EXTENDING THE LIFESPAN OF REINFORCED CONCRETE BRIDGE DECKS: HISTORICAL ANALYSIS, MODELING, AND IMPROVED CONTRACTING

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Extending the Lifespan of Reinforced Concrete Bridge Decks:
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AC : Average cover

AQL : Acceptable quality limit

C : Chloride concentration

Ca : Average cover along a deck

Cd : Design cover

C_F : Cracking fraction

Cn : Normalized cover

C_s : Surface chloride concentration

C_{sh} : Upper limit of Cs

C_{sl} : Lower limit of Cs

C_t : Chloride threshold

C_0 : Initial chloride concentration in concrete

C_1 : Chloride concentration at the surface (Cs equivalent)

D : Chloride diffusivity in concrete

DC : Design cover

$DF(t)$: Damage function at time t

D_h : Upper limit of D

D_l : Lower limit of D

$DM(t)$: Damage at time t

$DM_C(t)$: Damage function for the cracked portion of the deck at t

E : Maximum acceptable deviation between the true and sample averages

f_{cs} : Probability distribution function for Cs

f_D : Probability distribution function for D

F_x : Cumulative probability distribution for cover

I : Moran's I statistic

$I_{expected}$: Expected value for the Moran's I statistic

LSL : Lower specification limit

n : Number of data points

n_a : Number of trials where I for the simulated deck matched or exceeded that of the real deck

n_b : Number of trials where I for the simulated deck was less than that of the real deck

PF : Pay factor

$P_{simulated}$: Significance value from Monte Carlo simulation

PWL : Percent within limits

RQL : Rejection quality level

S_0 : Sum of spatial weights

t : A time

t_i : Corrosion initiation period

t_p : Corrosion propagation period

t_{sl} : Service life of a bridge deck

USL: Upper specification limit

$w_{i,j}$: Assigned spatial weights for i and j

x : Concrete cover

x_i : Cover at location i

x_j : Cover at location j

z_i : Difference in cover between x_i and the mean

z_j : Difference in cover between x_j and the mean

ζ : Standard deviation of a lognormal distribution

λ : Mean of a lognormal distribution

μ :	Mean
σ :	Sum of spatial weights
σ_{center} :	Standard deviation of the results of a process about a target value
$\sigma_{combined}$:	Standard deviation accounting for process and center deviations
$\sigma_{process}$:	Inherent standard deviation of a process
σ_0 :	A priori estimate of the standard deviation for a process
φ :	The concentration at distance and time
§ :	Section in a code or document

SUMMARY

Bridges are vital to the safe and efficient conveyance of people and goods around the world. For this reason they are considered critical structures. Despite their importance to society, bridges are often compromised by a wide range of deficiencies that require significant rehabilitation, replacement, and maintenance to remain in service. Understanding and mitigating deficiencies within each bridge component is crucial to extending the service life of the bridge. In order to gain insights into the degradation of reinforced concrete bridge decks, specifically, the bi-annual inspection reports of recently decommissioned decks in Georgia were analyzed. The findings indicated that corrosion or corrosion-related mechanisms were often present. Therefore, chloride-induced corrosion models best predict the degradation of decks in Georgia. To address uncertainty in the deck environments, the deck's degradation was modeled under a variety of conditions based on both literature values and the damage information from inspection reports. After establishing a baseline set of key corrosion parameters, alternative construction practices and materials were modeled and evaluated. The models predicted that alternative reinforcement had the greatest impact on service life. Incorporating supplementary cementitious materials in the deck mix designs, reducing surface cracking, improving top mat cover control, and applying surface coatings also appreciably affected the projected service lives. One way to implement these findings in practice is through contracting mechanisms that promote quality of construction. This research explores the use of various contracting mechanics to achieve extensions of service life through cover control. To demonstrate this approach, a sample adjustable payment plan for improved cover control was created.

CHAPTER 1

INTRODUCTION

1.1 Motivation

Bridges are vital to the safe and efficient conveyance of people and goods around the world. For this reason they are considered critical structures. Despite their importance to society, bridges are often compromised by a wide range of deficiencies that require significant rehabilitation, replacement and maintenance to remain in service. Such bridges are often called “structurally deficient” [1]. In 2019, the Federal Highway Administration (FHWA) estimated that the replacement cost of all the nation’s structurally deficient bridges to be about $50 billion dollars [1]. If those structurally deficient bridges were repaired, instead of rehabilitated or replaced, the cost is estimated to be $35 billion dollars [1]. In addition to these rehabilitation and repair costs, bridges also incur large annual maintenance costs. One such cost is the currently estimated $13.6 billion per year direct cost attributed to the corrosion of highway bridges [2]. As the national average age of bridges increases, the corresponding maintenance costs also are expected to increase. A recent FHWA survey reported that 39 percent of the national bridge inventory exceeds the predominant 50-year expected lifespan, and 15 percent of the inventory is between 40 and 49 years old [3]. While repairing or replacing bridges is an enormous expense, it also represents an economic opportunity. FHWA estimates that for every dollar spent on road, highway, and bridge improvements the expected return is over five dollars. This return is due to reduced vehicle maintenance costs, more efficient travel, and decreased road and bridge maintenance costs [3].

Due to these economic impacts, and safety concerns, significant research and implementation efforts are directed toward reducing damage, reducing maintenance costs, and prolonging the lifespans of bridges. This dissertation is part of this broad effort, with a focus on improving the quality and extending the service life of reinforced concrete bridge deck through developing and implementing science-based contracting methods.

Reinforced concrete bridge decks degrade from a variety of mechanisms such as corrosion, freeze-thaw, and abrasion [4-7]. The susceptibility of reinforced concrete decks to these mechanisms depends on the quality of their construction (e.g., construction practices, material selections), the environmental conditions (e.g., salt exposure, large volume of traffic), the monitoring, and interventions by the owners. To estimate the susceptibility to these mechanisms and its effect on the lifespan of decks, a series of bridge deck service life models were developed. These models incorporate historical construction and biannual inspection reports from in-service and recently decommissioned decks from the State of Georgia. While the data and work here is based on the historical records and construction practices of a single state, this study is broadly applicable to other areas with similar conditions and construction practices.

The results from the models created in this research are used to inform contracting practices for new construction, and maintenance practices for existing decks. The goal of these contracting mechanisms was to promote adoption of promising technologies and practices. A sample science-based contractual provision was created to demonstrate how the outputs and findings from the models may be implemented to practice.

1.2 Thesis Organization

The thesis is organized into eight chapters. Each chapter is comprised of an overview, followed by the main contents, and ends with conclusions. After this introductory first chapter, Chapter 2 discusses the available literature on prolonging the lifespan of reinforced concrete bridge decks, as well as service life modeling, and the legal landscape for public works contracts. Chapter 3 presents the dissertation’s research aims and objectives, which address the knowledge gaps and technical needs identified in Chapter

2. In Chapter 4, the mechanisms of bridge deck degradation are investigated by analyzing historical bi-annual bridge inspection reports for recently decommissioned bridge decks in Georgia. Chapter 5 exhibits the investigation into top mat cover control, a key construction practice, which serves as the technical foundation for the sample contract provision. Chapter 6 contains the service life modeling, with particular emphasis on establishing a baseline performance for decks to compare alternative construction practices and materials against. In Chapter 7 the findings from Chapters 5 and 6 are implemented in a demonstrative contractual provision for new construction. Finally, the main conclusions of this work are summarized in Chapter 8.

CHAPTER 2

LITERATURE REVIEW

2.1 Deck Degradation Mechanisms

Understanding the causes of reinforced concrete bridge degradation is important for extending service life. Numerous degradation mechanisms that have been observed acting on reinforced concrete bridge decks including wear and abrasion, freeze-thaw, moisture and thermal cycling, fatigue, and corrosion [4-7]. The predominant degradation mechanism depends on deck factors, such as its environment exposure, the quality of its materials and construction, the amount of vehicular travel facilitated, monitoring, and interventions by owners. Depending on the degradation mechanism, specific countermeasures can be used to either delay that form of degradation in existing decks, or inhibit it in new bridge decks. The most common rehabilitation techniques for existing decks include: patching, complete deck overlays, protective electrical systems such as cathodic protection, conductive asphalt, or using thin bonded overlays [4]. While these techniques are effective against some degradation mechanisms such as corrosion, they are significantly less effective against other forms of degradation (e.g., alkali silica reaction). Specific degradation mechanisms that were observed on bridge decks in Georgia is discussed in Chapter 4 of this dissertation.

2.2 Broad Research Areas in the Literature

When examining the literature on the extension of bridge deck service life, three broad areas of research emerged. Those three areas are: 1) novel and alternative construction materials, 2) improved construction practices, and 3) early degradation detection and monitoring. For the purposes of this chapter, the effects on service life for these areas will be treated independently, though it is acknowledged that in practice a deficiency in one of those areas may be offset by exemplary performance in another. For example, the depreciating effect on service life of poor workmanship may in some cases be offset by the use of higher quality materials. These interactions are discussed in more detail in Chapter 6.

2.2.1 Novel and Alternative Construction Materials

A significant body of research exists on the topic of novel and alternative construction materials for use in bridge decks. In particular, significant research has been performed and implemented on novel and improved concrete mix designs [8-17], reinforcement [8, 18-21], and surface coatings [4, 6, 22, 23]. However, relatively few innovations have been incorporated into bridge deck construction. One obstacle to the adoption of these novel and alternative materials in construction may be the increased financial and lifecycle costs as compared to the existing methods and technology, a concern which should be evaluated through economic frameworks [24].

2.2.2 Improved Mix Designs

There have been innovations to concrete mix designs that significantly increase the expected service life of reinforced concrete structures. These innovations involve the addition of novel and alternative materials in the mix design, such as the inclusion of high resistivity and polymer modified concretes which decreases chloride ingress and corrosion coupling, the use of admixtures (e.g., calcium nitrate, a corrosion inhibitor), or the addition of supplementary cementitious materials (e.g., fly ash, metakaolin, or slag) which beneficiate the concrete [8]. Some other recently developed materials include phase-change materials, micro-reinforced polypropylene fibers, and limestone fines [9-11]. Phase-change materials reduce the length and depth of freezing events, which limits damage from freeze-thaw cycling [9]. Micro-reinforced polypropylene fibers increase abrasion resistance [10]. Limestone fines reduce water permeability [11]. While the function of each material varies, in general they either reduce the permeability of the concrete (i.e., less corrosion and freeze-thaw by inhibiting ingress of aggressive agents) [11-14] or improve the mechanical properties (e.g., strength and abrasion resistance) [10, 15, 16].

The supplementary cementitious materials (SCM) that are frequently used as partial replacements to Portland cement (PC) in concrete are fly ash (FA), and blast furnace slag (BFS). In general, the amorphous silica in SCM reacts, in the presence of water, with calcium hydroxide (CH) in the concrete to form additional calcium silicate hydrate (C-S-H, the main strength giving phase in concrete) [25-27]. This process reduces chloride ingress by creating a more refined concrete matrix with a finer, and less continuous, system of capillary pores [27]. While the overall function of these SCM is similar, they have varying properties and origins.

FA is a by-product of energy production in coal plants, consisting of the silica rich residue remaining after coal combustion [28]. FA is often used because it is less expensive than PC (around 50 percent of the cost of cement). FA consists of spherical particles, which have less friction than angular particles, and can reduce the water demand for the mix [28]. A limit of 25 percent replacement of cement by FA is the theoretical maximum quantity which could fully react with the available CH in concrete [28]. However, mixes with greater amounts of FA have proven durable because the FA functions as a water-reducer and filler [28].

BFS is an industrial by-product of metal production (typically iron), which has been in use in concrete since the early 1900s [27]. Aside from forming later age C-S-H, slag also inhibits chloride ingress by binding chlorides [27]. BFS is near the same cost as PC.

Due to its ability to its inherent cementitious properties, and comparable cost, BFS is often permitted to replace the most cement.

SCMs have also been shown to significantly increase deck service life. To illustrate the point, the work of Balakumaran et al. can be examined [17]. In a 2017 study of Virginia bridges, the work found that, on average, a bridge deck constructed without SCM will likely initiate corrosion in as little as four to eight years of service under ordinary traffic and environmental conditions, compared to 1-17 years for SCM concrete [17]. Despite this expected increase in service life, the mix designs prescribed in some state Departments of Transportation (DOT) appear to underutilize these materials. To better make the case for the use SCMs and other novel and alternative materials, the expected effects on the service life of bridge decks constructed using concrete with these materials was evaluated in this dissertation.

2.2.3 Alternative Concrete Reinforcement

The main alternatives to traditional low-carbon steel rebar construction to extend the deck service life are epoxy-coated rebar (ECR), fiber reinforced polymer rebar (FRP), metallic composite rebar, stainless steel rebar (SS), and stainless clad rebar (SCR) [8, 18, 21, 29-31]. These alternative materials extend the service life of decks through enhanced corrosion resistance. A recent state of the art report by the National Association of Corrosion Engineers describes the history, field performance, and benefits and drawbacks of some of these alternative rebar [31]. Of these alternatives, the most common used in deck construction is ECR due to its low relative cost to expected performance.

For ECR, the corrosion resistance is directly related to the robustness of the epoxy coating, which has been shown to incur damage during handling and installation [29]. ECR has presented poor field performance in bridges in some states (e.g. Florida) [19], and adequate to good performance in others [8, 20]. The poorer performance in Florida was observed with early formulations of the ECR coating, and more modern ECR has proven more durable [31]. Studies have shown that the performance of ECR is also linked to differences in exposure conditions (e.g., average temperature, salt exposure), with the poorer performance seen in more aggressive environments which would warrant ECR use. Consequently, alternatives to ECR are desirable, particularly those that are less sensitive to handling defects, which results in premature corrosion [8].

SS and SCR have appeared as promising alternatives, with the former being a solid rebar comprised of a stainless steel alloy, and the latter being a rebar comprised of a stainless steel layer metallurgical bonded around a carbon steel core. The evaluation of the field performance of SS and SCR rebar were the subject of recent work by the Michigan Department of Transportation (MDOT) [8]. In a field comparison between a deck constructed with SS and one with ECR in Michigan, no deterioration of the stainless steel reinforced deck was observed, whereas the ECR reinforced deck showed minor deterioration including patching near joints [8]. In another study, McDonald et al. estimated the difference in service life of ECR, SS, or metallic clad rebar as compared to plain low-carbon steel rebar. Through electrochemical assessments of ECR, SS, and metallic clad rebar, it was estimated that bridge decks constructed with low-carbon steel reinforcement, ECR, and SS have an service lives of around 9 years, 36 years, and 75 to 100 years, respectively [21]. Despite good field performance of SS decks, the primary concern with SS and SCR continues to be the relatively high initial cost. When comparing the performance of ECR, SS, and SCR, the study by MDOT estimated the breakeven point between cost and performance at 83 years of maintenance-free service, or when material costs exceed 24 percent of construction costs [8]. To reduce cost, SCR was developed, with the hope that the corrosion durability of SS could be achieved for significantly lower cost. This has not proven true, as the material cost in initial version of SCR were near identical to that of SS, and SCR appears to no longer be in commercial production. The similar cost and uncertain supply has resulted in the disuse of SCR [31].

The final category of rebar evaluated are metallic composite rebar, such as dual phase ferritic-martensic steel rebar (DP). These rebar are proprietary metallic rebar which contains chromium and presents elevated corrosion resistance compared to that of plain low-carbon steel rebar [29, 32].

The performance of all these rebar alternatives was estimated in the service life modeling undertaken in Chapter 6 of this dissertation.

2.2.4 Overlays

A very common rehabilitation practice to extend bridge deck service life is to apply an overlay or waterproofing membrane on top of the deck [4]. This practice began as early as the 1950s with the introduction of coal-tar epoxies. Significant advances were seen with moisture tolerant epoxies developed in the 1970s [22]. Since the 1970s, new material formulations have been developed, though typical overlay compositions are still epoxy, copolymer, or bitumen based [4]. An example of a new material formulation in recent work featured the use of ultra-high performance concrete (UHPC) to form a thin overlay, which has the advantage of higher tensile strength and lower permeability than that of the traditional concrete substrate [23].

For best results, all overlays are applied when the deck has minor to moderate deterioration but is likely to experience a sharp increase in deterioration in the near future, or if the deck is not in need of immediate replacement [23]. The primary purpose of the overlay is to resist abrasion, prevent ingress of chlorides into the concrete, resist freezing and thawing, and adhere well to the deck concrete [4]. The main concerns with these types of interventions are cracking of the overlay or bond failures with the underlying substrate

[6]. Orta and Bartlett investigated the reliability of concrete deck overlays, with an emphasis on modeling the tensile stresses that develop from the restrained shrinkage of the overlay by the substrate [6]. Through their modeling, it was estimated that for a concrete overlay between 2.75 and 7.87 inch thick over 50 years, there is a 30-50 percent of cracking prevalence over its service life, regardless of the substrate system [6]. Furthermore, the study found that if an overlay is between 2.75 and 5.90 inch thick and has not cracked within a year, then the probability of cracking due to restrained shrinkage for the remainder of its service life is very low [6]. Taken together, these two findings suggest a propensity for early cracking in overlays, and a substantial likelihood that an overlay will crack over its intended service life. If the overlays cracks, it no longer functions as a barrier to the ingress of aggressive elements, and it may actually promote degradation. Despite these concerns, field performance has shown that if the reliability of a particular overlay material is well-established, overlays may be used prior to or after degradation has manifested [4]. Service life modeling in this work considers the effects of widespread overlay use on bridge decks.

2.3 Improved Construction Practices

There appears to be a lack of recent research on improved construction practices for bridge decks, particularly in the area of cover control. The research in this area is focused on three main topics: how specifications for cover translate into practice, how to quantifying cover variability in decks, and, ultimately, seeking to understand which cover specifications lead to the longest service life [33]. Further research in this area may significantly extend the service life of decks.

Expanding the literature review to include general construction practices of bridges as a whole, there does appear to be research on efforts to improve construction quality and practices, generally through contracting. For example, multiple DOTs have employed adjustable payment plans, which correlate the pay to quality of the work, to ensure asphalt roadway quality or proper painting of steel bridges [34]. Other adjustable payment examples include a similar plan developed for the density of hot mix asphalt [35] and the compressive strength of the deck concrete [36]. There appears to be an opportunity to utilize the underlying methodologies from these examples to improve other construction practices, such as cover control, to extend bridge deck service life.

2.4 Early Degradation Detection and Monitoring

Establishing a system to evaluate the condition of decks is an important part of monitoring and detecting degradation. Customarily, the condition of bridge decks is assessed through the Federal Highway Administration's National Bridge Inventory (NBI) condition ratings [37]. The NBI condition ratings are given on a scale from zero to nine. A rating of zero represents a bridge in a failed condition, while nine is indicative of excellent condition, though in practice only ratings one through eight are typically assigned. From a discussion with Georgia Department of Transportation (GDOT) personnel, it was stated that a bridge is typically replaced when the deck's NBI condition rating is a four, which was validated in the evaluation of the NBI ratings of decks undertaken in this work (see Appendix A). A NBI rating of four is given when a bridge component has "advanced section loss, deterioration, spalling or scour" [37]. In practice, however, it is unclear whether or not that rating has truly been reached due to the subjective nature of the assessment and general inconsistency in ratings [38]. Multiple solutions have been provided over the years, including relating the NBI rating to a quantitative metric, such as the percentage of the deck surface that has spalled or been patched [38], though no such changes have been implemented.

The literature provides a variety of techniques and methods for determining the health of reinforced concrete bridge decks that serve as alternatives to the NBI system.

Some frequently used non-destructive techniques (NDT) for monitoring the health of bridge decks include: half-cell or linear polarization measurements, pachometer concrete cover surveys, ground penetrating radar surveys, and acoustic techniques such as the impact echo [4]. Some newer technologies include a technique based on stress waves, which can help track the damage caused to decks from repeated traffic loads [5] and resistivity testing, which has been shown to be able to effective in assessing the overall condition of decks in-situ [39]. The goal of these promising technologies broadly is to detect degradation earlier, which could result in reduced maintenance costs from more timely repairs and greater confidence in the condition of the decks.

2.5 Corrosion Modeling

Corrosion is a primary degradation mechanism of reinforced concrete bridge decks, particularly for decks in areas with de-icing salt and marine exposure [40-42]. Ordinarily, for reasons greatly expanded on in other sources [43-45], the steel reinforcement in concrete is protected and corrodes at a negligible rate, a rate frequently described as “passive.” This passivity is the result of the highly alkaline environment inside concrete at the typical steel potentials, which facilitates the formation of a stable oxide layer [43], typically called the “passive layer.” Active corrosion, that is to say corrosion that is deemed harmful, is generally the result of either the acidification of the concrete surrounding the steel reinforcement, or the destabilization of the passive layer (in the presence of oxygen) by a sufficient concentration of chloride or other aggressive ions [45]. Chloride-induced corrosion is typically found in bridges and other marine and coastal structures [41].

