory Testing

Visual Assessment

Each core was visually inspected and photographed for the purpose of documentation. The surface crack widths and the crack depths at both sides of the cores were measured and recorded.

Material Testing

Disks containing reinforcement of 1-in (25-mm) thickness were cut from the cores. The concrete portions on both sides of the rebar were separated out and were labeled “A” and “B.” These pieces were tested for density, absorption, and voids, which provide moisture saturation and pore volume at the rebar depth, in accordance with ASTM C642 (ASTM, 1997a).

Chloride Content

Powdered concrete samples were obtained from cores using a drill bit 0.5 in (12.7 mm) in diameter at multiple locations directly over and parallel to the reinforcement at increments of ¼-in (6-mm) depth. Acid-soluble potentiometric titration was performed in accordance with ASTM C1152/C1152M (ASTM, 1997b). Slag-containing concrete typically contains sulfide, which will affect the chloride titration results. Thus, a 3 ml 30% hydrogen peroxide solution was added before the titration to oxidize the sulfide to sulfate. Using the chloride content depth profile, the diffusion coefficients were calculated from Fick’s second law of diffusion. In addition, electrical resistance to chloride ion penetration was determined by the rapid chloride permeability test in accordance with ASTM C1202 (ASTM, 2005).

Data Analysis

Grouping of Data

The bridges in the testing pool were built at different times using technologies and materials from different construction eras. Thus, the data were divided into appropriate groups to remove any bias and to compare the differing factors. Because of the apparent differences in the materials, construction era, and service age, the 37 bridges were divided into three subgroups:

1. *Built between 1968 and 1971, 0.47 specified w/c ratio, bare steel, no SCM—10 decks (hereinafter referred to as “1968-71 No-SCM decks”).* This subgroup was taken

from a previous study of 129 bridges (Newlon, 1974), of which 50% had not been overlaid after an average age of 33 years and, thus, is a representative sample of the better performing decks built in this era.

2. *Built between 1984 and 1991, 0.45 specified w/c ratio, ECR, no SCM—16 decks (hereinafter “1984-91 No-SCM decks”).* As a random sample of the decks built throughout Virginia, this subgroup is a representative sample of decks built in this era without SCM. The decks were about 15 years old at the time of the field survey.
3. *Built between 1984 and 1991, 0.45 specified w/c ratio, ECR, SCM (fly ash / slag)—11 decks (hereinafter “1984-91 SCM decks”).* This subgroup is representative of decks built in this era in Virginia using a SCM. The decks were about 15 years old at the time of the field surveys.

The subgrouping aided in isolating and identifying the influences of the above-mentioned parameters on crack formation. Originally, the two 0.45 w/c ratio subgroups were to be of equal size as identified from VDOT’s inventory. However, the final number of bridge decks in each category changed after the petrographic analyses.

Statistical Analysis

Statistical analyses of the data were conducted using SAS JMP software. Analysis methods included probability distribution fitting, the use of histograms, one-way analysis of variance, the matched pairs test, Student’s *t*-test, the dependence test, and regression modeling. Unless otherwise stated, the significance level for such tests was $\alpha = 0.05$ in this study.

The dependency of a factor on various parameters can be assessed using regression analysis. Once lists of parameters that were theoretically related to the dependent variable were selected from the database, stepwise regression was used to filter parameters based on significance in correlation. A combination of forward and backward selections was used. From the output of the stepwise regression, a standard least-squares regression model was formed. The R-squared value and predicted versus actual plot were used to identify statistically significant models.

The following parameters formed the pool from which the appropriate parameters were selected for regression modeling:

- environmental zone
- year built/replaced or age
- moisture saturation percentage
- diffusion coefficient
- background chloride
- reinforcement bar size
- annual average daily truck traffic
- presence of crack
- crack width range

- crack depth range
- concrete pore volume
- chloride permeability
- speed limit
- span continuity
- skew angle
- superstructure composite moment of inertia
- road classification
- geographic zone
- reinforcement type
- surface chloride
- chloride at reinforcement depth
- concrete cover depth
- surface area rust
- annual average daily traffic
- surface crack width
- crack depth
- apparent concrete density
- concrete resistivity
- span length
- load cycle
- superstructure type
- beam spacing
- sip forms.

The superstructure composite moment of inertia was calculated by using the dimensions of the decks and beams presented in the bridge plans.

RESULTS AND DISCUSSION

Literature Review

Classification of Cracks

Concrete cracks occur when the induced tensile stress exceeds the tensile strength of concrete. Cracking of concrete is typically classified based on the root cause and orientation. Classification aids in the correlation of the occurrence of types of cracks and their cause. Identification of the cause of the cracking allows for the adjustments to construction activities that are the root causes of the cracking and the selection of the proper maintenance activities to minimize their impact on the performance of the structure. The following are the classifications of cracks based on origin and orientation.

By Origin

Plastic Shrinkage Cracks. Plastic shrinkage cracks appear when the concrete is in the plastic state (Krauss and Rogalla, 1996). Cracks develop when the surface of the concrete dries because of the rapid evaporation of surface moisture before the internal moisture can rise and replace it (Schmitt and Darwin, 1995). Plastic cracks have random patterns and are found frequently in low-slump dense and latex-modified concrete rather than the regular concretes (Babaei and Hawkins, 1987). These cracks are wide at the surface and shallow in depth but may increase in size as restraints increase over time. Plastic cracks can be reduced or avoided by preventing moisture loss from the surface by using appropriate curing techniques according to the specific weather conditions.

Settlement Cracks. Settlement cracks, also called subsidence cracking, appear when the concrete continues to settle after placement and finishing, and the top reinforcement acts as a restraint, causing concrete to sag between the rebars (Schmitt and Darwin, 1995). These cracks, as is to be expected, form over and run parallel to the topmost rebars (Babaei and Hawkins, 1987). Dakhil et al. (1975) showed that the probability of settlement cracks occurring increases with a decrease in the concrete top cover, an increase in concrete slump, and an increase in rebar size. Dakhil et al. (1975) also suggested that when U.S. No. 4 or No. 5 rebars are used with a 2-in cover and concrete with a 2-in slump, the probability for settlement cracking approaches zero.

Drying Shrinkage Cracks. Drying shrinkage cracks occur when the recently cured concrete loses moisture to the environment. They appear as random and transverse cracks (Babaei and Hawkins, 1987). The drying shrinkage strain of plain, unrestrained concrete at 73 °F and 50% relative humidity can vary from 500 to 1000 $\mu\epsilon$ (Krauss and Rogalla, 1996). Krauss and Rogalla (1996) stated that in modern low permeability concrete, a large strain differential can develop because of slow diffusion of internal moisture toward the surface. Proper proportioning of the concrete mixture can reduce drying shrinkage, such as an increased aggregate proportion, which reduces shrinkage (Schmitt and Darwin, 1995).

Deflection Cracks.

Dead Load Deflection Cracks. Self-weight can cause transverse cracks over the supports (negative moment) in continuous unshored structures (Babaei and Hawkins, 1987; Krauss and Rogalla, 1996). These cracks can be prevented by including the deflection of false-works in calculations and by selecting a proper construction sequence to reduce self-weight deflections (Krauss and Rogalla, 1996).

Live Load Deflections. Live load stresses are not commonly expected to cause cracking of concrete, except in negative moment regions of certain types of bridges (Schmitt and Darwin, 1995). Krauss and Rogalla (1996) reported that heavy truck traffic appeared to lengthen existing cracks.

Vibrations. Babaei and Hawkins (1987) reported that the dynamic effects of traffic-induced vibrations can cause concrete to crack or cause existing cracks to propagate lengthwise

or depthwise. But, Krauss and Rogalla (1996) reported that bridge deck widening did not cause any problems because of vibrations, and they estimated that the maximum tensile stress in decks subjected to traffic-induced vibrations was 100 psi (0.7 MPa). Frosch et al. (2003) found through testing that transverse cracks were not influenced by the live loads or associated vibrations.

By Orientation

Concrete cracks can be generally divided based on orientation and pattern. The type of crack might indicate their cause.

Linear Cracks. Linear cracks, as the name suggests, roughly pertain to straight-line shapes. These are single continuous cracks running in a specific direction, frequently found parallel at regular intervals, and typically indicate restraints for shrinkage in the direction perpendicular to the crack (Kelly, 1981). They can be further divided into three subcategories based on their orientation with respect to the direction of traffic on the bridge deck, as follows.

Longitudinal Cracking. Longitudinal cracks can develop by differential soil settlement or restraints to transverse concrete shrinkage, perhaps by transverse composite action. These cracks form above longitudinal reinforcement in solid slab bridges and above void tubes in hollow slab bridges (Schmitt and Darwin, 1995). Notably deeper and wider longitudinal cracks have been identified in adjacent box girder bridges with differential deflection issues (Balakumaran, 2012).

Transverse Cracking. Transverse cracks running directly above the reinforcement are the result of subsidence of the concrete around the rebar. Other transverse cracks are formed mainly because of the restraints posed by the girder-deck longitudinal composite action on the shrinkage and thermal strains (Krauss and Rogalla, 1996; Saadeghvaziri and Hadidi, 2002).

Diagonal Cracking. Diagonal cracks are at a slope to the direction of traffic. They are commonly associated with the ends of skewed bridge decks and single piers (Portland Cement Association [PCA], 1970). Again, these cracks could be related to the restraints for concrete shrinkage or to external loads (Schmitt and Darwin, 1995).

Pattern Cracks. Pattern cracks including map cracks, crazing, checking, and plastic shrinkage have a random orientation (ACI Committee 201, 2008). Map cracks or pattern cracks are often uniformly distributed cracks that extend in all directions and are indicative of restraints by the inner layers of concrete on the surface (Kelly, 1981). The cracks typically are thin, spread out, and not considered hazardous to the durability of the bridge deck.

Factors Influencing Cracking

Concrete Material Properties

Slump. Slump is an indicator of the water content of the specific concrete mixture (ASTM, 2012). The PCA recommended limiting the slump to 2 to 3 in (50.8 to 76.2 mm), while

striving to use the lowest possible slump, because a higher slump would lead to settlement cracking (PCA, 1970). Schmitt and Darwin (1995) found that cracks increased as the slump increased and that the zero-slump concrete exhibited exceptional cracking in contrast to the trend. Krauss and Rogalla (1996) found that slump did not have any relationship with cracking tendency by means of restrained ring tests. Saadeghvaziri and Hadidi (2002) came to the same conclusion.

Cement Paste Content. Concrete shrinkage occurs primarily because of moisture loss from the cement paste; the aggregates resist shrinkage. This leads to the notion that increasing either the cement content or water content might lead to increased shrinkage cracks. Krauss and Rogalla (1996) reported that among specimens with a variable water and cement content, the low cement and w/c ratio specimens took the longest time to crack; the high cement, low w/c ratio specimens cracked the earliest; and the intermediate cement and w/c ratio specimens cracked at intermediate times. In addition, Xi et al. (2003) observed that concrete with a low cement content and a high w/c ratio performed well with regard to prevention of cracks.

Cement Content. When portland cement reacts with water, heat of hydration is released. A value of 110 calories per gram of cement is approximately calculated from the heat of hydration of the Type I components (Whittman, 1982), whereas Type II cement appeared to produce 75 calories of heat per gram (Krauss and Rogalla, 1996). In addition, autogenous shrinkage occurs as the cement hydrates. When the hydration reaction decreases, concrete is subjected to autogenous thermal shrinkage stresses. Thus, cement content might correlate with the formation and extent of shrinkage cracks. In a study for the Minnesota Department of Transportation (DOT), French et al. (1999) stated that higher cement contents lead to higher cracking and recommended a maximum of 650 to 660 lb/yd³ (386 to 392 kg/m³), which is the same limit suggested in a study for the New Jersey DOT (Saadeghvaziri and Hadidi, 2002). In a study for the Colorado DOT, Xi et al. (2003) suggested limiting cement content to 470 lb/yd³ (279 kg/m³). Krauss and Rogalla (1996) reported that another study by Horn et al. (1975) found no correlation between cement content and extent of cracking.

Water Content. An excessive water content may induce cracking for the same reason as increased slump, i.e., settlement cracking. Schmitt and Darwin (1995) noted an increase in cracking with an increase in water content; very high and very low water contents were associated with more cracking. Krauss and Rogalla (1996) did not find any relationship between water content and time for cracking. They recommended that the factors of water content and cement paste volume should be assessed together since the combined effects on cracking were more evident.

Water–Cementitious Material Ratio. Xi et al. (2003) found that the use of a low cement content and a relatively high w/c ratio will reduce cracking and suggested that a w/c ratio of around 0.4 be used. Schmitt and Darwin (1995) found a mild increase in cracking with an increase in the w/c ratio, but the samples represented a limited range of w/c ratios. Consistent with the effect of cement paste on cracking, both a very high and a very low w/c ratio appeared to increase cracking. French et al. (1999) came to the same conclusion and suggested that low w/c ratios be used. Saadeghvaziri and Hadidi (2002) suggested using water reducers and limiting the w/c ratio to 0.40 to 0.45. PCA (1970) recommended a maximum w/c ratio of 0.40 for severe exposure regions and 0.45 for areas with no deicer usage.

Silica Fume. Silica fume particles are 100 times smaller than portland cement particles and thus significantly increase the surface area of cementing particles, resulting in higher water demands to improve workability. The smaller particles hydrate faster and thus reduce internal water permeability. Concrete with silica fume bleeds less (Krauss and Rogalla, 1996) and thus an evaporation rate as low as 0.05 lb/ft²/hr (0.25 kg/m²/h) can lead to crack formation (Kosmatka and Wilson, 2011). Xi et al. (2003) suggested keeping the silica fume content within 5% by weight of cement.

Aggregates. Aggregates provide local restraints for cement paste shrinkage. Increasing aggregate size and content, and thus reducing the volume of cement paste, result in decreased autogenous and thermal stresses during hydration. Babaei and Hawkins (1987) found that softer aggregates yielded to concrete shrinkage and thus recommended stiffer aggregate types. However, French et al. (1999) found that aggregates with a higher modulus of elasticity will increase the overall modulus of elasticity of concrete, which will provide restraint for shrinkage. The study recommended maximizing coarse and fine aggregates. Krauss and Rogalla (1996) also recommended aggregates with a lower modulus of elasticity.

Air Content. Air entrainment in concrete reduces water content and allows the internal water to expand and contract freely during freeze-thaw conditions, thus improving durability. Schmitt and Darwin (1995) found that concrete cracking decreases with an increase in air content and that concrete with greater than 6% air content performed the best. French et al. (1999) also recommended using mixtures with an air content between 5.5% and 6.0%. However, Krauss and Rogalla (1996) did not find a significant difference in cracking tendency between the air-entrained and non-air-entrained samples.

Compressive Strength

Cracks form when the stresses exceed the tensile strength of concrete. Characteristically, an increase in compressive strength results in an increase in tensile strength. However, the advantage of higher compressive strengths and, subsequently, higher tensile strengths achieved by increasing the cement content and lowering the w/c ratio, can, most of the time, be cancelled out by causing higher amount of cracking. Many studies have reported that there is a definite increase in crack occurrence with an increase in compressive strength and recommend a limit to the early strength gain (Frosch et al., 2003; Krauss and Rogalla, 1996; Saadeghvaziri and Hadidi, 2002; Schmitt and Darwin, 1995).

Concrete Temperature

Higher temperatures can increase the rate of hydration causing early thermal stresses. Krauss and Rogalla (1996) found that the minimum concrete temperatures specified by agencies at placement were between 45 °F (7 °C) and 60 °F (16 °C) and the maximum limit on concrete temperature was typically around 90 °F (32 °C). They recommended casting the concrete 10 °F (5 °C) to 20 °F (10 °C) below the ambient temperature when the temperature is above 60 °F (16 °C).

Deck Design Parameters

Reinforcement Details

Reinforcement can act as a stress raiser near the top portion of the concrete by reducing the cross section and can lead to subsidence cracking. Using large rebars, consequently at wider spacings, will not help for the reasons mentioned here.

Top Reinforcement Size. Several instances of cracks at equal spacings found over the transverse rebars reinforce the ideas of stress raisers and subsidence. Dakhil et al. (1975) found that increased rebar sizes would result in increased subsidence cracking. In their study, Schmitt and Darwin (1995) could not come to a significant conclusion based on limited data but found an increasing trend of greater cracking with larger rebar sizes. Several other studies also recommended limiting the transverse reinforcement size (French et al., 1999).

Top Reinforcement Spacing. Reinforcement spacing, independently, might not be instrumental in the formation of cracks because cracks are typically related to the size of the rebar. French et al. (1999) showed that maximizing the rebar spacing and minimizing the rebar size will improve crack prevention. The study recommended using No. 5 at 5.5 in (140 mm) or No. 6 at 6.5 or 7 in (165 or 178 mm) spacing. Frosch et al. (2003) found that crack widths increased with increased rebar spacing and recommended using a maximum rebar spacing of 6 in (152 mm) to limit crack width at 16 mils (406 μm).

Concrete Cover Depth

Concrete cover over the top reinforcement can delay the onset of corrosion by chloride diffusion according to Fick's second law of diffusion. Subsidence cracking is less pronounced with more cover, although the longitudinal shrinkage reinforcement cannot effectively counter concrete shrinkage with a higher cover. Schmitt and Darwin (1995) found that a cover of 2.5 in (64 mm) showed the least cracking compared to a cover of 2 or 3 in (51 or 76 mm) but could not come to a definite conclusion.

Deck Thickness

Higher differential shrinkage can be expected in relatively thicker decks, whereas thicker decks can store more moisture that can reduce the drying out of concrete. French et al. (1999) recommended avoiding thin decks (159 mm or 6.25 in), since increased cracking was observed. Krauss and Rogalla (1996) found that thinner decks cracked more and suggested a minimum thickness of 8 to 9 in (203 to 229 mm), whereas Xi et al. (2003) suggested a thickness of at least 8.5 in (216 mm). ElSafty and Jackson (2012) conducted a finite element modeling study and found that increasing deck thickness can reduce deck stresses.

Bridge Design Factors

Superstructure Stiffness

Larger girders spaced closer together increase the stiffness of a superstructure, which might lead to a higher degree of restraint for shrinkage of concrete. Since longer span lengths typically involve the use of larger girders, such spans are likely to undergo higher transverse cracking (Krauss and Rogalla, 1996). For this reason, Xi et al. (2003) recommended the use of smaller girders. French et al. (1999) recommended reducing girder restraints by increasing girder spacing. They found that when girders were observed to be more flexible, the stiffness from cross frames and splices to the overall superstructure stiffness were more evident; otherwise, they did not contribute significantly.

Span Length

Schmitt and Darwin (1995) did not find any influence of span lengths on cracking intensity. In addition, Krauss and Rogalla (1996) found that span length has only a minor influence on cracking. In the finite element analysis study by ElSafty and Jackson (2012), increasing span lengths did not influence tensile stresses except under truck loading.

Composite Action

Composite action is a preferred design method since it increases structural stiffness and strength while saving in terms of material costs. Krauss and Rogalla (1996) found that increased composite action can increase the likelihood for transverse cracking; however, using non-composite design will not necessarily help since it usually involves large girders. French et al. (1999) concluded that superstructures with appreciable shear stud restraints had extensive transverse cracking and thus recommended using fewer rows of shear studs and smaller and shorter studs.

Continuity / End Condition

Continuity in superstructure design can increase the restraint against concrete shrinkage, which is known to lead to crack formation. French et al. (1999) recommended using simply supported spans or expansion joints in continuous spans to reduce excessive restraints. Schmitt and Darwin (1995) found that when the end conditions were fixed, intense cracking was observed, especially near the supports.

Girder Material

The specific heat capacity of concrete is higher than that of structural steel; thus, steel tends to heat up faster than concrete when subjected to an equal amount and rate of heat energy. This thermodynamic property indicates that the thermal expansion of a concrete deck and steel girders might not be in synchronization, resulting in shrinkage restraints. Several studies found increased cracking in concrete decks supported by steel girders compared to concrete girders (French et al., 1999; Krauss and Rogalla, 1996; Xi et al., 2003).

Skew

The skew of a bridge is the angle between the line parallel to the abutment and the line running perpendicular to the direction of traffic. Schmitt and Darwin (1995) did not find any relationship between crack intensity and skew for monolithic decks, although it was suspected that increasing skew angle might increase cracking for two-layer decks. Other studies did not find any influence of skew angle on cracking (Krauss and Rogalla, 1996; Saadeghvaziri and Hadidi, 2002).

Stay-in-Place Forms

SIP forms, which are left in place for quicker construction, are alternatives to conventional removable forms. Cady and Carrier (1971) found less cracking in decks with SIP forms, and the increased moisture availability to the concrete was stated as the reason. In contrast, other studies found that SIP forms can introduce higher differential shrinkage in the cross section by both moisture profile disparity and shrinkage restraint at the bottom layers of concrete (Krauss and Rogalla, 1996; Frosch et al., 2003).

Construction Practices

Surface Finishing

Xi et al. (2003) recommended completing surface finishing and texturing as soon as possible to allow the final cure of the concrete. Krauss and Rogalla (1996) found that mechanical grooving causes less damage to the concrete compared to rake tining.

Curing

Proper curing of concrete is essential for development of strength prevention of shrinkage crack formation. Ineffective curing was the most common reason reported by transportation agencies for excessive cracking in the PCA study (PCA, 1970). The literature suggested that the curing water over hardened concrete should not be cooler than the concrete by more than 20 °F (11 °C) (Kosmatka and Wilson, 2011). Krauss and Rogalla (1996) reported that delayed curing increased the number of cracks. Further, they found that curing was more important with mixtures with a high cement content and a low w/c ratio.

Environmental Conditions

Air Temperature

Placing concrete in hot weather can induce thermal stresses in addition to drying shrinkage stresses. Krauss and Rogalla (1996) stated that concrete in a plastic condition can adjust to changing air temperatures without developing stresses but that temperature changes can induce stresses in hardened concrete. They also reported that the maximum air temperatures at the time of casting, specified by the state transportation agencies, varied from 80 °F to 90 °F (27

°C to 35 °C) and the minimum air temperatures ranged from 35 °F to 50 °F (2 °C to 10 °C) for proper curing.

Evaporation Rate

Kosmatka and Wilson (2011) advised that when the rate of evaporation is greater than 0.2 lb/ft²/hr (1 kg/m²/h), provisions such as wind-screening or fogging are necessary. Also, they specified that cracking is possible at evaporation rates greater than 0.1 lb/ft²/hr (0.5 kg/m²/h).

Chloride Diffusion Parameters

Surface Chloride

Surface chloride concentration can be defined as the amount of chlorides that has diffused from the outer (top, in the case of a deck) surface, which is likely the maximum amount at a one-dimensional depth unless cracking leads to higher chloride accumulation near the bottom of the crack. Usually, the top 1/8 to 1/4 in of concrete is discarded because of the inconsistency associated with that depth and the background chloride content will be excluded. This parameter is used in the diffusion model to predict the service life of the bridge decks. No correlation between surface chloride concentration and diffusion coefficients has been reported (Balakumaran, 2012). Depending on the pore volume and saturation of concrete, only a certain amount of chlorides can diffuse, irrespective of the surface chlorides.

Diffusion Coefficient

Diffusion coefficient is back-calculated from Fick's second law of diffusion using known chloride concentrations at various depths of deck concrete and age of sample. This parameter represents the rate of chloride diffusion and the quality of the concrete and is used in the service life modeling. The influence of cracks on diffusion coefficients needs investigation so that the effect of cracks on the durability of bridge decks can be understood.

Crack Width and Depth

It is intuitive to assume that increased crack widths would allow increased diffusion of chlorides into the concrete. But some researchers do not think that crack widths have any significant influence over chloride diffusion, whereas some have reported that there is a crack width threshold below which diffusion is not different from that in uncracked surfaces. The *AASHTO LRFD Bridge Design Specifications* attempt to keep concrete crack widths below 0.017 in (0.43 mm) by controlling the reinforcement spacing (American Association of State Highway and Transportation Officials, 2012). Xi et al. (2003) reported that the critical crack width is between 0.1 mm (0.004 in) and 0.2 mm (0.008 in). Krauss and Rogalla (1996) reported observation of water leakage through cracks as narrow as 0.002 in (0.05 mm). Ismail et al. (2008) mechanically induced cracks in concrete specimens in the laboratory. They concluded that chloride diffusion is independent of cracking when cracks are wider than 0.2 mm (0.008 in); no diffusion occurred through cracks narrower than 0.06 mm (0.002 in). Mangat and Gurusamy (1987) stated that cracks with surface widths less than 0.2 mm (0.008 in) will not contribute to

increased chloride diffusion. In their study, increased crack widths did allow increased chloride diffusion, but the effect became significant when the cracks were wider than 0.5 mm.

Rodriguez and Hooton (2005) conducted a laboratory investigation of crack specimens with smooth and rough surfaces, with OPC and a 25% slag mixture with a w/c ratio of 0.40. Crack openings ranged from 0.003 to 0.027 in (0.09 to 0.69 mm). Water-cured saturated specimens were immersed in a 2.82 mol/L chloride solution. The results showed that the chloride penetration of the cracks is a two-dimensional diffusion mechanism. The average diffusion constants were 403 mm²/yr (0.625 in²/yr) and 98 mm²/yr (0.152 in²/yr) for the OPC and slag mixtures, respectively. Also the chloride penetration was independent of crack width.