A common framework for corrosion service life modeling of reinforced concrete structures (adaptable to bridge decks) describes the service life (t_{sl}) as consisting of an initiation period (t_i), where there is negligible damage due to the passivity of the steel in the concrete, followed by a propagation period (t_p) where corrosion is ongoing and which ultimately leads to the end of its service life [46] (Figure 1). Researchers focus on prolonging the initiation period to extend the service life of the deck, since the propagation period has been found to be as short as five to seven years under most models [36].

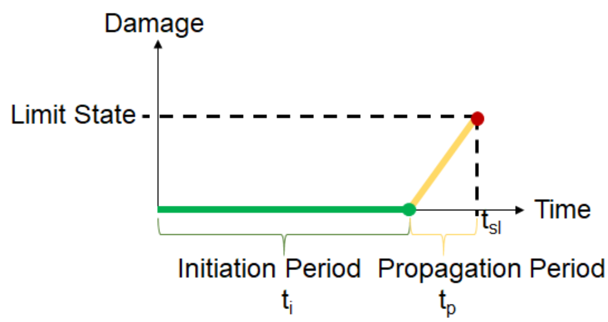

Figure 1. Service life framework.

For the initial modeling presented here, the initiation period is defined as the time needed for the chloride concentration at the surface of the rebar to initiate corrosion, and the propagation period is the time from initial corrosion to the deck reaching its limit state (often assumed to be five years). A significant portion of the work described above, particularly the novel and alternative materials, uses methods that increase the service life either by increasing the needed concentrations of chloride to initiate corrosion (e.g., SR), or to extend the propagation time (e.g., application of overlays).

The modeling presented in this dissertation utilizes pure one-dimensional diffusion of chloride ions from the external surface of the deck to the rebar surface, which omits many of the complex interactions in the system, but may still be useful when there exists significant uncertainty of the system parameters (e.g., composition of the pore solution). The accuracy of such a simple approximation has been the subject of an investigation [47], where the increased accuracy of the predictions from more sophisticated models is weighed against the corresponding demands for system information. Titi and Biondini found good agreement between the one-dimensional diffusion model and more complex service life models when the width of the concrete cross-section is significantly greater than the thickness (in accordance with ratios found in bridge decks) [48].

2.5.1 Diffusion Modeling

For systems under pure diffusion in one direction, the concentration at some time and distance can be described by Fick's Second Law in Equation 1.

$$\frac{\partial \varphi}{\partial t} = D\frac{\partial^2 \varphi}{\partial x^2} \qquad \text{Eq. 1}$$

In the equation, φ is the concentration at some distance *x* and time t, and *D* is a proportionality constant (frequently called diffusivity). For the case at bar, the concentration of interest is that of the chloride ions in the concrete, and the distance of interest is the concrete cover. If the bridge deck is treated as a semi-infinite media, with a constant chloride surface concentration, one-dimensional diffusion, and constant diffusivity, the error function solution is yielded, as shown in Equation 2. In the equation, *C* is the concentration of chlorides, C_1 is the concentration of chlorides at the surface, C_o is the initial concentration of chlorides initially throughout, *x* is the depth of interest, and *t* is time of interest [49].

$$\frac{C - C_1}{C_0 - C_1} = \mathrm{erf}\frac{x}{2\sqrt{Dt}} \qquad \text{Eq. 2}$$

In such an arrangement, the concentration of chlorides in the concrete evolve over time until there exists an equilibrium of concentration throughout the concrete. Of interest is when the concentration of chlorides at rebar depth exceeds the chloride threshold (i.e., the concentration of chloride above which corrosion initiates). This point is illustrated in

Figure 2, which shows that the threshold (red dashed line) was exceeded at the hypothetical 2 inch rebar depth in approximately 6.5 years.

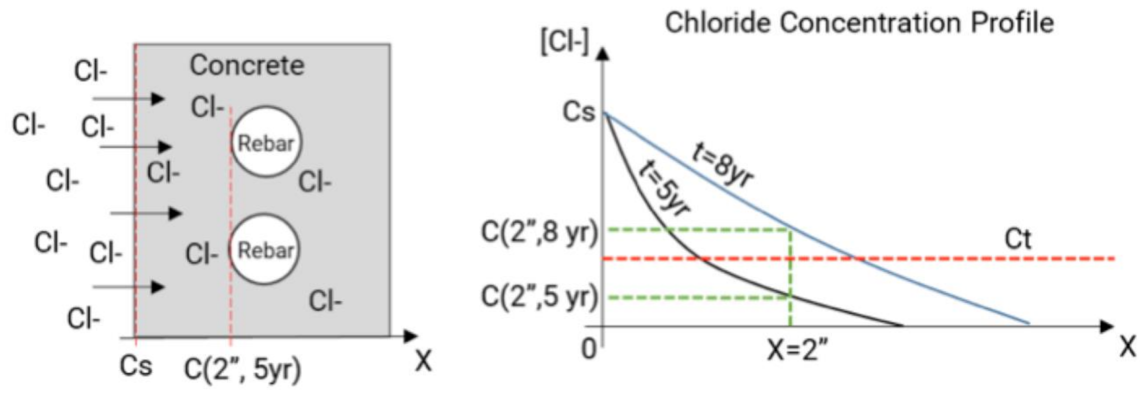

Figure 2. Chloride ingress and the resulting chloride profiles for 2 inch rebar depth.

Equation 2 can be simplified further by assuming that $C_0 = 0$ (i.e., no chlorides in the concrete initially), setting C equal to the chloride threshold C_t, x equal to the concrete cover, and rearranging the equation to solve for t. With those alterations, the value of t is that which corresponds to the exceedance of the chloride threshold at rebar depth, which should mark the end of the initiation period. This form, represented in Equation 3, is used in many practical service life models [20, 42, 48, 50, 51]:

$$t_i = \frac{x^2}{4D\left(\mathrm{erf}^{-1}\left(1 - \frac{C_t}{C_s}\right)\right)^2} \qquad \text{Eq. 3}$$

The service life could therefore be computed as the summation of the result of Equation 3 and an estimate of the propagation period. For the purposes of the work undertaken here, the propagation period was assumed to be five years which is within the ordinary range given by [44].

For the diffusion model presented in this dissertation, the most recent GDOT mix design requirements were used to approximate the parameter values with relationships derived from the literature. Reasonable estimates for the main parameters of C_t, C_s, and D ascertained from the literature are given in Table 1, Table 2, and Table 3, respectively.

The values found in the references were all converted to a single system of units, and in the case where the reference provided a range based on percentage of a mix constituent, the value for the Class A and Class D mixes commonly used in deck construction, were computed and provided as a range.

Table 1. Chloride threshold ranges

Value	Units	Comments	Reference(s)
1.22-1.3	lb/yd^3	Estimate based on 0.2% by weight of the cement	[20, 40, 42]
1.97	lb/yd^3	Default value from Life-365tm	[52]

Table 2. Surface chloride concentration ranges

Value	Units	Comments	Reference(s)
21.8	lb/yd^3	Based on 0.1% by weight of concrete, converted from 12950 ppm Cl-. Data supported by compiling the results of coring in 4 states (73 bridges, 688 cores)	[20, 40, 42]
8.31	lb/yd^3	80 Iowa bridge decks	[52]

Table 3. Apparent diffusivity ranges

Value	Units	Comments	Reference(s)
0.147	in^2/yr	For w/c=0.45, average ambient temperature of 60F based on work by (Page, Short et al. 1981) on mortar specimens	[40, 53]
0.489	in^2/yr	For cement class CEM I 42.5 R with w/c=0.45	[42]
0.240	in^2/yr	Average D measured in a set of Pennsylvania bridges with cover 75 mm (2.95”), w/c<=0.43, minimum cement 400 kg/m3, 15% fly ash by mass of cementitious materials max	[54]
0.298	in^2/yr	Determined by non-steady state migration tests (NT Build 492) on 100 mm wide, 50 mm thick cores on sound specimens	[55]
0.050	in^2/yr	Based on analysis from concrete cores taken from 80 Iowa bridge decks	[20]
0.522-0.614	in^2/yr	Based on an equation from the fitting of 10 separate studies using w/cm ratio as the primary input.	[54, 56]

The wide range in values for the parameters is to be expected given the variety of experimental and/or field evaluations on which they are based. To incorporate this uncertainty, all diffusion modeling was performed using 12 permutations derived from Tables 1-3, with combinations of the C_t values (1.22 and 1.97 lb/yd^3), C_s (8.31 and 21.8 lb/yd^3), and D (min, avg, max: 0.050, 0.301, 0.614 in^2/yr). These permutations represent a variety of deck characteristics and exposure conditions in Georgia, which may be applicable to other similar areas in the country. For the most accurate modeling, sampling for these key parameters should be undertaken for the bridge population being modelled, which, in this case, was only possible for the cover thickness.

2.5.2 Cover Thickness

The thickness of cover concrete over the top reinforcement mat in bridge decks significantly impacts bridge durability. Cover that is too thin may lead to earlier and more severe corrosion and wear, which degrades the driving surface and shortens the deck's life [57, 58]. Conversely, cover that is too thick may lead to cracking, which exacerbates the deck's degradation through other mechanisms such as corrosion, salt scaling, and/or freeze/thaw cycling [57, 59]. Premature deck degradation necessitates more frequent inspections, earlier and additional maintenance and repair, and eventually, replacement [57, 58]. While the effect of minor variations in cover control for any one bridge may impact its performance (ranging from ride quality to service life), the aggregated effects over a large bridge inventory can be significant in terms of increased costs for maintenance and eventual reconstruction and in traffic delays. As a result, state agencies have re-evaluated their requirements for concrete cover over the years, in an effort to ensure bridges meet design service lives with minimal maintenance.

Since the early 2000s, most states have specified a cover thickness of 2.5 or 2 inch [60], which is within the 2 to 3 inch range that studies [61-63] show to be optimal. However, states with more aggressive exposure environments, including Florida and New York, specify thicker covers to ensure adequate field performance. The New York State Department of Transportation (NYSDOT) Bridge Manual [64] provides a clear example of a shifting cover specification over recent decades, as summarized in Figure 3 and Table 4. This example shows increasing cover requirements over time for uncoated bars and also adjustments – specifically decreases in cover – for coated and corrosion-resistant bars, as those materials technologies were introduced.

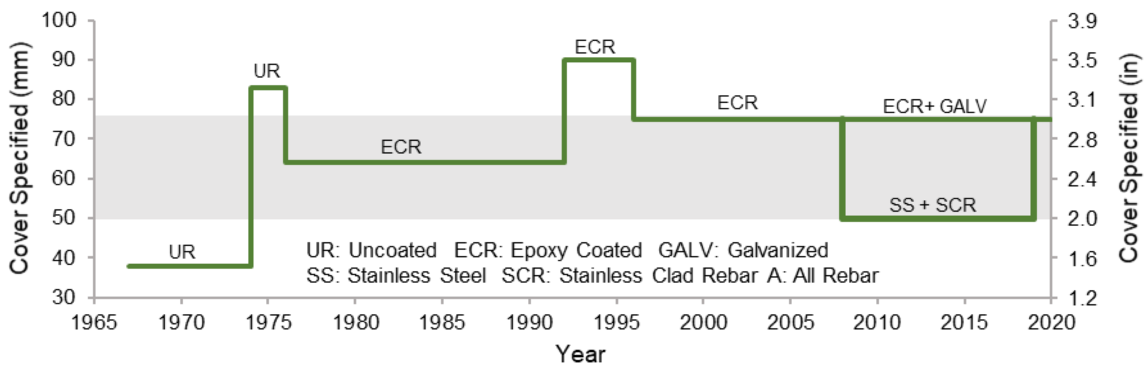

Figure 3. Changes in the NYSDOT Bridge Manual top mat cover specification over time and with reinforcement type. The figure shows the cover thickness specification (green outline) for the rebar type and the theorized optimal cover range (gray shaded region).

Table 4. NYSDOT specified cover and commentary [64]

Year	Rebar Type	Cover Specified	Commentary
1967	Uncoated	1.5 inch	
1974	Uncoated	3.25 inch	Change due to chloride penetration and durability concerns
1976	Epoxy-coated	2.5 inch	Introduction of epoxy-coated rebar
1992	Epoxy-coated	3.5 inch	Change due to chloride penetration and durability concerns
1997	Epoxy-coated	3 inch	
2008	Epoxy-coated or Galvanized	3 inch	
	Stainless steel or Stainless clad	2 inch	
2019	Epoxy-coated, Galvanized, Stainless steel, Stainless clad	3 inch	Permissible as of 2019

Researchers have examined how specifications for cover translate into practice, quantifying cover variability in decks and ultimately seeking to understand which specifications lead to the longest service [33]. Some of the earliest work quantifying cover control in field conditions was performed by Newlon in 1974, which surveyed 117 Virginia bridge decks (10,170 cover measurements) and found a normal distribution with a mean of 2.40 inch and standard deviation of 0.49 inch, much greater than specified [33]. In the same year Weed examined the cover of 17 reinforced concrete bridge decks in New Jersey [65]. Nine of the seventeen bridges were built with a design cover of 1.5 inch, and the remaining eight were built with 2 inch cover. The observed cover was normally distributed for both sets with means of 1.66 and 1.84 inch, and standard deviations of approximately 0.375 inch. If 2 inch was considered the minimum cover thickness over the entire bridge deck to promote deck durability, Weed found that a 2.5 inch specified design cover would result in 90 percent of the cover along the deck exceeding 2 inch. For full cover thickness compliance, a 3.125 inch design cover would be needed. Weed also suggests that if the standard deviation could be reduced to 0.25 inch, then a design cover of 2.75 inch would be sufficient. In 2003, Weyers surveyed 21 Virginia bridges, observing a normal distribution in cover with an average of 2.56 inch and a standard deviation of 0.358 inch, for a design cover of 2.5 inch [66]. A unique aspect of this work was investigating whether or not a pay incentive could be used to deliver 0.5 inch of extra cover. A further 30 bridges received this incentive for extra cover. When comparing both sets of bridges, cover distributions were near identical with mean 2.56 inch and standard deviation 0.358 inch for bridges with the pay incentive and mean 2.60 inch, standard deviation 0.378 inch for those without. During the intervening quarter century between those two studies, the variation among the cover measured for a deck remains very similar, suggesting a lack of improvement in cover control, despite implementing contracting incentives, as well as improvements in assessment methods [67-70].

Both the Weed and Weyers studies represent “snapshots” of bridge cover conditions among a relatively small set of bridges at a particular time. To better understand trends in cover thickness and variability, data analysis over time is necessary. This information, and the modeling which it facilitates, is useful for policymaker as a way to assess construction quality and inform contract adjustments.

2.6 Applicable Contract Law

One of the ways in which the results from service life modeling can be implement in the real-world to extend the lifespans of bridges is as the basis for contractual provisions. The aim would be to incentivize and reward contractors that utilize novel technologies and better construction practices, while deterring those that do not. To serve as an illustrative example, the case of breach of contract for improper cover control will be discussed. The same principles that apply to this example are also applicable to other construction practices, such as ensuring a desired compressive strength in deck concrete. There are questions of the legality of such proposed contractual provisions, which is the subject of the remainder of this subsection. As an important note, none of the opinions and conclusions presented here are to be interpreted as legal advice, or the unlawful practice of law, but rather an academic discussion. Bearing that in mind, a literature review of the basic elements of contracts as it pertains to public works, should begin with the topic of contractual remedies.

2.6.1 Remedies

Assuming that a valid contract was formed, which stipulates the required cover control, the material failure to meet those requirements may constitute a breach of contract. It is therefore important to discuss the general remedies available for such a breach of contract. As the data gathered for this dissertation (see Chapters 4-6) come from Georgia, the law of Georgia will be used as the default jurisdiction, though the law of other states is discussed as well. The Georgia Code Title 13 Chapter 6, Damages and Cost generally states that permissible damages broadly fit into five categories which are covered in sections § 13-6-6 through § 13-6-10 [71] and are given in Table 5.

Table 5. Georgia code damages and remedies

Damage Type	Relevant Section	Language
Nominal	GA Code § 13-6-6	In every case of breach of contract the injured party has a right to damages, but if there has been no actual damage, the injured party may recover nominal damages sufficient to cover the costs of bringing the action.
Liquidated	GA Code § 13-6-7	If the parties agree in their contract what the damages for a breach shall be, they are said to be liquidated and, unless the agreement violates some principle of law, the parties are bound thereby.
Remote or Consequential	GA Code § 13-6-8	Remote or consequential damages are not recoverable unless they can be traced solely to the breach of the contract or unless they are capable of exact computation, such as the profits, which are the immediate fruit of the contract, and are independent of any collateral enterprise entered into in contemplation of the contract.
Expenses Necessary for Compliance	GA Code § 13-6-9	Any necessary expense, which one of two contracting parties incurs in complying with the contract may be recovered as damages.
Exemplary	GA Code § 13-6-10	Unless otherwise provided by law, exemplary damages shall never be allowed in cases arising on contracts.

A breach of contract for improper cover control would not be a nominal damage as the owner (state) incurred actual damage (i.e., a diminished asset). Such a breach could be eligible for remote or consequential damage, though it would be difficult to ascertain that the injuries suffered were solely the cause of poor cover. Expenses necessary for compliance may apply but only in narrow circumstances, such as if the DOT hires another firm to correct the improper work and seeks to recover the costs against the original contractor. As noted in the table, exemplary damages which are intended to be punitive, are prohibited. Of the damages listed, the most applicable to a breach for improper cover control is liquidated damages because the actual damages resulting from the breach are difficult to quantify with certainty. For example, estimating the cost the state incurs for an average cover of 0.25 inch less than design across a deck varies depending on many factors such as economic conditions and deck characteristics, particularly when the damage occurs many decades after construction. In such cases, the state could ensure appropriate cover control by pursuing liquidated damages against the contractor if an agreed upon price per inch of cover were stipulated in the contract. However, there are two obstacles. The first is that the cost for liquidated damages needs to be agreed on in advance and justified. This is where the service life modeling is vital, serving as a means to justify the financial cost to the improper cover. The second issue, is that liquidated damages may not be permissible by state law for public works contract, a concern which is investigated in Section 2.6.2.

2.6.2 Public Works Contracts - Georgia

Based on the analysis of damages above, it is unsurprising that the Georgia Code only mentions liquidated damages in relations to public works projects. The most relevant statutes can be found in §13-10-70 of the Georgia Code, “Liquidate damages for late completion and incentives for early completion,” [72] which offers the following guidance:

> *“Public works construction contracts may include both liquidated damages provisions for late construction project completion and incentive provisions for early construction project completion when the project schedule is deemed to have value. The terms of*

the liquidated damages provisions and the incentive provisions shall be established in advance as a part of the construction contract and included within the terms of the bid or proposal."

Section 13-10-70 emphasizes that liquidated damages must be an agreed upon pre-estimate of damages, and also provides for both an incentive for early construction as well as a disincentive for late completion. It is important to have a corresponding incentive for every provision with a disincentive as the courts have ruled that provisions that function solely as a penalty are prohibited (which is expanded on in the next section). Section 13-10-70 is a clear example of an incentive/disincentive (I/D) contract mechanism, intended to reduce the construction time for public works projects. However, improving cover control is not related to construction speed, but rather construction quality. It is therefore useful to examine the statutory language used in other states, which may permit such an application of liquidated damages to construction quality concerns.

2.6.3 Public Works Contracts – Other States

The state codes and statutes of Virginia, Texas, Indiana, Florida, Ohio, Utah, and California were examined. In general, it appears that the states defer to the Universal Commercial Code (UCC), in whole or with modification, to serve as the general basis for their contracting laws, with specific amendments by statute. The overall consensus in regards to liquidated damages is that there may be no penalty clauses in contracts without the prospect of receiving a bonus. So long as a liquidated damage provision affords the opportunity for a benefit it may be permissible.

Virginia, Texas, and Indiana provide no specific statutory requirements for public works contracts. In examining Florida statutes, the most relevant section is FL Stat § 337.18 [73], which allows for liquidated damages, and in the case where time is of the essence, an incentive payment is permissible. Ohio § 5525.20 provides the following incentive and disincentive provisions for critical construction projects [74]:

"…the director of transportation may include incentive and disincentive provisions in contracts the director executes for projects or portions or phases of projects that involve any of the following:

(1) A major bridge out of service;

(2) A lengthy detour;

(3) Excessive disruption to traffic;

(4) A significant impact on public safety;

(5) A link that completes a segment of a highway.

As used in this section, 'incentive and disincentive provisions' means provisions under which the contractor would be compensated a certain amount of money for each day specified critical work is completed ahead of schedule or under which the contractor would be assessed a deduction for each day the specified critical work is completed behind schedule. The director also may elect to compensate the contractor in the form of a lump sum incentive for completing critical work ahead of schedule."

Utah gives wide latitude to the remedies which are permissible in section 63G-6a-1210 [75]:

"Contract provisions for incentives, damages, and penalties.

A procurement unit may include in a contract terms that provide for:

(1) incentives, including bonuses;

(2) payment of damages, including liquidated damages; or

(3) penalties."