Yoon (2012) conducted a laboratory study on OPC concrete and high performance concrete mixtures with a w/c ratio of 0.50 and 0.20, respectively. The depth of cracks induced in the specimens ranged from 0.00078 to 0.00472 in (0.02 to 0.12 mm), and the specimens were tested at 28 days after 2 days of moist curing and 26 days at a relative humidity of 65%. The cracked surface was exposed to chloride using the Nordtest rapid chloride migration test method. For the high performance concrete, the higher chloride ingress protection is significantly lost once the concrete cracks. The rate of chloride diffusion increased with increasing crack width and decreased with increasing crack depth. With increasing tortuosity of the cracks, the chance of being unconnected at the crack tips increases, thus restricting chloride ingress.

Kato et al. (2005) assessed and subsequently modeled the chloride ingress characteristics of cracked concrete for continuous wet and wet/dry exposures with concrete mixtures with a w/c ratio of 0.50 and 0.39; cracks were induced in the concrete beams using the four-point loading method after 28 days of moist curing. A chloride solution of 3% NaCl was used for both chloride exposure conditions. Crack widths ranged from 0.00275 to 0.00394 in (0.07 to 0.10 mm) before testing and the cracks closed to a range of 0.00118 to 0.00275 in (0.03 to 0.07 mm) after testing. Modeling results showed that the concrete chloride content decreased with increasing depth and increasing lateral distance from the crack surface for both chloride exposure conditions. The modeling also showed that the chloride content of the concrete increased from a surface crack width of 0.00078 to 0.00295 in (0.02 to 0.075 mm) but remained relatively constant from a width of 0.00295 to 0.00787 in (0.075 to 0.20 mm), regardless of the crack depth, which ranged from 1.18 to 3.54 in (30 to 90 mm).

Crack Repair

Cracks are generally repaired for both aesthetic and durability reasons. Different agencies follow different guidelines for deciding if cracks need to be repaired and, if so, which kind of cracks needs to be repaired. In the survey by ElSafty and Jackson (2012), 60% of the states did not have a crack-sealing program and 24% used epoxies and methacrylates. VDOT's *Guide Manual for Causes and Repair of Cracks in Bridge Decks* states that cracks that are greater than 0.2 mm (0.0079 in) in width about 6 months after casting must be repaired (VDOT, 2009).

Pattern cracks are typically filled by flooding with gravity fill polymers. Linear cracks may be repaired by gravity fill polymers including epoxies, methyl methacrylates, high

molecular weight methacrylates, and polyurethanes; carbon fiber mesh; and epoxy injection. The decision regarding the repair method is made based on crack type, crack frequency, and whether the cracks are active or passive. In a highly cracked deck, a thin bonded epoxy overlay might be necessary to increase durability. However, addressing the active cracks is a necessary step before using any type of overlaying; otherwise, the overlays would simply reflect the cracks after a period of time. Typically, active cracks are reflected in deck overlays.

Bridge Deck Studies

Concrete Cover Depths

Concrete cover over steel reinforcement is the foremost element in delaying chloride-induced corrosion damage. Figure 2 shows the distributions of cover depths for the subgroups, as measured with a magnetic pachometer.



Figure 2. Histograms of Concrete Cover Depths: (a) 1968-71 No-SCM Decks; (b) 1984-91 No-SCM Decks; (c) 1984-91 SCM Decks

By visual observation, these histograms appear like normal distributions. The Shapiro-Wilk test for goodness of fit was used to determine if the distributions were statistically normal. As shown in Figure 2, cover depths from the structures built in 1984-91 are normally distributed, whereas the distributions of those from the older structures were not statistically normal (based on p -value = 0.0429 versus the significance level, $\alpha=0.05$), although the shape of the distribution appears to be approximately normal. Mean values indicate that the newer structures were built with greater clear concrete cover depths. In the 1968-71 era, the specification for clear concrete cover depth was nominally 1.69 in, more specifically, 2 in minus one-half of the diameter of the top rebar (typically No. 5). The 1984-91 era cover depth specification was 2.5 in with an acceptable tolerance of 0.5 in, for a nominal average clear concrete cover depth of 2.75 in. This would represent an expected increase in average cover depth of approximately 1 in. However, a comparison of the means of the subgroups in Figure 2 shows that the change in cover depth specification resulted in the increase in clear cover of about 0.4 in for this set of bridges.

Surface Chloride

Figure 3 presents the distribution of surface chloride concentrations for the three subgroups.

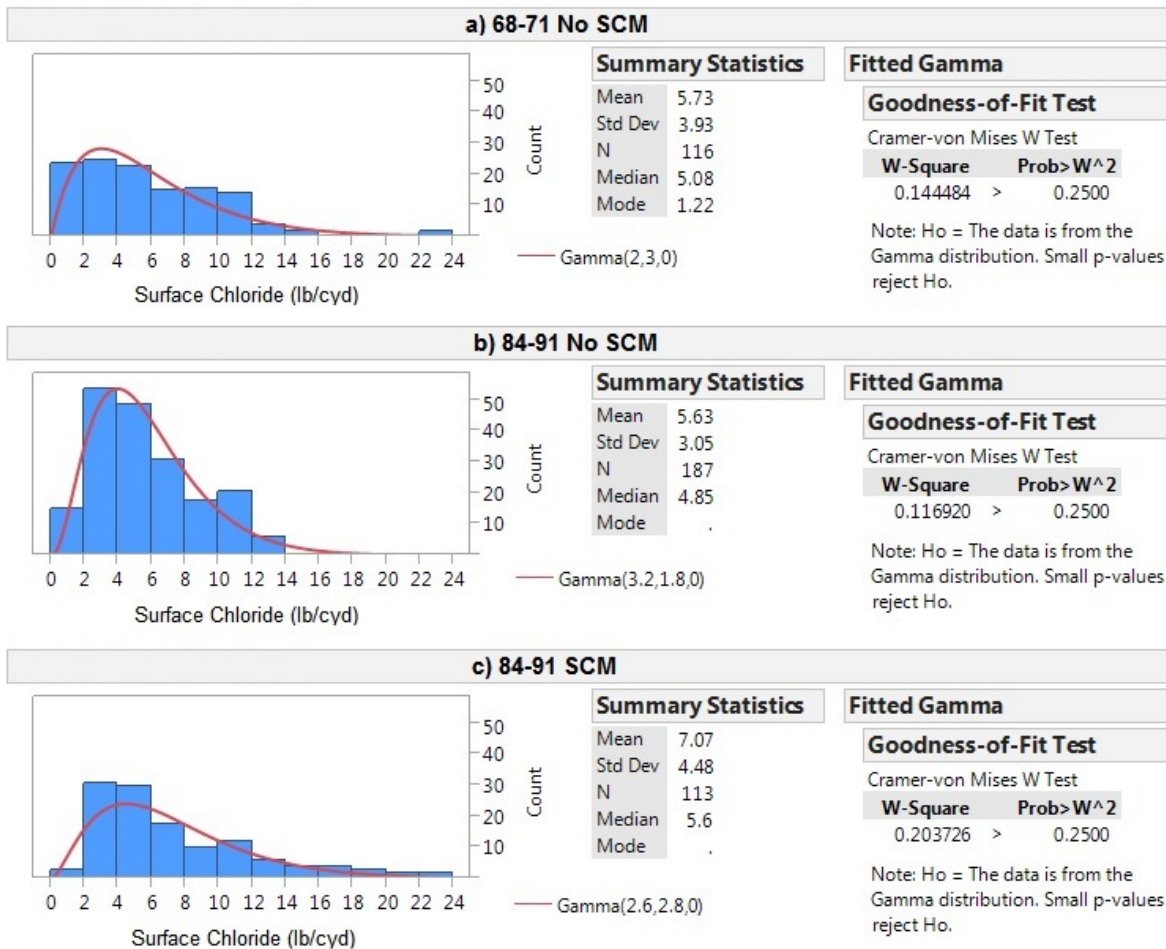


Figure 3. Histograms of Surface Chlorides: (a) 1968-71 No-SCM Decks; (b) 1984-91 No-SCM Decks; (c) 1984-91 SCM Decks

Diffusion Coefficients

For each core sample location, the distribution of chloride concentrations as a function of depth was analyzed by curve fitting a one-dimensional solution of Fick's second law of diffusion to determine the effective diffusion coefficient over the period the deck has been in service. Figure 4 presents the distribution of diffusion coefficients for three subgroups.

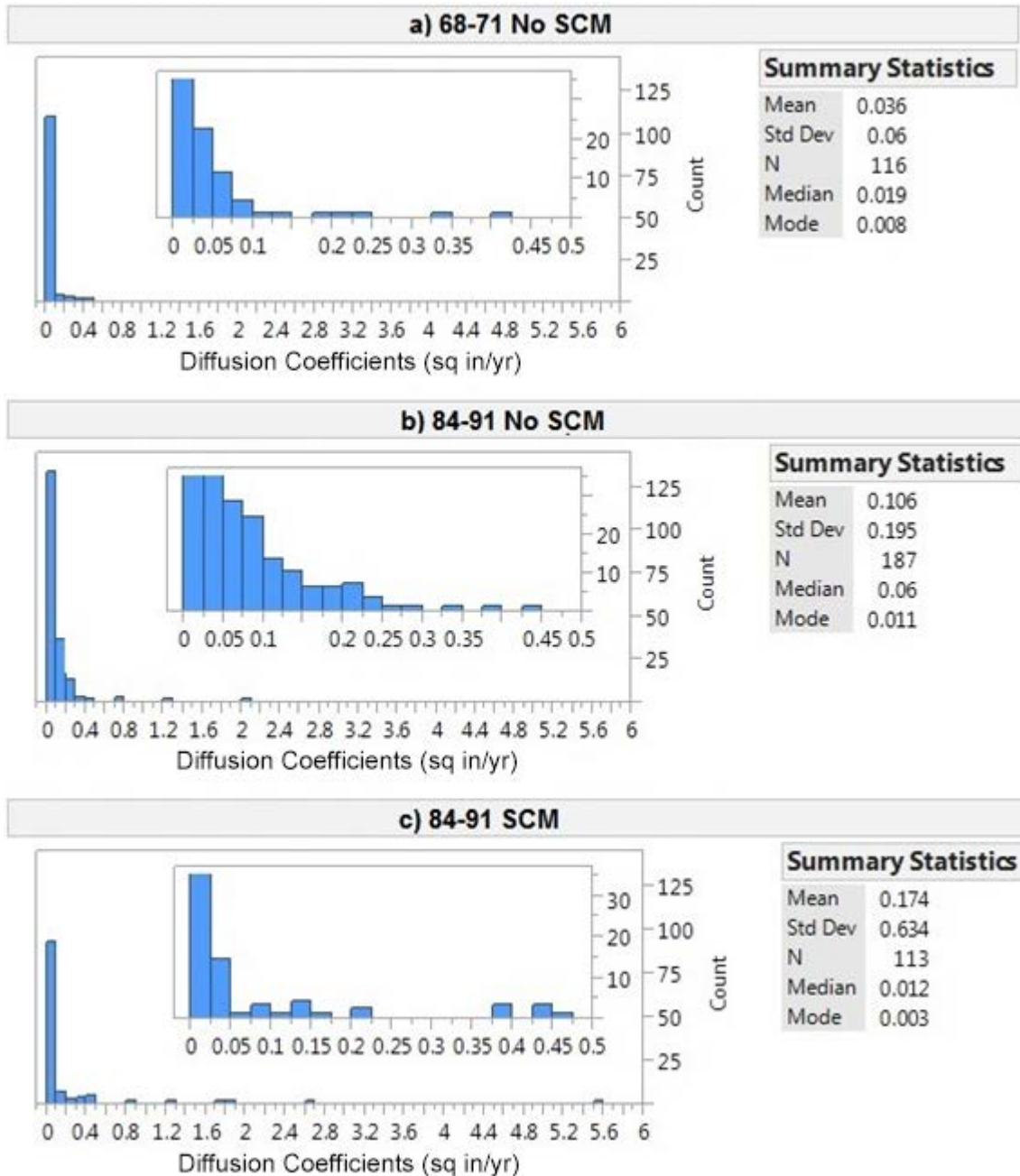


Figure 4. Histograms of Diffusion Coefficients: (a) 1968-71 No-SCM Decks; (b) 1984-91 No-SCM Decks; (c) 1984-91 SCM Decks

Distribution characteristics and the magnitude of surface chloride concentrations do not vary greatly among the subgroups. Histograms may conform to gamma distribution functions. A Cramer-von Mises test for goodness of fit was used to determine if the histograms are gamma distributions. As shown in Figure 3, surface chlorides from all three groups of structures are gamma distributions. The parameters for the gamma distributions are presented to the right of the histograms in the form of Gamma (x, y, z). These represent the shape, scale, and threshold for the equations, respectively.

Since the majority of observed diffusion coefficients is at the low end of the scale, an inset histogram is provided for distributions of coefficients less than $0.5 \text{ in}^2/\text{yr}$ ($323 \text{ mm}^2/\text{yr}$) as a subset of each subgroup. High variability prevents fitting of probability density distributions, although by visual observation the distributions appear to be lognormal.

As shown in Figure 3, the 1968-71 No-SCM subgroup has a significantly lower mean diffusion coefficient ($0.036 \text{ in}^2/\text{yr}$) than the 1984-91 No-SCM ($0.106 \text{ in}^2/\text{yr}$) and the 1984-91 SCM ($0.174 \text{ in}^2/\text{yr}$) subgroups. The histograms represent the diffusion coefficients of uncracked and cracked deck areas combined. Thus, a direct comparison of the histogram results would not be a simple reflection of the abilities of the respective bulk (intact) concretes to resist the ingress of chloride.

Table 3 presents the summary statistics for the three subgroups. Observations include a small number of outliers, 14 of 416 values that were removed for calculating the basic statistics. The number of observations for all six conditions is sufficiently large to be representative of field conditions.

Interestingly, the variability of all six subsets shown in Table 3 is very high, with coefficients of variation greater than 100%. The average diffusion coefficients for the cracked conditions were significantly higher than those for the uncracked conditions. The ratios of diffusion coefficients, cracked to uncracked, were 4.5, 3.0, and 10 for the 1968-71 No-SCM, 1984-91 No-SCM, and 1984-91 SCM deck subgroups, respectively.

The rank order of diffusion coefficients for the uncracked condition from lowest to highest is as follows:

1. 1984-91 SCM decks
2. 1968-71 No-SCM decks
3. 1984-91 No-SCM decks.

For the cracked condition, the ranking is as follows:

1. 1968-71 No-SCM decks
2. 1984-91 SCM decks
3. 1984-91 No-SCM decks.

Table 3. Statistical Parameters for Diffusion Coefficients for Uncracked and Cracked Samples Within Deck Subgroups

Parameter	Subgroup					
	1968-71 No-SCM		1984-91 No-SCM		1984-91 SCM	
	Uncracked	Cracked	Uncracked	Cracked	Uncracked	Cracked
Average, in ² /yr (mm ² /yr)	0.020 (13)	0.091 (59)	0.060 (39)	0.18 (116)	0.014 (9)	0.14 (90)
Std. Dev., in ² /yr (mm ² /yr)	0.027 (17)	0.102 (66)	0.070 (45)	0.21 (135)	0.023 (15)	0.15 (97)
Range, in ² /yr (mm ² /yr)	0.001-0.19 (1-123)	0.02-0.42 (13-271)	0.001-0.26 (1-168)	0.03-0.78 (19-503)	0.001-0.14 (1-90)	0.009-0.47 (6-303)
Number of observations	91	25	137	49	80	32
Portion of subgroup, %	78	22	74	26	73	27
Number of outliers	0	0	1	2	3	5

Chlorides at Depth of Top-Mat Reinforcement

Chloride concentrations in the concrete at the level of the rebars can be useful to assess the relationship between chloride content and corrosion and also can indicate the rate of chloride diffusion from the surface. Figure 5 presents the histograms for the three subgroups.

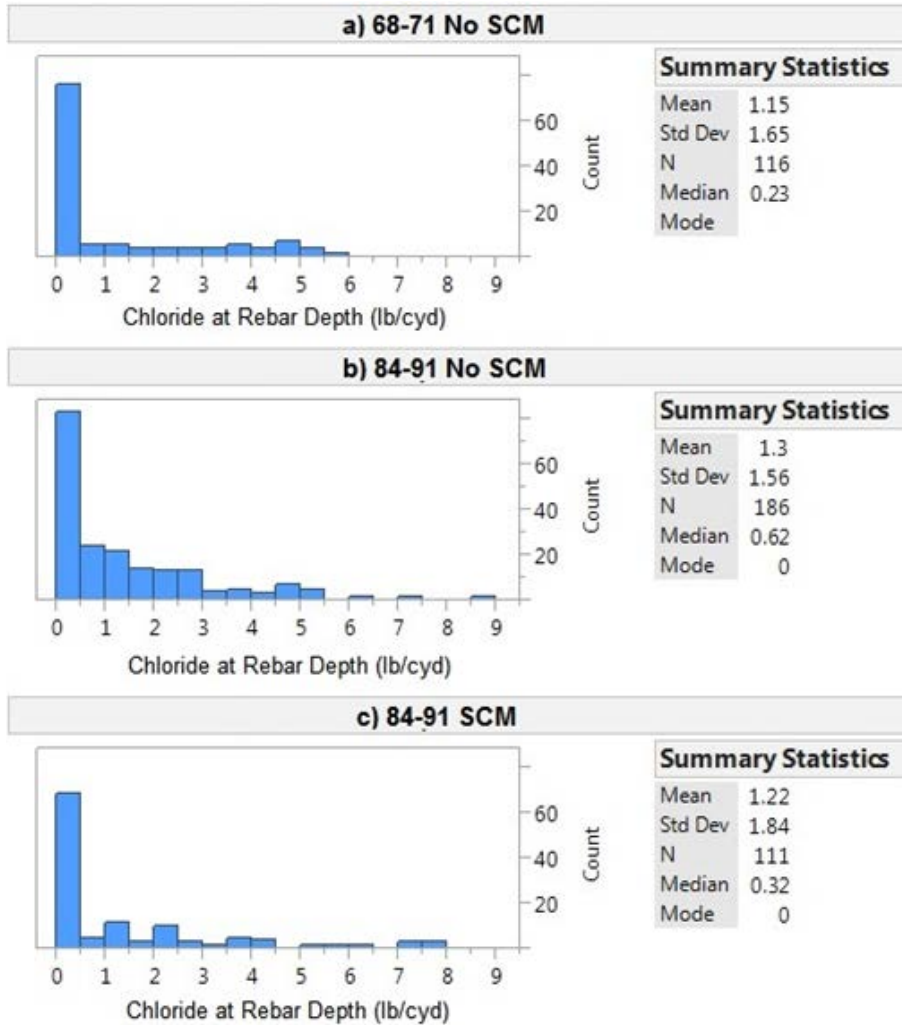


Figure 5. Histograms of Chloride at Reinforcement Depth: (a) 1968-71 No-SCM Decks; (b) 1984-91 No-SCM Decks; (c) 1984-91 SCM Decks

Ranges and magnitudes of chloride concentrations at the depth of the top-mat reinforcement did not appear to vary greatly among the subgroups, even though the 1968-71 No-SCM subgroup was about 18 years older. The histograms in Figure 5 represent the combined total of the uncracked and cracked conditions. Thus, a direct comparison of the histogram results is not appropriate and may result in incorrect observations.

Cracking Frequency

The crack survey results, in terms of linear feet of cracks per square foot of visually surveyed deck area, were separated by crack orientation relative to the direction of traffic: transverse cracks, longitudinal cracks (parallel to the direction of traffic), and diagonal cracks. The basic statistics regarding crack frequencies are presented in Table 4. The limited number of data points did not allow distribution functions to be fit to the data.

From Table 4, the 1984-91 No-SCM decks had cracked more than the decks in the other two subgroups. A single outlier deck with higher longitudinal cracking created bias in the statistical terms, so the values after the outlier was removed are presented in a separate column and represent a significant decrease in mean total cracking and closer agreement with the other two subgroups.

Table 4. Basic Statistics of Crack Frequencies

Statistic	Subgroup			
	1968-71 No-SCM N = 10	1984-91 No-SCM N = 16	1984-91 SCM N = 11	
Longitudinal Crack Frequency (ft/ft²)				
Mean	0.059	0.259	0.163 ^a	0.100
Std. Dev.	0.075	0.402	0.121 ^a	0.085
SEM	0.024	0.100	0.031 ^a	0.026
Median	0.028	0.145	0.139 ^a	0.080
Transverse Crack Frequency (ft/ft²)				
Mean	0.110	0.073		0.079
Std. Dev.	0.151	0.090		0.055
SEM	0.048	0.023		0.017
Median	0.038	0.034		0.074
Diagonal Crack Frequency (ft/ft²)				
Mean	0.022	0.007		0.013
Std. Dev.	0.041	0.011		0.038
SEM	0.013	0.003		0.012
Median	0.003	0.000		0.000
Total Crack Frequency (ft/ft²)				
Mean	0.191	0.340	0.246 ^a	0.192
Std. Dev.	0.165	0.415	0.187 ^a	0.093
SEM	0.052	0.104	0.048 ^a	0.028
Median	0.135	0.217	0.213 ^a	0.176

SEM = standard error of the mean.

^a After one outlier was removed (1.7 ft/ft²).

Chloride Permeability

The ability of concrete to permit or resist chloride ingress is often referred to as permeability, which was determined in accordance with ASTM C1202. Since specimens need to be 2 in thick for this test, only a few selected uncracked core specimens that qualified were used. Figure 6 presents the distribution of the electrical charge passed, which is indicative of the chloride permeability of cores from each bridge deck. The decks built with SCM had significantly lower permeability to ions compared to that of the other groups.

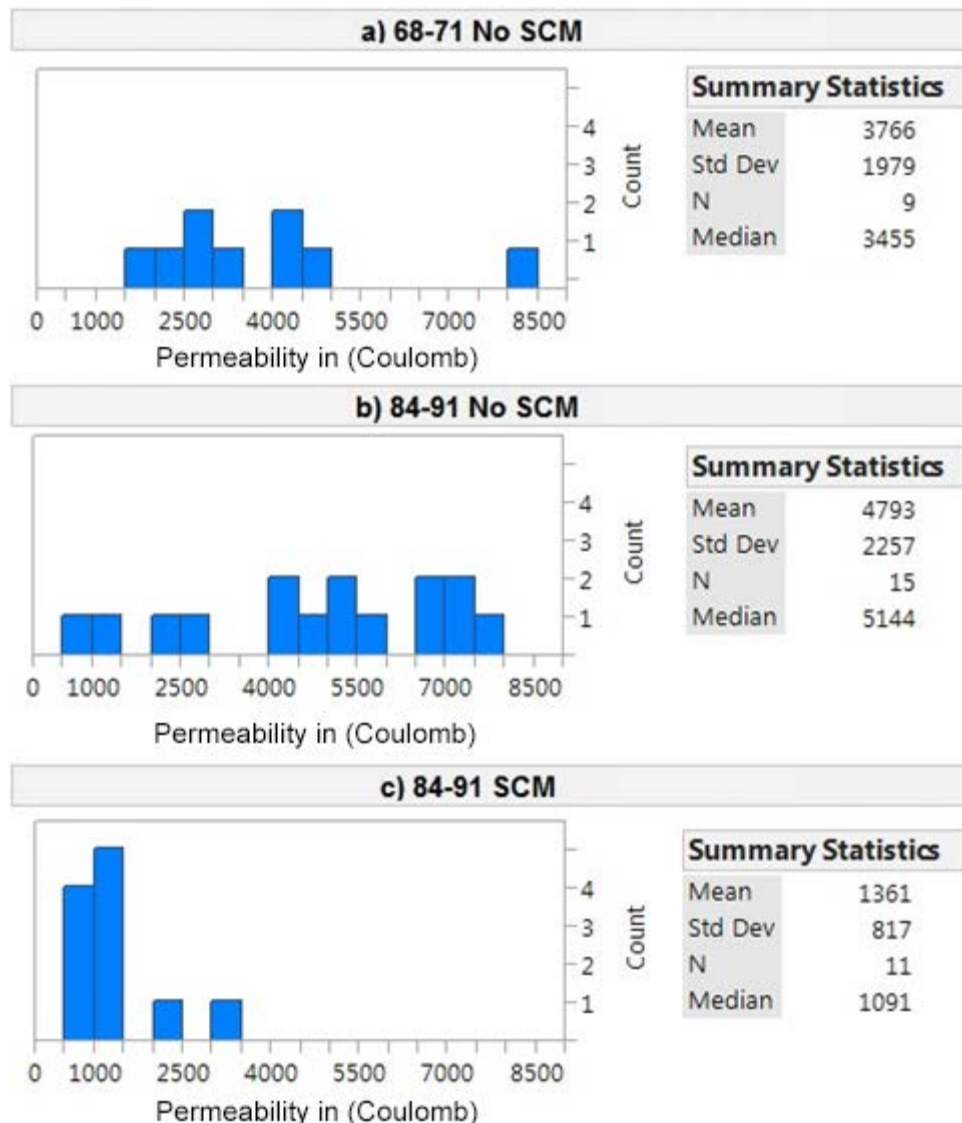


Figure 6. Histograms of Chloride Permeability: (a) 1968-71 No-SCM Decks; (b) 1984-91 No-SCM Decks; (c) 1984-91 SCM Decks

Concrete Pore Volume

Concrete pore volume is related to the performance of concrete; higher pore volumes may allow higher chloride ingress rates, resulting in earlier corrosion damage. Concrete with a

saturated pore system will not fare well against “freeze-thaw” expansion of water in improperly air-entrained concrete. Figure 7 shows the distribution of the pore space volumes of cores from all subgroups, as determined in accordance with ASTM C642.

After one bridge deck that showed a pore volume of more than 25% was omitted, all values remained below 19%. The mean pore volume of the 1984-91 No-SCM subgroup was greater than that of the other two subgroups, again indicating lower quality concrete.

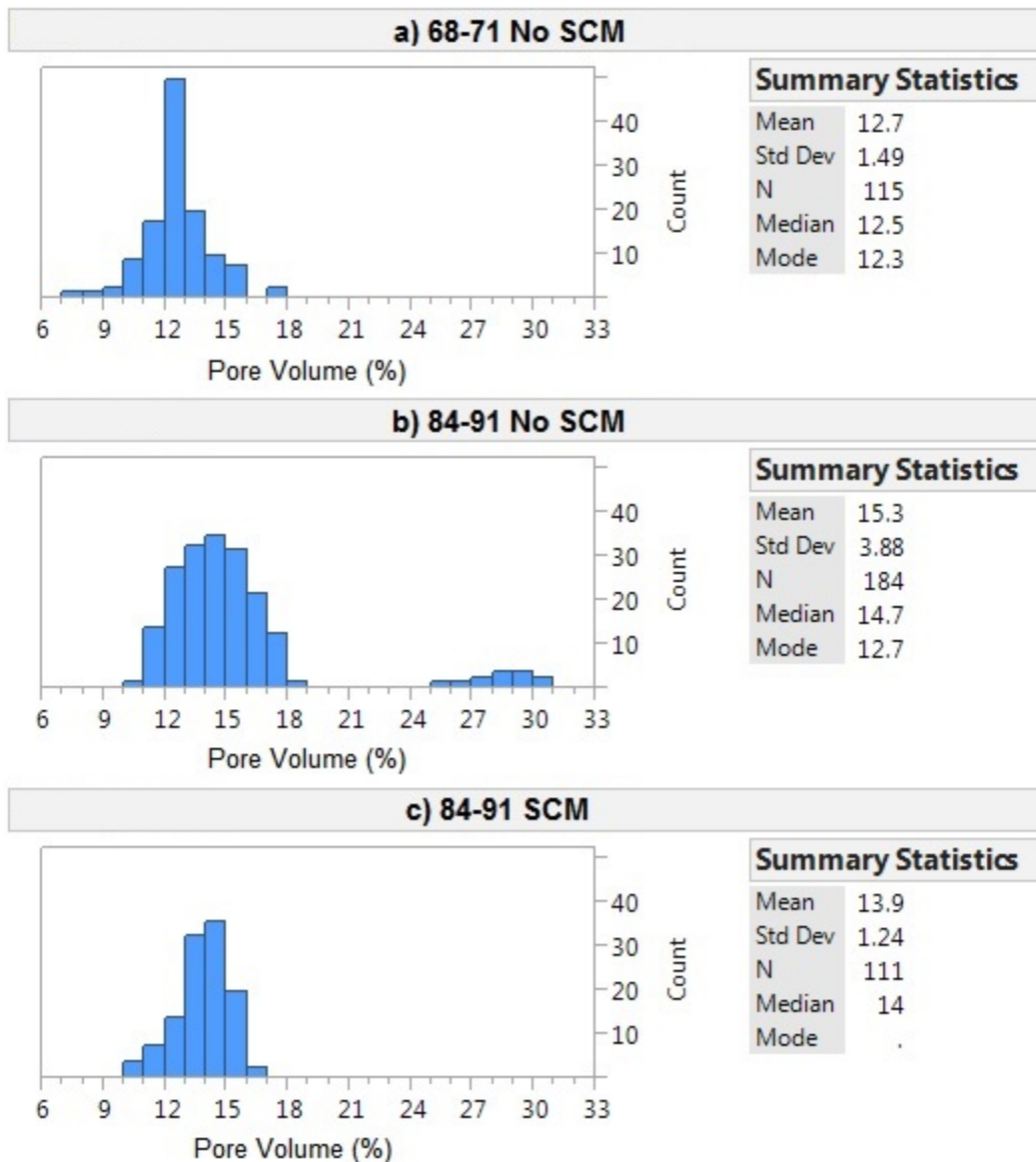


Figure 7. Histograms of Concrete Pore Volume: (a) 1968-71 No-SCM Decks; (b) 1984-91 No-SCM Decks; (c) 1984-91 SCM Decks

Concrete Moisture Saturation

Figure 8 presents the distribution of the percentage of moisture saturation for the three subgroups. Percent saturation of the concrete was determined from sliced concrete samples at the rebar depth in accordance with ASTM C642 (ASTM, 1997a). The decks built with SCM had a higher degree of saturation. This is not necessarily an indicator of poorer quality concrete but rather of the fineness of the pore system. Concrete with a fine pore system or smaller pores will hold proportionally more water than concretes with larger pores, which dry out quicker.

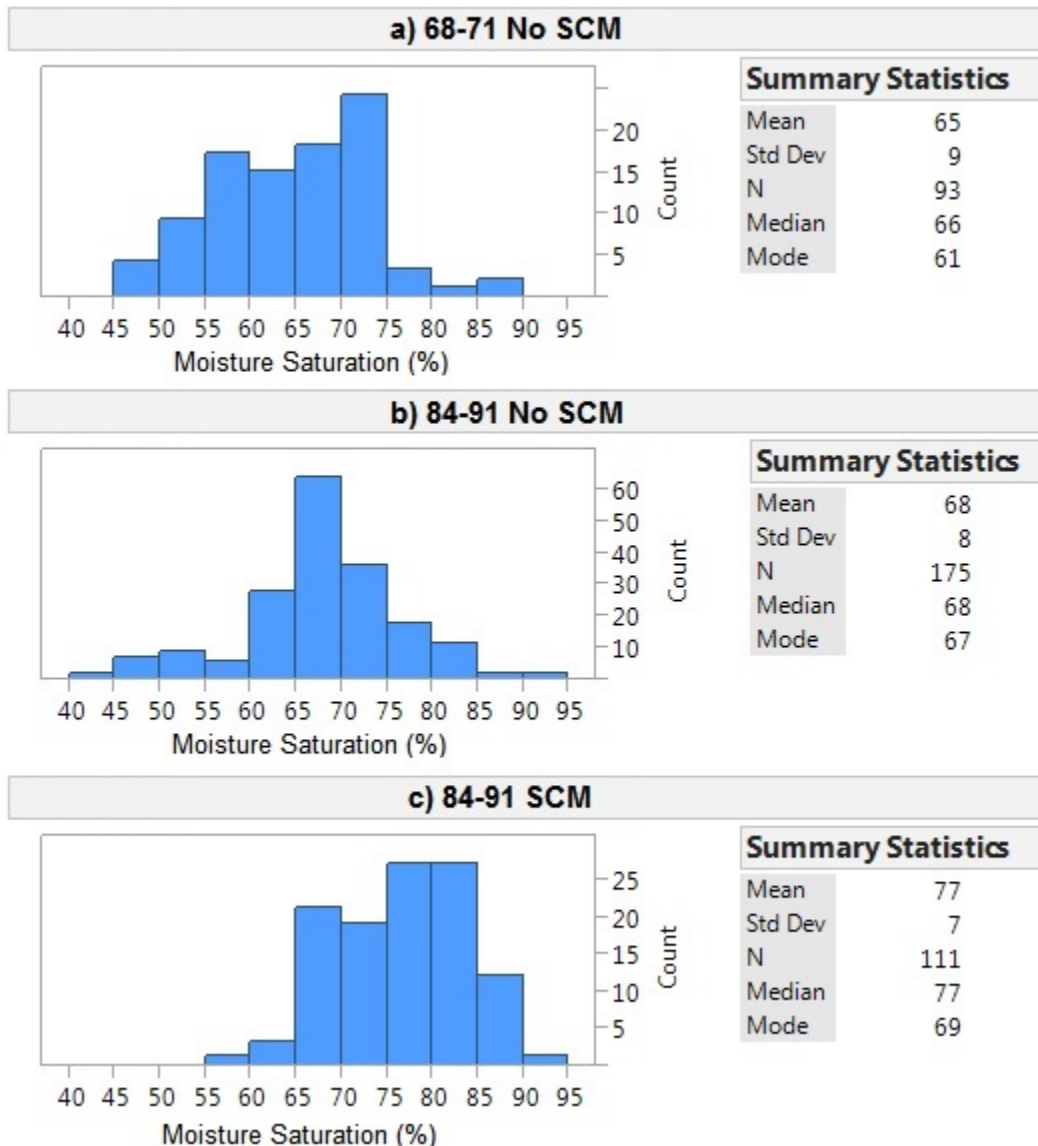


Figure 8. Histograms of Moisture Saturation: (a) 1968-71 No-SCM Decks; (b) 1984-91 No-SCM Decks; (c) 1984-91 SCM Decks

Analysis

Crack Dimensions

The measured widths and depths of cracks were divided into ranges for identifying the influence on various diffusion parameters. VDOT specified that cracks wider than 0.2 mm (0.007 in) should be repaired. However, only a few cracks were narrower than 0.2 mm (0.007 in). Schiessl and Raupach (1997) concluded in a laboratory study where cracks of widths 0.1 mm (0.004 in), 0.2 mm (0.007 in), 0.3 mm (0.012 in), and 0.5 mm (0.02 in) were studied that crack widths from 0.3 to 0.5 mm (0.012 to 0.020 in) allowed more corrosion. However, concrete quality and cover depth were factors of more importance. This prompted a splitting of the dataset into three crack width ranges: less than 0.3 mm (0.012 in), 0.3 to 0.5 mm (0.012 to 0.020 in), and greater than 0.5 mm (0.020 in). Crack depth is important, as rebars may be exposed. The crack depths were divided into three ranges: less than 1 in (25.4 mm), representing shallow cracks; 1 to 2 in (25.4 to 50.8 mm), representing medium cracks; and greater than 2 in (50.8 mm), representing cracks closer to the top reinforcement mat.

Independence of Crack Width and Crack Depth

Determining the depth of cracks is not easy, whereas the surface width of cracks can be more easily measured. To determine if crack depth and width were correlated, Pearson’s chi-square test was conducted, as shown in Figure 9.

The relationship between the surface crack width ranges and crack depth ranges is shown as a mosaic plot in Figure 9. The dimensions of the tiles are proportional to the number of samples in each of the permutations. For example, there were 33 samples with cracks narrower than 0.3 mm and shallower than 1 in. A trend of increasing crack depth with increasing crack width is shown, and Pearson’s chi-square test showed that surface crack width and crack depth are dependent that ($p = 0.0107 < 0.05 = \alpha$) with a weak R^2 of 0.06, which means that crack width explained only 6% of the variance in crack depth.

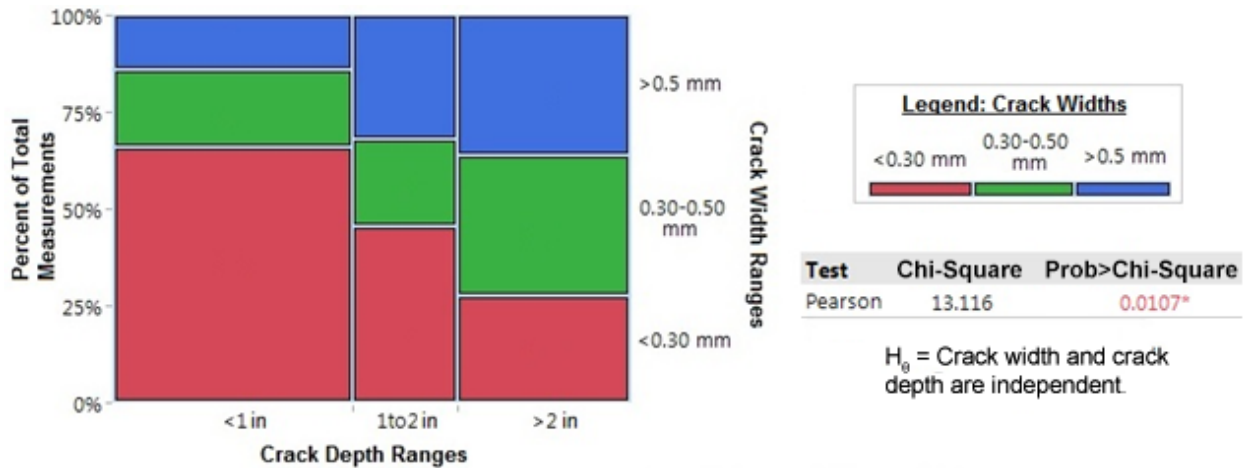


Figure 9. Independence Test on Crack Width and Crack Depth

Crack Types and Formation

Figure 10 shows the composition of the cracks by orientation on an area plot for each bridge deck. It can be seen that longitudinal cracks occupied 55% of the total crack length, transverse cracks occupied about 40%, and diagonal cracks occupied the remaining 5%.

Figure 11 presents a scatter plot of diffusion coefficients less than $400 \text{ mm}^2/\text{yr}$ and corresponding surface crack widths. It can be seen that crack widths even narrower than 0.15 mm (0.006 in) corresponded to diffusion coefficients that are comparable to crack widths up to 0.5 mm (0.02 in). It seems that surface crack widths wider than 0.5 mm (0.02 in) result in lower diffusion coefficients, which might be due to washing action from runoff water; however, the presence of fewer wider cracks becomes evident from the histogram to the right in Figure 11.

Figure 12 presents a scatter plot of diffusion coefficients less than $400 \text{ mm}^2/\text{yr}$ and corresponding crack depths. The very wide spread prevented identification of any correlation.

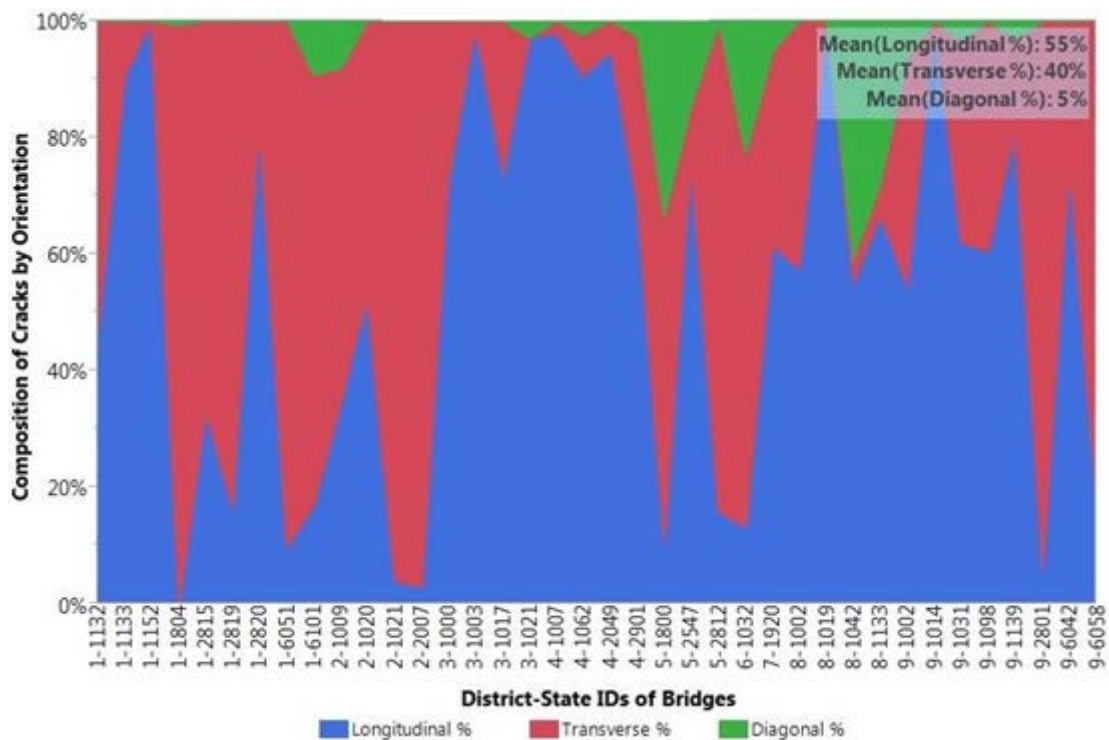


Figure 10. Cracks Based on Orientation

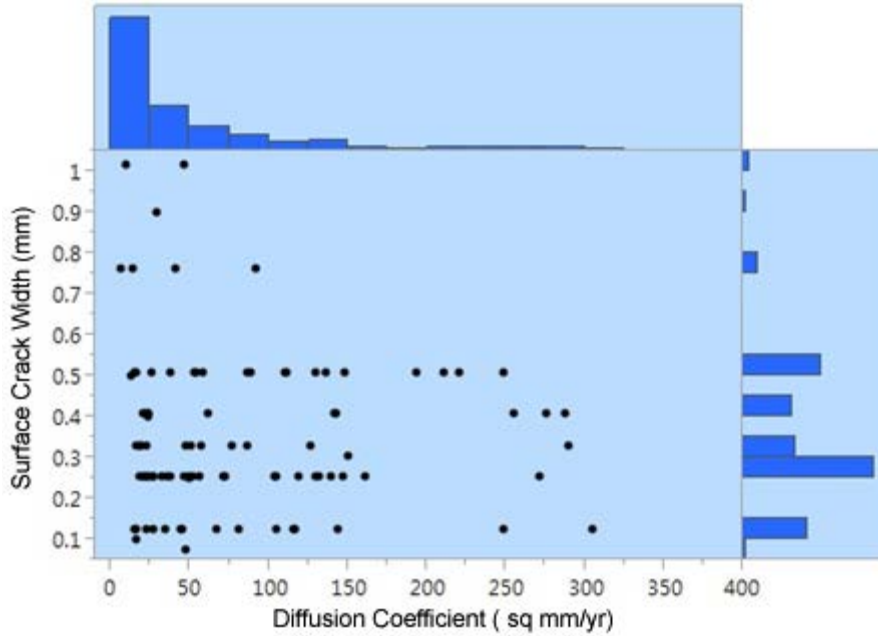


Figure 11. Scatter Plot and Histograms for Diffusion Coefficient Versus Surface Crack Width

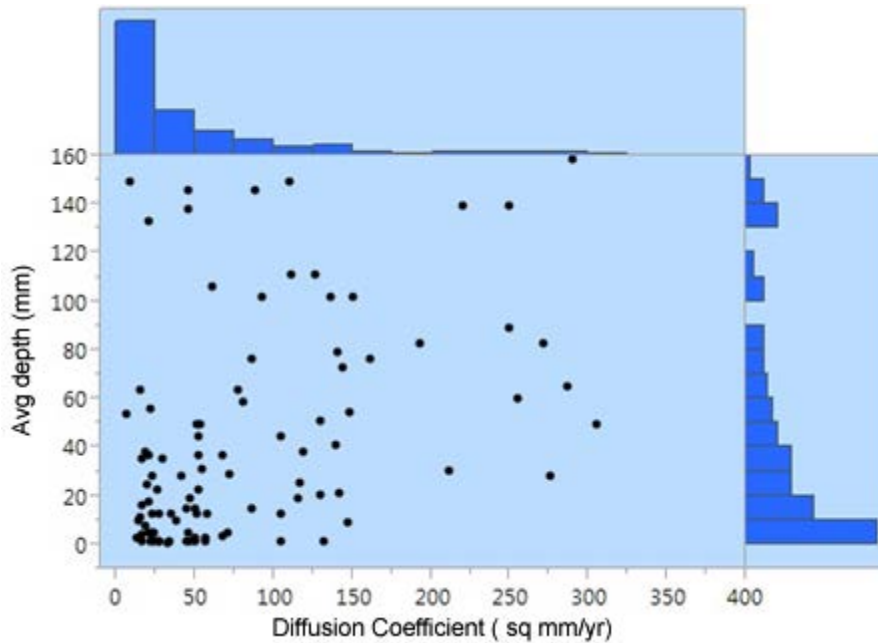


Figure 12. Scatter Plot and Histograms for Diffusion Coefficient Versus Crack Depth

Figure 13 presents the one-way analysis of diffusion coefficients for cracked and uncracked locations for the three subgroups. The connecting letters reports on the right side of the plots show that the cracked locations have significantly higher chloride diffusion compared to uncracked locations. From this analysis, it is clear that cracks have a significant influence on chloride diffusion. The breakdown of the influence of cracks on diffusion is discussed later.

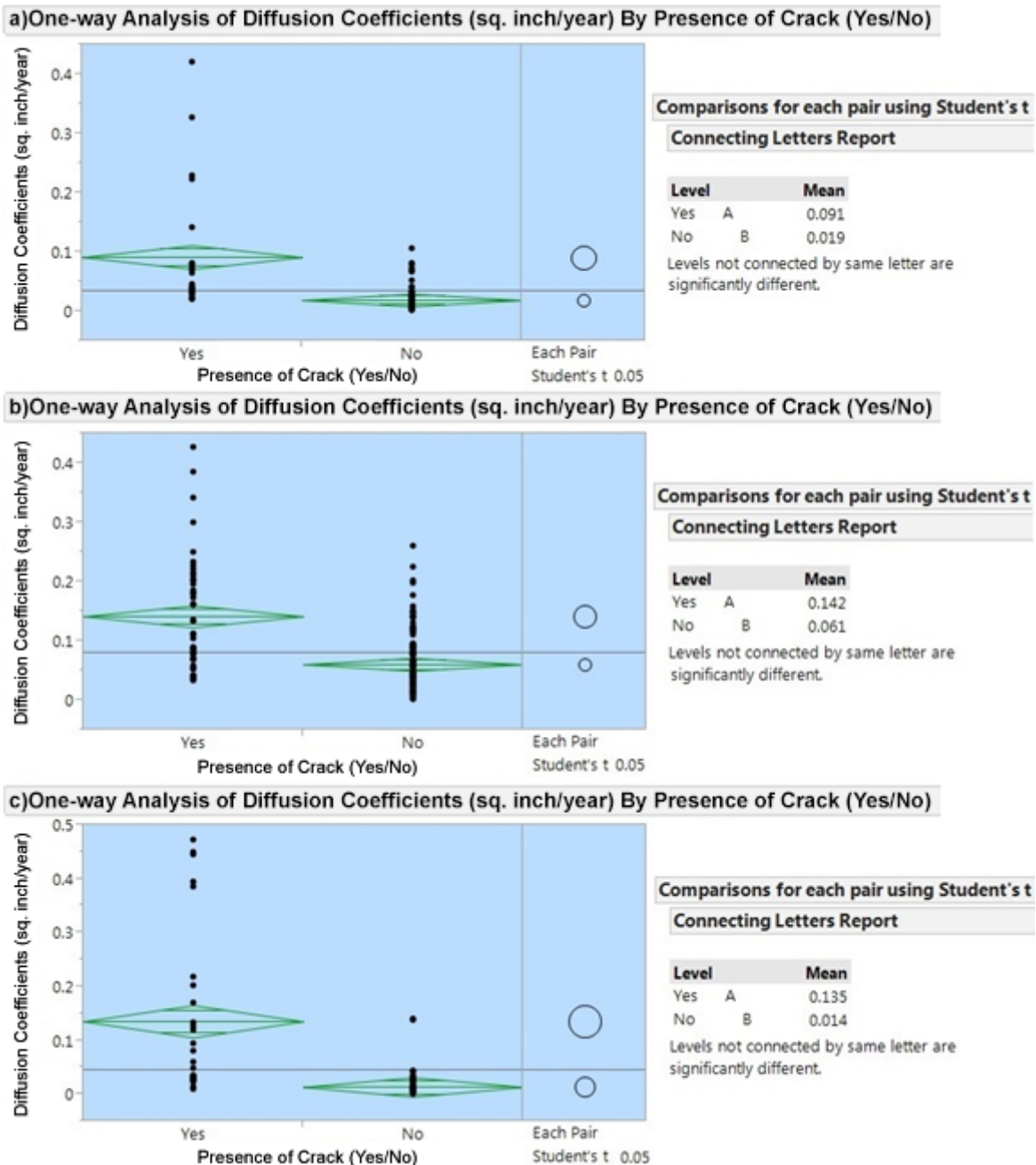


Figure 13. Diffusion in Cracked and Uncracked Locations in Subgroups: (a) 1968-71 No-SCM; (b) 1984-91 No-SCM; (c) 1984-91 SCM One-way

Figure 14 presents the one-way analysis of total crack frequency by specified w/c ratio. It appears that the w/c ratio did not have any influence on cracking. Figure 14 shows green diamond-shaped statistic indicators, where the long horizontal line in the center indicates the group mean and top and bottom of the diamonds indicate 95% confidence intervals. In addition, the short horizontal lines above and below the central mean line are for intergroup comparison purposes and are called overlap lines. The degree of overlap between two datasets can be understood from these lines.