California appears to have very explicit language in terms of incentivizing early construction and reducing costs or inconvenience to the public. Two relevant examples are those given in CA Pub Count Code § 7101 [76] and CA Civ Code § 1671 [77]:

"The state or any other public entity in any public works contract awarded to the lowest bidder, may provide for the payment of extra compensation to the contractor for the cost reduction changes in the plans and specifications for the project made pursuant to a proposal submitted by the contractor. The extra compensation to the contractor shall be 50 percent of the net savings in construction costs as determined by the public entity. For projects under the supervision of the Department of Transportation or local or regional transportation entities, the extra compensation to the contractor shall be 60 percent of the net savings, if the cost reduction changes significantly reduce or avoid traffic congestion during construction of the project, in the opinion of the public entity. The contractor may not be required to perform the changes contained in an eligible change proposal submitted in compliance with the provisions of the contract unless the proposal was accepted by the public entity."

"…(b) Except as provided in subdivision (c), a provision in a contract liquidating the damages for the breach of the contract is valid unless the party seeking to invalidate the provision establishes that the provision was unreasonable under the circumstances existing at the time the contract was made."

In summary, there appears to be no significant statutory barriers in Georgia to using liquidated damages to remedy a breach do to improper cover control, much like the other states examined. It appears that most states have similar language related to the permission of liquidated damages for public works contracts. If the state wishes to pursue liquidated damage claims for breaches related to construction quality, the passage of a law which specifically permits such an application may be needed. In cases with where statutory law do not answer a question of permissibility such as this, common law (the law derived from custom and jurisprudence) may sometimes clarify. Section 2.6.4 examines the relevant case law for public works contracts.

2.6.4 Relevant Georgia Case Law

To investigate the common law landscape for public works contracts, a search of relevant case law was undertaken for the State of Georgia. Two relevant cases were found to be applicable to this effort.

The first case, Southeastern Land Fund v. Real Estate World (237 Ga. 227 1976) [78] considered whether a provision in a real estate sales contract constituted an enforceable liquidated damage provision or a penalty. The provision in question stipulated that $5,000 paid in earnest money to the seller was partial liquidated damages in the case of default, as a means to collect the proceeds of the indebtedness owed. When the buyer defaulted, the seller sued for more damages than the $5,000, claiming that they were entitled to pursue any and all legal remedies including, but not limited to, the $5,000. The case reaffirmed the requirements for a liquidated damages provision: "First, the injury caused by the breach must be difficult or impossible of accurate estimation; second, the parties must intend to provide for damages rather than for a penalty; and third, the sum stipulated must be a reasonable pre-estimate of the probable loss" [79]. This case addressed the intent of the parties for the second requirement, wherein the court found that in this particular case the seller intended to retain the right to other damages rather than liquidated damages, and so the provision was unenforceable. The court made clear that liquidated damages can be enforced in addition to other remedies given explicit language in the contract, otherwise the provision may instead be a penalty, and thus unenforceable.

The second case was Fortune Bridge Co. v. Department of Transportation (242 Ga. 531 1978) [80]. Fortune Bridge Company was awarded a $1M contract to build three bridges and a roadway for U.S. 19 in Georgia within a period of 620 days. The bridges were eventually constructed with a delay of about a year, and consequently GDOT withheld $73,000 ($200/day). The case appeared in front of the Supreme Court of Georgia, where the liquidated damages provision was upheld due to the inability to calculate the actual and consequential damages of a breach. This case supports the notion that a liquidated damages clause may be enforceable for cover deficiencies if GDOT was unable to accurately determine the actual damages in advance.

Neither the examination of statutory nor common law explicitly permit or prohibit the use of liquidated damage provisions to remedy a breach of contract for improper construction quality. While it may be possible to use liquidated damage provision, alternative contractual methods which have been successfully used for construction quality applications may be better suited.

2.7 Contracting Methods and Implementation

State transportation agencies have employed contracting methods to achieve construction goals, such as reduced construction time, reduced project cost, or quality assurance. Some of the main contracting methods used to achieve those goals are given in Table 6. The strengths and weaknesses of the methods have tailored their application to public works projects. An in-depth discussion of the provisions and how they relate to this concrete cover is given in the next subsections.

Table 6. Contracting methods used by state agencies

Method	Characteristics	Typical Uses
Incentives/Disincentives	Calculate a per day cost for early project delivery or delay related to the direct and indirect cost of project.	To achieve faster project delivery. Often in urban projects with high cost for delays. Used in highway construction and refurbishing.
Warranties	Requires contractor to repair or replace work if it fails to meet expected service life.	In cases where there is an interest in ensuring quality, examples include warranties on asphalt pavement. Generally short-term projects.
Design-Build-(Finance)-Operate-Maintain Frameworks	Contract features a requirement that the contractor operates and maintains the asset after construction. Shifts risk to the contractor and incentivizes quality construction.	In cases where it is feasible to shift operation and maintenance of the infrastructure to the contractor.
Acceptance/Adjustable Plans	Contract stipulates a testing regime that the work is subject to. The results of the testing can lead to a pass/fail judgment for acceptance plans, or a reduction/increase in payment due for the adjustable plans.	Used in pavement construction or other cases where quality assurance is the primary goal.

2.7.1 Incentive/Disincentives (I/D)

Incentive/Disincentive provisions are generally intended to reduce construction time and, in some cases, cost. The liquidated damages provisions mentioned in many state codes are the embodiment of an I/D provision, representing the benefit or cost incurred for changes in the delivery date of the project. In the case of the California Code, another form of I/D provision is given. In this case, it is solely an incentive provision, whereby the cost savings are split between the state and the contractor. For the particular application of improving cover control existing I/D provisions are not well suited. This is because the goal is to improve construction quality instead of construction speed or cost.

2.7.2 Warranties

As noted by [81], multiple states use warranties for quality assurance in a variety of public works applications ranging from asphalt density in roadways to paint thickness in steel bridges. The advantages of warranties are a guaranteed product/process quality (for the warranty period) and that the contractor is freed to optimize the construction process, which may result in more innovation and reduced cost. For warranties in the public works environment, the length of the warranty period (an important parameter to optimize) is often between two and twenty years. This period of time would be inadequate for the case of cover control where the damage is evident after longer periods of time, generally toward the end of the deck service life (e.g. 50 years). This observation also renders a short-term warranty, such as for the first ten years of service ineffective as well, as the short-term performance is not a good predictor of the performance over the full life of the deck. Furthermore, it is unclear which element the contractor would warranty, whether it would be the whole deck against defects for the warranty period, or perhaps just the cover concrete itself. In both cases, the required warranty period and scope would prove impractical. For example, if damage is observed 40 years after the construction, the contractor may no longer be in business, making recovery of costs difficult. For these reasons, warranties are not well suited for ensuring cover control.

2.7.3 Design-Build-(Finance)-Operate-Maintain (DBOM) Frameworks

In recent years, the legislature in the State of Georgia has passed a law that allows the private finance and operation of infrastructure (including bridges) [82]. The legislation sets requirements that GDOT annually identify projects that “afford the greatest gains in congestion mitigation or promotion of economic development” that would be appropriate for a public-private partnership (P3). The goals of the P3 initiative is to seek “innovative project delivery and innovative financing solutions from the private sector to meet the State's transportation needs." DBOM would represent an example of such a P3 arrangement. For this application, the public (or private industry) finance the construction of the bridge, with a separate entity (e.g. contractor or consortium) designing-building-operating-maintaining the structure for a period of time. While there are no known P3 projects in the state that is likely to change in the near future.

The Confederation Bridge provides an example of a large-scale P3 bridge project [83]. The Confederation Bridge was completed in 1997, having been entirely funded through a private consortium. In return for constructing the bridge, the consortium receives tolls on the bridge as well as an annual payment ($44M for 33 years) from the Canadian government. In 2032, at 35 years of service, the bridge will revert to government ownership, but in the intervening time the consortium is responsible for the operation and maintenance of the structure. This provides an incentive for quality construction as all repair and operation costs are borne by the consortium in this period.

If GDOT or other DOTs were to transition to a DBOM framework like that seen in the Confederation Bridge for future construction, the impacts of insufficient cover will be the concern of a separate ownership entity. There are, however, general public policy and legal concerns that will need to be addressed before DBOM frameworks become widely adopted for bridge construction.

2.7.4 Acceptance/Adjustable Plans

An acceptance plan is one that stipulates a testing regime for the parameter of interest or product, and then based on the results and an acceptance threshold, accepts or rejects the product. An adjustable payment plan uses the same methodology except instead of the binary acceptance or rejection decision, the payment for the product is adjusted based on the results. Both of these methodologies have been used by multiple DOTs [34]. These methodologies could be readily adapted for cover control. In fact, the status quo is in effect an acceptance payment plan. If the cover control is found to be inadequate and cannot be remediated, the engineer is empowered to reject the work similar to an acceptance plan.

For both acceptance and adjustable payment plans the testing methodology is paramount. In the case at bar, the testing methodology would involve sampling the deck surface to determine the cover distribution. The required number of samples and how the locations are chosen (i.e., randomly) would need to be stipulated. More samples would yield greater certainty that unacceptable work is not being unintentionally accepted or that acceptable quality work is mistakenly rejected. For an adjustable payment plan, more sampling would reduce the likelihood of underpaying the contractor for cover control that is at the acceptance limit. It could also be combined with a bonus in the payment scheme for work over the acceptable limit that offsets the risk to the contractor.

For examples for testing methodologies for both of these contracting methods, GDT 73 Method C (random selection of roadway concrete samples) [84] or SOP 46 (procedure for calculating pay reduction for failing roadway and bridge approach smoothness) [85] can be examined.

The methodology from GDT 73 Method C could be readily adapted for cover control compliance testing with the simple substitution of a span of the deck as the lot boundary (area which is evaluated) and measuring the cover thickness instead of the thickness of the roadway. The method provides tables for randomly selecting the locations for the depth checks within a subsection of the work termed a “sub lot.” The method utilizes an adjustable payment plan to link the payment the contractor receives for portion of the roadway to the roadway thickness measured from cores.

In SOP 46, the pay reduction for substandard road smoothness is computed by subtracting the ratio of the specified roadway smoothness to the actual road smoothness for each failing mile section from one [85]. For the purposes of cover control, the ratio of the actual average cover to the design cover could be subtracted from the full pay value as an example. The pay factor reduction is then used to de-rate the payment for all square yards of product in the failing mile section(s).

Acceptance/adjustable payment plans appear promising for construction quality assurance applications, such as cover control. The results of the service life modeling can serve as the basis for devising a link between the cover measured and expected performance of the deck. This methodology could be readily adapted to other similar construction quality applications.

2.8 Knowledge Gaps and Technical Needs

There appears to be knowledge gaps in the literature, which guided the direction of this research. The first gap is a more complete understanding of how reinforced concrete decks degrade in real world conditions, which is uniquely addressable in this work by the availability of bi-annual inspection reports for in service bridges.

The next gap is understanding the barriers to implementation of many of the novel and alternative materials described, not just in economic terms, but from a legal and political perspective as well. Combining those perspectives with the technological advantages may prove beneficial for future work.

Another gap is clearly seen in a lack of research in improved bridge deck construction practices. Poor workmanship can significantly decrease expected service life of bridge decks, and prevent realization of the gains that better materials and technology are expected to provide. Ensuring that the as-built product meets the designer's intent, while accommodating for the inherent variability in construction, will enable greater longevity in bridge decks. Of particular concern is the lack of research on ensuring concrete cover control. Proper cover control enhances durability by reducing the likelihood of multiple degradation mechanism such as premature corrosion, freeze-thaw damage, and excessive abrasion.

Implementing more accurate and robust early monitoring technologies in bridge deck assessments, as opposed to the current reliance on visual inspection and NBI ratings represents another gap. While a variety of promising technologies have been identified, they are not generally used in a preventive manner, but rather in response to observed deck distress. The predominant assessment remains visual inspections, which may result in more costly repairs and less confidence in the health of decks.

Estimating the extension to the service life of bridges from changes to the materials, construction practices, and monitoring may allow for prioritizing improvements in those areas. The aim would be to achieve the largest benefits for the cheapest or least intrusive costs, resulting in a healthier bridge inventory.

Finally, there is a lack of research on implementing the findings from service life modeling into the bridge construction contracts, so as to have an impact on the condition of the bridge inventory. Addressing these key gaps is the thrust of the work presented in this dissertation.

CHAPTER 3

RESEARCH AIMS AND OBJECTIVES

The overarching aim of this dissertation is to extend the service life of reinforced concrete bridge decks. The literature review in Chapter 2 identified key areas where knowledge and understanding were lacking, and these gaps served as the foundation for the objectives. Each objective is accompanied with a corresponding initial hypothesis:

Objective 1: To understand bridge deck degradation in Georgia and select an appropriate service life limit state.

Hypothesis 1: The main cause of degradation for bridge decks in Georgia is chloride-induced corrosion, and the end of service life is reached when sufficient surface damage is observed.

Objective 2: To model the service life of decks.

Hypothesis 2: The expected extension of service life for bridge decks with improved construction practices, novel and alternative materials, and earlier degradation monitoring and detection can be estimated from corrosion-based service life modeling.

Objective 3: To compare different construction materials and practices to estimate their impact on service life.

Hypothesis 3: Construction practices, most notably cover control, have significant impact on service life.

Objective 4: To implement the findings into practical contracting language.

Hypothesis 4: Improvements to bridge deck construction practices, and by extension service life, can be achieved through enforceable and effective contractual mechanisms that take into account legal, technical and economic considerations.

CHAPTER 4

BRIDGE DECK DEGRADATION

4.1 Chapter Overview

The objective of the work in this chapter was to understand bridge deck degradation in Georgia, and select an appropriate service life limit state. The objective was accomplished by analyzing all available inspection reports of inactive (also called decommissioned or deleted) bridges to track their degradation. This work served as the basis for selecting an appropriate degradation mechanism (corrosion), and devising a corresponding service life model for predicting the degradation of a bridge deck over its service life.

4.2 Approach

Many factors impact time initiation and rate of deck degradation. These include materials and mixture proportions, the quality of the deck's construction, the intended service conditions (rural, highway, etc.), and the environmental exposure. A bridge in a rural environment may experience less vehicular-induced wear on the deck than an equivalent highway bridge. Therefore, modeling the service life of a rural bridge assuming the degradation mechanism is abrasion may not prove informative. Harsh environments where chloride exposure is high may result in degradation from corrosion, and so the service life model of decks in those conditions becomes a corrosion model.

To determine the predominant degradation mechanisms of reinforced concrete bridge decks, the final bi-annual inspection reports of decommissioned decks were analyzed. As noted in Chapter 2, these inspections are performed based on FHWA guidelines, where the bridge is visually inspected and a NBI rating is assigned to each bridge component (deck, superstructure, and substructure) based on visual inspections.

These reports provide the conditions of the decks at removal or replacement (which represents the end of their service lives), and the frequencies of the various forms of damage observed by inspectors. From these, the main degradation mechanisms were identified. Aside from determining the main degradation mechanisms, these reports are useful for selecting an appropriate bridge deck limit state which represents the conditions that result in removal or replacement in service life modeling. A couple concerns warrant discussion from relying on these bi-annual reports.

The first concern is the subjective nature of the visual inspection, where the judgment of the inspector classifies both the type and severity of damage. For example, the inspector delineates “heavy” scaling versus “light” scaling on the deck. A case could be made that similar classifications of damage such as heavy and light scaling could be combined into a single type of damage “scaling” to reduce subjectivity. Identification of cracking and its extent (e.g., transverse vs longitudinal vs minor cracking) is another area where subjective judgement is apparent in the inspection reports. To avoid adjudicating whether or not certain combinations of damage types and severities are equivalent, each type was treated independently in the analysis.

The second concern is that the deck’s condition is not necessarily the only cause for decommissioning a bridge. It is entirely possible for a bridge to be removed or replaced for other reasons such as roadway expansion, the conditions of the substructure and superstructure, external events (e.g., fire). It may be that the deck may not have reached its true potential service life and may not have manifested all the damage that it would otherwise. This concern is alleviated to some degree by the advanced age of most decks at the time of removal (average of 60.5 years old, standard deviation of 12.9 years), the use of relative frequencies of the damage types amongst the decks, and the observed NBI ratings at decommissioning (less than 17 percent had a rating of good, very good, or excellent).

4.3 Methods

4.3.1 Predominant Degradation Mechanisms

The Georgia Asset Management System (GAMS) was used to source the bi-annual bridge inspection reports utilized. From GAMS, it was found that 524 bridges (including culvert structures) have been decommissioned since March of 2014 through the end of 2018. This sample set was used for all analysis because decommissioned bridge reports prior to March 1st, 2014 are unavailable electronically through GAMS and cannot be sourced in paper form. A filtering process was performed that excluded culvert structures, bridges without a concrete deck, and/or bridges with reports that had significant omissions (such as year built, missing inspection reports, etc.). After filtering, the number of bridges available for analysis was reduced to 341.

The main bridge inspection parameters extracted from the reports are presented in Table 7. By examining the latitude and longitude of the decks, it was apparent that the decks are well distributed across the state (see Figure 4), and do not represent just one area or set of environmental conditions. In addition to the parameters seen in Table 7, the damage types and severity were extracted verbatim from the reports as well. For these 341 decks, the inspection reports noted damage a total of 1137 times (with 58 distinct damage types), for an average of around 3.3 damage types per deck.

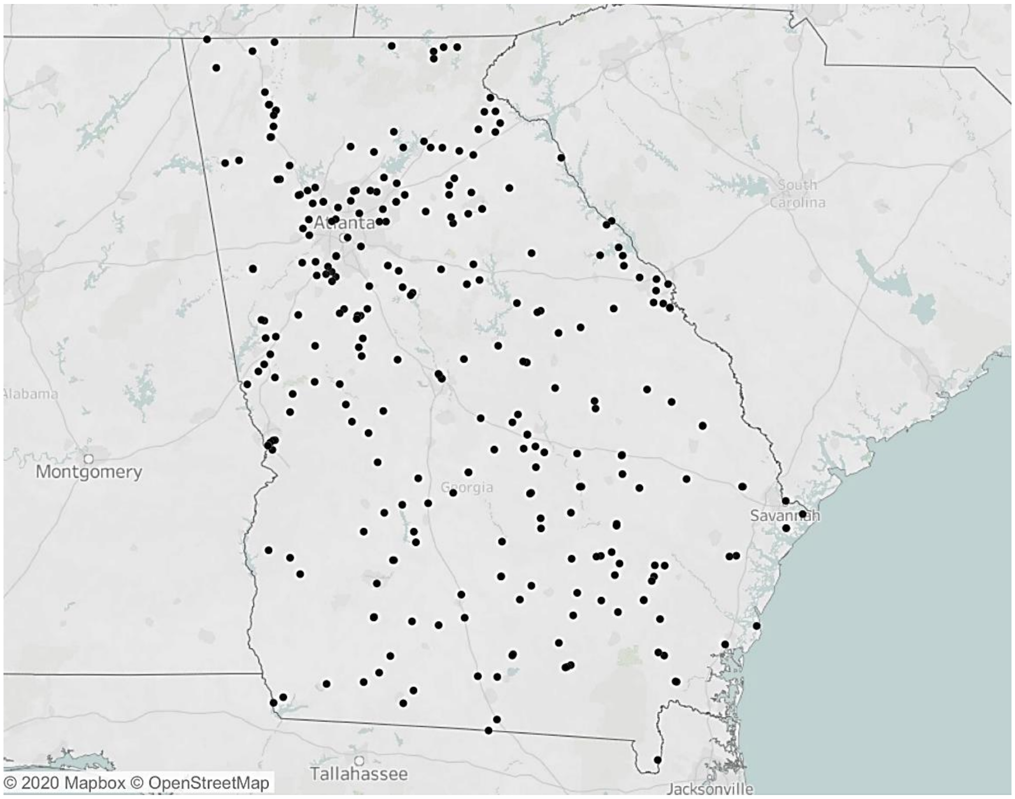

Figure 4. Spatial distribution of a sampling of the decommissioned decks.

Table 7. Bridge inspection parameters

Parameter	Definition
Bridge Serial Number	The identifier used by maintenance personnel to identify each bridge in the inventory. The form is xxx-xxxx-x, with the first three values being the county number.
Latitude	The latitude coordinate of the bridge in degrees, minutes, seconds format
Longitude	The longitude coordinate of the bridge in degrees, minutes, seconds format
Year Built	The year when bridge construction completed
Year Replaced	If a bridge was replaced by another, the year that it was replaced
Service Under Type	The type of service that the bridge spans over, such as a waterway or a highway
Service On Type	The facility carried by the bridge, such as a highway or a country road
NBI Rating	The condition rating for each subcomponent of the bridge (deck, superstructure, substructure) on a 0-9 scale. The higher the NBI rating, the better the condition of that component.

4.4 Results and Discussions

As expected, there was significant variability in damage for each of the major bridge components (see Figure 5). Ranking the damage in terms of total prevalence (summation of the prevalence in each major bridge component) yields Figure 5.

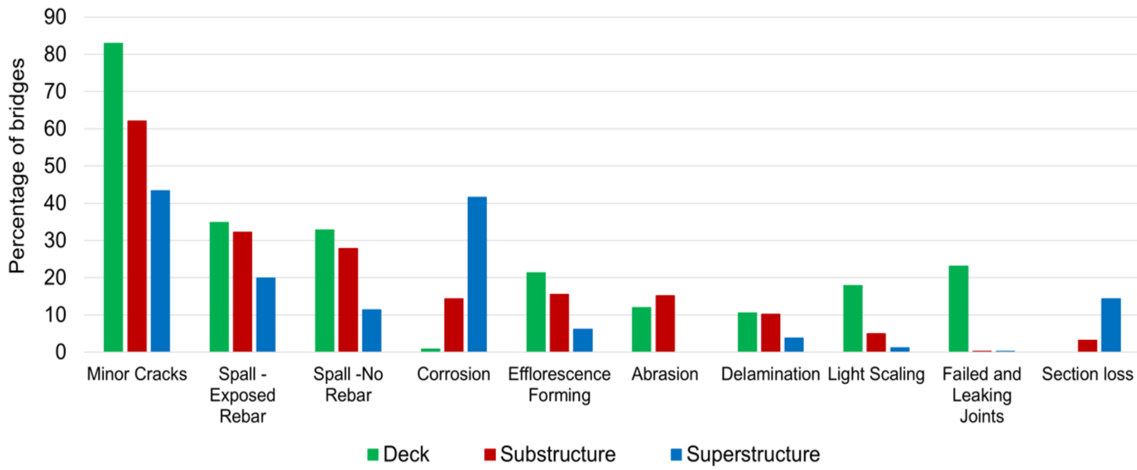

Figure 5. Top ten most prevalent bridge damage ranked by total prevalence

When examining Figure 5 it is important to recall the potential for interaction between certain damage types. For example, although essentially none of the bridge inspection reports expressly note corrosion on or in the deck, concrete spalling is frequently the result of corrosion, and exposed rebar in all likelihood will corrode. Both of these formed of damage were common on the decks.

With that consideration in mind, the primary damage types noted amongst the decks appear to result from corrosion and mechanical wear. In terms of corrosion related damage, 32 percent of decks noted spalling with exposed rebar in their reports and a similar proportion experienced spalling without rebar exposure (the groups are overlapping to some degree), 10.6 percent experienced delamination, and 7.6 percent had heavy cracking. Furthermore, 3.5 percent noted exposed rebar from thin cover, which likely resulted in immediate reinforcement corrosion. Compared to the relative prevalence of corrosion related damage types, abrasion and wear was observed in only 12 percent of the sampled decks.

The most frequent damage, minor cracking, was seen in 83 percent of decks. The cause of minor cracking can be attributed to a variety of sources, such as thermal expansion, mechanical stress, and early onset of environmental degradation (e.g., freeze/thaw, alkali-silica reaction). Though prevalent, minor cracking may or may not be a durability concern [17]. Monitoring of crack growth is necessary to make this determination. Unfortunately, the inspection methodology currently in place does not assess crack growth over time.

Figure 5 also shows some further limitations of the qualitative nature of the inspection, which in most cases are entirely visual. For example, virtually no section loss is observed in the decks in Figure 5, despite a significant proportion of decks that spalled and exposed rebar. It is highly unlikely that spalled decks with exposed rebar would not experience section loss, but rather the inspectors were unable to observe the section loss occurring within the decks, or perhaps the expansive nature of corrosion products may have obfuscated the underlying thinning of the rebar. Overall, the results in Figure 5 support the notion that a corrosion model may be required to adequately forecast the degradation of Georgia bridges because of the prevalence of spalling with exposed rebar in decks, and the high frequency of corrosion in the super and substructures.

Regarding deck degradation alone, Figure 6 presents the ten most prevalent forms of damage observed in the decommissioned bridge decks. These data further support the proposition that a corrosion model may be important in forecasting the service lives of Georgia bridge decks because majority of these damage types are associated with corrosion, either by resulting from corrosion, leading to the early onset of corrosion, or being present in highly corrosive conditions.

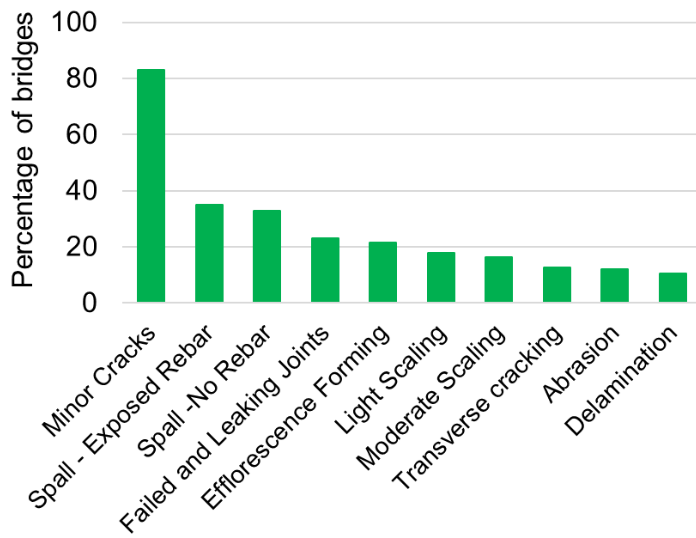

Figure 6. Deck damage ranked by prevalence

Despite the data limitations, some important findings can be made. First, a majority of decommissioned bridge decks had some form of spalling, which depending on the location and severity could disrupt the ability of traffic to safely pass. Second is the prevalence of scaling, which may be the result of chloride exposure from de-icing activities. The presence of abrasion on a significant number of bridges (>10 percent) represents the other significant degradation mechanism: mechanical wear. Based on the findings from this investigation, it appears that neither a freeze-thaw model, a mechanical stress model, nor an alkali-silica reaction model will provide meaningful insights into the degradation of Georgia's bridges. Rather, a corrosion model is believed to prove best for the intended application, in alignment with past approaches and findings [20, 40].

4.4.1 Selecting a Bridge Deck Limit State

Based on the inspection report analysis, it is proposed that the limit state for a bridge deck should be quantified based on the percentage of the surface experiencing delamination, spalling, and patching (DSP). To test this proposition, and quantify the bridge deck limit state, the inspection reports from 65 decommissioned bridges were pseudo-randomly selected from 341 decommissioned decks used in the preceding section. As an initial screening, any bridge where the final inspection did not report the quantity (ft^2) of at least one of the following deck defects: DSP, abrasion and wear (AW), or cracking was excluded. These defects represent the candidate criteria for defining the limit state. It should be noted that delamination, spalls, and patching are not separately counted when assigning the square footage affected along the deck, but combined into the single category, DSP, in the reports. For the purposes of the service life modeling that follows, the combination of those damage types is not impactful, because of the difficulty in predicting which of those specific forms of damage will manifest on any given deck.

Table 8. DSP areas in the bridge decks sampled

Bridge #	Total Deck Area (ft^2)	Damaged Area (ft^2)	Damaged Area (%)
3	10098	2.63	2.63
5	2412	83.33	83.33
7	108936	0.01	0.01
8	9116	0.34	0.34
9	8676	50.62	50.62
14	4590	1.33	1.33
16	8304	38.91	38.91
18	6919	0.16	0.16
21	24000	0.72	0.72
30	806	0.74	0.74
35	6600	0.61	0.61
36	42452	6.10	6.10
37	21358	0.24	0.24
39	5090	0.20	0.20
41	2400	1.17	1.17
46	9576	0.01	0.01
48	31220	0.54	0.54
49	31232	0.41	0.41
52	2781	35.96	35.96
54	2814	1.28	1.28
55	10098	0.53	0.53

Table 8 shows, for each bridge, the sum of DSP, on the basis of square footage affected, as well as percent of area affected. Table 8 features the 21 decks among 65 that exhibited DSP, while AW was found in 27 decks and cracking in 54 decks. The square footage for cracking and abrasion were less reliable and informative, with the full deck square footage often being described as affected, despite no corroborating evidence in the reports. Therefore, DSP, which affects about a third of the decks analyzed in smaller and more meaningful quantities, was selected to represent the limit state.

To facilitate comparison among these 21 decks, the affected area was converted into a percentage of the deck surface, and both the average deck area with defects and the median were calculated. The results show that the average deck area with DSP when the bridge is replaced or removed is 10.75 percent (*n* of 21, S.D. of 22), with a median of 0.72 percent. If outliers are removed, which are defined as being greater than two standard deviations away from the mean (only Bridge #5 qualifies), then the new average becomes 7.1 percent (*n* of 20, S.D. of 14.8), with a median of 0.66 percent. As discussed earlier, it is unclear whether or not the bridges sampled were replaced because of deck deficiencies. For that reason, the decks with higher percentages of damage are more likely the cause of replacement, but for those with very little damage there may be significant deck service life remaining.

These results can be compared to those found in [86], which surveyed the opinion of engineers across the country who make rehabilitation decisions for bridge decks. The authors found that the end of service life for a deck without overlays or similar interventions was the point when the level of damage from DSP was between 5.8 and 10 percent of the entire deck surface or 9.3 to 13.6 percent of the worst damaged lane surface. The averages observed here, 10.75 percent (without removing outliers) and 7.1 percent (removing one outlier) of the deck DSP match well with the range provided in the literature. To address the uncertainty in this estimate, a range of limit states may be better than selecting a single value. Consequently, for service life modelling the limit states were defined as when five percent and 10 percent of the deck surface is expected to experience delamination/spalls/patching.

4.5 Chapter Conclusions

The first finding from this chapter is a ranking of the prevalence and forms of damage observed on the various sections of decommissioned bridge decks. Spalling with exposed rebar is the most prevalent damage if minor cracking is excluded, and most of the degradation on the decks were either directly caused by corrosion or related to corrosion. Consequently, a service life model based on corrosion may prove best for forecasting the service life of bridge decks in Georgia.

The second finding is that limit states of 5 and 10 percent of the deck surface damaged by spalling are representative of the conditions of a deck at removal or replacement.

CHAPTER 5

TOP MAT COVER INVESTIGATIONS

5.1 Chapter Overview

Aside from the bi-annual inspection reports in Chapter 4, the other data sourced for this research is cover surveys for bridge decks built in Georgia. Control of concrete thickness over the top reinforcement mat in concrete bridge decks during construction is central for prolonging service life and minimizing maintenance costs. The objective of this work was to investigate the current cover control practices and to characterize the cover control delivered based on a series of analyses of historical data. A sampling of deck cover surveys from bridges built in the late 1970s to recent years was analyzed to determine the current cover control delivered. Greater than 90 percent of the sampled bridge decks had an average cover within 0.25 inch of their design cover. This tolerance appears constant over the last 40 years. Furthermore, the cover data was best approximated by a normal distribution or lognormal distribution, and may exhibit spatial interdependence. These findings were used in modeling the durability of bridges, which incorporate cover thickness as a key parameter. Additionally, these findings were also applied to contracting strategies (Chapter 7) that policymaker can use to assess construction quality and inform contract adjustments.

5.2 Data Collection and Aggregation

The historical cover surveys used in this research are derived from Georgia's inventory of 14,689 bridges. From these, 103 randomly sampled Georgia bridges built between 1980 and 2018 (Figure 7) were analyzed, representing a total of approximately 12,500 individual cover measurements.

Over this period of years, the records indicate that bridge decks in Georgia were designed with covers between 2 and 3 inches, but with the overwhelming majority having a design cover of either 2.25 or 2.75 inch. Bridges south of the Fall line (see Chapter 6) are designed with 2.25 inch cover and those north with 2.75 inch cover. Northern bridges are more likely to experience freezing and thawing cycles and will be more likely to be subjected to deicing during service.

Figure 7. Histogram showing the bridge sampling by year of construction.

The surveys were obtained from archival records collected and archived by the GDOT Office of Materials and Testing. Each cover survey features bridge and project identification information, the total number of cover measurements taken per deck, the cover values, the average cover, the standard deviations, and (for select bridges) a two-dimensional representation of the cover distribution. Cover depth was measured by electromagnetic cover meter, with an expected accuracy of +/- 0.19 inch [87]. All reports were made at the time of construction by technicians from the GDOT Office of Materials and Testing. When the observed cover diverges more than 0.25 inch from the design cover, the deck is normally cored to confirm the cover measurements, and in some cases corrective action is documented. A more comprehensive discussion of the cover control process in Georgia can be found in [88].

The research objectives were addressed through a series of analyses. Those analyses include: a time-series evaluation where changes in cover control over the 40 year period were plotted and tracked, spatial evaluations such as cover mapping and spatial autocorrelation calculations which characterize the cover distribution across the surface, and cover distribution fitting for use in informed policy setting and service-life modeling applications. For each analysis a unique sample of the full 103 survey data set was generated in response to different selection criteria. For example, the cover mapping data set required that the surveys have a two-dimensional map of the cover as measured, which was not needed for other analyzes. The number of surveys for each data set was based on the difficulty in meeting the selection criteria, with a minimum of ten decks.

5.3 Approach and Findings

5.3.1 Time-Series Evaluation

A time series evaluation was performed for all 103 cover surveys to determine their average and standard deviation. These bridges were constructed over the forty-year period from 1980 through 2020. As a first step, the data was aggregated without considering when the bridges were built, as seen in Figure 8. This figure shows the deviations from the design cover among these bridges. Greater than 90 percent of the surveyed decks have an average cover within 0.25 inch of their design cover, with a preference or skew toward excess cover.

These data were also used to examine for trends in cover control over this period. In the state of Georgia over the last 40 years, the design cover has varied between 2 and 3 inches generally in 0.25 inch increments.

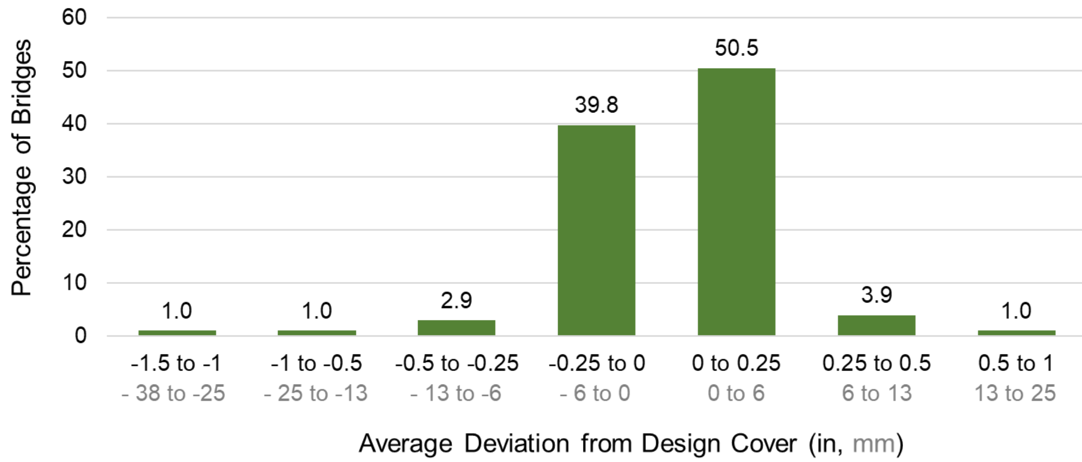

Figure 8. Average deviation from design cover for the 103 randomly sampled bridges.

To compensate for these changes in the design parameter specifications, a normalized average cover (Cn) was calculated as the difference between the average cover (Ca) and the design cover (Cd) according to Equation 4:

$$\mathrm{Cn} = \mathrm{Ca} - \mathrm{Cd} \qquad \text{Eq. 4}$$

This form of normalization has been used in prior work [65] and preserves the sense of scale of the measurements at a minimal cost to the accuracy of the plot in Figure 9. Figure 9 displays the normalized average cover (Cn) and the standard deviation. To visually differentiate the markers of bridges constructed in the same year, overlapping markers were slightly offset from each other along the horizontal axis. A linear fit to the data set aids in the visualization of possible trends in 'cover accuracy' over this period and is denoted with a dashed line.

The flatness of the fitted trend-line in Figure 9 suggests that cover accuracy has not changed significantly over time. Cover accuracy is defined as the magnitude of *Cn*, with *Cn*=0 being the most accurate cover control. This finding was not expected, given the advancement of construction technology, though technological adoption may have been limited [89, 90]. This is also surprising as any technological improvements over such a large inventory of bridges would aggregate into substantial savings in maintenance and service life extension.

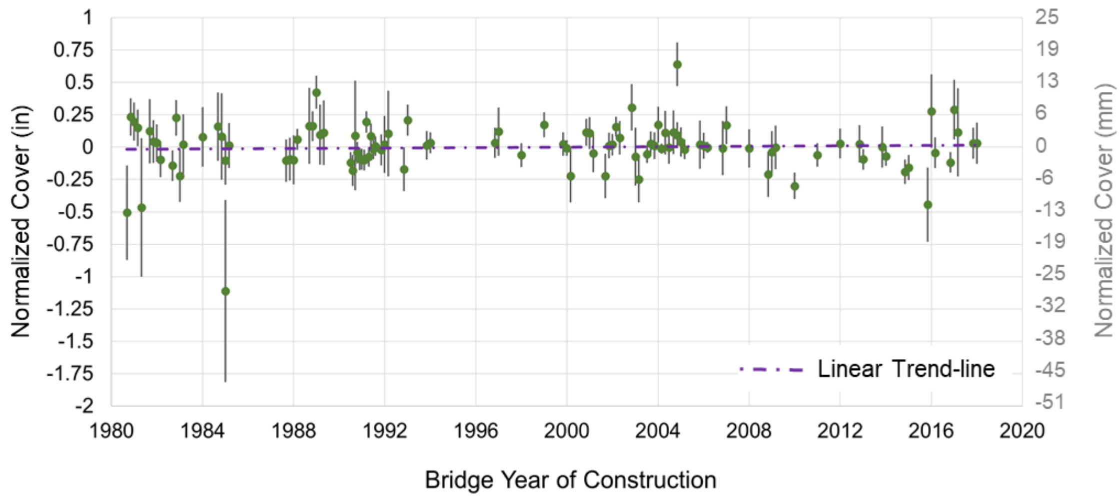

Figure 9. Normalized average cover for the 103 randomly sampled bridges with linear best-fit of the data.

It is also notable that the trend-line is very close to zero. While the aggregated data show a slight skew toward excessive cover (see Figure 8), it does not appear that there is a trend toward increasing (or decreasing) cover over this period. This observation does not support the idea that contractors intentionally pour excess cover to compensate for grinding, nor that a general industry shift in practice has occurred.

The variability in cover within each deck is evident in the whiskers associated with each of the data points. Some whiskers show significant variability, as much as 0.7 inch with an average of 0.16 inch. Figure 10 shows the average standard deviation for the decks in Figure 9 each year. It appears that the standard deviation decreased from the

1980s to 1990s, but thereafter there are no discernible trends – neither increases nor decreases in variation - over time.

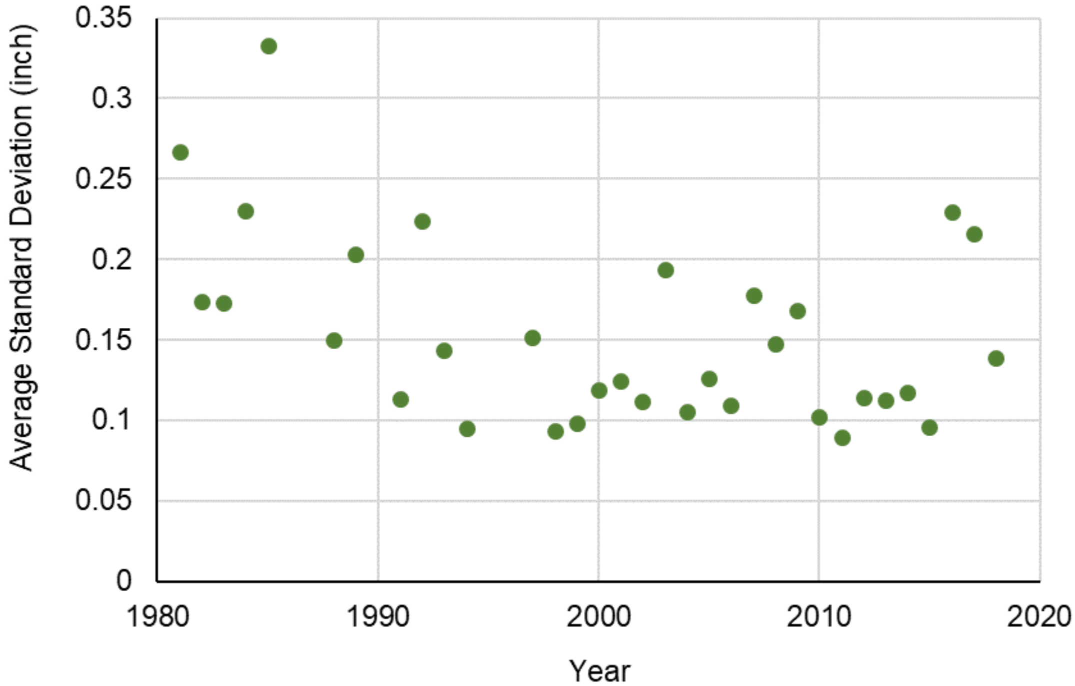

Figure 10. The average standard deviation for the sampled bridges per year of construction.