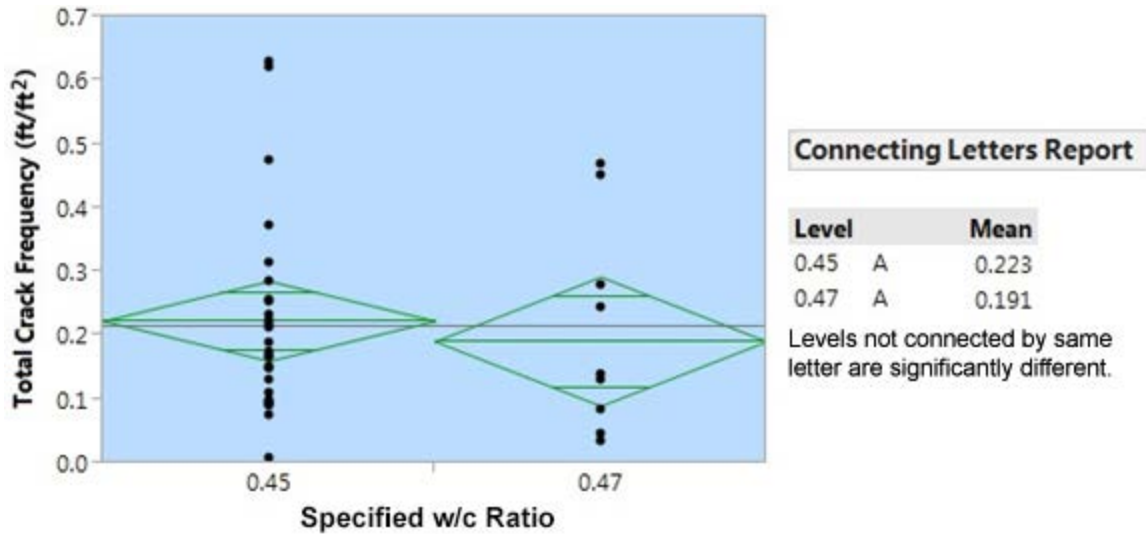


Figure 14. Total Crack Frequency by Specified Water–Cementitious Material (w/c) Ratio

A flexible superstructure may have transverse cracks, compared to a rigid superstructure. Figure 15 shows that steel superstructures have more transverse cracks than concrete superstructures; however, the difference is not significant.

A continuous structure can provide more resistance to shrinkage compared to a simple structure and thus have more transverse cracks. Figure 16 shows the one-way analysis between simple and continuous structures and the occurrence of transverse cracks. It appears that continuous structures are associated with a higher occurrence of transverse cracks, although the difference in occurrence was not statistically significant.

The presence of SIP forms may also affect crack formations, since the bottom of the deck will remain in a higher state of saturation whereas the top portion loses moisture and will shrink more. Figure 17 shows that transverse cracks decrease and longitudinal cracks increase in the presence of SIP forms, although not significantly.

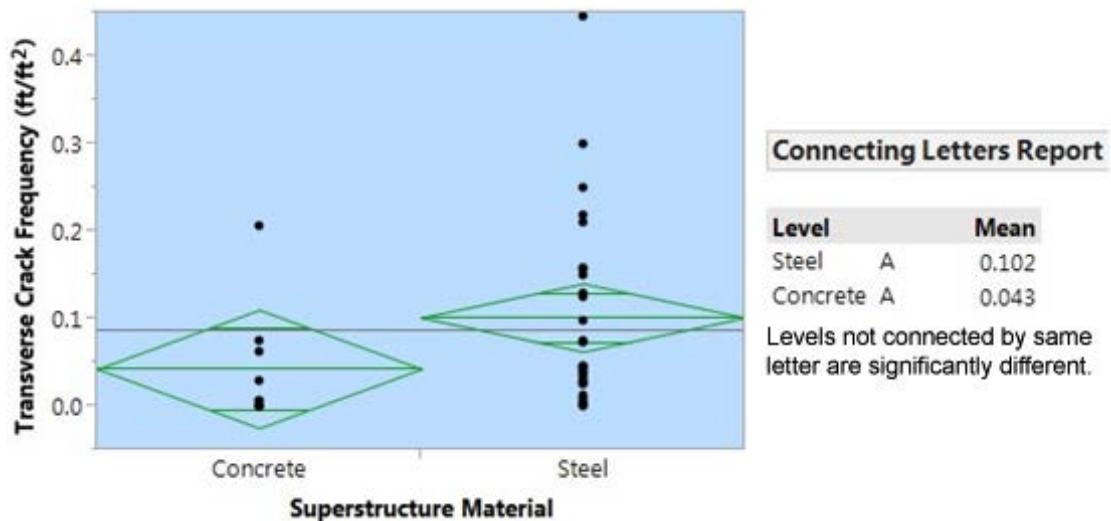


Figure 15. Transverse Crack Frequency by Superstructure Material

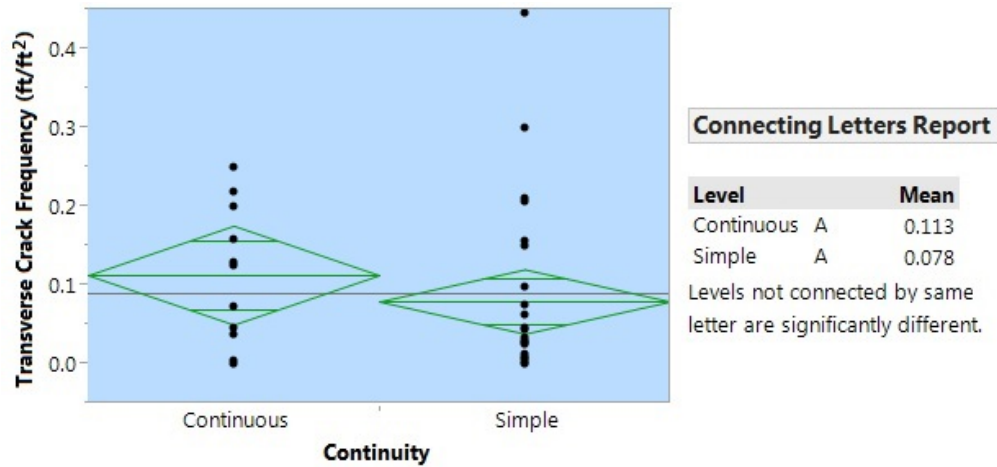


Figure 16. Transverse Crack Frequency by Span Continuity

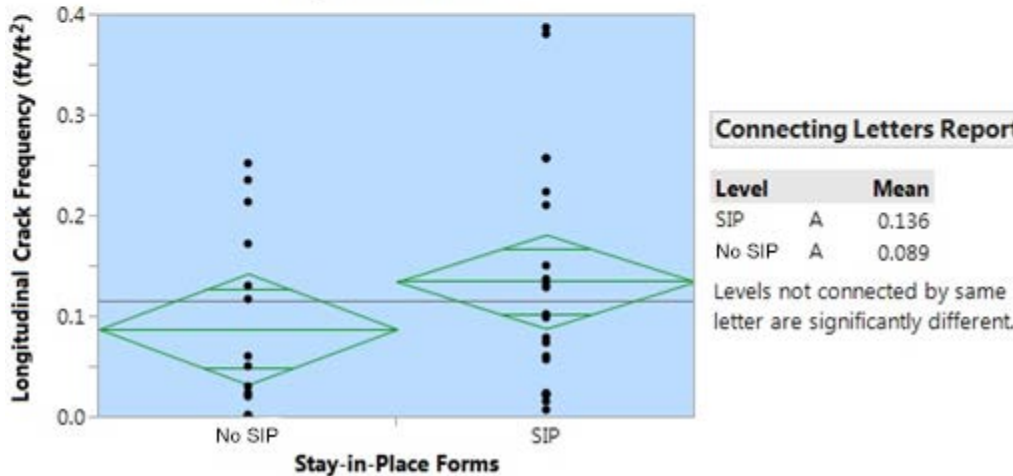
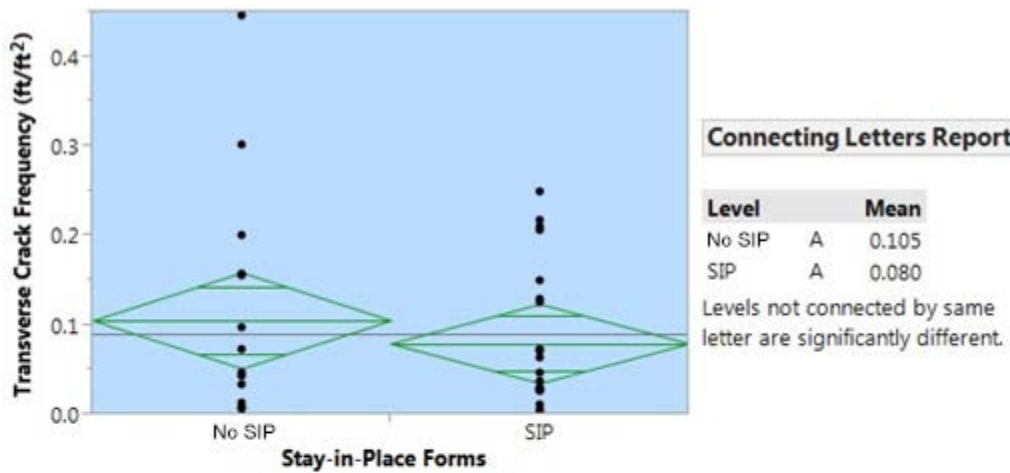


Figure 17. Transverse and Longitudinal Crack Frequency for Stay-in-Place (SIP) Forms

Reinforcement type influences the type of cracking. Figure 18 shows that transverse cracks occur more in the decks with bare steel rebars, albeit not significantly. Longitudinal cracking was significantly higher in decks with ECR, which might be due to low bond strength between the concrete and ECR.

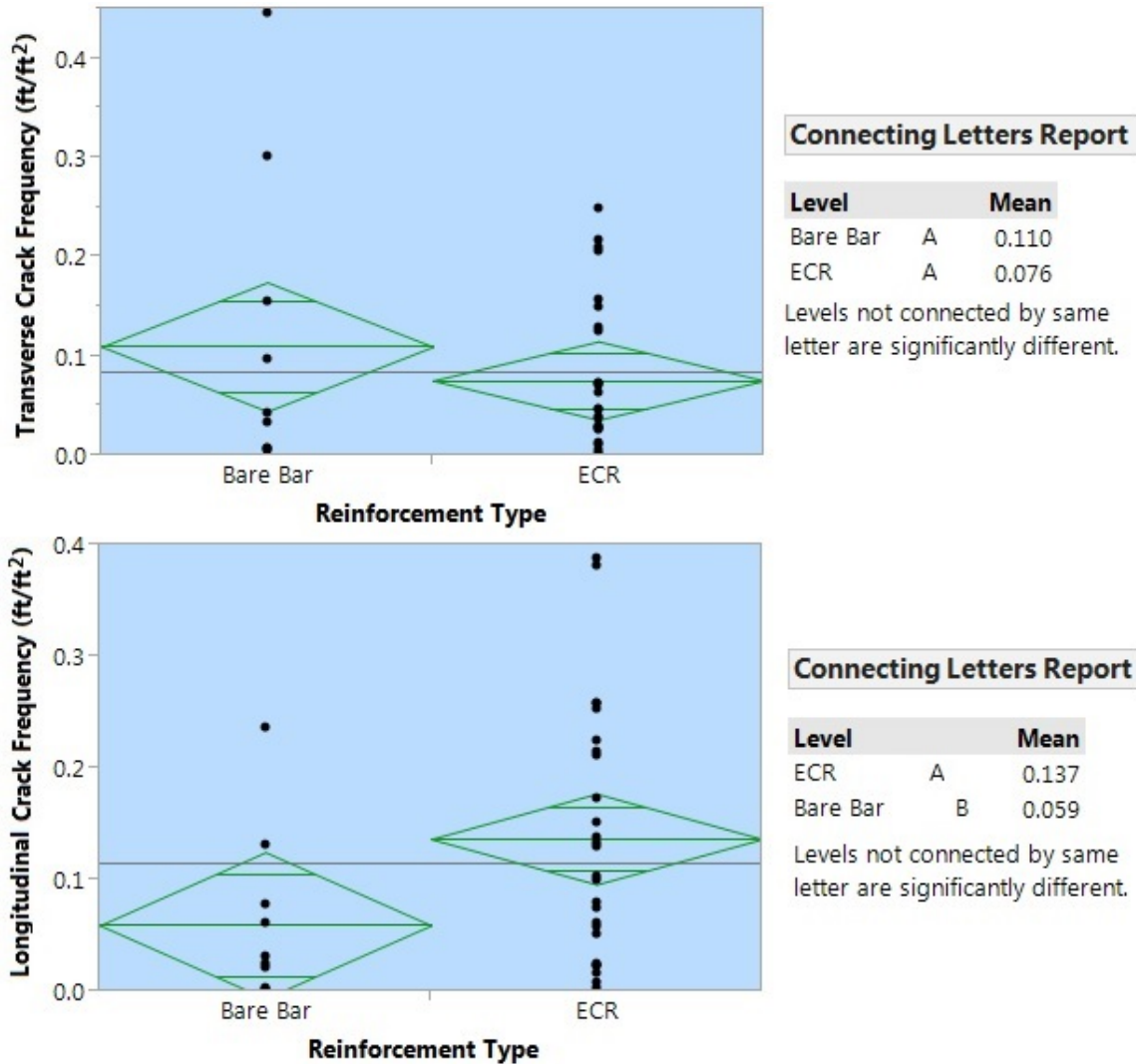


Figure 18. Transverse and Longitudinal Crack Frequency by Reinforcement Type. ECR = epoxy-coated reinforcement.

Analysis of Subgroups

Data from subgroups were analyzed separately to assess if factors defining the subgroups such as age of concrete, specified w/c ratio, pozzolans, reinforcement type and others have any influence. The material properties collected from the decks, including permeability, concrete pore volume, and moisture saturation are presented in Table 5. Use of SCM appears to improve vastly the resistance to chloride penetration based on the permeability values. The older structures have fewer water-permeable voids compared to the newer structures, even the decks with SCM, although not by a large margin. This might be related to the quality of construction and cement properties in the 1960s era compared to the newer structures. Later constructions used a Type I/II cement rather than the Type II cement used earlier. Moisture saturation did not vary much, but the decks with SCM exhibited a higher percentage saturation, which is probably a result of the pozzolan-modified concrete matrix having smaller pores.

Table 5. Material Properties of Subgroups

Property	1968-71 w/c = 0.47 No-SCM	1984-91 w/c = 0.45 No-SCM	1984-91 w/c = 0.45 SCM
Permeability (C)			
Mean	3,797	4,793	1,361
Std. Dev.	1,884	2,257	817
Median	3,455	5,144	1,091
COV	50%	47%	60%
Pore Volume (%)			
Mean	12.7	15.3	13.9
Std. Dev.	1.5	3.9	1.2
Median	12.5	14.7	14
COV	12%	25%	9%
Moisture Saturation (%)			
Mean	65	68	76
Std. Dev.	8.6	6.9	4.1
Median	66	68	75
COV	13%	10%	5%

COV = coefficient of variation.

The product of the pore volume and percent moisture saturation is the in-place moisture content, resulting in moisture contents of 8.2%, 15%, and 10.6% for the 0.47 No-SCM, 0.45 No-SCM, and 0.45 SCM subgroups, respectively. The amount of moisture and the fineness of the concrete pore system govern the rate of ion ingress. These results support the permeability results.

Figure 19 shows the traffic information on the subgroups. Annual average daily traffic (AADT), truck traffic, and speed limit information were obtained from the state bridge inventory. For the 1968-71 No-SCM decks, the traffic count is fewer compared to other subgroups, but the load cycle is higher because of the age difference of approximately 18 years. The newer structures show similar traffic volume and speed limits.

Table 6 shows the results of the comparisons of group means. Student's *t*-test was conducted on the data from the three subgroups and the results in terms of connecting letters are provided in Table 6. Connecting letters allocate the same alphabets for the subgroups that do not differ significantly from each other. From Table 6, it can be noted that the subgroups are ordered in descending order but have "A" as the connecting letter. This is true for all four traffic-related factors. Thus, it can be concluded that the traffic conditions did not vary significantly between the subgroups.

Figure 20 shows the comparison of damage conditions between the subgroup decks surveyed in 2003. It is clear that the 1968-71 No-SCM decks have undergone more damage than the other two subgroups, which is most likely related to their longer service life of over 18 years compared to the other subgroups and the associated chloride exposure conditions. In addition, it appears that most of the damage has occurred in the form of delamination.

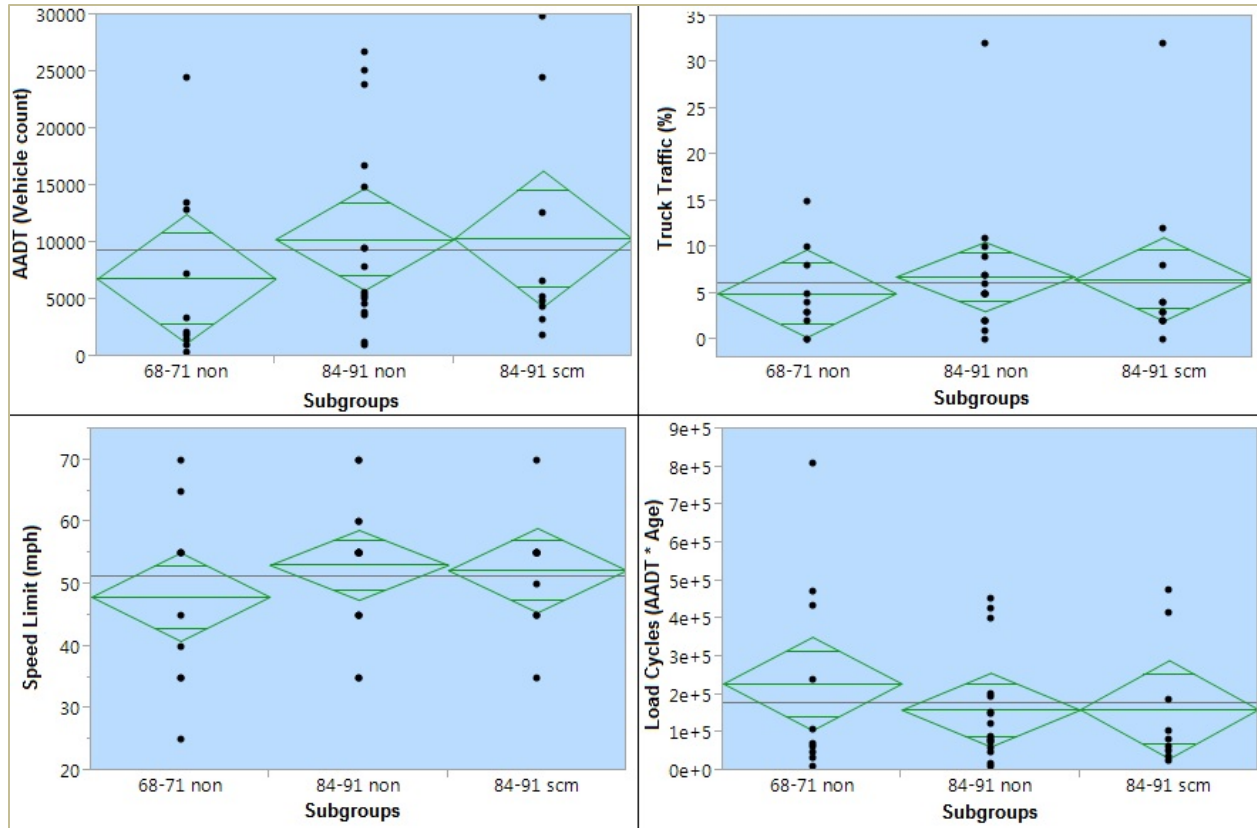


Figure 19. Comparison of Traffic Data Among Subgroups. AADT = average annual daily traffic; non = no SCM; SCM = supplementary cementitious material.

Table 6. Mean Comparisons of Traffic Data Among Subgroups

Connecting Letter Reports of Traffic Factors											
AADT			AADTT (%)			Speed Limit (mph)			Load Cycle (AADT*Age)		
Level		Mean	Level		Mean	Level		Mean	Level		Mean
1984-91 SCM	A	10,352	1984-91 No-SCM	A	6.8	1984-91 No-SCM	A	53	1968-71 No-SCM	A	229,586
1984-91 No-SCM	A	10,311	1984-91 SCM	A	6.5	1984-91 SCM	A	52	1984-91 SCM	A	161,554
1968-71 No-SCM	A	6,833	1968-71 No-SCM	A	5.0	1968-71 No-SCM	A	48	1984-91 No-SCM	A	160,454

AADT = annual average daily traffic; AADTT = annual average daily truck traffic; SCM = supplementary cementitious material.

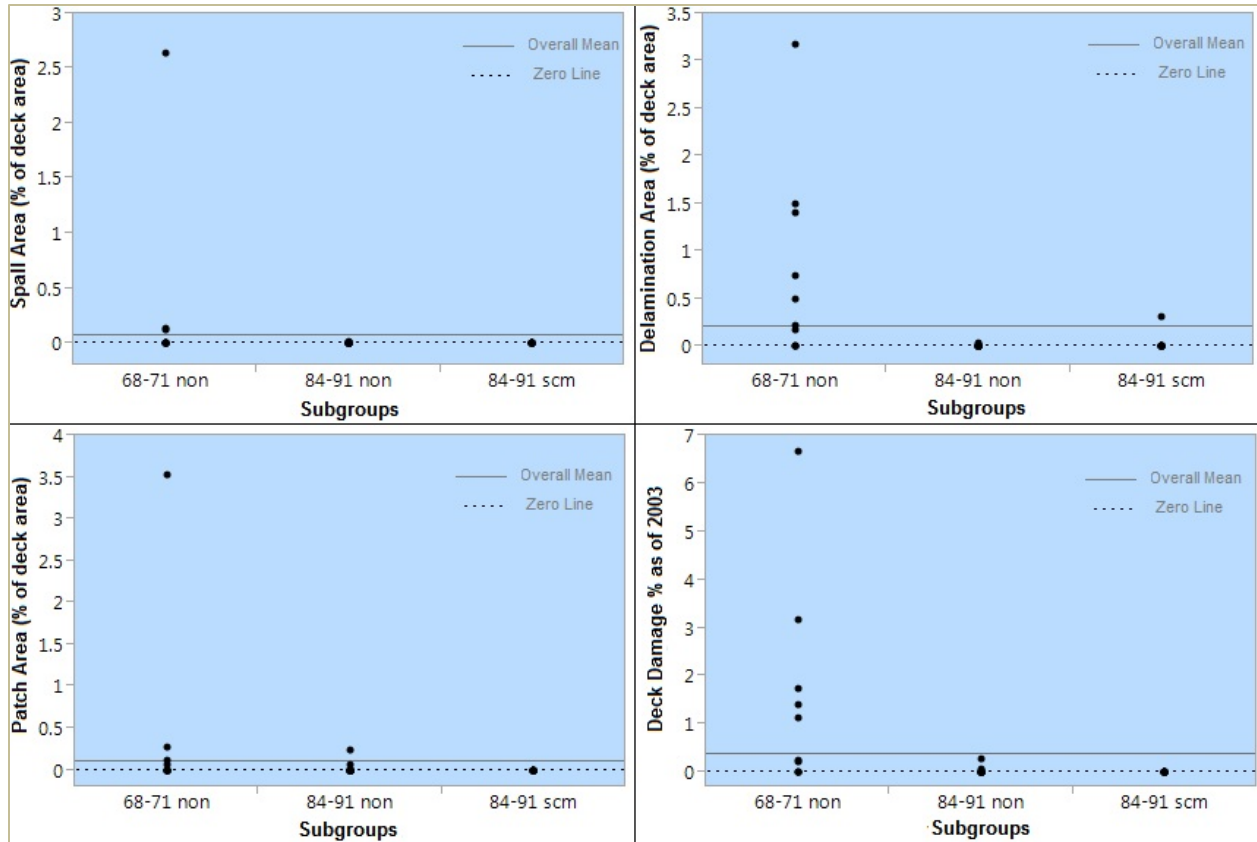


Figure 20. Damage Conditions of Decks by Subgroup (as of 2003). Non = no SCM; SCM = supplementary cementitious material.

1968-71 No-SCM Bridge Decks

General. Determining the influence of concrete cracking on parameters related to chloride diffusion and subsequent corrosion might lead to results regarding whether and when cracks need to be repaired. For this purpose, the matched cores collected from the corresponding cracked and uncracked locations over an adjacent rebar were used. The paired t -test was conducted on the data obtained from companion cores with uncracked and cracked locations. This test was conducted to determine if the presence of cracks influenced the diffusion parameters or in the case of some parameters such as cover depth, the influence of cover depth on cracking. The Wilcoxon signed rank test was chosen since the requirement of normal distribution for paired t -test was not met. Table 7 presents the paired t -test results.

The paired t -test was conducted by taking pairs of cracked and uncracked data, where each pair consisted of data from companion cores as explained in the “Methods” section; their difference was checked against zero (i.e., null hypothesis). The actual estimates are the mean values of the differences between data from the corresponding cracked and uncracked locations. The data are assumed to be normally distributed for the t -test; thus, the Shapiro-Wilk goodness-of-fit test was conducted for all distributions. Only the cover depth differences passed the test. Thus, except for the cover depths, the Wilcoxon signed rank test was conducted, which assumes a non-parametric distribution to calculate the p -values. The calculated two-tailed and one-tailed p -values are presented in Table 7.

Table 7. Crack Influence on Diffusion Parameters: 1968-71 No-SCM Decks

Parameter	Hypothesized Value	Actual Estimate	Prob. > T Two Tail	Prob. > T Right Tail	Prob. < T Left Tail
Surface chloride (lb/yd ³)	0	-0.580	.228	.886	.114
Cl- at rebar depth (lb/yd ³)	0	0.985	.021	.0105	.990
Diffusion coefficient (in ² /yr)	0	0.069	<.0001	<.0001	1.000
Surface rust area (%)	0	2.115	.016	.008	.992
Moisture saturation (%)	0	5.156	.006	.003	.997
Cover depth (in)	0	0.024	.548	.274	.726
Pore volume (%)	0	0.140	.390	.195	.805

Values in bold indicate statistically significant differences between the corresponding cracked and uncracked locations since they are less than the significance level of 0.05.