Similar to the observation around coverage accuracy, the apparent consistency of the standard deviation over the last 30 years suggests that any improvements in construction and monitoring have not translated into measurable improvements in cover control over this period. It may be that a standard deviation around 0.15 inch represents the inherent variability in cover control given current technology and practices.

5.3.2 Cover Mapping

The previous section provided insights into the average cover and variability among a bridge inventory over time, but does not describe how cover varies across the deck surface. Weyers et al. [86] determined, through a survey of engineers who make rehabilitation decisions for bridge decks, that most decks are removed from service at or before the point where approximately 10 percent of the deck surface is damaged. Examples of damage include spalling, delamination, and patching. If the exposure conditions and quality of materials are assumed to be consistent along the deck surface, then the portions of the deck with the thinnest cover could be the main parameter used to determine premature failure. That is, the deviation between the design cover and that of the thinnest 10 percent of the deck is likely more informative to the durability of the deck than the average cover. In addition, understanding how cover varies across the deck may indicate the cause of the variability. For example, if a deck has uniformly thin cover on half the surface, then it may be inferred that a systemic issue occurred during construction, for instance that a screed rail was installed too low. Conversely, whether cover variation is prone to clustering (i.e., with clear areas of low or high cover) or is randomly distributed across the deck will inform which cover sampling method is appropriate.

Cover variability across the deck surface can be visualized through two-dimensional cover maps, from select cover surveys in the database. For this analysis, 11 cover surveys were pseudo-randomly selected among the 103 bridge datasets. In this context pseudo-random selection refers to first randomly sampling and then rejecting bridges that did not have two-dimensional cover information. The selected decks were predominantly from the early 1980s and late 2010s, giving a good range of data over decades.

The two-dimensional data were extracted from the surveys and plotted to form a cover map, divided into 16 colors. For each map, the color range was set to +/- 1 inch from the design cover. The map linearly interpolates between the cover data points, which are represented by the vertices of the grid (assumed 10 ft square grid). The percentage of the deck area that is within each cover level (e.g., 1.5 to 1.625 inch) was determined by a

MATLAB® script which divides the number of pixels for each cover level (determined by pixel color) by the total number of pixels.

Figure 11 presents examples of the cover maps generated. Each portion of the figure represents a different bridge and displays a type of cover distribution commonly seen. Figure 11a shows the results from SR 72 bridge (deck constructed 2016, SR72 over South Fork Broad River, Madison County, GA) and exemplifies a near uniform cover distribution without significant areas of thin (red) or thick (blue) cover. This type of cover map, characterized by uniformity, was observed in 5 out of the 11 decks (46 percent). Figure 11b shows a deck (constructed 2018, SR22 over Bailey Branch, Crawford County, GA) with more heterogeneity, including a concentrated area of thin cover with the remainder of the deck having relatively good cover control.

This pattern suggests a localized issue with placing the rebar. Similar minimal areas of poor cover control were observed in 3 out of 11 decks (27 percent). Figure 11c (constructed 1985, widening of Northlake Parkway and I-285, DeKalb County, GA) presents the final major type which is similar in appearance to Figure 11a, but with large adjacent areas of deficient cover, and was seen in 3 out of the 11 decks (27 percent). A deck with fully systemic poor cover suggests either miscalibration of the cover meter, failed formwork, poor rebar placement, or incorrect screed rail placement.

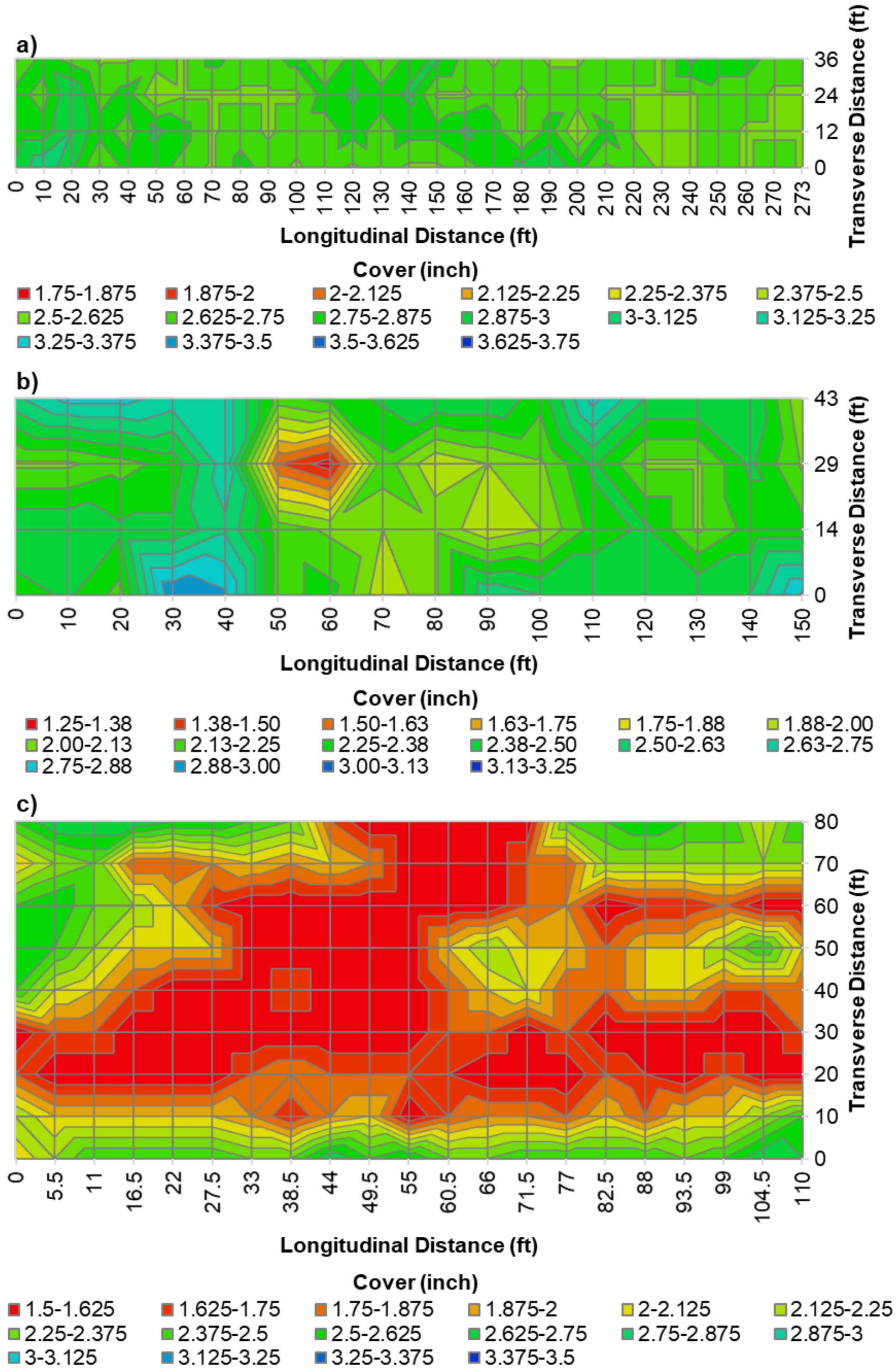

Figure 11. Cover maps for select decks with red representing thinner than specified cover, green representing the design cover, and blue representing thicker than specified cover. a) A deck with uniformly consistent cover as seen in 46% of decks b) localized area of poor cover as seen in 27% of decks c) uniformly thin cover as seen in 27% of decks.

The relative proportion of the deck that is within a designated cover threshold can also be calculated from these maps. This information is represented as a cumulative distribution function (CDF) for the cover, as seen Figure 12. Figure 12 shows that, on average, approximately 40 percent of the deck area of the sampled bridges are below design cover, and 60 percent are above design cover. Additionally, approximately 20 percent of deck area is more than 0.5 inch below design cover.

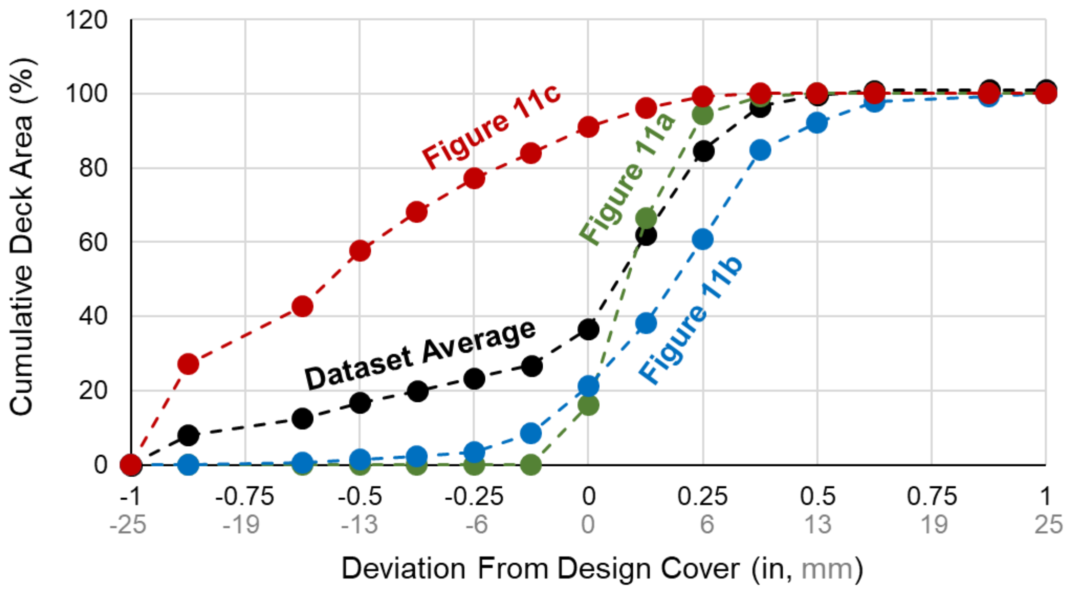

Figure 12. Cumulative Distribution Function (CDF) of bridge deck area per cover deviation for all 11 decks (black), for Figure 26a (green), for Figure 26b (blue), for Figure 26c (red).

As mentioned previously, 10 percent of deck surface with the thinnest cover ultimately determines the durability and performance of the deck. From Figure 12, acknowledging that this only represents the selected 11 decks, the thinnest 10 percent of cover is between 1 and approximately 0.85 inches less than the design cover. For a typical 2.75 inch design cover, 10 percent of the cover would be between 1.75 and 1.9 inches. Using the common error function solution to Fick's Second Law described in Chapter 2 and Chapter 6 (diffusion model), the service life of a concrete bridge deck due to corrosion can be approximated according to Equation 3 (Chapter 2) using the parameter values in Table 9.

Table 9. Input parameters for modeling

Cover (in)	C_s mean (lb/yd^3)	Diffusivity (in^2/yr)	C_t (lb/yd^3)	t_p (yr)
2.75	8.31	0.05	1.97	5

Using Equation 3, portions of the deck with 2.75 inch cover thickness are expected to last 59 years, versus 30 years for the 1.9 inch cover portions, and 27 years for the 1.75 inch cover portions.

Figure 12 also presents a clear distinction between the CDF for decks with better cover control (see lines for Figure 11a and Figure 11b) versus decks with poor cover control (Figure 11c), which have CDFs with a significant skew toward thinner covers.

5.3.3 Spatial Autocorrelation

The two-dimensional maps and CDF provided insight into the proportions of each deck within discrete cover ranges. The visual representation is helpful for understanding patterns and potential underlying causes for cover variation over a deck surface in a qualitative manner. However, a qualitative analysis is needed to compare the degree of cover interdependence between decks. One method to investigate this interdependence is to evaluate the data for spatial autocorrelation. Spatial autocorrelation is the relationship between the distance and value of data points within a data set, with a positive autocorrelation indicating that data points in close proximity are closer in value then those farther away. A common test for assessing spatial autocorrelation is computing the Moran's I statistic as shown in Equation 5 [91, 92].

$$I = \frac{n}{S_0} \frac{\sum_{i=1}^{n} \sum_{j=1}^{n} w_{i,j} z_i z_j}{\sum_{i=1}^{n} {z_i}^2} \qquad \text{Eq. 5}$$

In the equation, I is Moran's I statistic, n is the number of points, z_i is the difference between the value at x_i and the mean, z_j is the difference between the value at xj and the mean, $w_{i,j}$ is the assigned spatial weights for i and j, and S_0 is the sum of the spatial weights. The spatial weights represent the influence of the value of a neighboring point on I, which is chosen by the analyzer. In this case, the common approach of assigning a spatial weight of 1 for all nearest neighbors, and zero for all other data points was selected. The weights matrix (wi,j) was then row standardized to ensure that each neighbor had an equal influence on the value of the statistic, and I will be within the -1 to 1 interval.

For this application, I would compare the values of the cover at any point along the deck with the average of its neighbors. If there is clustering of cover values then I will be positive, with a maximum of value of one. If the cover is spatially random, then I will be approximately zero. If the data is spatially dispersed, I will be negative, with a minimum of -1.

The value of the I is not enough solely to assess the spatial autocorrelation of the data, rather the value of I should be compared against those of the null hypothesis. The null hypothesis is that the value of cover is completely spatially random. The results of this analysis were evaluated for significance through a Monte Carlo simulation ($P_{simulated}$) [93]. For the Monte Carlo simulation, I for 500 trials of randomly assigned cover values was determined, as well as a rudimentary significance value ($P_{simulated}$). $P_{simulated}$ is calculated according to Equation 6 [93], which is presented graphically in Figure 13.

$$p_{simulated} = \begin{cases} 2 * \dfrac{n_a + 1}{n + 1} \; when\ I \geq Iexpected \\ 2 * \dfrac{n_b + 1}{n + 1} \; when\ I < Iexpected \end{cases} \qquad \text{Eq. 6}$$

Where n_a is the number of trials where I for the simulated deck was greater than or equal to the I for the real deck, and n_b is the opposing count. $I_{expected}$ is the expected value for the Moran's I statistic via Equation 7 [92].

$$I_{expected} = \frac{-1}{(n-1)} \quad \text{Eq. 7}$$

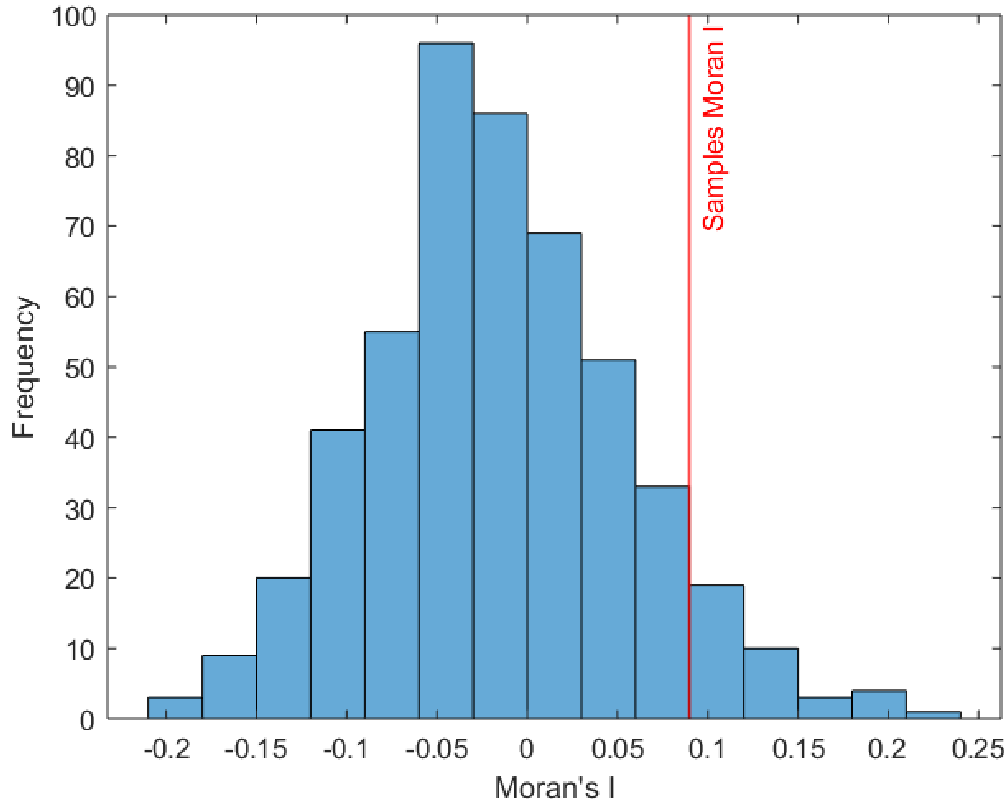

Figure 13. Results of 500 trials of a Monte Carlo simulation of I of a sample bridge deck. The expected I for this deck is -0.014, and the I for the data (0.090, marked with a red line) is more positive, representing greater than expected clustering. The Psimulated value for this deck is 0.15 suggesting there is a 15 percent probability that the sampled cover is the result of a completely spatially random distribution.

I was calculated for a sample of 36 bridge decks taken from the original set of 103. Without information to the contrary, each cover measurement was assumed to be 10 ft apart, taken in a grid. Table 10 presents the results from the Moran's I analysis of the 36 decks.

Table 10. Moran's I analysis results for the 36 decks

Bridge I.D.	I	I Expected	Variance	P Simulated	Reject Null
1	0.010	-0.026	0.0013	0.38	No
2	-0.017	-0.026	0.0014	0.70	No
3	0.345	-0.013	0.0003	0.00	Yes
4	0.314	-0.009	0.0002	0.00	Yes
5	0.280	-0.008	0.0001	0.00	Yes
6	0.458	-0.006	0.0001	0.00	Yes
7	0.332	-0.016	0.0005	0.00	Yes
8	0.520	-0.005	0.0000	0.00	Yes
9	0.518	-0.007	0.0001	0.00	Yes
10	0.298	-0.005	0.0000	0.00	Yes
11	0.811	-0.013	0.0003	0.00	Yes
12	0.515	-0.013	0.0004	0.00	Yes
13	-0.155	-0.031	0.0019	0.79	No
14	0.326	-0.021	0.0009	0.01	Yes
15	0.137	-0.010	0.0002	0.02	Yes
16	0.478	-0.023	0.0010	0.00	Yes
17	0.745	-0.017	0.0006	0.00	Yes
18	0.189	-0.024	0.0012	0.12	No
19	0.444	-0.021	0.0009	0.00	Yes
20	0.202	-0.014	0.0004	0.00	Yes
21	0.270	-0.014	0.0004	0.00	Yes
22	0.090	-0.014	0.0004	0.15	No
23	0.111	-0.007	0.0001	0.02	Yes
24	0.234	-0.006	0.0001	0.01	Yes
25	0.119	-0.024	0.0012	0.25	No
26	0.323	-0.011	0.0002	0.23	No
27	0.031	-0.053	0.0053	0.30	No
28	0.493	-0.016	0.0005	0.00	Yes
29	0.151	-0.017	0.0006	0.15	No
30	0.198	-0.006	0.0001	0.00	Yes
31	-0.084	-0.018	0.0006	0.25	No
32	0.269	-0.003	0.0000	0.00	Yes
33	0.010	-0.011	0.0003	0.48	No
34	0.216	-0.005	0.0001	0.00	Yes
35	0.114	-0.011	0.0003	0.09	No
36	0.254	-0.013	0.0004	0.00	Yes

From Table 10, it appears that for two-thirds of the decks, the null hypothesis of complete spatial randomness is rejected with a 95 percent confidence interval. In the vast majority of cases, the calculated *I* showed more clustering than expected. The implication of this finding is that cover sampling using a systematic method may result in biased findings. And for modeling applications, the cover measurements may not be treated as independent. The cover maps from Section 5.3.2 did not present such a clear majority of systematic cover, though the sample size was smaller and the assessment more subjective.

5.3.4 The Cover Distribution

The objective of this analysis was to determine a representative function describing the cover distribution for the deck inventory. This information is useful for modeling the durability of bridges which incorporate cover thickness as a key parameter, particularly when corrosion is considered as the primary degradation mechanism [42]. The information is also useful for policymaker as a way to assess construction quality and inform contract adjustments.

Cover has been approximated by multiple distributions, with a normal and lognormal distribution being very common [42]. It has been proposed that physical constraints (e.g., using polymer chairs, which is increasingly common since the mid-1980s) diminish the probability of inadequate cover; such practices would skew the probability distribution toward non-symmetric forms, specifically toward lognormal distributions. However, if the cover range is small, these physical restraints may not be consequential.

To identify candidates for the cover distribution, cover surveys taken from the cover survey dataset are plotted in Figure 14. The plots show a well-defined mean and a general “bell” shape, characteristic of both lognormal and normal distributions.

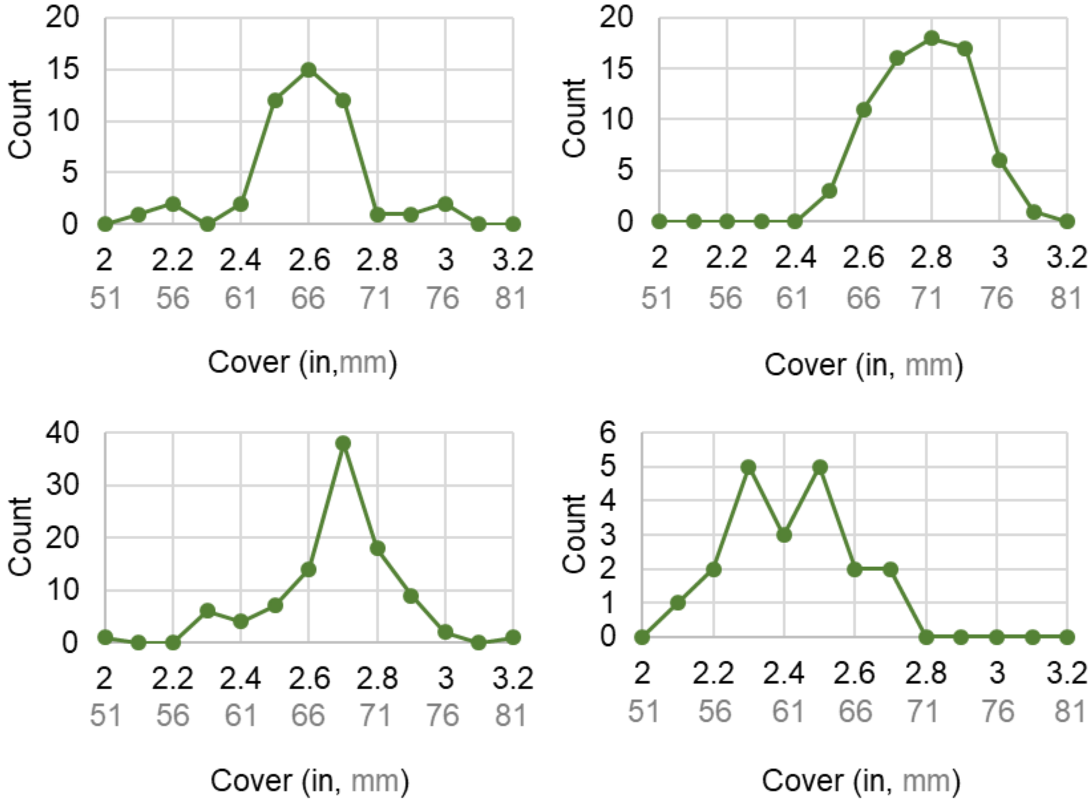

Figure 14. Example cover survey distributions for bridges from the cover survey dataset.

To compare the fit between a normal and lognormal distribution a data set of cover surveys for 36 individual spans (each from a separate bridge) with the same design cover (2.75 inch) were randomly selected among the 103 cover surveys. Next, a lognormal and normal distribution were fitted to each survey's data separately by means of a least squared error fitting regime, and an average error between the model and actual cover was calculated. The results are summarized in Table 11.

There does not appear to be a substantial difference in mean, standard deviation, and error between the fitted distributions when considering the average over the whole dataset. For both distribution the standard deviation is about 0.11 inch, and the average cover approximately 0.07 inch below design cover. The values from Table 11 can be compared to those from the literature (Table 4).

Table 11. Summary of results

<table>
<tr><th colspan="4">Normal Distribution</th><th colspan="6">Lognormal Distribution</th></tr>
<tr><th>Mean, μ</th><th>S.D., σ</th><th>C.O.V.</th><th>Error</th><th>Mean,μ</th><th>S.D., σ</th><th>Lambda, λ</th><th>Zeta, ζ</th><th>C.O.V.</th><th>Error</th></tr>
<tr><td>2.68 inch</td><td>0.11 inch</td><td>0.044</td><td>30.6</td><td>2.68 inch</td><td>0.12 inch</td><td>0.98</td><td>0.04</td><td>0.044</td><td>28.0</td></tr>
</table>

Table 12. Cover distribution from literature sources

<table>
<tr><th>Source</th><th>Year</th><th>Sample Size</th><th>Cover Distribution</th><th>Mean (inch)</th><th>S.D. (inch)</th><th>C.O.V.</th></tr>
<tr><td>Newlon [33]</td><td>1974</td><td>117 decks, 10,170 measurements</td><td>Normal</td><td>2.40</td><td>0.49</td><td>0.197</td></tr>
<tr><td rowspan="2">Weed [65]</td><td rowspan="2">1974</td><td>9 decks, 398 measurements</td><td>Normal</td><td>1.66</td><td rowspan="2">0.38</td><td>0.226</td></tr>
<tr><td>8 decks, 314 measurements</td><td>Normal</td><td>1.84</td><td>0.202</td></tr>
<tr><td rowspan="2">Weyers [66]</td><td rowspan="2">2003</td><td>21 decks, 2498 measurements</td><td>Normal</td><td>2.56</td><td>0.358</td><td>0.14</td></tr>
<tr><td>31 decks, 2670 measurements</td><td>Normal</td><td>2.60</td><td>0.38</td><td>0.145</td></tr>
</table>

From Table 12, it appears that a normal distribution was the most common form observed when sampling cover, despite the large intervening time between the sources. In addition, the coefficient of variation for the fitted data in Table 11 is significantly lower than those presented in Table 12. This is due to a substantial decrease in the standard deviation compared to the literature values, indicative of better cover control, the effects of which are modeled in Chapter 6. Table 11 also shows that for this dataset, the error for the lognormal distribution fit was approximately 8 percent less than that of the normal distribution fit, though this may not be statistically significant. Therefore, in Georgia, without a priori knowledge of the cover distribution on a bridge deck with a design cover of 2.75 inch, a normal distribution (mean 2.68 inch, standard deviation 0.11 inch) or lognormal distribution (Lambda 0.98, Zeta 0.04) may approximate the true cover distribution of the deck. This information will be used for the modeling in Chapter 6.

5.4 Chapter Conclusions

The result of current cover control practice is that greater than 90 percent of the 103 bridge decks sampled had an average cover within 0.25 inch of their design cover, with a slight preference toward thicker than average cover. The average cover presented is pre-grinding and the thinnest 10 percent of cover along the deck may have a more important role on ensuring the durability of the deck than the average cover. An analysis of historical cover surveys showed no significant improvement in cover control from the 1980s to present in terms of cover accuracy or cover spread. From analyzing the fit of distributions on a subset of 36 spans, it was proposed that for a design cover of 2.75 inch in Georgia, a normal distribution (mean 2.68 inch S.D. 0.11 inch) or lognormal distribution (Lambda 0.98, Zeta 0.04) may approximate the true cover distribution of the deck.

To understand the cover distribution along the deck, maps of cover for 11 sampled decks were generated that yielded the relative proportion of the deck within cover thresholds, with an average of 36 percent of the surface having the design cover thickness. A spatial autocorrelation analysis showed that the cover distribution may not be randomly distributed across the surface, which may impact cover sampling.

Small improvements to cover control over such a large bridge inventory may aggregate to significant impacts in terms of the durability and maintenance costs of the bridge inventory. Future work may attempt to integrate new technology earlier in the cover control process.

CHAPTER 6

SERVICE LIFE MODELING

6.1 Chapter Overview

The goal of the work in this chapter is to model and compare the expected service life of decks built with alternative materials (alternative rebar selection, mix designs, and service coatings) and improved construction practices (cover and cracking control). To accomplish this goal, three service life models were developed: diffusion model, probabilistic simulation, and full probabilistic model. The three models, collectively, were used to model the service life of decks in a variety of conditions, and are based on data from the literature (see Chapter 2) as well as data sourced for this project (e.g., cover surveys and inspection reports). The results from the modeling form the basis for recommendations in contracting to extend expected service life.

6.2 Approach

It was hypothesized that the expected extension to the service life of bridge decks from improved construction practices, the use of appropriate materials and mixture proportions for reinforced concrete, and alternative reinforcement can be estimated from corrosion-based service life modeling. To make meaningful comparisons between the results, a baseline prediction for the service live of decks from each model was determined. Ideally, the baselines would be based on the performance of in-service and recently decommissioned decks, with full knowledge of all the important modeling parameters (e.g., concrete mix, cover surveys, chloride exposure). In practice, however, no such database has been found for the Georgia bridges that have been selected for this research. As a result, significant uncertainty surrounds the modeling of deck service lives. In response to this uncertainty, a set of three corrosion-based models was created with differences in their complexity, modeling principles, and information intensity. For each of the models, a baseline deck performance and default key model parameters, which describe the decks (e.g., cover thickness, concrete quality, etc.) and surrounding environmental characteristics (e.g., chloride exposure) were determined.

The first two models (diffusion and probabilistic simulation) rely primarily on values from the literature for the main modeling parameters (see Chapter 2). The diffusion model uses the error function solution of Fick's Second Law (Chapter 2) to model the ingress of chloride from the environment, through the concrete, to the surface of the rebar for a uniform deck (i.e., invariant deck properties). The probabilistic simulation models the same chloride ingress process, but over a two-dimensional deck surface for a non-uniform deck. If the parameter values from the literature, on which these models are based, are good approximations for the true values in Georgia, then these two models may provide meaningful predictions.

The full probabilistic model, in contrast to the other two, attempts to find the main parameter values using a regression analysis of the degradation of select decks in real–world conditions (see Chapter 4). Since each deck may only provide very limited information, such as a bridge age and the corresponding amount of damage (e.g., spalling, delamination, and patching), the regression is performed on a group of similar decks. This group will be referred to as a "population." The similarity and number of the decks in the population, as well as the objectivity of the inspectors in characterizing the damage they observed, greatly impacts the accuracy of the predictive model.

After establishing the baseline performance and model parameters of the decks with each of these models, the modeling parameters were altered to simulate the use of alternative materials and construction practices. For example, to simulate building a deck with stainless steel rebar as opposed to ordinary low-carbon steel rebar, the chloride threshold was increased from the baseline value to that of stainless steel [21, 30]. This approach has been used successfully by other researchers for similar purposes [29]. It is worth noting that this approach assumes that the use of stainless steel rebar has no effect on the other properties of the system, such as the diffusivity of the concrete or the chloride exposure. This assumption is valid in most cases.

6.3 Establishing Performance and Model Parameters

The baseline for the predicted service life and model parameters of reinforced concrete bridge decks in Georgia was determined for each of the three models. Each are described briefly here, and in more detail in the following sections. The first model is a diffusion model. This model was a simpler corrosion analysis, whereby the service life of the deck is tied to the ingress of chlorides from the environment (a common approach). Simple modeling often has advantages over more sophisticated approaches when there is significant uncertainty in the modeling parameters [47]. The second model developed utilized a probabilistic simulation, which combines the diffusion of chlorides and parametric uncertainty by performing many trials with variation of the key model parameters. This method was chosen because of its affinity to uncertain systems, and the ability to model the deck degradation in two dimensions (i.e., a planar deck surface). The probabilistic simulation is more information intensive, but with appropriate parameter values will yield more accurate predictions than more simple models. The probabilistic simulation was only used for the cover accuracy evaluation, when the available two-dimensional cover survey data could be used. The final model, the full probabilistic model, relies on defined probability distributions for the key modeling parameters and was used for regressing those parameters from observed field performance. This model was the most complex, allowing for considerations of deck location as well as cracking.

6.3.1 The Diffusion Model

As described in Chapter 2, the diffusion model uses Fick's Second Law to predict the diffusion of chloride through the deck. For this model, the key parameters are C_t, C_s, D, x, and t_p. As described in Section 2.5.1, an appropriate selection for these values can be represented in part, by 12 permutations with combinations of the C_t values (min, max: 1.22, 1.97 lb/yd^3), C_s (min, max: 8.31, 21.8 lb/yd^3), D (min, avg, max: 0.050, 0.301, 0.614 in^2/yr), and a range of average cover values (1.7 - 3.7 inch). These 12 permutations represent variations in deck construction and environment. The variation in C_t represent the range found in the literature for corrosion resistance of plain low-carbon steel rebar. The variations in C_s, represent a less aggressive (e.g., inland) versus a more aggressive (e.g., marine) chloride exposure. The D values represent the range of concrete quality, from very good (0.050 in^2/yr) to poor quality (0.614 in^2/yr). The 12 permutations are given in Table 13.

Table 13. Diffusion model permutations

Permutation	C_t	C_s	D
1	1.22	8.31	0.050
2	1.97	8.31	0.050
3	1.22	21.8	0.050
4	1.97	21.8	0.050
5	1.22	8.31	0.301
6	1.97	8.31	0.301
7	1.22	21.8	0.301
8	1.97	21.8	0.301
9	1.22	8.31	0.614
10	1.97	8.31	0.614
11	1.22	21.8	0.614
12	1.97	21.8	0.614

The permutations in Table 13 were solved according to Equation 3, which is the common error function solution of Fick's Second Law, assuming one-dimensional chloride ion diffusion over a uniform deck. The error resulting from these assumptions are minimized when the deck conditions are very homogeneous across the surface, which is unlikely in real world conditions. The results from the 12 permutations are also represented graphically in Figure 15:

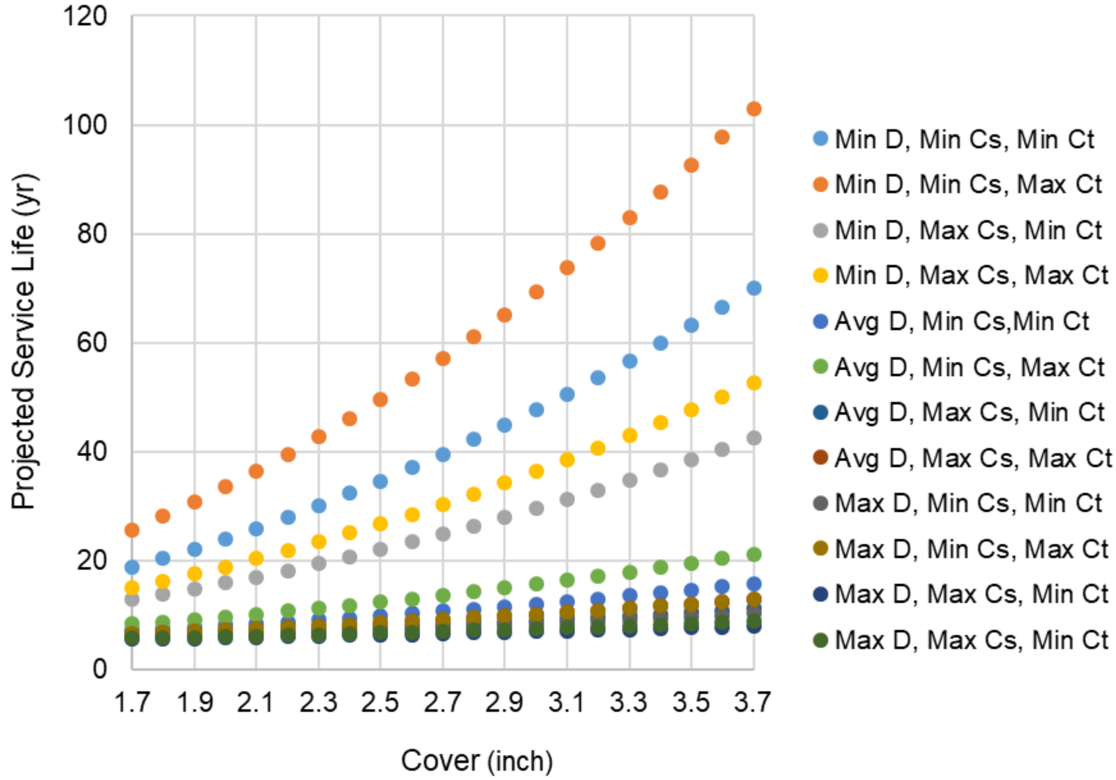

Figure 15. Results of diffusion modeling with varying parameter values.

From Figure 15, it appears that the envelope for the service life is approximately five to 100 years, for the cover and other parameters selected. From the sampling of the decommissioned bridges (see Chapter 4), the average service life from construction to bridge replacement or removal was found to be 60 years (*n* of 286, S.D. of 13 years). This service life is only achievable with the parameters (Min D, Min C_s, Max C_t, and 2.75 inch cover) represented by the orange points, and also those represent by the light blue points (Min D, Min C_s, Min C_t, and 3.3 inch cover). These two sets of parameters share low values for D and C_s. Between those two parameter sets, those corresponding to the orange curve are more appropriate (C_t 1.97, D 0.050, C_s 8.31) since cover has never been specified at greater than 2.75 inches in Georgia. Therefore, baseline performance and model parameters for this model were selected as: a service life of 60 years and C_t of 1.97 lb/yd^3, C_s of 8.31 lb/yd^3, D of 0.05 in^2/yr, and cover of 2.75 inches. These values are well within the ranges established in Chapter 2.

6.3.2 Probabilistic Simulation Model

While the results from the simple diffusion model are insightful, a more complex analysis was conducted to explore how non-uniformity in the bridge deck materials and structure (e.g., cover depth) impact corrosion performance over service life. These scenarios allow for better representation of a range of real-world conditions. Though many corrosion models exist that are more sophisticated than the diffusion model, the limited information available for the bridge inventory of Georgia inhibits their use for this application. Therefore, to address this level of uncertainty in the key parameters, a probabilistic simulation was developed in collaboration with IFSTTAR (The French Institute of Science and Technology for Transport) [94].

The methodology developed is presented schematically in Figure 16. The model uses the same set of corrosion parameters C_t, D, x, C_s, but unlike the diffusion model, incorporates variability. The bridge deck surface is subdivided into smaller surface areas called "cells." Each cell is an equal-sized subdivision of the deck, such that it contains at least one concrete cover measurement from a cover survey. Typical cell size was found to be a maximum of 10 feet by 10 feet, but may be smaller depending on the concrete cover survey grid.

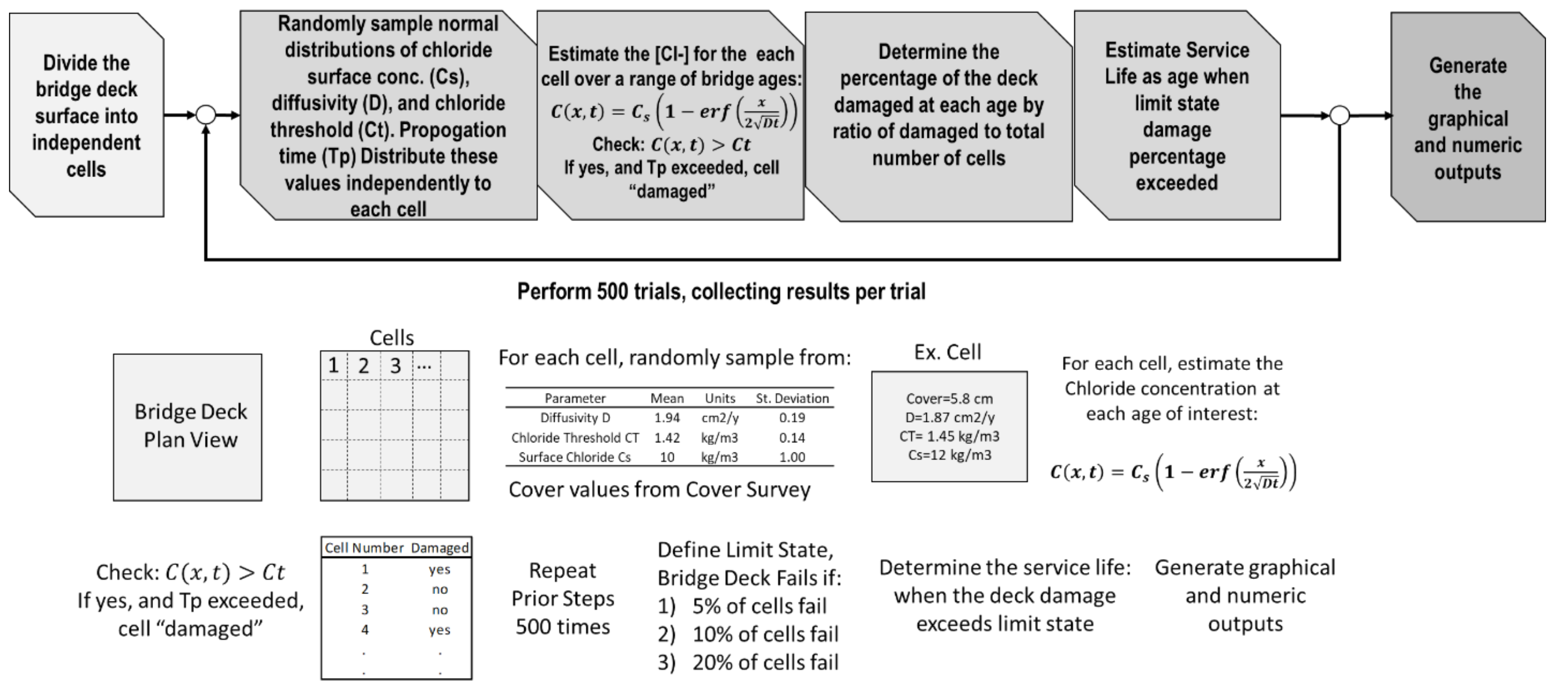

Figure 16. Probabilistic modeling methodology.