It can be concluded from the paired *t*-tests that the cracked concrete locations had significantly higher chloride contents at the depth of the rebar (hereinafter “rebar depth”), chloride diffusion coefficients, rebar rust, and concrete moisture saturation when compared to that of uncracked concrete. Diffusion coefficient and moisture saturation influence the time to corrosion initiation and the rate of subsequent corrosion. The other two parameters are the result of this influence. Peculiarly, the surface chloride concentration was higher in uncracked locations compared to the corresponding cracked locations, although the difference was not statistically significant. This might be because chlorides can move deeper with less effort in a cracked location than in an uncracked location, thus showing significantly higher chloride contents at the rebar depth in cracked locations.

Cover depths and concrete pore volumes were compared among the cracked and uncracked concrete locations; however, from the *t*-tests these factors were not significantly different. As a note, the smaller right-tailed *p*-values indicate that cover depths and concrete pore volumes were higher in cracked locations.

A comparison of the chloride diffusion parameters among three crack width ranges is presented in Figure 21. The chloride concentrations at the rebar depth appear to be higher under cracks greater than 0.3 mm in width but were not statistically significant.

From Figure 21, for surface chlorides, it can be noted that when cracks were narrower than 0.3 mm and when cracks were wider than 0.5 mm, less chlorides were found near the surface, compared to the cracks with a width between 0.3 mm and 0.5 mm. The narrower cracks may reduce the chloride diffusion because of obvious reasons, and the wider cracks may simply allow the ions to diffuse inward. This is supported by the chlorides at the rebar depth being lower in cracks narrower than 0.3 mm than in the wider cracks. If the two outlying values for diffusion coefficients in the <0.30 mm crack width range were removed, the diffusion coefficients at cracks wider than 0.5 mm would be higher than at the narrower cracks. Surface rust was not different for any of the crack width ranges.

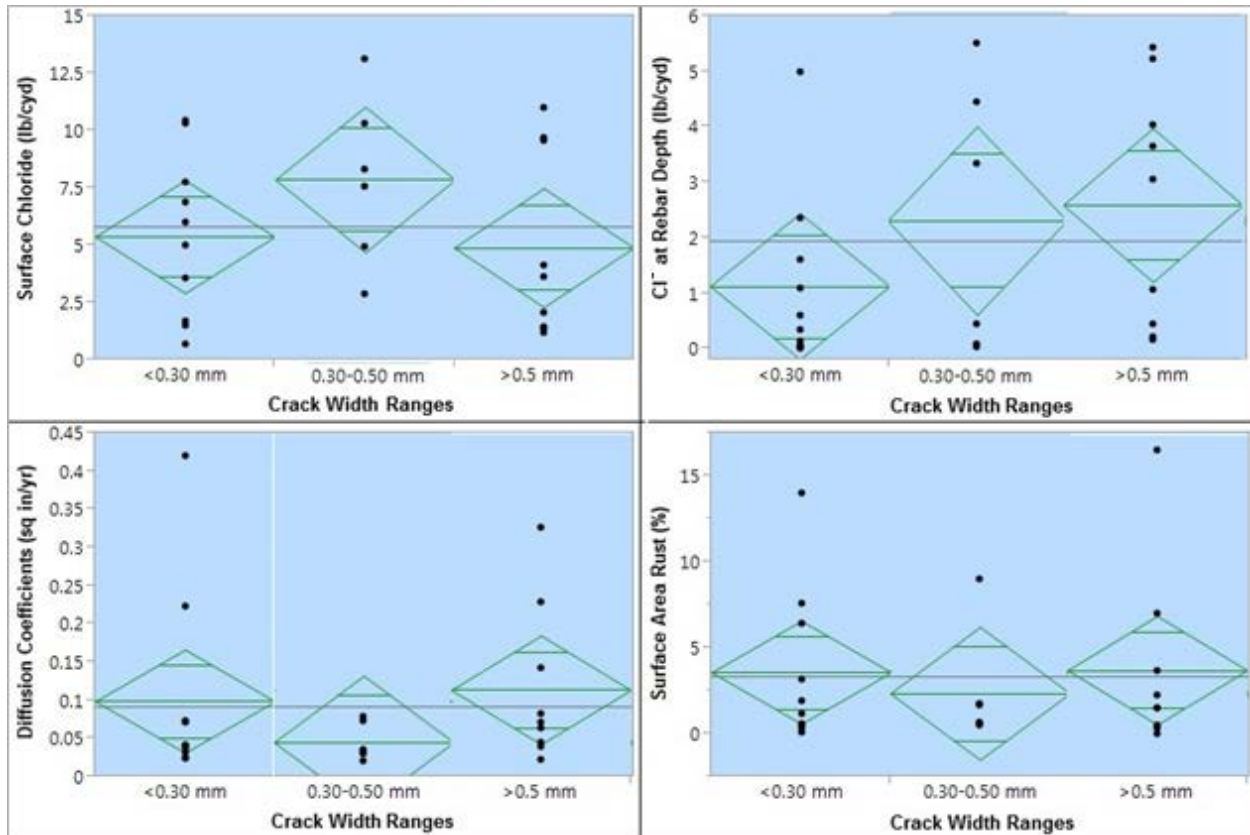


Figure 21. Chloride Diffusion Factors Among Crack Widths (1968-71 No-SCM Decks)

Table 8 presents the mean comparisons of the chloride diffusion factors among the crack width ranges. None of the diffusion parameters was significantly different for any crack width range, as indicated in Table 8. Figure 22 presents the comparison of diffusion parameters among the three crack depth ranges.

Diffusion coefficients were significantly higher for cracks deeper than 2 in than for the other two depth ranges, as shown in Figure 22. In addition, the surface rust reflects the higher diffusion coefficients in that range. Table 9 presents the connecting letters report for the mean comparisons shown in Figure 22. It can be noted that cracks deeper than 2 in had significantly higher diffusion coefficients and subsequent rust formation compared to cracks shallower than 1 in.

Table 8. Mean Comparisons of Chloride Diffusion Factors Among Crack Widths

Connecting Letter Reports of Chloride Diffusion Factors											
Surface Chloride (lb/yd ³)			Cl- at Rebar Depth (lb/yd ³)			Diffusion Coefficient (in ² /yr)			Surface Rust (%)		
Level		Mean	Level		Mean	Level		Mean	Level		Mean
0.30-0.50 mm	A	7.86	>0.5 mm	A	2.59	>0.5 mm	A	0.114	>0.5 mm	A	3.7
<0.30 mm	A	5.38	0.30-0.50 mm	A	2.31	<0.30 mm	A	0.099	<0.30 mm	A	3.6
>0.5 mm	A	4.89	<0.30 mm	A	1.13	0.30-0.50 mm	A	0.045	0.30-0.50 mm	A	2.4

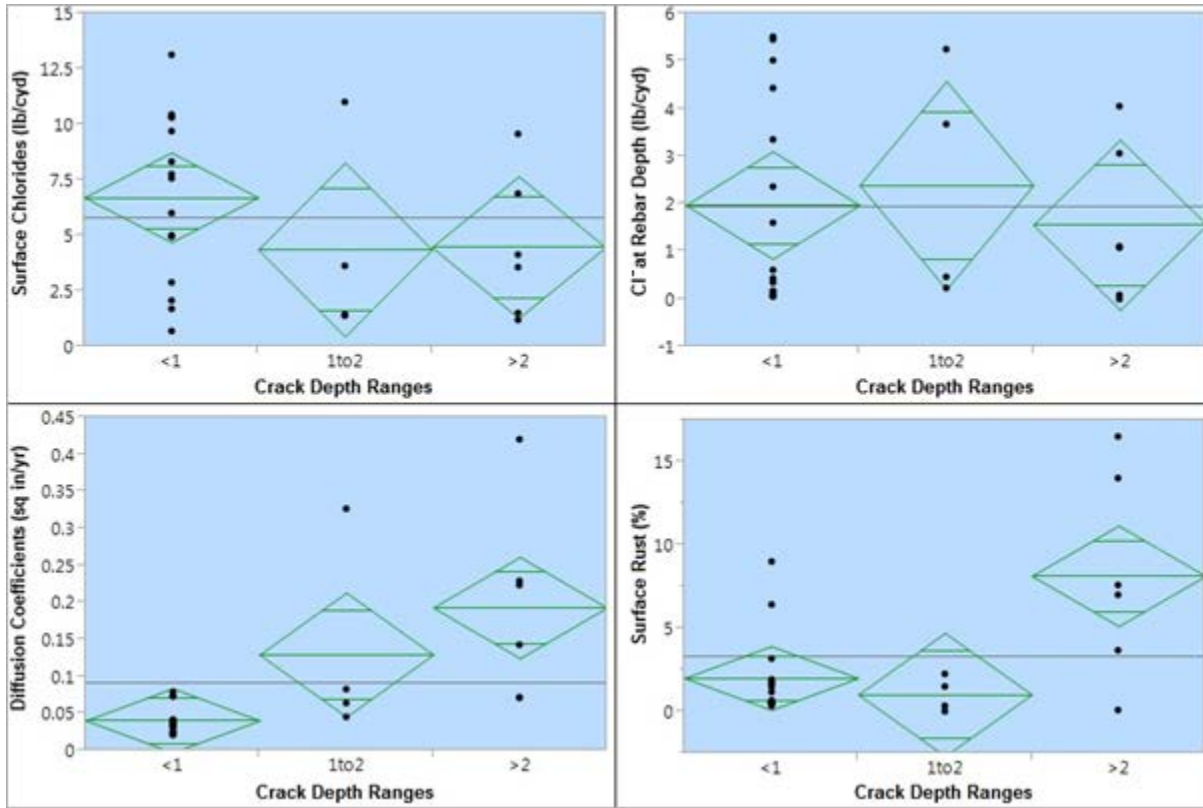


Figure 22. Chloride Diffusion Factors and Crack Depth (1968-71 No-SCM Decks)

Table 9. Mean Comparisons of Chloride Diffusion Factors and Crack Depths

Connecting Letter Reports of Chloride Diffusion Factors											
Surface Chloride (lb/yd ³)			Cl- at Rebar Depth (lb/yd ³)			Diffusion Coefficient (in ² /yr)			Surface Rust (%)		
Level		Mean	Level		Mean	Level		Mean	Level		Mean
<1	A	6.71	1 to 2	A	2.39	>2	A	0.193	>2	A	8.1
>2	A	4.47	<1	A	1.97	1 to 2	AB	0.129	<1	B	2.0
1 to 2	A	4.36	>2	A	1.56	<1	B	0.041	1 to 2	B	1.0

Regression Modeling. With a number of factors as presented in the “Methods” section affecting concrete cracking and subsequent chloride diffusion mechanisms, it would be helpful to learn which have the most effect. Once the parameters that are conceptually related to the dependent variable were selected, a stepwise regression method was used to filter for higher degree of correlation. Such models can be used to determine the relationships between the parameters rather than simply being used to predict the dependent variable.

In the evaluation of field structures, estimates of diffusion coefficients require much work to obtain and can be highly variable, even within a single structure, and service life estimates using the Fickian diffusion model are highly sensitive to even small changes in this parameter (Kirkpatrick et al., 2002). An attempt was made to model the diffusion coefficients for structures similar to the subgroup 1968-71 No-SCM decks (see Eq. 1). It can be understood from the model with a R^2 of 0.66 that diffusion coefficients are strongly affected by the concrete material properties and the depth of cracks.

$$\log D_c = 4.78 + 0.40\theta - 1.34\rho_{appr} + \begin{pmatrix} \text{if } D_{crk} < 1 \text{ in, } -0.25 \\ \text{if } D_{crk} = 1 \text{ in to } 2 \text{ in, } -0.09 \\ \text{if } D_{crk} > 2 \text{ in, } +0.34 \end{pmatrix} \quad [\text{Eq. 1}]$$

where

D_c = diffusion coefficients, mm²/yr
 ρ_{appr} = apparent density, g/cm³
 θ = concrete moisture saturation, %
 D_{crk} = depth of crack, in.

The regression equations for longitudinal crack frequencies with an R² of 0.99 (Eq. 2) and transverse crack frequencies with an R² of 0.65 (Eq. 3) are given here.

$$Cf_{lg} = 0.32 - 0.014CD - 0.002\theta - \frac{\mu_{cl}}{47824} + \begin{pmatrix} \text{if Zone} = \text{Hills, } -0.05 \\ \text{if Zone} = \text{Plains, } -0.05 \\ \text{if Zone} = \text{Coastal, } +0.05 \end{pmatrix} - 0.00056skew \quad [\text{Eq. 2}]$$

where

Cf_{lg} = longitudinal crack density, ft/ft²
 CD = average bridge deck concrete cover depth, in
 μ_{cl} = permeability, C
 $skew$ = bridge skew angle, degrees.

$$Cf_{tr} = -0.45 + \frac{\mu_{cl}}{84034} + 0.059S_{bm} + \begin{pmatrix} \text{if superstructure mat.} = \text{concrete, } -0.034 \\ \text{if superstructure mat.} = \text{steel, } +0.034 \end{pmatrix} \quad [\text{Eq. 3}]$$

where

Cf_{tr} = transverse crack density, ft/ft²
 S_{bm} = beam spacing, ft.

1984-91 No-SCM Bridge Decks

General. Bridge decks built more recently among the study bridges with ECR and no SCM in the concrete mixture form the 1984-91 No-SCM subgroup. Table 10 presents the results from the paired *t*-tests on the influence of cracking on various parameters.

From Table 10, the chloride concentration at the rebar depth, diffusion coefficient, and moisture saturation are significantly affected by concrete cracking, as indicated by *p*-values less than the significance level of 0.05. Similar to the results presented in Table 7, concrete cover depth does not have an appreciable influence on cracking. However, the presence of surface rust on rebars was not correlated with cracking, unlike for the 1968-71 No-SCM subgroup. The higher concrete pore volumes at cracked locations do not have any relationship with the presence of cracks.

Table 10. Crack Influence on Diffusion Parameters: 1984-91 No-SCM Decks

Parameter	Hypothesized Value	Actual Estimate	Prob. > t Two tail	Prob. > t Right tail	Prob. < t Left tail
Surface chloride (lb/yd ³)	0	0.10	0.677	0.338	0.662
Cl- at rebar depth (lb/yd ³)	0	1.15	<.0001	<.0001	1.000
Diffusion coefficient (in ² /yr)	0	0.17	<.0001	<.0001	1.000
Surface rust area (%)	0	0.08	0.893	0.446	0.554
Moisture saturation (%)	0	2.75	0.009	0.005	0.995
Cover depth (in)	0	0.06	0.375	0.187	0.813
Pore volume (%)	0	0.68	<.0001	<.0001	1.000

Values in bold indicate *p*-values less than the significance level of 0.05.

Figure 23 shows the comparison between diffusion parameters for the crack width ranges in this subgroup. It appears that crack width has no statistically significant influence on surface chloride, diffusion coefficient, and surface area rust. However, the chloride concentration at the rebar depth, Figure 23, appears to follow the same trend as that in Figure 22, which showed higher chloride concentrations for concrete with cracks greater than 0.3 mm in width.

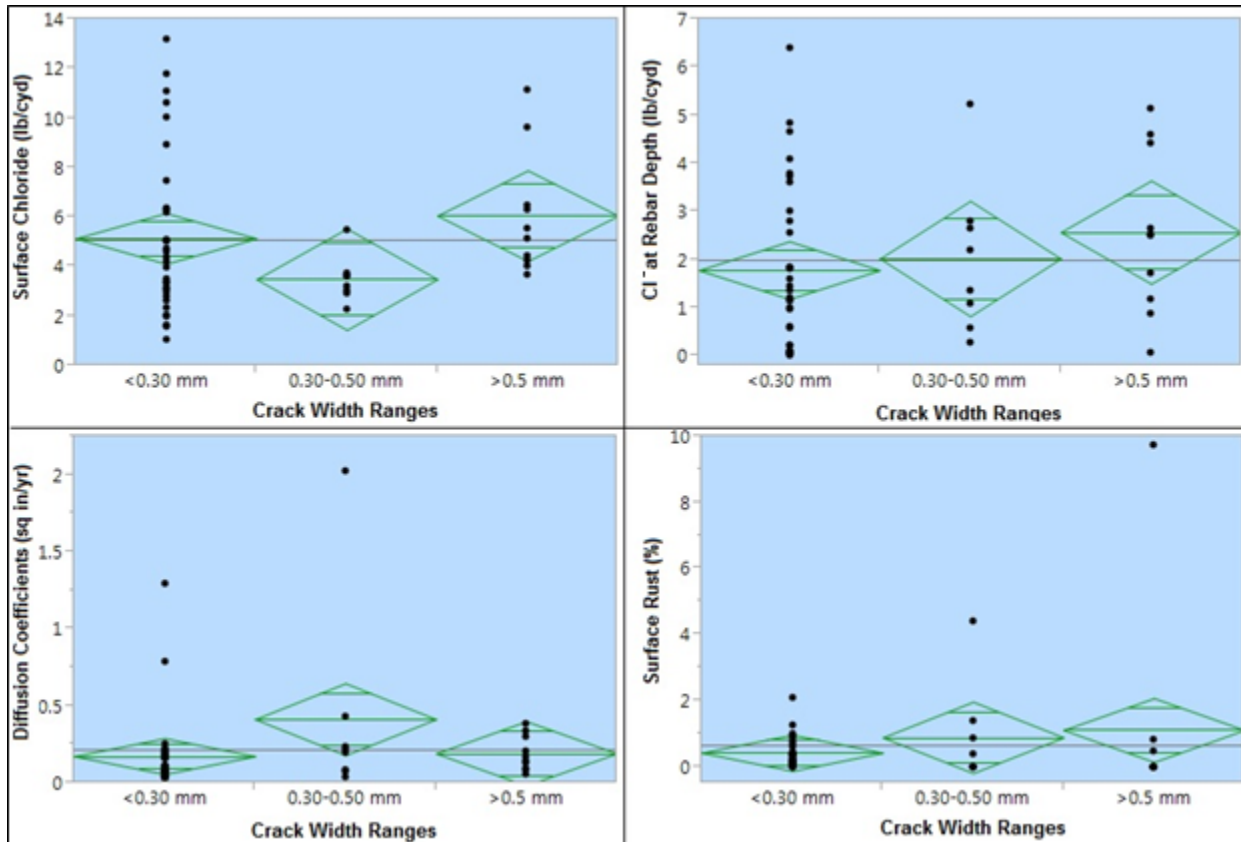


Figure 23. Chloride Diffusion Factors Among Crack Widths (1984-91 No-SCM Decks)

Table 11 presents the mean comparisons of the chloride diffusion factors among the crack width ranges. None of the diffusion parameters was significantly different for any crack width ranges.

Table 11. Mean Comparisons of Chloride Diffusion Factors and Crack Widths

Connecting Letter Reports of Chloride Diffusion Factors											
Surface Chloride (lb/yd ³)			Cl- at Rebar Depth (lb/yd ³)			Diffusion Coefficient (in ² /yr)			Surface Rust (%)		
Level		Mean	Level		Mean	Level		Mean	Level		Mean
>0.5 mm	A	6.05	>0.5 mm	A	2.57	0.30-0.50 mm	A	0.413	>0.5 mm	A	1.1
<0.30 mm	A	5.13	0.30-0.50 mm	A	2.02	>0.5 mm	A	0.191	0.30-0.50 mm	A	0.9
0.30-0.50 mm	A	3.49	<0.30 mm	A	1.78	<0.30 mm	A	0.173	<0.30 mm	A	0.4

Figure 24 shows the comparison of diffusion parameters among the three crack depth ranges. Diffusion coefficients and chloride concentrations at the rebar depth appear to be significantly increased with the increase in crack depths.

Table 12 presents the mean comparisons of the chloride diffusion factors and the crack depth ranges. As seen in Figure 24, deeper cracks significantly influenced the diffusion coefficient and chloride at the rebar depth.

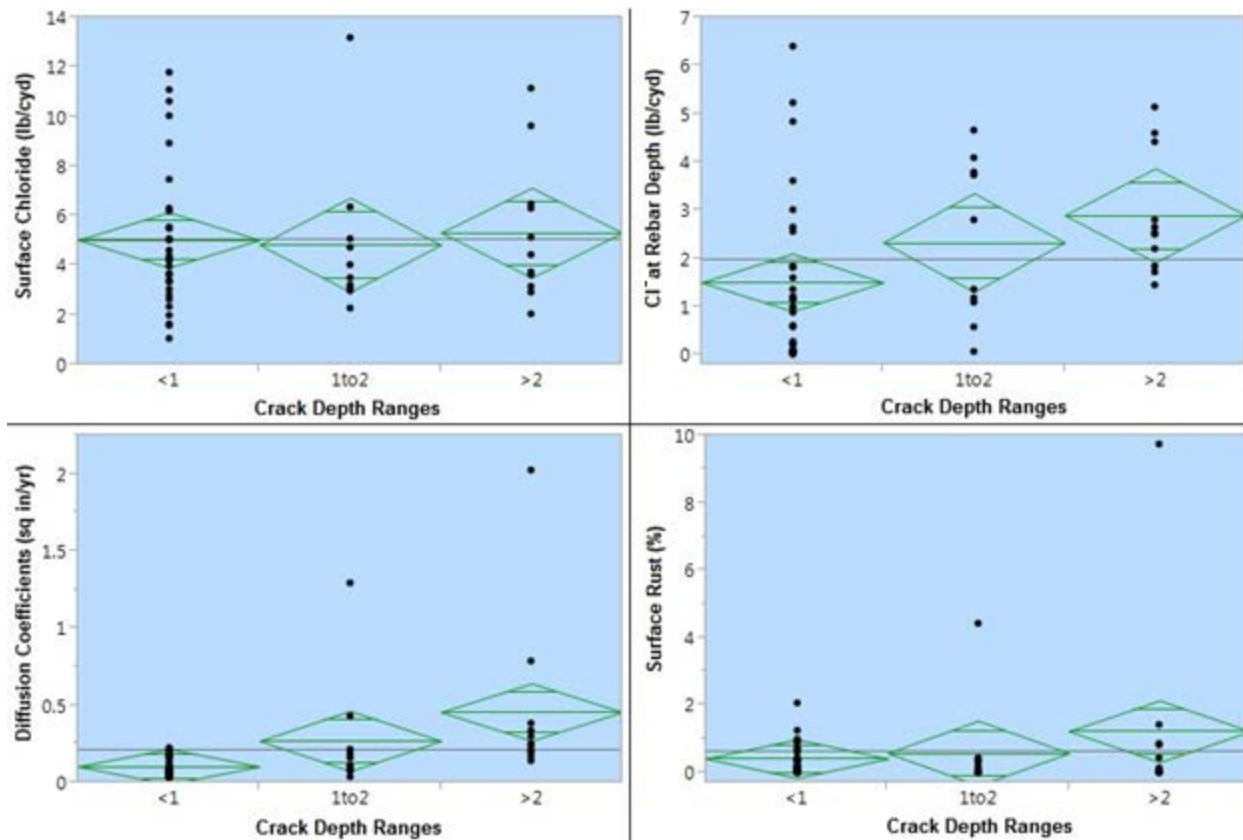


Figure 24. Chloride Diffusion Factors and Crack Depths (1984-91, No-SCM Decks)

Table 12. Mean Comparisons of Chloride Diffusion Factors and Crack Depths

Connecting Letter Reports of Chloride Diffusion Factors											
Surface Chloride (lb/yd ³)			Cl- at Rebar Depth (lb/yd ³)			Diffusion Coefficient (in ² /yr)			Surface Rust (%)		
Level		Mean	Level		Mean	Level		Mean	Level		Mean
>2	A	5.32	>2	A	2.90	>2	A	0.458	>2	A	1.2
<1	A	5.03	1 to 2	AB	2.33	1 to 2	AB	0.270	1 to 2	A	0.6
1 to 2	A	4.83	<1	B	1.51	<1	B	0.104	<1	A	0.4

Regression Modeling. Following are the regression models for diffusion coefficients with an R² of 0.47 ([Eq. 4]), longitudinal cracks with an R² of 0.63 ([Eq. 5]), and transverse crack frequencies with an R² of 0.63 ([Eq. 6]).

$$\log D_c = 1.08 - 0.05Age + 1.39\theta + 0.03n + 0.0039D_{crk} \quad [\text{Eq. 4}]$$

where

Age = age of bridge deck, yr
n = pore volume, %.

$$Cf_{lg} = 0.3 - 0.25CD + 0.045S_{bm} + 0.0073I_{sp} \quad [\text{Eq. 5}]$$

where

I_{sp} = superstructure moment of inertia calculated at midspan, ft⁴

$$Cf_{tr} = 0.504 - 0.22CD + 0.002skew + \begin{pmatrix} \text{if continuous superstructure, +0.014} \\ \text{if simple superstructure, -0.014} \end{pmatrix} \quad [\text{Eq. 6}]$$

1984-91 SCM Bridge Decks

General. Bridge decks built more recently with ECR and SCM (fly ash, slag) in the concrete mixture comprise the 1984-91 SCM subgroup. Table 13 presents the results of the paired *t*-tests for influence of cracking on various parameters. From Table 13, the surface chloride concentration, chloride concentration at the rebar depth, diffusion coefficient, and degree of moisture saturation are significantly higher for the cracked locations compared to the uncracked locations, where *p*-values were less than the significance level of 0.05.