For each cell, values randomly sampled from normal distributions of the corrosion parameters (e.g., surface chloride concentration, chloride threshold, apparent diffusivity, propagation time) are assigned. Next, the performance of the cells is evaluated at the bridge ages of interest (years 6 to 100 in 2 year increments) by way of Equation 3.

The concentration of chlorides at the rebar depth is then calculated for each cell and compared to the assigned chloride threshold. At each deck age evaluated, if the threshold is exceeded and the propagation period has been satisfied, then the cell is recorded as damaged, if not, it remains in an undamaged state.

The main parameter of interest, as found in Chapter 4, is the percentage of the deck surface that is predicted to be damaged (by spalls, delamination, and/or patching) at a given bridge age. To calculate the percent of the deck damaged, the total number of cells is divided by the number of damaged cells at each bridge age. For each age examined, the entire process described above is repeated 500 times (a sufficient number to achieve stable results). The results from the 500 iterations are then averaged for each bridge age.

The final step is to determine the ages at which the limit state criterion are met, namely when the bridge deck has experienced damage in five percent or ten percent of its surface area. The age of when these criteria are met becomes the predicted service life.

To determine the baseline performance and key modeling parameters, a data set of 20 bridges were randomly selected for evaluation from the 103 available decks with cover surveys (see Chapter 5). Each deck had a two-dimensional cover survey, which provides both the location and magnitude of the cover along the surface. The bridges were evaluated against four sets of values for the key parameters which will be referred to as "cases." The cases are based on the values taken from the literature, as well as the performance observed in the diffusion model (Figure 15). The cases were chosen to represent variations in chloride exposure and variation in steel reinforcement, as Georgia utilizes both epoxy-coated and plain rebar (C_t of 3.6 and 1.97, respectively) in deck construction. The value for the diffusivity was kept constant as the previous one-dimensional analysis suggest that 0.050 in^2/yr yields results most consistent with observed field performance. The propagation time was kept at five years in alignment with the one-dimensional analysis, and the three main values were assumed to have a normal distribution with a five percent standard deviation, which introduces a controlled level of variability. The values used in each case are given in Table 14. The results from the probabilistic simulation are presented in Table 15, and Figure 17 to Figure 20. For the Figures, each line represents the results for each of the 20 decks, which are designated by their average cover.

The wide range in performance is expected as minor variations in the modeling parameters can have significant effects on the predicted service life, especially over such a long evaluation period. The limit states of five and ten percent of the deck surface were selected to signify the end of the deck's service life.

Table 14. Parameters explored in probabilistic service life model.

Case	Cover mean	Cover S.D.	C_s mean	C_s S.D.	D mean	D S.D.	C_t	t_p
1	2.75	0.14	8.31	0.42	0.05	0.0025	1.97	5
2	2.75	0.14	8.31	0.42	0.05	0.0025	3.60	5
3	2.75	0.14	21.8	1.09	0.05	0.0025	1.97	5
4	2.75	0.14	21.8	1.09	0.05	0.0025	3.60	5

Table 15. Predicted service lives in years for the bridges.

Bridge	Cover Avg.	Cover S.D.	5% Damage Limit State				10% Damage Limit State			
			Case 1	Case 2	Case 3	Case 4	Case 1	Case 2	Case 3	Case 4
1	1.66	0.65	7	9	6	6	8	12	6	7
2	1.93	0.34	17	33	11	14	20	39	12	16
3	2.13	0.15	29	57	17	22	31	62	18	24
4	2.25	0.22	30	60	17	23	32	65	18	24
5	2.27	0.12	34	68	19	26	36	72	20	27
6	2.29	0.04	37	73	21	29	38	77	21	29
7	2.30	0.08	37	73	21	28	38	77	21	29
8	2.35	0.23	32	65	18	25	35	71	20	27
9	2.41	0.14	38	83	21	29	40	88	22	30
10	2.41	0.12	39	77	22	30	40	81	22	31
11	2.53	0.16	41	97	23	31	43	100*	24	33
12	2.65	0.16	43	88	24	33	46	95	25	35
13	2.65	0.14	45	90	24	34	47	96	26	36
14	2.69	0.09	48	97	26	37	50	100*	27	38
15	2.72	0.15	48	97	26	37	50	100*	27	38
16	2.73	0.12	48	99	26	37	51	100*	27	38
17	2.76	0.22	43	90	24	23	48	100	26	24
18	2.78	0.12	50	100*	27	23	52	100*	28	25
19	2.81	0.12	51	100*	28	39	54	100*	29	41
20	2.87	0.26	47	100*	26	36	50	100*	27	38

* Projected service lives in excess of the evaluation period of 100 years.

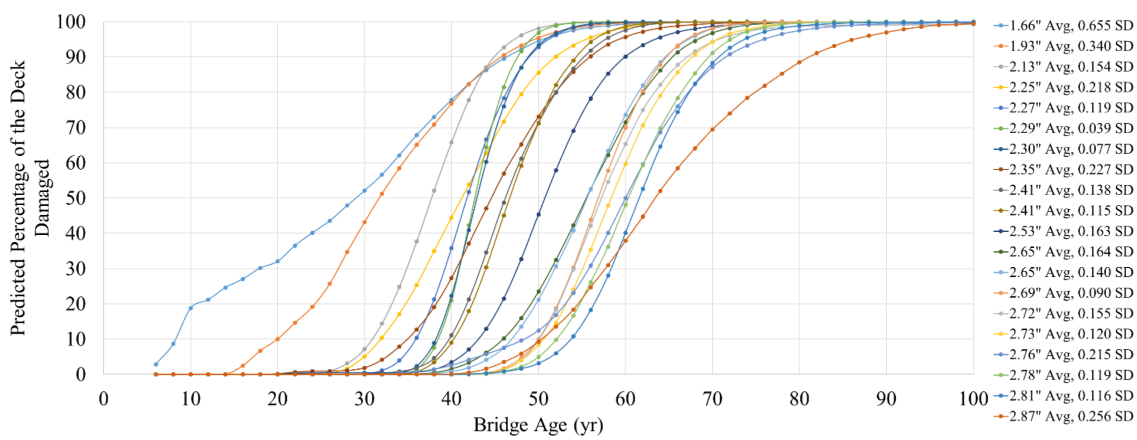

Figure 17. Case 1 results: predicted percent deck damage over time.

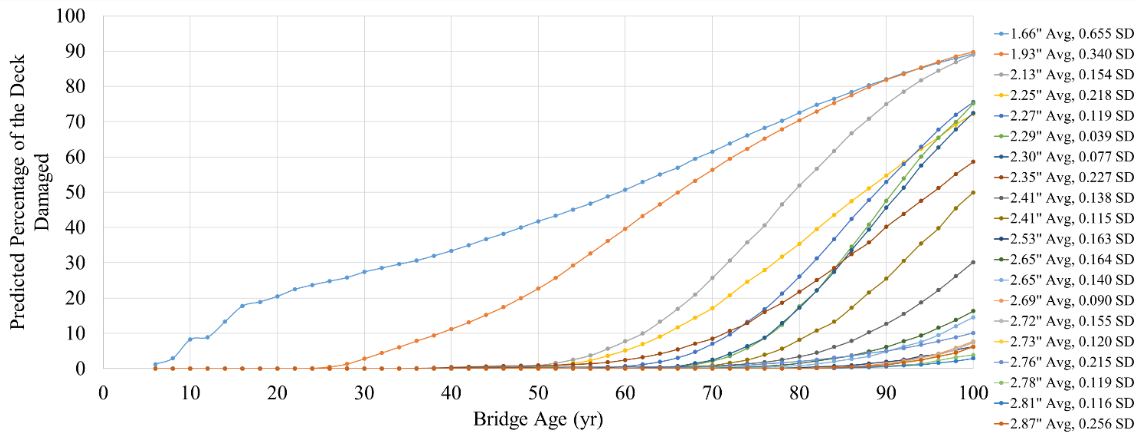

Figure 18. Case 2 results: predicted percent deck damage over time.

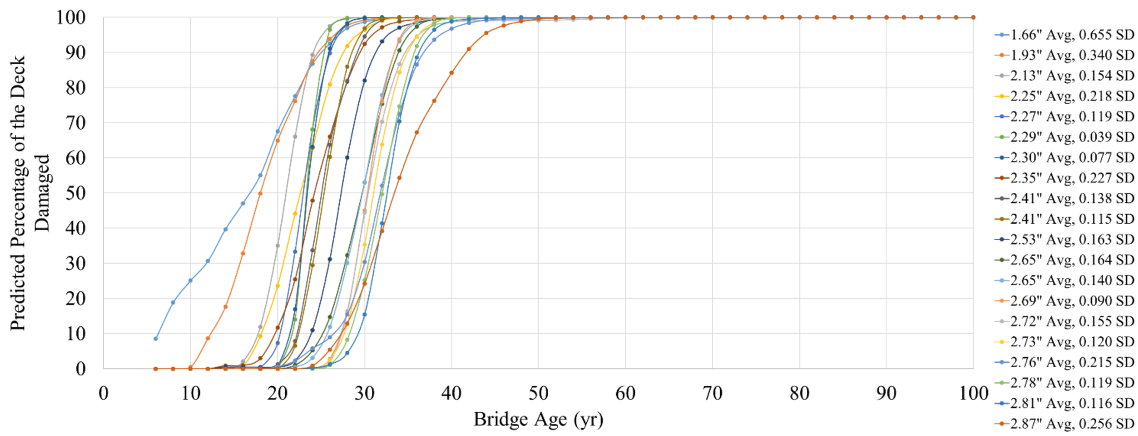

Figure 19. Case 3 results: predicted percent deck damage over time.

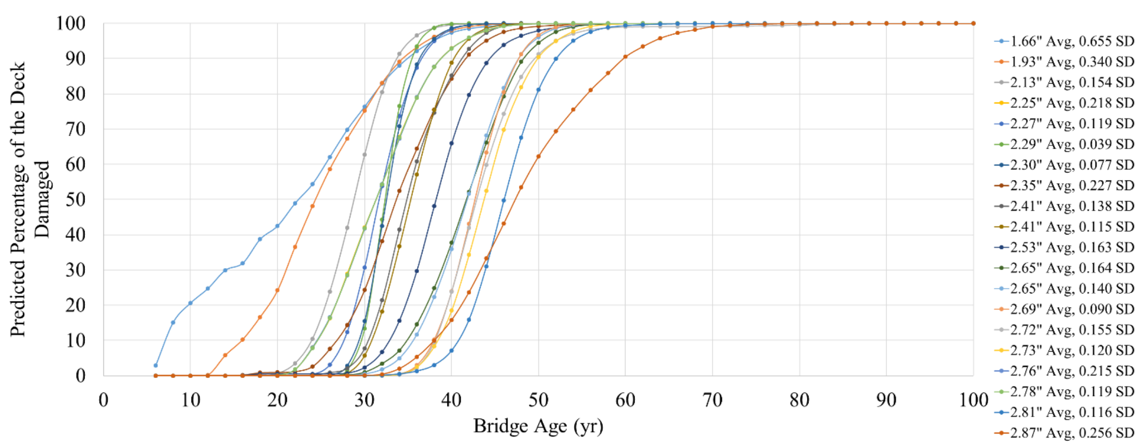

Figure 20. Case 4 results: predicted percent deck damage over time.

The results for the four cases were averaged for each limit state and then plotted according to the average cover for each bridge. The resulting data was then fit with a polynomial function, as shown in Figure 21. From the figure, the predicted service life changes on average about two years per 0.1 inch of cover, with a slight variation in the fitting between the five and ten percent limit states, with otherwise good agreement. The predicted service life for the common 2.75 inch cover is on the order of the 60 year average service life, similar to what was observed in the early portions of this study (Chapter 4).

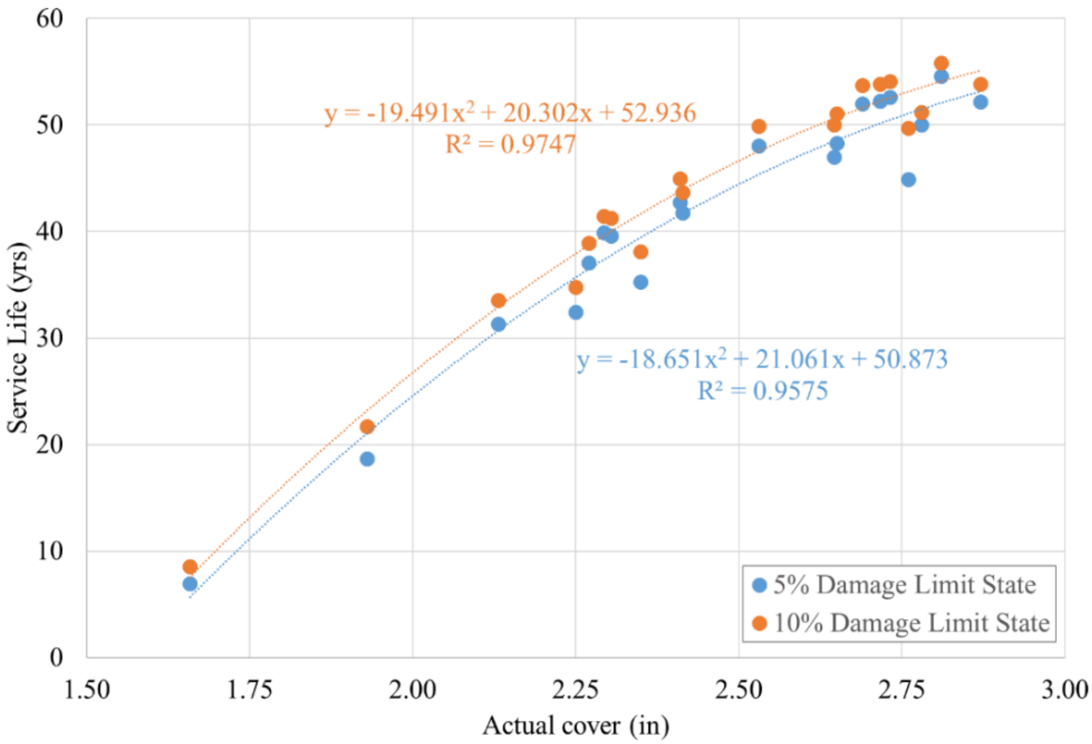

Figure 21. Results from the probabilistic simulation, plotting the predicted service life based on the sampled cover surveys and the five and ten percent limit states. The data is fitted with polynomial trend-lines, with the equations displayed on the figure.

The baseline performance that this model provides is an expected service life of 53 years (with the ten percent damage limit state), assuming 2.75 inch cover. The service life for other bridges in the state is then calculated using the fitted equations provided in Figure 21. The main advantages to this model are the ability to model non-uniformity in the model parameters across the deck, and the spatial context that the model provides. This model not only provides the amount of the surface that is damaged at a given time, but also which specific cells are damaged (corresponding to identifiable portions of the deck). However, to provide that spatial information, the model requires two-dimensional cover data. Within the database of cover surveys, no significant number of decks had both the requisite two-dimensional cover data and corresponding damage noted in its inspection report. That is likely due to the fact that all the decks in the database where built from the 1980s onward, and as a result, are not advanced enough in age to present noticeable damage.

While the probabilistic simulation could be used to perform a regression on the surface damage data from Chapter 4, it is cumbersome to do so. As a result, a new model was created specifically for the purpose of the regression procedure.

6.3.3 Full Probabilistic Model

The last model, the full probabilistic model, so named because it utilizes a fully probabilistic approach to service life modeling, is a large departure from the other two models. For a full description on the theory behind the full probabilistic model, the reader is directed to [95], with only the essential aspects being presented here. For this model, just as in the probabilistic simulation, the bridge deck is represented as a series of independent cells. The corrosion behavior of the cells is assumed to be independent of one another, which while not true, is a reasonable simplification for this purpose. In such an arrangement, if all other corrosion parameters are consistent, then the first cell to experience damage would be the one with the thinnest cover. If the probability distributions for all the key parameters are known and related to one another with the error function solution of Fick's Second Law, then the damage along the surface of the deck at a time *t* can be described according to Equation 8 [95]:

$$\mathrm{DM}(t) = \int_{\mathrm{Dl}}^{\mathrm{Dh}} \int_{\mathrm{Csl}}^{\mathrm{Csh}} \mathrm{F_x}\left(2\sqrt{\mathrm{D}(t - t_p)}\,\mathrm{erf}^{-1}\left(1 - \frac{\mathrm{C_t}}{\mathrm{C_s}}\right)\right) \mathrm{f_{cs}}(\mathrm{C_s})\mathrm{f_D}(\mathrm{D})\,\mathrm{dC_s}\,\mathrm{dD} \qquad \text{Eq. 8}$$

Where *DM(t)* is the damage at time *t*, D_h and D_l are the upper and lower limits of *D*, C_{sh} and C_{sl} are the upper and lower limits of C_s, F_x is the cumulative probability distribution for cover, f_{cs} is the probability distribution function for C_s, and f_D is the probability distribution function for *D*. Equation 8 can be modified to consider cracking, by duplicating the equation and changing and *D* terms to be that of cracked concrete instead. In such cases, the percentage of the deck *DF(t)* can be calculated according to Equation 9.

$$\mathrm{DF}(t) = (1 - \mathrm{C_F})\mathrm{DM}(t) + \mathrm{C_F}\mathrm{DM_C}(t) \qquad \text{Eq. 9}$$

Where C_F is the cracking fraction that represents the proportion of the deck surface influenced by detrimental cracking, and $DM_C(t)$ is the result of Equation 8 utilizing a D value representing that of cracked concrete.

As a basis for the cracking fraction, the work of Balakumaran et al. can be examined [17]. They found that between zero and 21 percent of the Virginia bridge decks they surveyed experienced cracking capable of diminishing the service life of the deck, which served as the range for C_F. Furthermore, in the cracked areas the diffusivity was found to increase by a factor of three in ordinary portland cement decks. Therefore, to incorporate the effect of cracking in the model, a portion of the deck (the crack fraction) was subjected to a diffusivity three times greater than that of the remainder of the deck.

From the bi-annual inspection reports (Chapter 4), the age and percentage of the deck damaged by deamination, spalling, or patching (DSP) was known for 32 bridges. The next objective was to group these bridges into populations for the regression analysis. Figure 22 shows the bridges' ages (i.e., last inspection year minus year built) and the corresponding percentage of the deck that was damaged.

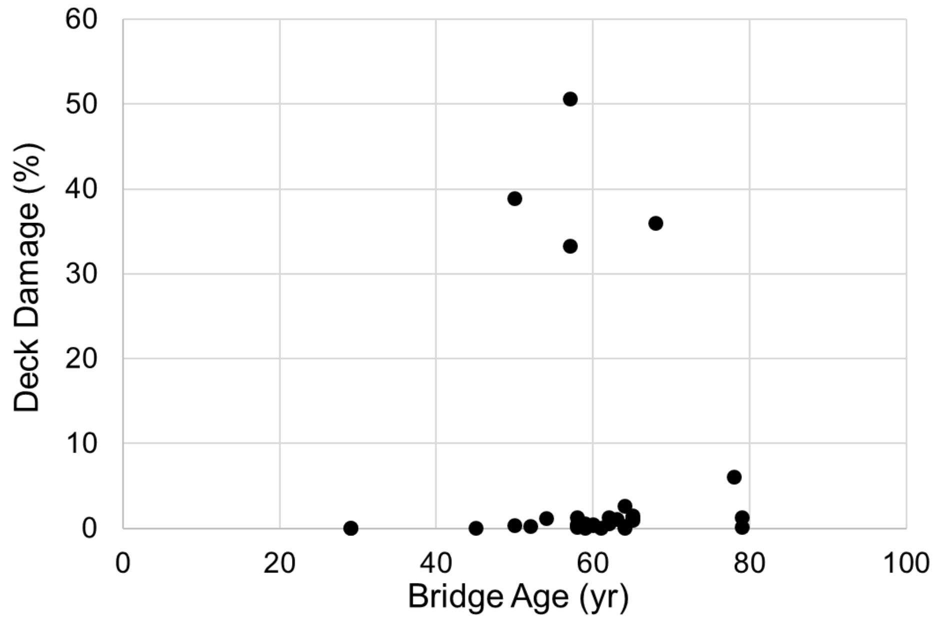

Figure 22. Bridge age and percent of the deck surface affected by spalls, delamination or patching for the 32 sampled decks.