Figure 25 presents the comparison of diffusion parameters among the crack width groups. It is understood from Figure 25 that cracks of widths greater than 0.3 mm result in higher chloride diffusion to the rebar depth, but they do not display statistical significance as shown in Table 14. All other results might have been affected by outliers.

Table 14 presents the mean comparisons of the chloride diffusion factors among the crack width ranges. None of the diffusion parameters was significantly different for any crack width range.

Table 13. Crack Influence on Diffusion Parameters: 84-91 SCM Decks

Parameter	Hypothesized Value	Actual Estimate	Prob. > t Two Tail	Prob. > t Right Tail	Prob. < t Left Tail
Surface chloride	0	2.91	0.000	0.000	1.000
Cl ⁻ at rebar depth	0	2.11	0.000	0.000	1.000
Diffusion coefficient	0	0.533	0.000	0.000	1.000
Surface rust area	0	2.62	0.279	0.139	0.861
Moisture saturation	0	8.88	0.000	0.000	1.000
Cover depth	0	0.02	0.410	0.205	0.795
Pore volume	0	0.29	0.438	0.219	0.781

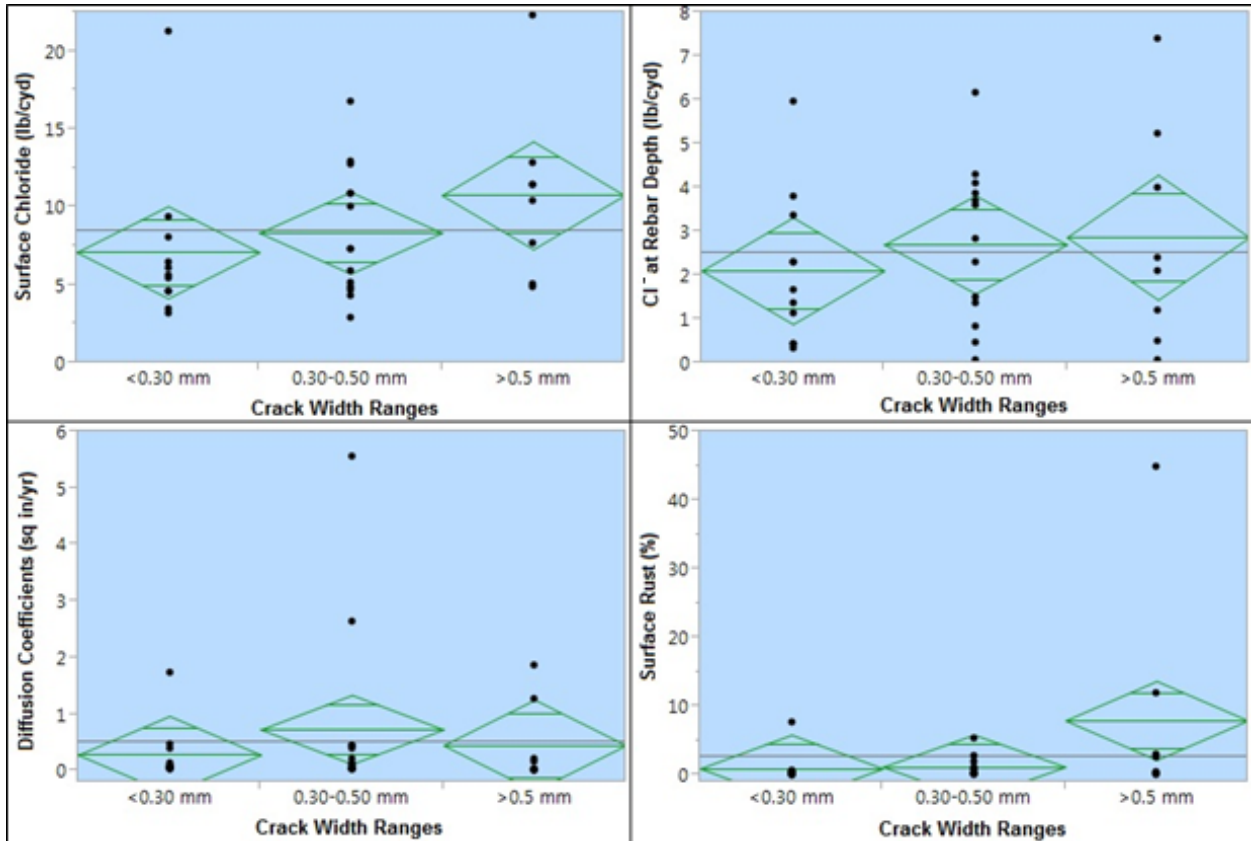


Figure 25. Chloride Diffusion Factors Among Crack Widths (1984-91 SCM Decks)

Table 14. Mean Comparisons of Chloride Diffusion Factors and Crack Widths

Connecting Letter Reports of Chloride Diffusion Factors											
Surface Chloride (lb/yd ³)			Cl ⁻ at Rebar Depth (lb/yd ³)			Diffusion Coefficient (in ² /yr)			Surface Rust (%)		
Level		Mean	Level		Mean	Level		Mean	Level		Mean
>0.5 mm	A	10.75	>0.5 mm	A	2.86	0.30-0.50 mm	A	0.730	>0.5 mm	A	8.0
0.30-0.50 mm	A	8.35	0.30-0.50 mm	A	2.70	>0.5 mm	A	0.450	0.30-0.50 mm	A	1.2
<0.30 mm	A	7.10	<0.30 mm	A	2.10	<0.30 mm	A	0.281	<0.30 mm	A	1.0

Figure 26 shows the diffusion parameters compared to the crack depth ranges for this subgroup. The only statistically significant correlation was for chloride concentration at the rebar depth and crack depth greater than 2 in. Higher variability associated with the chloride diffusion process was evident after analysis of the collected data.

Table 15 presents the mean comparisons of the chloride diffusion factors and the crack depth ranges. Chloride concentration at the rebar depth was statistically higher at cracked locations; however, the other parameters were affected by outliers.

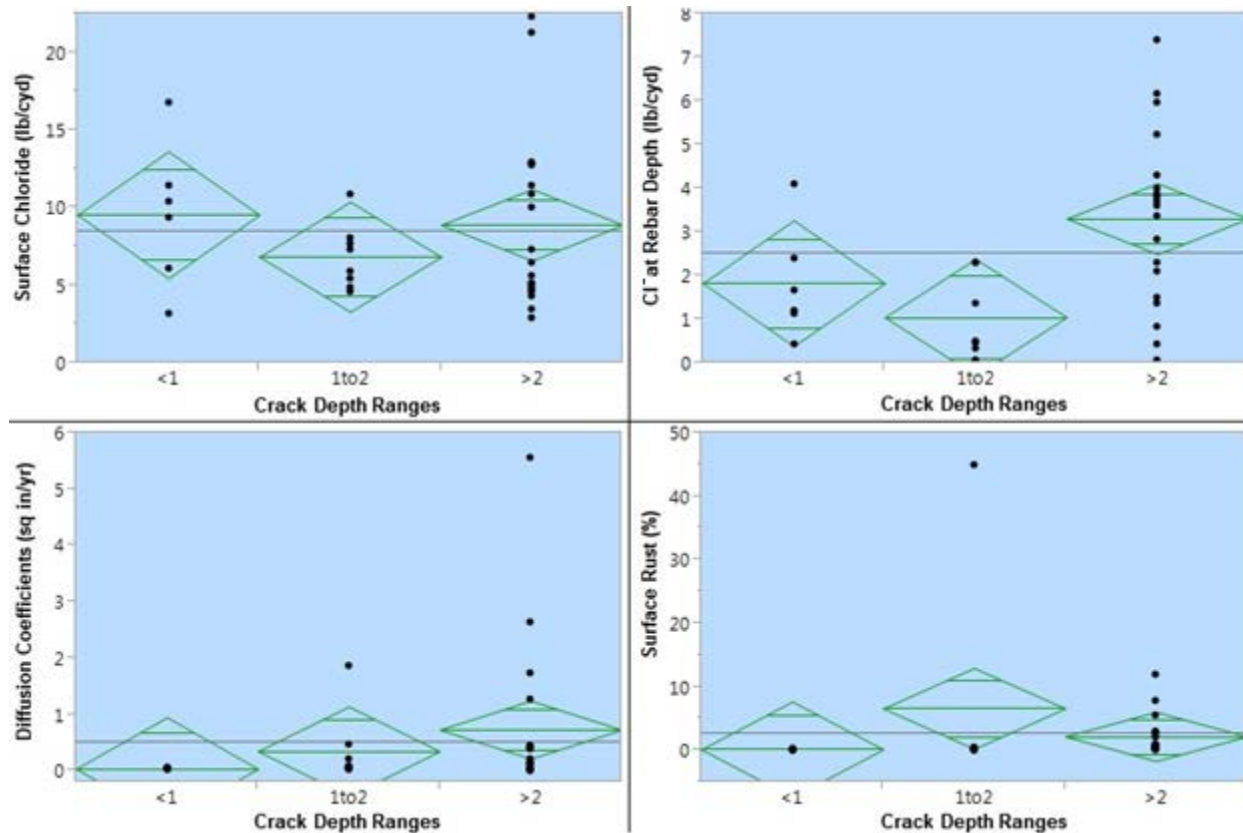


Figure 26. Chloride Diffusion Factors and Crack Depths (1984-91 SCM Decks)

Table 15. Mean Comparisons of Chloride Diffusion Factors and Crack Depths

Connecting Letter Reports of Chloride Diffusion Factors											
Surface Chloride (lb/yd ³)			Cl- at Rebar Depth (lb/yd ³)			Diffusion Coefficient (in ² /yr)			Surface Rust (%)		
Level		Mean	Level		Mean	Level		Mean	Level		Mean
<1	A	9.54	>2	A	3.31	>2	A	0.731	1 to 2	A	6.6
>2	A	8.90	<1	AB	1.83	1to2	A	0.349	>2	A	2.2
1 to 2	A	6.84	1 to 2	B	1.05	<1	A	0.037	<1	A	0.2

Regression Modeling. Following are the regression models for diffusion coefficients with an R^2 of 0.37 (Eq. 7), longitudinal crack frequencies with an R^2 of 0.80 (Eq. 8), and transverse crack frequencies with an R^2 of 0.99 (Eq. 9).

$$\log D_c = 1 - 0.18Age + 4.1\theta + \begin{pmatrix} \text{if } D_{crk} < 1 \text{ in, } -0.29 \\ \text{if } D_{crk} = 1 \text{ in to 2 in, } +0.0003 \\ \text{if } D_{crk} > 2 \text{ in, } +0.29 \end{pmatrix} \quad [\text{Eq. 7}]$$

$$Cf_{lg} = -0.25 + 0.008\theta - 0.035S_{bm} - 0.012K_{gr} + \begin{pmatrix} \text{if } SCM=\text{fly ash, } -0.09 \\ \text{if } SCM=\text{slag, } +0.09 \end{pmatrix} + \begin{pmatrix} \text{if Zone} = \text{Hills, } +0.05 \\ \text{if Zone} = \text{Plains, } +0.05 \\ \text{if Zone} = \text{Coastal, } -0.10 \end{pmatrix} [\text{Eq. 8}]$$

$$Cf_{tr} = 0.81 + \frac{\rho}{42855} - 0.01\theta + \frac{LC}{1.16e9} + \begin{pmatrix} \text{if Zone} = \text{Hills, } -0.004 \\ \text{if Zone} = \text{Plains, } -0.024 \\ \text{if Zone} = \text{Coastal, } +0.03 \end{pmatrix} + \begin{pmatrix} \text{if continuous super., } +0.06 \\ \text{if simple super., } -0.06 \end{pmatrix} \quad [\text{Eq. 9}]$$

where

ρ = Wenner four-point resistivity, ohm-m

LC = approximate load cycle calculated as a product of AADT and years of service and number of days in a year (365).

Influence of Moisture Saturation

Moisture supports the diffusion of chlorides and acts as an electrolyte that is necessary for the corrosion mechanism to progress, and the moisture content of concrete varies often. Concrete with a lower specified w/c ratio has higher resistance to moisture permeability; however, it is also true that once moisture gets in, moisture depletion takes a long time, as more energy is required to drive moisture from very fine pores than from larger pores.

Figure 27 shows the difference between moisture saturation in the concretes with a specified w/c ratio of 0.45 and the concretes with a specified w/c ratio of 0.47. Concrete with a specified w/c ratio of 0.45 had a higher moisture saturation.

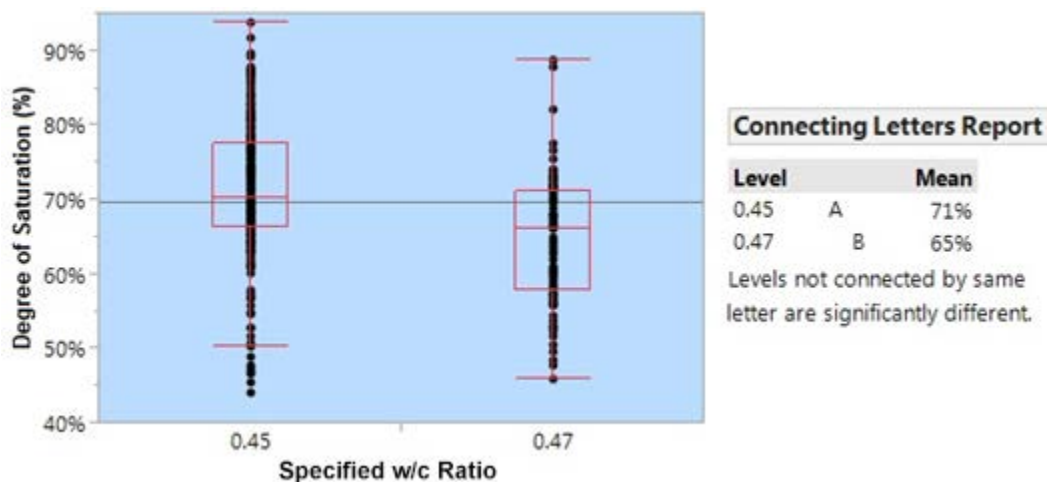


Figure 27. Saturation by Specified Water–Cementitious Material (w/c) Ratio

Figure 28 shows the pore volume ratio by specified w/c ratio, i.e., 0.45 and 0.47. The pore volume ratio was higher for concrete with a specified w/c ratio of 0.45. This might be the case because the actual water content in the mixtures might have been higher than the specified water content.

Figure 29 presents the effects of SCM on degree of saturation. The addition of SCM can typically reduce permeability. Figure 29 shows that concrete with fly ash had the highest saturation; slag concrete had moderate saturation; and concrete with no SCM had the lowest saturation. This can be explained by the fact that fewer pore spaces can be filled with a smaller amount of water. Further, it is to be expected that concrete cracks will allow significantly more moisture to diffuse quickly, as shown in Figure 30.

Saturation will primarily depend on the availability of moisture from the environment. Figure 31 shows the one-way analysis of saturation percentage by geographic zone, which shows a significant contribution from the environment, probably in the form of precipitation.

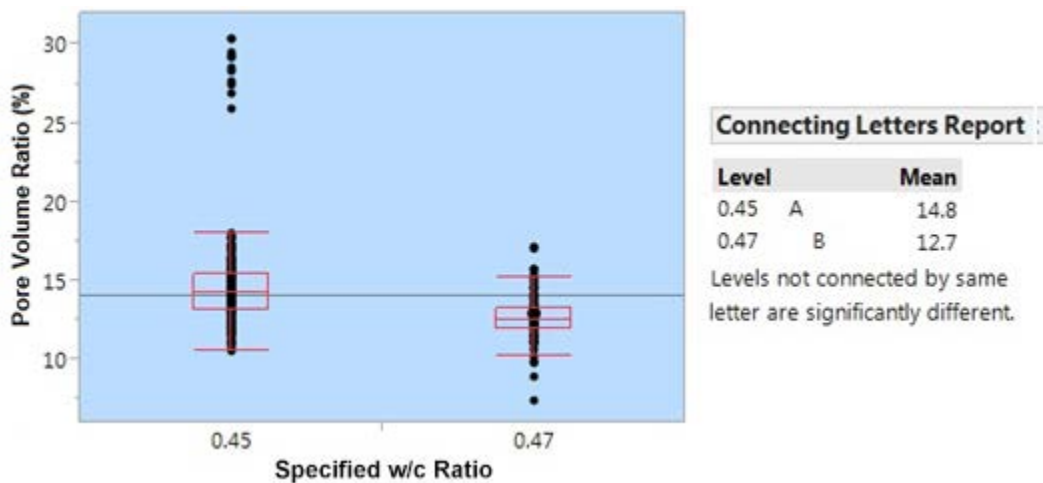


Figure 28. Pore Volume Ratio by Specified Water–Cementitious Material (w/c) Ratio

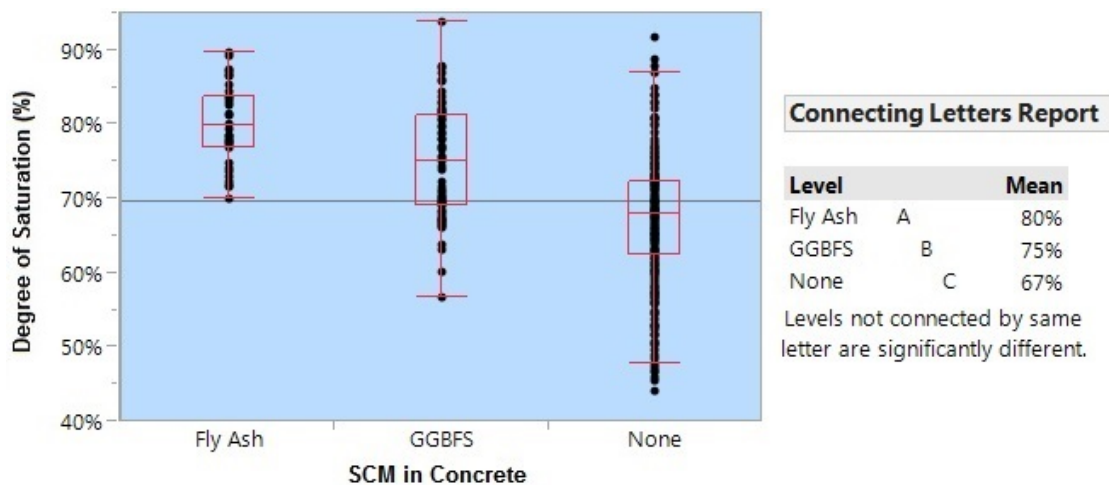


Figure 29. Degree of Saturation by SCM. SCM = supplementary cementitious material; GGBFS = ground granulated blast furnace slag.

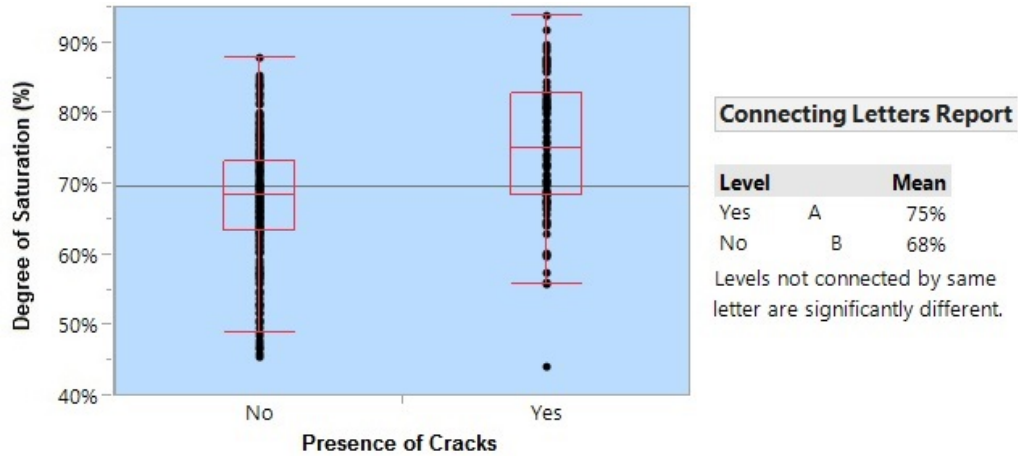


Figure 30. Degree of Saturation in Cracked and Uncracked Concrete

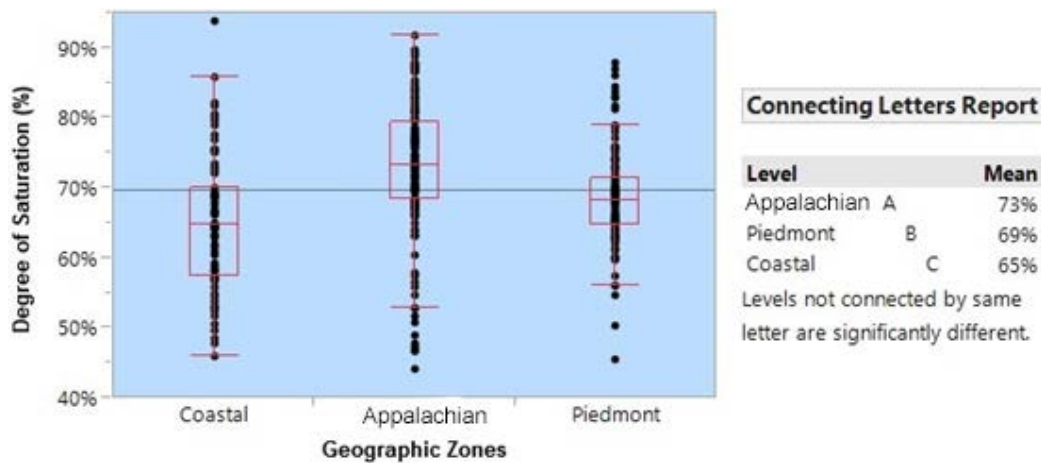


Figure 31. Degree of Saturation by Geographic Zone

Figure 32 presents the outcome of the analysis of the influence of crack width on moisture saturation. It appears that there is almost no difference in saturation above and below the crack width threshold.

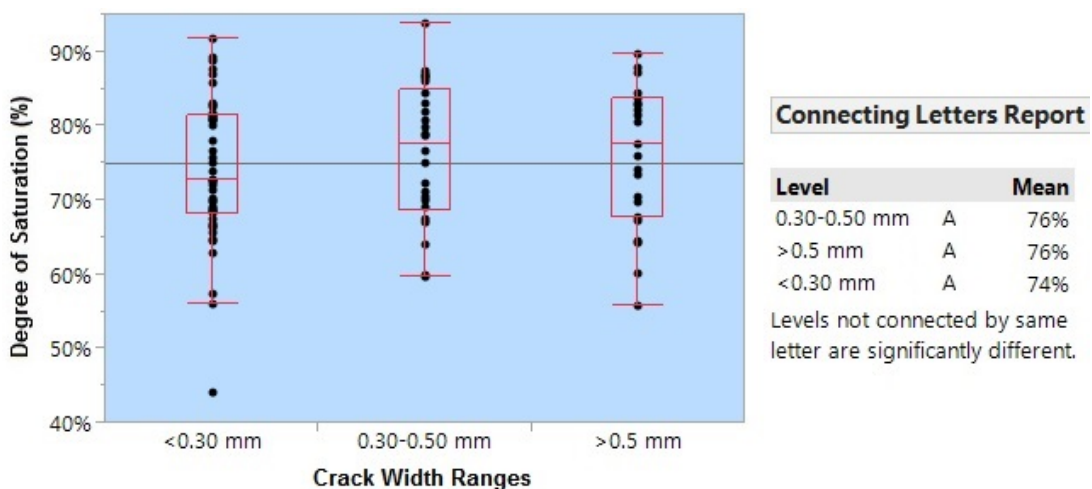


Figure 32. Degree of Saturation by Crack Width

Figure 33 presents the influence of crack depth on moisture saturation. It appears that moisture saturation increases as crack depth increases. The saturation of cracks deeper than 1 in (25.4 mm) was significantly higher compared to that of shallower cracks.

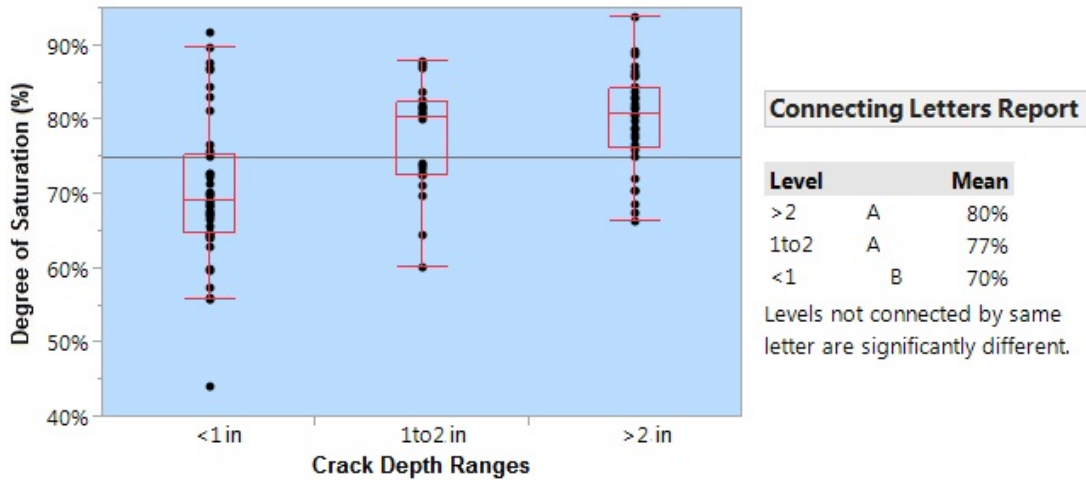


Figure 33. Degree of Saturation by Crack Depth

Service Life Estimation

Determining the degree to which the service life of bridge decks is affected by concrete cracking can be useful for predicting rates of deterioration and establishing maintenance and replacement strategies for an aging network. Service life was estimated in accordance with Equation 10 (Fitch et al., 1995).

$$\begin{aligned} \text{Service life} = & \text{Time to corrosion initiation} + \text{Time for cracking (0\% to 2\% deck damage)} \\ & + \text{Time for end of functional service life (EOFSL) (2\% to 12\% deck damage)} \end{aligned} \quad [\text{Eq. 10}]$$

where the percentage of deck damage represents the combined area of spalls, delaminations, and patches.

Typically, chlorides diffuse from the top deck surface through the concrete to reach the steel reinforcement mat. As this process continues over time, sufficient chloride accumulates around the reinforcement to initiate corrosion. The time period is defined as the time to corrosion initiation. The ingress of chloride through the concrete is a process that involves a number of transport mechanisms but may be effectively modeled using Fick's second law of diffusion. Once corrosion initiates, corrosion products (rust) take the place of the displaced base iron at the surface of the reinforcement. The corrosion products have been shown to consume 3 to 6 times the volume of the original iron (Liu and Weyers, 1998a), and their accumulation causes tensile stresses in the surrounding concrete. Liu and Weyers (1998b) modeled the time taken for accumulation of sufficient corrosion products to cause the concrete to crack. This time to cracking phase is represented by development of 2% surface area damage in Equation 10. Beyond cracking of concrete, rate of corrosion will depend on several factors, including spall formation, surface crack widths, depth of cracks, crack repairs, and others. The time for EOFSL

has been modeled using two methods: field observation of corrosion damage propagation (Fitch et al., 1995; Williamson, 2007) and continuous diffusion of chloride to the reinforcement level using Fick's second law of diffusion (Balakumaran, 2012). The diffusion method relies on the diffusion of chlorides to the reinforcement and ignores the supposedly increased rate of corrosion following the initial corrosion damage. Thus, the interpretation of the model results must be approached with these limitations in mind.

Table 16 presents the linear crack frequency and percent of deck area affected by cracking for all bridges. Crack frequency is presented as a ratio of feet of linear crack per square foot area of deck. For example, for the first bridge in the table (Federal Structure No. 16305), a crack frequency of 0.452 is indicated, which can be visualized as an average of 45.5 linear feet of cracks in each 100 square foot (10 ft × 10 ft) area. The area of deck influenced by a crack was further estimated as follows. For an average cover depth of 2 in and an assumed crack angle of 45° to form a spall, for a deck area of 1 ft², the area affected by a crack 1 ft long would be as follows:

$$\frac{((\tan 45^\circ \times (\frac{2}{12}) ft) \times 2 \times 1 ft)}{1 ft \times 1 ft} = 0.33 \frac{ft^2}{ft^2} \text{ or } 33.3\%$$

Thus, a 1 ft/ft² crack frequency influences 33.3% of 1 ft² of deck area. For each of the subgroups, the mean cover depth was used to calculate the crack-influenced deck area, which was 2.06 in for the 1968-71 No-SCM decks; 2.38 in for the 1984-91 No-SCM decks; and 2.46 in for the 1984-91 SCM decks, as presented in Figure 2.

The proportion of diffusion coefficients associated with cracked concrete ranged from 22% to 27% of the total diffusion coefficient samples in this study, whereas the percent of deck influenced by cracking was less in all but one case of 58% (see Table 16). The proportion of crack diffusion coefficients must match the crack influence area or the influence of cracking will be overestimated in all cases except for the 58% case.

The 1968-71 No-SCM decks had a crack influence area range of 1% to 16%. For the 1984-91 No-SCM decks, the crack influence area varied between 0.03% and 21% if the one outlier was omitted (58%). For decks with SCM, crack frequencies varied between 3% and 12%.

Service life estimations by a probabilistic chloride diffusion model for individual bridge decks will not provide reliable results because of the relatively small number of surface chloride and diffusion constants per deck (Balakumaran, 2012). Thus, the three groups as a whole were analyzed separately. To put the crack frequency and corresponding influenced areas into perspective, Figure 34 illustrates the deck areas with 3%, 6%, and 12% areas of crack influence. These maps were drawn to scale in terms of crack lengths compared to the deck surface area. For a constant cover depth of 2 in, the 3%, 6%, and 12% deck influence would correspond to 0.09, 0.18, and 0.36 ft/ft² of cracks, respectively.

Table 16. Crack Survey Results

VDOT District No.	Federal Structure No.	Year Built/ Replaced	Specified w/c Ratio	SCM	Crack Frequency (ft/ft ²)	Crack-Influenced Deck Area (%)
1	16305	1988	0.45	None	0.474	16
1	16307	1988	0.45	None	0.287	10
1	18526	1987	0.45	Fly ash	0.254	9
1	22469	1969	0.47	None	0.451	16
1	22374	1986	0.45	Fly ash	0.225	8
1	22356	1986	0.45	Fly ash	0.150	5
1	22380	1986	0.45	None	0.168	6
1	17624	1990	0.45	None	0.233	8
1	19192	1969	0.47	None	0.131	5
2	8364	1986	0.45	None	0.098	3
2	7775	1988	0.45	Fly ash	0.077	3
2	12144	1970	0.47	None	0.034	1
3	23033	1991	0.45	GGBFS	0.110	4
3	5711	1988	0.45	None	0.213	8
3	4191	1990	0.45	GGBFS	0.176	7
3	12419	1971	0.47	None	0.244	10
4	5142	1990	0.45	None	1.739	69
4	3577	1969	0.47	None	0.086	3
4	5972	1968	0.47	None	0.139	6
4	23233	1991	0.45	None	0.220	9
5	20240	1970	0.47	None	0.280	11
5	21945	1984	0.45	None	0.131	5
5	23098	1991	0.45	GGBFS	0.153	6
6	18067	1971	0.47	None	0.469	19
7	12919	1991	0.45	None	0.620	25
8	1683	1988	0.45	GGBFS	0.373	15
8	15217	1984	0.45	None	0.009	0.03
8	15361	1990	0.45	GGBFS	0.315	13
8	1046	1987	0.45	None	0.093	4
9	158	1987	0.45	GGBFS	0.190	8
9	11079	1987	0.45	None	0.258	11
9	11033	1990	0.45	None	0.629	26
9	114	1988	0.45	None	0.096	4
9	11061	1987	0.45	None	0.165	7
9	19948	1970	0.47	None	0.046	2
9	6742	1969	0.47	None	0.033	1
9	23121	1991	0.45	GGBFS	0.089	4

w/c ratio = water–cementitious material ratio; SCM = supplementary cementitious material; GGBFS = ground granulated blast furnace slag.

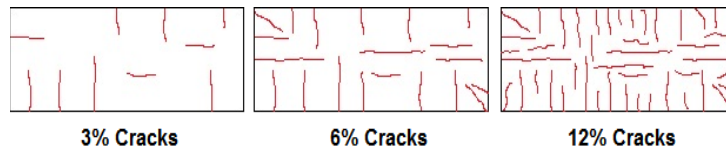


Figure 34. Illustration of 3%, 6%, and 12% Areas of Crack Influence in Decks

The service life model makes use of concrete cover depths, surface chloride concentrations, diffusion coefficients, and chloride threshold concentrations for corrosion initiation in the service life estimation. A maximum of nine sets of cover depth, surface chloride concentration, and diffusion coefficient data from uncracked concrete surfaces and about three sets of values (a few uncracked cores turned out to have cracks by observation in the lab) from cracked surfaces were available from each of the 27 bridges for use in the service life model. The collective 12 data points for each deck (Balakumaran, 2012) are fewer than desired for statistical significance, so the analysis involved sampling from aggregated data. As the influence of the cracks on the diffusion period was of primary interest in this study, a common triangular distribution for chloride concentrations at corrosion initiation was assumed, with a minimum, maximum, and mode of 0.6, 4.0, and 1.3 lb/yd³ (0.36, 3.27, and 0.77 kg/m³), respectively, based on previous studies of Virginia bridges (Balakumaran, 2012; Balakumaran et al., 2013). The service lives resulting from chloride diffusion and corrosion propagation were estimated for the decks using the bridge analysis program developed in a previous study (Balakumaran, 2012).

Diffusion coefficient was the only model parameter affected significantly by cracking. In order to determine the influence of cracking on the service life of a structure, diffusion coefficients for cracked and uncracked surfaces were grouped for comparative analysis. However, an important consideration was to find an appropriate proportion of diffusion coefficients from cracked surfaces and uncracked surfaces so that the influence of cracking on the overall service life would not be understated or overstated. It was also necessary to have a sufficient number of discrete input values for each parameter in the model to predict reasonable results, since too few samples can provide unrealistic results.

As discussed previously, a crack survey of each deck provided data on crack lengths, crack orientations, and surface crack widths at every foot along the length. For this analysis, the deck area affected by the cracks was needed to apportion diffusion coefficients. As mentioned previously, the deck area affected by a crack 1 ft long was estimated to be 0.33 ft². This factoring was used to determine the proportion of the deck area affected by cracking.

In each of the three groups, the low, median, and high values of crack frequency were identified to describe the range of observations. The proportions of diffusion coefficient values for cracked versus uncracked concrete used to estimate the service life were selected for these ranges. Another issue of concern was the selection of discrete diffusion coefficients for cracked concrete from the existing pool. Since the relationship between diffusion coefficients and surface crack widths was not robust, instead of randomly selecting a pre-calculated number of diffusion coefficients from the available set, the researchers selected groups of values in three ranges for comparison representing low, high, and median values. The pool of diffusion coefficients for cracked concrete was sorted in ascending order. In the first round of analysis (low), the researchers selected the required number of discrete values starting from the low value working up. In the second round of analysis (high), the researchers selected discrete values from the highest working down. In the last round (median), the researchers used values selected from the median and alternating upward and downward from there to represent the middle of the pool of diffusion coefficients. Service life estimates were generated using input values from each range to bracket the possible service life predicted for each type of concrete.

The diffusion parameters for the uncracked concrete were used in the estimation of the service life of the three groups of decks. Table 17 presents the results of the service life estimation. Time to corrosion initiation and cracking (up to 2% damage) represents the time taken for the corrosion to initiate in 2% of the top-mat reinforcement, which can be taken as the time when the damage begins to appear as delamination and spalls. The decks with SCM can last more than twice as long as decks with no SCM before corrosion initiation in 2% of the top-mat reinforcement, a point at which the earliest visible signs of corrosion damage have been noted.

In this analysis, the influence of cracks on the time to cracking phase of corrosion deterioration and subsequent time to EOFSL cannot be specifically assessed, as many parameters other than chloride ingress may influence these phases of deterioration. However, the continued ingress of chloride will occur in sound areas of the deck even as deterioration occurs at the sites of early initiation. As a surrogate for a more robust model to address corrosion-induced cracks, delamination, and spalls directly, the time for further diffusion of chloride and initiation (up to 12% of the top-mat reinforcement) is demonstrated. This has been documented to correspond to the point at which many bridge engineers conduct rehabilitative overlay or replacement (Fitch et al., 1995).

As discussed in the “Methods” section, the decks with a 0.47 w/c ratio were a subset that showed superior performance from a larger set of decks. The superior performance was characterized by the lack of a need for overlaying those decks even after 32 to 35 years of service, when the testing was conducted. The reason could be better resistance to chloride ingress through reduced pore volume (Figure 7) and lower moisture saturation (Figure 8) in the decks with a 0.47 w/c ratio. The decks with a 0.45 w/c ratio had major differences in performance with the presence of SCM (unknown quantities of fly ash or slag). Younger decks with a 0.45 w/c ratio with no SCM showed much earlier corrosion initiation than older decks with a 0.47 w/c ratio. Even though the specified w/c ratio was lower for the decks with a 0.45 w/c ratio, the void volume was higher than that of decks with a 0.47 w/c ratio. This might be because of an unintended water usage that was higher than the specified amount.

Table 17. Service Life of Uncracked Bridge Deck Groups (in Years)

Group	0.47 w/c, No-SCM		0.45 w/c, No-SCM		0.45 w/c, SCM	
	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
Time to corrosion initiation (0% to 2% deck damage)	16		8		28	
Time to diffusion (0% to 12% deck damage)	47		20		105	
Input variables	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
Cover depth (in)	2.04	0.41	2.44	0.45	2.44	0.36
Surface chloride (lb/yd ³)	5.71	3.99	5.84	3.07	6.47	4.21
Diffusion coefficient (in ² /yr)	0.021	0.027	0.065	0.076	0.035	0.119

w/c = water–cementitious material ratio; SCM = supplementary cementitious material.

The time to corrosion initiation in 12% of rebars in the absence of cracks followed the same trend as the time to corrosion initiation. The input parameters for the model are presented in Table 17. As discussed previously, diffusion coefficients influenced the service life more than the other parameters. Even though the mean diffusion coefficient of 0.45 w/c ratio SCM concrete was one-half as much as 0.45 w/c ratio no-SCM concrete, the variability was quite high. Variability in diffusion coefficients may represent the unpredictability of the concrete pore interconnectivity in SCM concrete.

Table 18 presents the range of diffusion coefficients excluding the outliers (2.026 in²/yr for the 0.45 w/c No-SCM decks and 5.560 in²/yr for the 0.45 w/c SCM decks) for the three categories of crack frequencies and three levels of diffusion at cracked locations. Cracks had a significant influence on diffusion (Figure 13); however, a number of factors influenced the degree to which diffusion was affected. Since surface crack width and crack depth did not have a strong correlation with diffusion (Figure 11, Figure 12, Figure 21 through Figure 26), the remaining factors influencing diffusion were outside the scope of this study. Thus, the diffusion coefficients for cracked locations were divided into low, median, and high based on their magnitude and were used for the analysis of service life estimates.

As shown in Table 18, different crack frequencies for the low, median, and high categories were selected based on the existing data from the three groups. From Table 17, where the time to corrosion initiation for uncracked concrete is presented, the researchers understood that the order of subgroups in terms of performance was as follows from high to low:

- 0.45 w/c, SCM
- 0.47 w/c, No-SCM
- 0.45 w/c, No-SCM.

Table 18. Diffusion Coefficient Ranges for Service Life Predictions

Cracked/ Uncracked	Crack Frequency	Diffusion at Cracks	Crack-Influenced Deck Area and No. of Data Points					
			0.47 w/c, No-SCM (yr)		0.45 w/c, No-SCM (yr)		0.45 w/c, SCM (yr)	
			Freq., No.	D _c (in ² /yr)	Freq., No.	D _c (in ² /yr)	Freq., No.	D _c (in ² /yr)
Cracked	Low	Low	2%, 2	0.020–0.022	3%, 4	0.033–0.037	3%, 3	0.009–0.023
		Median		0.042–0.045		0.110–0.136		0.081–0.095
		High		0.327–0.420		0.386–1.297		1.748–2.651
Cracked	Median	Low	5%, 5	0.020–0.029	9%, 14	0.033–0.078	7%, 6	0.009–0.025
		Median		0.042–0.071		0.084–0.172		0.050–0.119
		High		0.143–0.420		0.202–1.297		0.450–2.651
Cracked	High	Low	16%, 17	0.020–0.073	25%, 46	0.033–0.386	15%, 14	0.009–0.059
		Median		0.029–0.143		0.036–0.428		0.031–0.202
		High		0.037–0.420		0.037–1.297		0.126–2.651
Uncracked	--	--	91	0.0015– 0.186	137	0.0015– 0.741	80	0.0015–0.897

w/c = water–cementitious material ratio; SCM = supplementary cementitious material.

As presented in Table 18, a predetermined proportion of uncracked versus cracked diffusion data points was used in the service life estimation. The results of the estimations of the time to corrosion initiation in 2% and 12% of rebars among the three subgroups in a total of six plots are presented in Figure 35.

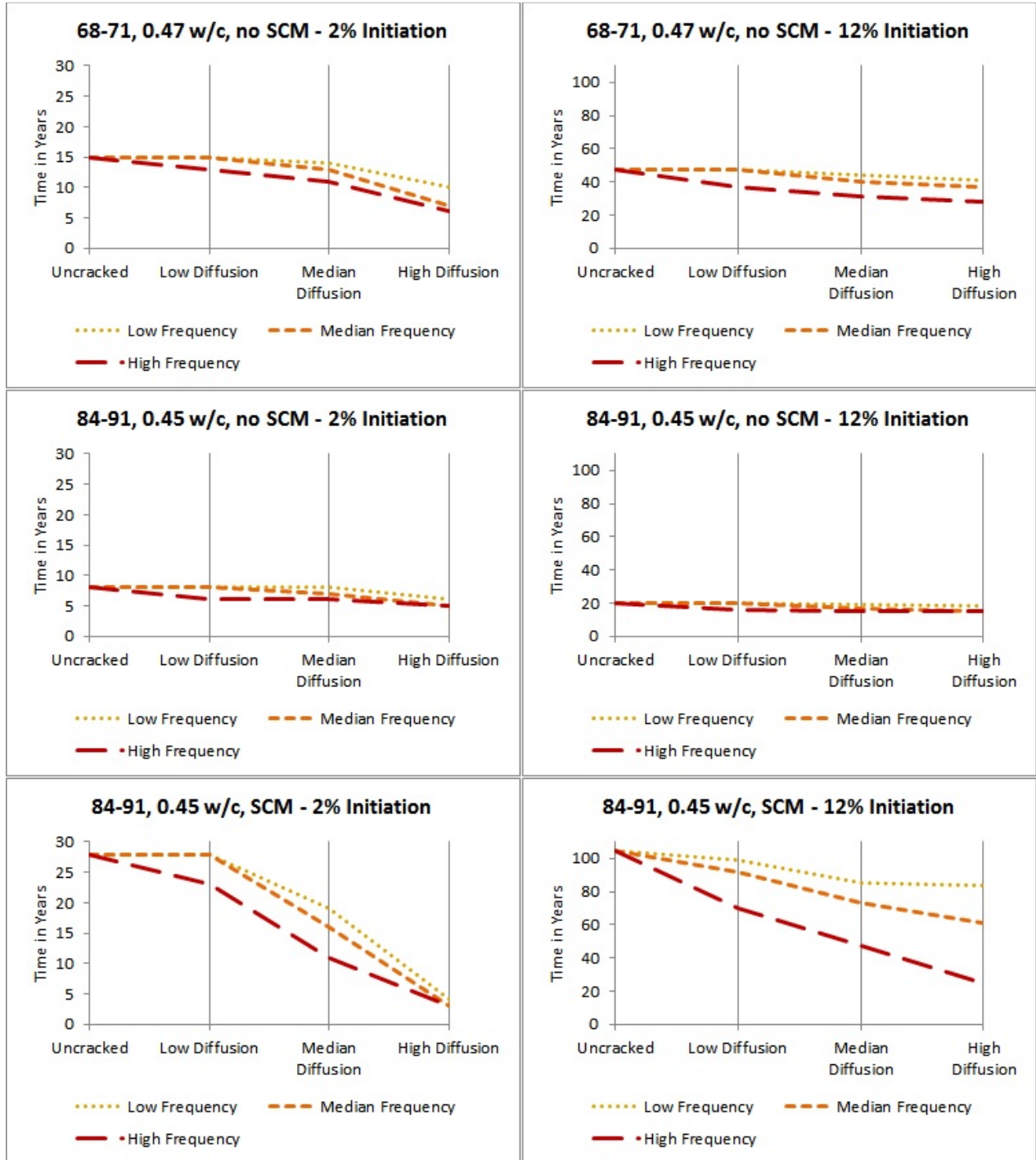


Figure 35. Crack Influences on Service Life Estimations Among Subgroups

Each plot is drawn with varying degrees of chloride diffusion through cracks, which starts from diffusion through uncracked concrete, against the estimated time to corrosion initiation. Plots consist of three lines representing low, median, and high crack frequencies for each subgroup.

Corrosion Initiation in 2% of Rebars

1968-71, 0.47 w/c ratio, No-SCM. Up until the median crack frequency (5% deck area) for up to the median level of chloride diffusion through the cracks, the time to corrosion initiation in 2% of rebars did not vary much from that for the uncracked condition. In highly cracked decks, the time to corrosion initiation decreased almost linearly with increasing degree of chloride diffusion through cracks. If cracks allow high chloride diffusion, corrosion initiation will happen much earlier compared to milder diffusion rates.

1984-91, 0.45 w/c ratio, No-SCM. The total range of reduction in time to corrosion initiation in 2% of rebars was the lowest of all three subgroups. This could be because of the low quality of the concrete. Therefore, the cracks did not pose a significant threat to the durability of the structure.

1984-91, 0.45 w/c ratio, SCM. The total range of reduction in time to corrosion initiation in 2% of rebars was the highest in this subgroup. This could be because of the higher quality of the concrete with added SCM. Low diffusion of chlorides through cracks did not affect the service life by a significant amount. However, the median diffusion and high diffusion of chlorides through cracks drastically reduced the time to corrosion initiation in 2% of rebars. At high diffusion of chlorides through cracks, the frequency of cracks becomes irrelevant. This lack of influence of crack frequency on corrosion initiation is because initiation is assumed when corrosion begins in 2% of rebars, which is less than the minimum crack-influenced area of 3% in 0.45 w/c ratio groups, as shown in Table 18.

Corrosion Initiation in 12% of Rebars

1968-71, 0.47 w/c ratio, No-SCM. The reduction in time to corrosion initiation in 12% of rebars appears similar to the reduction in corrosion initiation in 2% of rebars. High crack frequencies resulted in higher reductions in time to corrosion initiation in 12% of rebars.

1984-91, 0.45 w/c ratio, No-SCM. The reduction in time to corrosion initiation in 12% of rebars was the lowest among the subgroups. Again, this was because of the low quality of the concrete. Thus, the presence of cracks did not affect durability by a significant amount.

1984-91, 0.45 w/c ratio, SCM. The total range of reduction in time to corrosion initiation in 12% of rebars was the highest in this subgroup. There is an apparent change in service life with increasing crack frequency and increasing chloride diffusion through the cracks. This is because of the higher quality of the concrete. The presence of cracks had the highest impact on the service life of this subgroup.

Appendix A gives service life estimates for decks with corrosion initiation of 2% and 12% of rebars by subgroup.

Corrosion Initiation in Recent Bridge Deck Concrete

Bridge decks in Virginia have been built with high performance concrete, with SCMs and a lower w/c ratio, since the mid-1980s and different types of corrosion-resistant reinforcement since mid-2000s. These measures are expected to increase the service life of bridges without the need of constant maintenance. However, high performance concrete does crack and expose the reinforcement to chloride and other harmful chemicals. The key question to be answered is whether the rate and amount of chloride diffusion experienced in Virginia had detrimental effects on the service life of the bridge decks.

Service life estimation was conducted using the chloride diffusion properties of the concrete from the 1984-91 SCM construction era. The chloride threshold for corrosion initiation for corrosion-resistant rebar, ASTM A1035 (low carbon, chromium steel bars) (ASTM, 2016) and ASTM A955 (stainless steel bars) (ASTM, 2017) were taken as triangular distributions with 1.37, 21.91, 4.90 lb/yd³ and 4.06, 65.1, 14.56 lb/yd³, respectively, for minimum, maximum, and mode. Service life estimations indicated excellent corrosion resistance for decks with low crack frequencies for corrosion-resistant rebars. With medium crack frequencies, corrosion-resistant rebars had corrosion initiation at 56 years if cracks allowed higher amounts of chloride diffusion. For higher crack frequencies, corrosion-resistant rebars had corrosion initiation as early as 30 years when crack frequency was high and the cracks allowed higher chloride diffusion. However, for stainless steel reinforcement, it did not matter if the crack frequency was high and if higher chloride diffusion was allowed through the cracks: the corrosion initiation in 2% of rebars did not occur within 150 years of service.

Tables A5 and A6 in Appendix A present detailed information on the time to corrosion initiation for 2% and 12% of rebars, respectively.

Simplifying Inputs

One of the steps that would simplify the service life prediction of bridge decks is finding a strong dependency of one or more input parameters on easily accessible factors. The geography of a bridge has considerable influence on the exposure conditions and required amount of deicing salt application and thus the service life of bridge decks to an extent.

Surface Chloride Concentrations

Surface chloride concentrations depend on a number of factors such as the amount and number of deicing salt applications; concrete quality; and factors such as precipitation, traffic, and wind to blow away the salt before diffusion. Surface chloride concentrations were determined for each geographic zone (Figure 36).

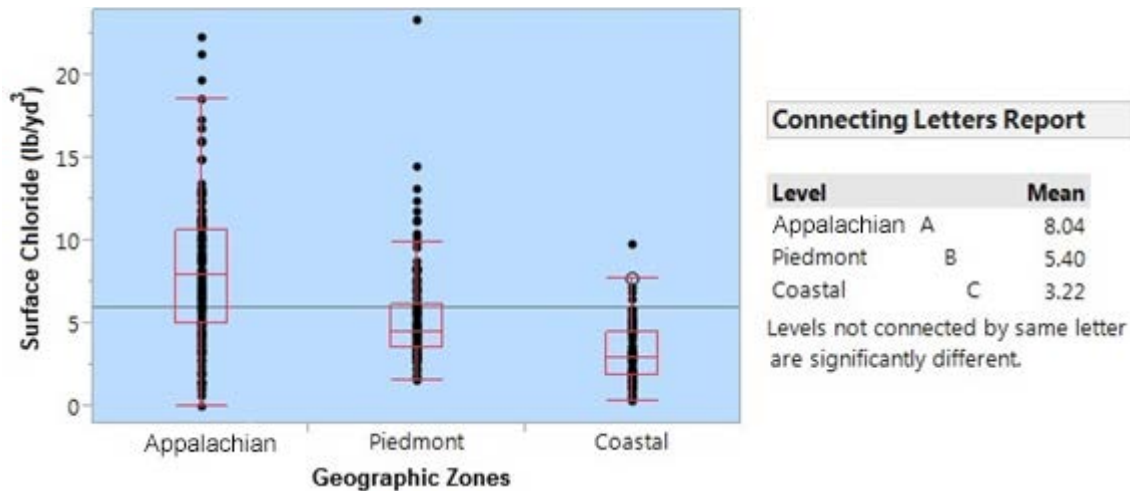


Figure 36. Surface Chloride by Geographic Zone

The decks in the Appalachian zone had the highest surface chloride concentration; those in the Piedmont zone had moderate surface chloride concentrations; and those in the Coastal zone had the lowest surface chloride concentrations. The *t*-test results in the form of the connecting letters report shown in Figure 37 indicated that the concentrations in the zones were significantly different. Thus, geographic zones can be used for modeling surface chloride concentrations.

Since the decks in the different zones had significantly different surface chloride concentrations, in order to substitute or reduce the field data collection, a parametric bootstrapping was conducted. A parametric bootstrapping involves generating random values from a defined probability distribution. Thus, the surface chlorides from the three zones were tested for goodness of fit. Figure 37 presents the histograms of surface chlorides for each geographic zone and the gamma distribution fitting. Gamma distribution was chosen because it is used to represent the processes where the intervals between events are relevant. Surface chloride, as it is the accumulation of chloride from winter maintenance activities, may be modeled as a gamma distribution. Table 19 presents the gamma distribution parameters and the Cramer-von Mises goodness-of-fit test results.

From Table 19, the *p*-values higher than the significance level of 0.05 from the goodness-of-fit tests indicate that the three distributions can be modeled using a gamma distribution. The parameters of the gamma distribution are also presented in Table 19. From this finding, a set of surface chloride values can be generated using the appropriate parameters based on geographic zone for running the service life model, instead of collecting an extensive set of field data.

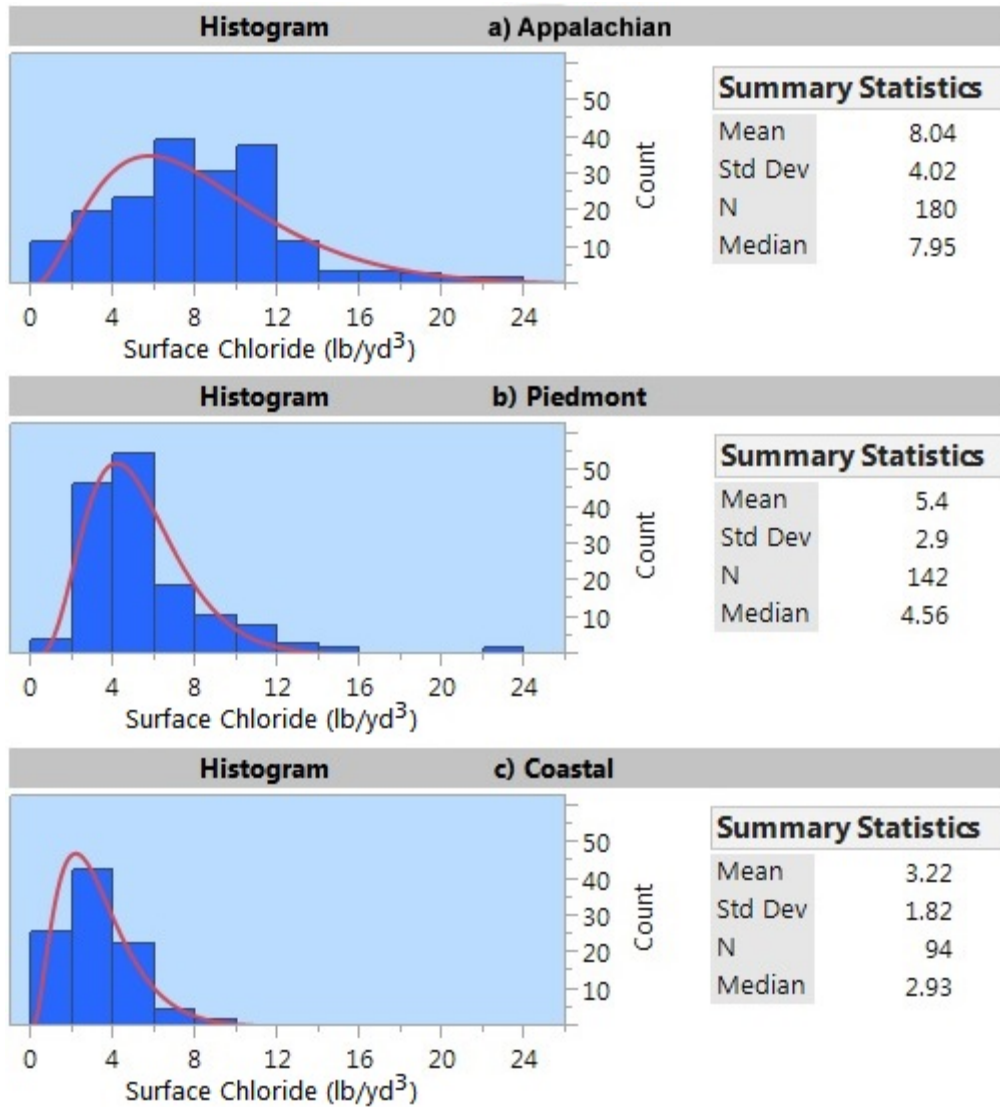


Figure 37. Surface Chloride Distribution by Geographic Zone

Table 19. Surface Chloride Gamma Distribution Fitting

Geographic Zone	Shape Parameter	Scale Parameter	Threshold Parameter	Cramer-von Mises <i>p</i> -value
Appalachian	3.10	2.70	0	0.1391
Piedmont	4.70	1.10	0	0.2406
Coastal	2.90	1.10	0	0.2500

SUMMARY OF FINDINGS

- On average, about 75% of the cracks in the bridge decks evaluated were less than 0.2 mm in surface width.
- Longitudinal cracks were more prevalent, followed by transverse and diagonal cracks.
- Geographic zone, crack width, crack depth, and use of SCM in the concrete had a significant influence on the moisture saturation of decks.
- Concrete mixtures with a specified w/c ratio of 0.45 had a higher volume of pore spaces and saturation than mixtures with a specified w/c ratio of 0.47, which may be the result of a higher water usage than specified and/or the differences in the modern fine-grinded cement properties.
- Surface chloride concentration had a significant correlation with geographic zone, enabling simplified input in service life models.

Influence of Cracking

- The chloride diffusion coefficient, chloride concentration at the rebar depth, and degree of concrete moisture saturation were significantly higher in cracked locations than in uncracked locations.
- Cracks with a width greater than 0.3 mm (0.012 in) allowed faster chloride diffusion, but the difference was not statistically significant, whereas the chloride concentration at the rebar depth was significantly higher than for concrete with narrow cracks and no cracks. Cracks narrower than 0.15 mm (0.006 in) had relatively higher diffusion coefficients.
- Crack depth had a significant influence on chloride diffusion. Concrete samples with crack depths greater than 51 mm (2 in) had significantly higher diffusion coefficients, chloride concentrations at the rebar depth, and areas of surface rust on rebars than concrete samples with shallow cracks or uncracked locations.
- The estimated service life of the decks was affected by the presence of cracks to varying degrees. Exposure conditions and the in-place concrete mixture characteristics determined the extent to which the service life was affected.
- Decks for which the presence or absence of cracks had no influence on durability had significantly higher pore spaces within the bulk concrete.
- Cracks deeper than 1 in (25.4 mm) increased moisture saturation significantly.

Influence on Cracking

- There were fewer transverse cracks in the decks of bridges with concrete girders in the superstructure versus steel girders.
- The w/c ratio was not related to the occurrence of cracking.
- The occurrence of transverse cracking was higher in continuous span structures.
- Decks with SIP forms had a lower occurrence of transverse cracks and a higher occurrence of longitudinal cracks, but the difference was not statistically significant.
- The occurrence of longitudinal cracking was higher in decks with ECR.
- Decks built in 1984-91 without SCM had more cracking than decks built in the same time period with SCM. This is because of the higher cement content and lower w/c ratio.

Cracking Influence on Service Life

- Chloride diffusion did not have a strong correlation with surface crack width; however, chloride diffusion was significantly faster for cracked locations than for uncracked locations.
- The durability of decks without SCM was significantly lower than for decks with SCM; however, the service life of decks built without SCM was affected less severely by the incidence of cracks.
- Decks with SCM performed well when the crack frequency was less than or equal to 0.1 ft/ft².
- The service life of decks with SCM depended on the degree to which the cracks affected chloride diffusion.

CONCLUSIONS

- *Cracked concrete allows significantly greater chloride diffusion than uncracked concrete.*
- *Decks with bare steel reinforcement and ECR built in the 1984-91 era with a crack frequency less than 0.1 ft/ft² may not need repairs.*
- *Surface crack width is not highly correlated with the rate of chloride diffusion, whereas crack depth is highly correlated. Even at a narrow crack width of 0.15 mm, cracks allow significant chloride diffusion to occur.*

- *The service life of bridge decks built with low permeability concrete ($\sim < 1000$ coulombs) with SCM is adversely affected significantly by the presence of cracks, whereas the service life of bridge decks built with OPC concrete is not, primarily because the high permeability of OPC concrete, with or without the presence of cracks, results in a shorter service life for OPC bridge decks.*
- *Decks without SCM with material properties similar to those presented in Table 5 may not need crack repair; however, a low permeable deck protection system should be considered for slowing the moisture diffusion.*
- *The service life of decks built with VDOT's current concrete mixture specification with ASTM A1035 (MMFX-2) or ASTM A955 (stainless steel) reinforcement will be much longer than that of decks constructed with bare steel reinforcement. Further, the time to corrosion initiation with stainless steel reinforcement was much later than 150 years of service. With high crack frequencies and harsh environmental exposures, the time to corrosion initiation with MMFX-2 reinforcement can be as soon as 30 years.*
- *ASTM A1035 (MMFX-2) reinforcement will undergo corrosion initiation when a median crack frequency of approximately 0.170 ft/ft^2 and above allows high diffusion of chloride. At the extreme conditions of a high crack frequency of 0.37 ft/ft^2 and above and higher chloride diffusion through the cracks, corrosion initiation can happen as early as 30 years of service.*
- *ASTM A955 (stainless steel) reinforcement will not undergo corrosion initiation under the chloride diffusion conditions in Virginia before 150 years even if the higher crack frequency observed in this study is the case in concrete decks.*

RECOMMENDATIONS

1. *VDOT's Structure and Bridge Division should make updates to VDOT's Guide Manual for Causes and Repair of Cracks in Bridge Decks (VDOT, 2009) or make additions to VDOT's Manual of the Structure and Bridge Division, Part 2, Chapter 32 (VDOT, 2012) in accordance with the findings of this study. A list of recommended changes is provided in Appendix B.*
2. *The Virginia Transportation Research Council (VTRC) and VDOT's Structure and Bridge Division should undertake a follow-up study of the frequency and characteristics of concrete cracking in bridge decks built with ASTM A1035 (MMFX-2) and ASTM A955 (stainless steel) reinforcement, which were modeled in this study. Such decks were not physically evaluated in the current study because of the lack of these structures during the study period.*

BENEFITS AND IMPLEMENTATION

Benefits

Implementing Recommendation 1 will aid VDOT decision making regarding crack repair based on the concrete mix design and reinforcement type of the bridge decks of interest. Crack repair methods are expensive to carry out; labor-intensive; and, quite often, ineffective because of delayed action. Implementing the recommended changes to VDOT's *Manual of the Structure and Bridge Division* provided in Appendix B will enable decision makers to select appropriate crack repair methods for various conditions in bridge decks. The results of this study showed the degree of the influence of cracks on the service life of bridge decks. By implementing the changes provided in Appendix B, the service life of decks with concrete cracking can be extended, thus saving immense time and money spent on premature replacements.

Implementing Recommendation 2 will provide valuable information regarding chloride diffusion in the bridge decks. VDOT has been building bridge decks with high performance concrete and corrosion-resistant reinforcement recently, and this trend will continue in the future to extend the service life of decks. However, more information about the cracking characteristics of these decks is needed.

Implementation

With regard to Recommendation 1, VTRC will work with the design and bridge management teams in VDOT's Structure and Bridge Division to modify guidance in VDOT's *Manual of the Structure and Bridge Division*. This is expected to be completed by August 2018.

With regard to Recommendation 2, VTRC will work with its Bridge Research Advisory Committee in the fall of 2018 to initiate a research project to characterize the cracking in new bridge decks with corrosion-resistant reinforcement.

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APPENDIX A

SERVICE LIFE ESTIMATIONS

Table A1. Time to Corrosion Initiation in 2% of Rebars

Degree of Crack Frequency	Category	1968-71, 0.47 w/c, No-SCM (yr)	1984-91, 0.45 w/c, No-SCM (yr)	1984-91, 0.45 w/c, SCM (yr)
Uncracked	Uncracked	15	8	28
Low frequency	Low diffusion	15	8	28
	Median diffusion	14	8	19
	High diffusion	10	6	4
Median frequency	Low diffusion	15	8	28
	Median diffusion	13	7	16
	High diffusion	7	5	3
High frequency	Low diffusion	13	6	23
	Median diffusion	11	6	11
	High diffusion	6	5	3

w/c = water–cementitious material ratio; SCM = supplementary cementitious material.

Table A2. Time to Corrosion Initiation in 12% of Rebars

Degree of Crack Frequency	Category	1968-71, 0.47 w/c, No-SCM (yr)	1984-91, 0.45 w/c, No-SCM (yr)	1984-91, 0.45 w/c, SCM (yr)
Uncracked	Uncracked	47	20	105
Low frequency	Low diffusion	47	20	99
	Median diffusion	44	19	85
	High diffusion	41	18	84
Median frequency	Low diffusion	47	20	92
	Median diffusion	40	17	73
	High diffusion	37	15	61
High frequency	Low diffusion	37	16	70
	Median diffusion	31	15	47
	High diffusion	28	15	25

w/c = water–cementitious material ratio; SCM = supplementary cementitious material.

Table A3. Reduction in Years for 2% Corrosion Initiation for Cracked Deck Subgroups

Degree of Crack Frequency	Diffusion at Cracks	Reduction in Time for 2% Corrosion Initiation					
		0.47 w/c, No-SCM (yr)		0.45 w/c, No-SCM (yr)		0.45 w/c, SCM (yr)	
Low	Low	0	(0%)	0	(0%)	0	(0%)
	Median	-1	(-7%)	0	(0%)	-9	(-32%)
	High	-5	(-33%)	-2	(-25%)	-24	(-86%)
Median	Low	0	(0%)	0	(0%)	0	(0%)
	Median	-2	(-13%)	-1	(-13%)	-12	(-43%)
	High	-8	(-53%)	-3	(-38%)	-25	(-89%)
High	Low	-2	(-13%)	-2	(-25%)	-5	(-18%)
	Median	-4	(-27%)	-2	(-25%)	-17	(-61%)
	High	-9	(-60%)	-3	(-38%)	-25	(-89%)

w/c = water–cementitious material ratio; SCM = supplementary cementitious material.

Table A4. Reduction in Years for 12% Corrosion Initiation for Cracked Deck Subgroups

Degree of Crack Frequency	Diffusion at Cracks	Reduction in Time for 12% Corrosion Initiation					
		0.47 w/c, No-SCM (yr)		0.45 w/c, No-SCM (yr)		0.45 w/c, SCM (yr)	
Low	Low	0	(0%)	0	(0%)	-6	(-6%)
	Median	-3	(-6%)	-1	(-5%)	-20	(-19%)
	High	-6	(-13%)	-2	(-10%)	-21	(-20%)
Median	Low	0	(0%)	0	(0%)	-13	(-12%)
	Median	-7	(-15%)	-3	(-15%)	-32	(-30%)
	High	-10	(-21%)	-5	(-25%)	-44	(-42%)
High	Low	-10	(-21%)	-4	(-20%)	-35	(-33%)
	Median	-16	(-34%)	-5	(-25%)	-58	(-55%)
	High	-19	(-40%)	-5	(-25%)	-80	(-76%)

w/c = water–cementitious material ratio; SCM = supplementary cementitious material.

Table A5. Time to Corrosion Initiation in 2% of Rebars

Degree of Crack Frequency	Category	ASTM A1035 Rebar (yr)	ASTM A955 Rebar (yr)
Uncracked	Uncracked	150+	150+
Low frequency	Low diffusion	150+	150+
	Median diffusion	150+	150+
	High diffusion	100+	150+
Median frequency	Low diffusion	150+	150+
	Median diffusion	100+	150+
	High diffusion	56	150+
High frequency	Low diffusion	100+	150+
	Median diffusion	64	150+
	High diffusion	30	150+

Table A6. Time to Corrosion Initiation in 12% of Rebars

Degree of Crack Frequency	Category	ASTM A1035 Rebar (yr)	ASTM A955 Rebar (yr)
Uncracked	Uncracked	150+	150+
Low frequency	Low diffusion	150+	150+
	Median diffusion	150+	150+
	High diffusion	150+	150+
Median frequency	Low diffusion	150+	150+
	Median diffusion	150+	150+
	High diffusion	150+	150+
High frequency	Low diffusion	150+	150+
	Median diffusion	150+	150+
	High diffusion	150+	150+

APPENDIX B

RECOMMENDATIONS FOR CHANGES TO VDOT'S *MANUAL OF THE STRUCTURE AND BRIDGE DIVISION*, PART 2, CHAPTER 32: MAINTENANCE AND REPAIR

The following are the recommendations regarding bridge deck cracks.

1. Since the width of surface cracks was found to have minimal correlation to chloride diffusion, all linear cracks should be treated as detrimental to the service life of bridge decks.
2. *Definitions*
 - 2.1. *Crack frequency*: Crack frequency is defined as the cumulative length of linear cracks in a bridge deck divided by the surface area of the bridge deck, presented as ft/ft² or m/m², irrespective of the orientation and size. Crack frequency should be calculated for a single worst lane of a bridge deck as a representative sample.
 - 2.2. *Background chloride*: Background chloride is the initial amount of chloride content present in the concrete, introduced as a part of any of the ingredients, before the bridge deck is open for traffic. For a newly constructed deck, this value can be found from concrete samples saved for testing. This value can be estimated for older decks without records by determining chloride content in deep concrete samples, below the top reinforcement mat at depths of about 4 inches (100 mm). Background chloride is typically between 0.2 and 0.5 lb/yd³ of concrete. Samples should be taken in traffic lanes away from cracks, expansion joints, and construction joints.
 - 2.3. *Active cracks*: Active cracks are those that exhibit significant cyclical dimensional changes, particularly width and depth. Dimensional changes may be due to temperature, live load, or other periodic load events.
 - 2.4. *Type B patching*: Type B patching, according to VDOT's *Road and Bridge Specifications* (2016), Section 412.03.b.2, consists of removing the deck concrete from the existing surface to a depth at least 1 inch below the top reinforcement mat and patching with concrete.
3. *Exceptions*:
 - 3.1. Linear cracks with a surface width of 0.43 mm (0.017 inch) and greater should be sealed (6.1) to avoid freeze-thaw damage from stored moisture and carbonation and for the purpose of aesthetics (AASHTO, 2012)(AASHTO, 2012).
 - 3.2. Full-depth cracks should be addressed to prevent moisture leakage into bridge components such as girders, columns, pier caps, and abutments. The cause of the full-depth cracks should be investigated to prevent further cracking.
 - 3.3. Active cracks warrant an investigation into their cause to prevent further propagation.
 - 3.4. Structural cracks warrant an investigation into their cause to develop proper remedial action.

4. *Guidance on treating concrete cracks:*

4.1. *ASTM A955 Steel (VDOT Class III):*

4.1.1. Partial-depth cracks need not be addressed for the purpose of protection from chloride-induced corrosion unless any of the exceptions in (3) applies.

4.2. *ASTM A1035 Steel (VDOT Class I):*

4.2.1. Decks observed with a crack frequency lower than 0.2 ft/ft², or 20 feet per 100 square feet of deck surface, need not be addressed unless any of the exceptions in (3) applies.

4.2.2. Decks observed with a crack frequency higher than 0.2 ft/ft², or 20 feet per 100 square feet of deck surface, should be sealed (6.1). This may result in a two-fold increase in the time to corrosion initiation for bridge decks built with ASTM A1035 steel reinforcement, assuming that the cracks have properties that allow higher than typical chloride diffusion.

4.3. *Bare Steel and Epoxy-Coated Steel Reinforcement:*

4.3.1. Decks, irrespective of age, with crack frequencies lower than 0.1 ft/ft², or 10 feet per 100 square feet of deck surface, need not be addressed unless any of the exceptions in (3) applies.

4.3.2. Decks observed with a crack frequency higher than 0.1 ft/ft², or 10 feet per 100 square feet of deck surface, should be addressed as described.

4.3.2.1. For decks that have experienced more than 1 year of service, concrete should be evaluated according to the evaluation procedure in (5).

4.3.2.1.1. If the chloride content at the rebar depth is less than 1.0 lb/yd³ and the chloride content near the surface at an 0.5-inch depth is sufficiently low to indicate a relatively low future penetration rate, then the cracks should be sealed (6.1). An engineering decision may be made to apply a preventive overlay (6.2) to delay the onset of chloride-induced corrosion deterioration.

4.3.2.1.2. If the chloride content is between 1.0 and 2.0 lb /yd³ at the rebar depth and the chloride content near the surface at a 0.5-inch depth is sufficiently high to indicate a relatively high future penetration rate after crack sealing, it must be recognized that crack sealing alone will not be effective, especially if there are even higher chloride contents at the rebar depth. Depending on the permeability of the concrete, the deck should be scheduled for type B repair: (a) when the chloride content is greater than 2.0 lb/yd³ for concrete permeability greater than 4000 coulombs; (b) when the chloride content is greater than 3.0 lb/yd³ for concrete permeability between 2000 and 4000 coulombs; and (c) when the chloride content is greater than 4.0 lb/yd³ for concrete permeability lower than 2000 coulombs.

5. *Evaluation Procedure:*

5.1. *Chloride Profile:* Chloride concentrations should be determined at a minimum of three sample locations in a bridge deck or a minimum of one location for every 600 square feet of deck surface area, whichever is greater. At every location, concrete sampling should be performed over visible crack locations in a wheel path and at a minimum of two depths, as measured from the concrete surface, at 0.5 inch (13 mm) and at top

reinforcement depth. As an option, an additional chloride sample may be collected from a depth of 1.5 inch. SHRP Report S-360 contains a numerical technique to project future chloride diffusion using the present chloride values at 0.5 (13 mm) and 1.5 inch (37 mm).

- 5.1.1. Concrete samples for chloride profiles may either be a wet drilled concrete core with a minimum diameter of 2 inches or be field-powdered concrete. The topmost 0.25 inch of surface concrete should be ignored to avoid issues.
 - 5.1.1.1. Using a water-lubricated diamond core bit, concrete cores are to be drilled to a minimum depth of 4 inches, broken loose, and removed, and the surface-dried samples are to be wrapped to preserve the in-place moisture.
 - 5.1.1.2. Powdered samples should be drilled using a carbide drill bit with a diameter at least 1.5 times the maximum coarse aggregate size. Powdered concrete samples may be collected using a vacuum apparatus or a different collecting device and placed in individual labeled plastic sample bags or canisters. Care should be taken to prevent cross-contamination between concrete samples.
 - 5.1.1.3. Chloride contents should be determined using acid-soluble titrations performed in accordance with ASTM C1152. The background chloride concentration should be subtracted from all measured chloride concentrations.

6. *Crack Repair:*

- 6.1. *Crack sealing:* The recommended treatment for cracks less than 0.2 mm in width is the application of the gravity-fill polymer method. The recommended treatment for cracks greater than or equal to 0.2 mm is epoxy injection or, if warranted by the crack density, an overlay. The effectiveness of the crack sealing, with the exception of the gravity-fill polymer treatment, should be evaluated by selective destructive coring of the sealed cracks. A minimum of three destructive cores should be collected from decks with up to 20 ft/ft² of cracks. For every additional increment of 5 ft/ft² crack frequencies beyond 20 ft/ft², one additional destructive core should be collected for evaluation.
- 6.2. *Preventive deck overlaying:* To treat widespread cracking, a thin epoxy overlay or a similar overlay with high resistance to chloride permeability should be placed after the cracks are addressed.
- 6.3. *Rigid overlaying:* Rigid overlaying should be applied by removing critically chloride-contaminated concrete, through milling and/or hydro-demolition, to a depth below the top reinforcement mat in order to clear the corrosive environment around the reinforcement. Latex-modified concrete overlay, silica fume overlay, rapid-set latex-modified overlay or an equivalent rigid concrete overlay may be applied to extend the service life