From Figure 22, a very clear delineation in performance is found between the decks. Twenty eight of 32 bridges were well below the stated acceptable ten percent damage threshold. The remaining four decks exceeded 30 percent surface damage. This suggests that at least two populations of decks exist within this dataset. To aid in separating the decks based on similar environmental conditions and performance, the damage for each county was overlaid on top of the main topographical divides for the state [96].

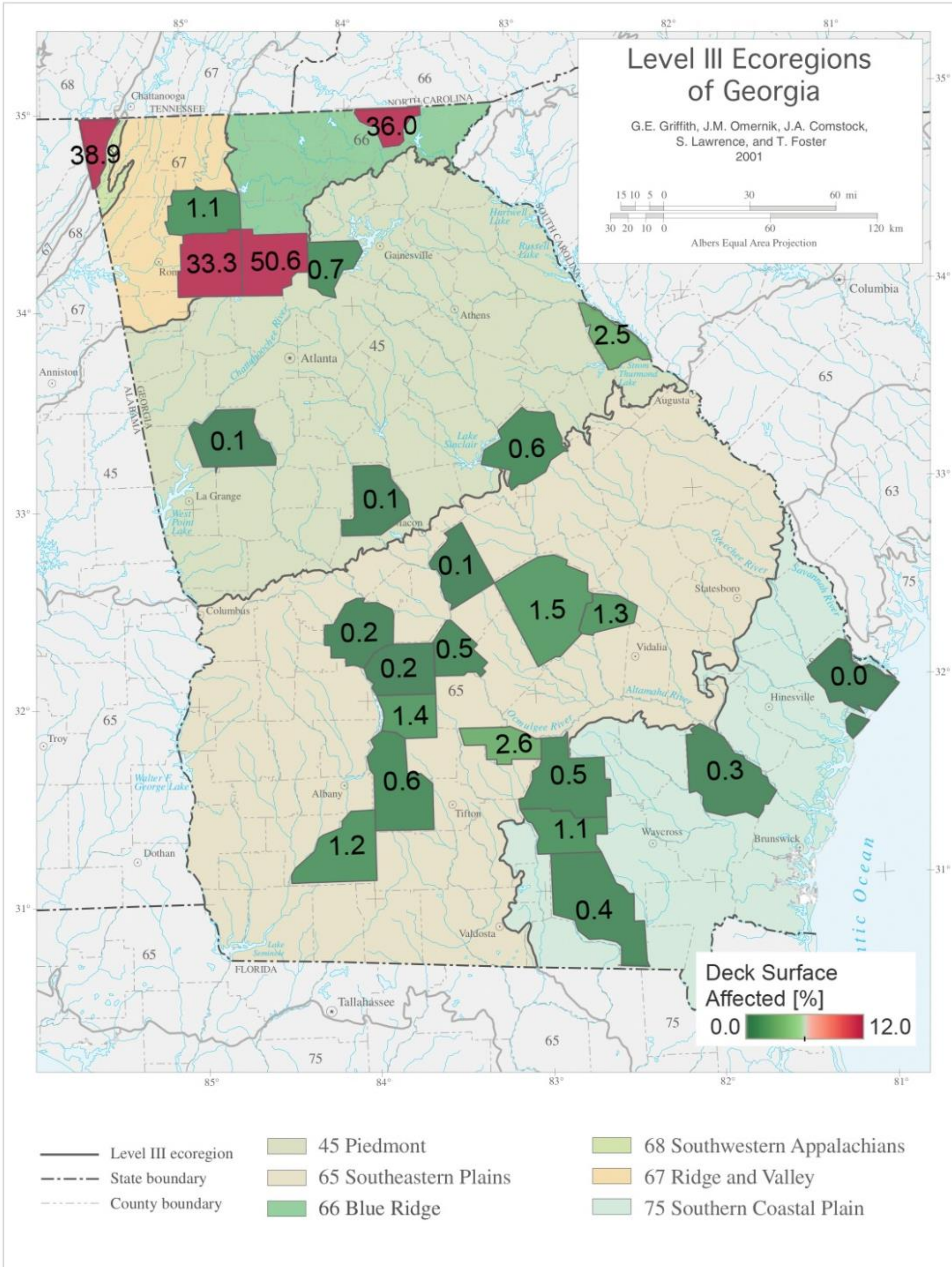

Figure 23. Map overlaying the deck damage by county.

As the specific latitude and longitude for the decks were unavailable, the bridge data was assigned per county. If multiple bridges were in the same county an average of the damage levels was reported. The damage level was then color coded with dark red indicating the most damage and dark green the least.

From Figure 23, the four locations with the most deck damage, despite being contemporary in age, were clustered in the northernmost portion of the state. Specifically they are in the areas identified as Southwestern Appalachians, Blue Ridge, and Ridge and Valley. This portion of the state may experience more de-icing activities, which may result in greater chloride exposure and more damage. These bridges may also experience freeze/thaw cycles, which may induce cracking and scaling, increasing diffusivity. As a result, the bridges from these areas were grouped into a population named “the Mountain region,” and consisted of a total of five decks. In the center of the map is a demarcation called the “fall line,” which separates the Piedmont and Southeastern Plains. It is current GDOT practice for bridges north of the fall line to require epoxy-coated rebar and a 2.75 inch cover, and those south to be plain low-carbon steel rebar with a 2.25 inch cover. This implies that north of the fall line is a more aggressive environment. For that reason, the remainder of the decks were split into two populations, one population called “the Central region,” which comprised of all decks north of the fall line but south of the Mountain region, and another call the “Southern region,” which consists of all decks south of the fall line.

Having separated the decks into three populations, the baseline values for all the key corrosion parameters for each region were extracted through regression. The cover, diffusivity, and surface chloride concentration were assumed to have Gaussian distributions, while the chloride threshold, propagation time, and crack fraction were fixed values. A broad range of acceptable values, representing the upper and lower bounds for all the parameters, was assigned based on literature values and convention. These ranges are provided in Table 16.

Table 16. Parameter ranges for regression.

Parameter	Units	Lower Bound	Upper Bound	Basis
Mean cover, X	in	1.0	2.5	Based on common range for the era of construction (1.5-2.0) +/- 0.5 inches
Cover standard deviation, X S.D.	in	0.05	1	Based on cover survey data in Chapter 5
Surface chloride Mean, C_s	lbs/yd^3	2	40	Based on values from [17, 29] for minimum up to practical maximum (full chloride saturation capacity of concrete)
Surface chloride standard deviation, C_s S.D.	lbs/yd^3	0.01	10	Sufficiently wide range to encompass all practical values
Diffusivity mean, D	in^2/yr	0.05	1	Based on findings in Chapter 2
Diffusivity standard deviation, D S.D.	in^2/yr	0.01	1	Extension of the range observed in [17]
Chloride threshold, C_t	lbs/yd^3	0.45	2.17	Extension of the range from Chapter 2 +/- 10%
Propagation time, t_p	yr	3	9	Extension of the range seen in [17]
Crack fraction, C_F	%	0	21	Based on the observed extremes in crack frequency in [17]

Having specified the acceptable ranges, the regression procedure was as follows. First 300 sets of uniformly random values (within the upper and lower bound) were created for each of the three regions which represent the key model parameters (e.g., cover, diffusivity). These sets of nine values are described as “seeds” from here onward. Next, the sum of squared errors between the output of Equation 9 with these seeds as the inputs, and the field damage for each region were calculated. The five seeds with the lowest initial sum of errors for each region were then selected. The purpose of this selection process is to increase the odds that the fitting algorithm, which is sensitive to selection of the initial seed, will converge. An interior point fitting algorithm [97] was applied using the selected seeds as the initial values, and the Table 16 values as the constraints, which attempted to achieve the lowest final sum of errors between the model’s prediction and the observed field damage. Following this transformation, one seed per region was selected as the baseline set of parameters, and the corresponding damage function as the baseline deck performance. The results from this process for each region are provided in Table 17 through Table 25 and Figure 24 through Figure 26. For all these tables, the same base English units were maintained. With exception of the cover (inch), t_p (yr), and crack fraction (%), the units are lb/yd^3.

The Mountain Region

Table 17. Initial seeds for the Mountain region.

Cover mean	Cover S.D.	C_s mean	C_s S.D.	D mean	D S.D.	C_t	t_p	Crack fraction
1.92	0.82	2.82	0.13	0.43	0.22	1.36	7.94	15.73
1.90	0.06	11.76	3.11	0.27	0.73	1.57	8.58	11.42
1.81	0.97	4.84	0.48	0.31	0.62	1.13	7.24	8.14
2.32	0.86	8.28	0.19	0.43	0.35	1.74	3.77	20.17
2.39	0.49	19.01	0.72	0.53	0.39	1.05	3.04	16.16

Table 18. Transformed seeds for the Mountain region.

Cover mean	Cover S.D.	C_s mean	C_s S.D.	D mean	D S.D.	C_t	t_p	Crack fraction	Sum of errors
2.24	0.71	2.38	0.50	0.05	0.27	1.74	7.14	20.97	1577.41
1.52	0.05	2.05	0.36	0.09	1.00	1.91	3.40	20.82	1472.88
2.50	0.97	3.15	0.71	0.12	0.15	2.04	7.31	7.05	1593.81
1.01	0.99	2.00	0.34	0.55	0.52	1.97	3.11	19.84	1426.57
1.60	0.12	2.30	0.41	0.53	1.00	2.17	3.02	20.93	1468.06

Table 19. Comparison of the transformed seeds in the Mountain region

Field Data		Seed 1	Seed 2	Seed 3	Seed 4	Seed 5
Age	**Deck Damage (%)**	**Deck Damage (%)**				
57	33.3	32.8	31.7	30.6	31.5	31.7
57	50.6	32.8	31.7	30.6	31.5	31.7
50	38.9	29.5	29.7	27.1	30.4	29.9
63	1.1	35.3	33.1	33.4	32.4	33.1
68	36.0	37.1	34.1	35.6	33.0	34.1

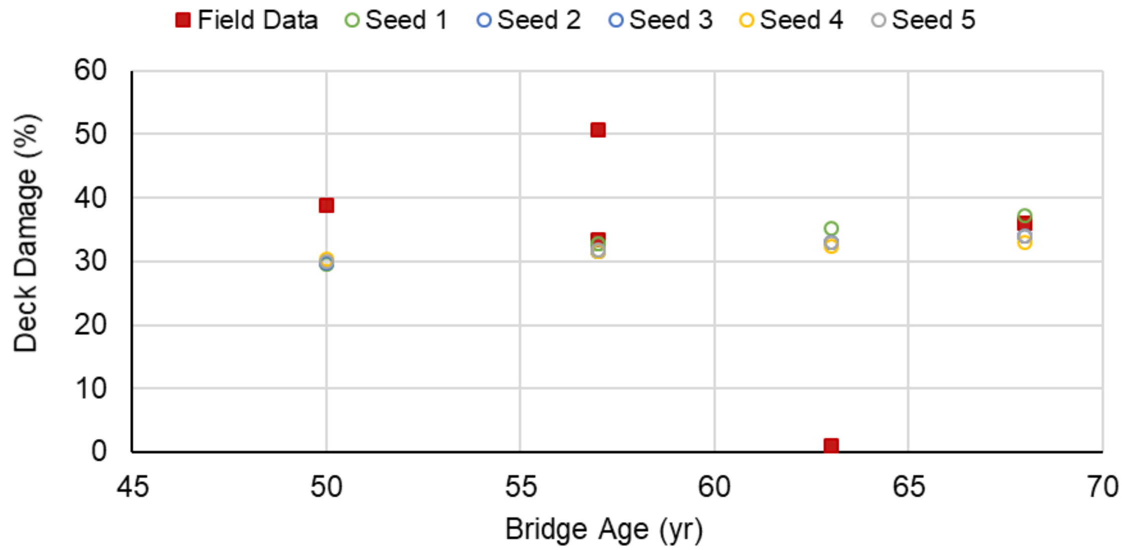

Figure 24. Graphical representation of the results in Table 14, showing the fitting of the model for each seed (colored circles) against the field data (red squares) Mountain.

Figure 24 shows that despite differences in their specific parameter values, the seeds yielded very similar fits to the field data. This is more pronounced in regions with fewer data points and more scatter. From the bridges' ages, it appears that most bridges in this population were built in the 1950s and 1960s. These two decades were a period where less cover, poorer quality concrete, and the use of plain low-carbon steel rebar was common practice. Considering these practices, Seeds 1 and 3 are improbable. It is unlikely that the average cover was 2.25 or 2.5 inch, and the diffusivity (0.05 and 0.12 in^2/yr) are likely too small given less SCM use and higher water-to-cement ratio concrete. Similarly, Seed 2 is improbable given the low diffusivity (0.09 in^2/yr) of the concrete, as well as the uncharacteristically low cracking fraction for its population (7 percent). Seeds 4 and 5 are the best fit, as represented by the lowest sums of errors, but differ in their description of the system. Seed 4 represents a deck with low and highly variable cover, low chloride exposure, ordinary to elevated permeability concrete, reinforcement with typical corrosion resistance, concrete which spalls readily after corrosion initiates, and substantial cracking. Seed 5 represents ordinary cover thickness with low variability, more chloride exposure, ordinary to elevated permeability concrete, rebar with elevated corrosion resistance, concrete which spalls readily after corrosion initiates, and maximal cracking. Considering these two seeds, Seed 5 best represents this population and will be used as the baseline parameters for this region.

The Central Region

Table 20. Initial seeds for the Central region.

Cover mean	Cover S.D.	C_s mean	C_s S.D.	D mean	D S.D.	C_t	t_p	Crack Fraction
1.89	0.39	7.34	1.29	0.08	0.41	1.93	8.34	12.60
2.30	0.31	27.31	0.76	0.10	0.87	1.98	8.89	18.86
2.28	0.19	18.84	0.18	0.14	0.68	2.07	5.78	20.13
2.39	0.46	25.23	0.77	0.07	0.14	0.70	5.86	8.63
2.39	0.21	12.87	0.05	0.46	0.39	2.16	4.58	4.32

Table 21. Transformed seeds for the Central region.

Cover mean	Cover S.D.	C_s mean	C_s S.D.	D mean	D S.D.	C_t	t_p	Crack Fraction	Sum of errors
2.39	0.06	3.32	0.03	0.06	0.02	1.72	7.06	0.65	0.40
2.27	0.10	3.01	0.09	0.10	0.02	1.85	5.86	0.51	0.40
2.21	0.11	2.63	0.06	0.19	0.03	1.88	6.13	0.49	0.40
2.23	0.11	2.39	0.02	0.31	0.02	2.04	6.38	17.53	0.54
2.19	0.11	2.39	0.03	0.41	0.05	1.92	5.53	0.48	0.40

Table 22. Comparison of the transformed seeds in the Central region.

Field Data		Seed 1	Seed 2	Seed 3	Seed 4	Seed 5
Age	Deck Damage (%)	Deck Damage (%)				
29	0.05	0.06	0.04	0.03	0.00	0.03
29	0.06	0.06	0.04	0.03	0.00	0.03
62	0.72	0.61	0.62	0.62	0.51	0.62
59	0.57	0.52	0.51	0.52	0.23	0.51
65	1.21	0.86	0.86	0.85	1.01	0.85
78	6.10	6.10	6.10	6.10	6.08	6.10
64	0.24	0.75	0.76	0.76	0.82	0.76

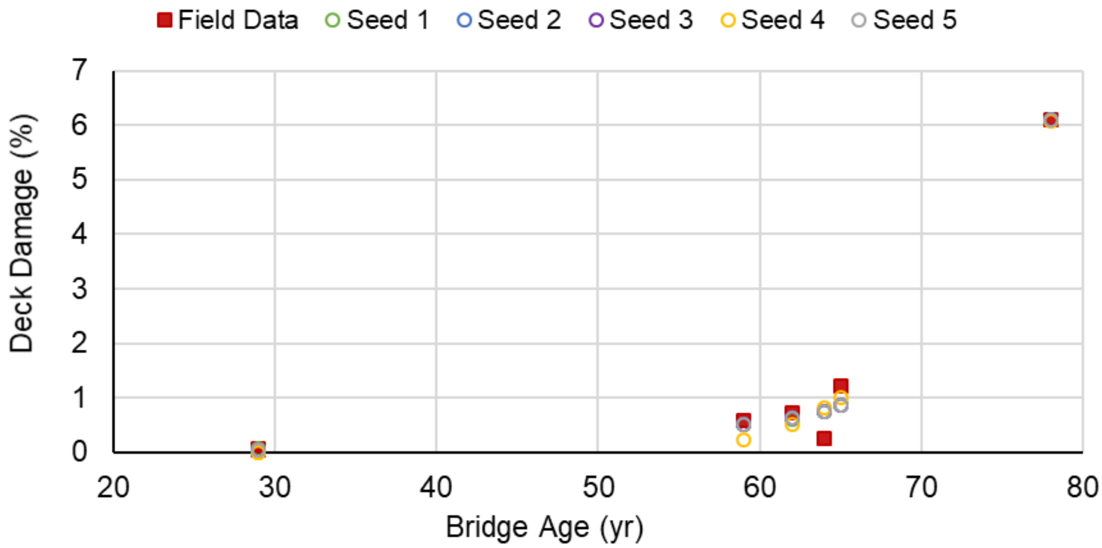

Figure 25. Graphical representation of the results in Table 17, showing the fitting of the model for each seed (colored circles) against the field data (red squares) in the Central region.

This region has few points, but little scatter, which resulted in a set of very similar seeds and a good visual fit of the model. Most of the decks in this population were built in the 1950s. The seeds collectively represent decks with higher than ordinary cover for their era with very low variability, low chloride exposure, very low to ordinary permeability concrete, normal reinforcement corrosion resistance, average resistance to spalling, and virtually no cracking (with the clear exception of Seed 4). Seed 1 had the best fit (i.e., lowest sum of errors) and was thus selected as the baseline set of parameters for this region, with the corresponding damage being the baseline deck performance for the Central region.

The Southern Region

Table 23. Initial seeds for the Southern region.

Cover mean	Cover S.D.	C_s mean	C_s S.D.	D mean	D S.D.	C_t	t_p	Crack Fraction
2.45	0.93	2.51	0.21	0.47	0.06	1.92	7.94	10.08
1.18	0.16	3.27	0.61	0.79	0.73	2.00	4.16	6.51
2.49	0.76	3.97	0.21	0.82	0.69	2.04	8.05	12.17
2.26	0.21	20.02	1.69	0.25	0.45	1.29	8.84	13.57
1.68	0.96	2.94	0.02	0.56	0.79	0.87	8.50	12.15

Table 24. Transformed seeds for the Southern region.

Cover mean	Cover S.D.	C_s mean	C_s S.D.	D mean	D S.D.	C_t	t_p	Crack Fraction	Sum of errors
2.50	0.19	2.60	0.01	0.46	0.01	2.17	3.01	1.02	7.66
2.10	0.59	2.10	0.24	0.14	0.08	2.09	6.19	11.24	8.03
2.07	0.54	2.07	0.14	0.23	0.48	2.10	7.32	16.85	8.01
1.46	0.38	2.01	0.13	0.92	0.89	2.13	8.56	0.90	8.00
1.93	0.54	2.10	0.10	0.33	0.43	2.08	6.29	11.80	8.03

Table 25. Comparison of the transformed seeds in the Southern region.

Field Data		Seed 1	Seed 2	Seed 3	Seed 4	Seed 5
Age	Deck Damage (%)	Deck Damage (%)				
65	1.06	0.89	0.79	0.79	0.79	0.78
64	2.63	0.87	0.76	0.77	0.77	0.76
64	0.03	0.87	0.76	0.77	0.77	0.76
60	0.35	0.73	0.68	0.68	0.68	0.68
60	0.46	0.73	0.68	0.68	0.68	0.68
61	0.08	0.77	0.70	0.70	0.70	0.70
62	1.33	0.81	0.72	0.72	0.73	0.72
59	0.08	0.69	0.66	0.66	0.66	0.66
79	1.37	1.02	1.10	1.09	1.09	1.09
58	0.16	0.65	0.64	0.64	0.64	0.64
65	1.50	0.89	0.79	0.79	0.79	0.78
79	0.20	1.02	1.10	1.09	1.09	1.09
54	1.17	0.45	0.56	0.56	0.56	0.56
58	0.54	0.65	0.64	0.64	0.64	0.64
58	0.41	0.65	0.64	0.64	0.64	0.64
58	1.28	0.65	0.64	0.64	0.64	0.64
45	0.10	0.09	0.39	0.40	0.38	0.40
52	0.25	0.35	0.52	0.53	0.52	0.53
50	0.37	0.26	0.48	0.49	0.48	0.49
62	0.57	0.81	0.72	0.72	0.73	0.72

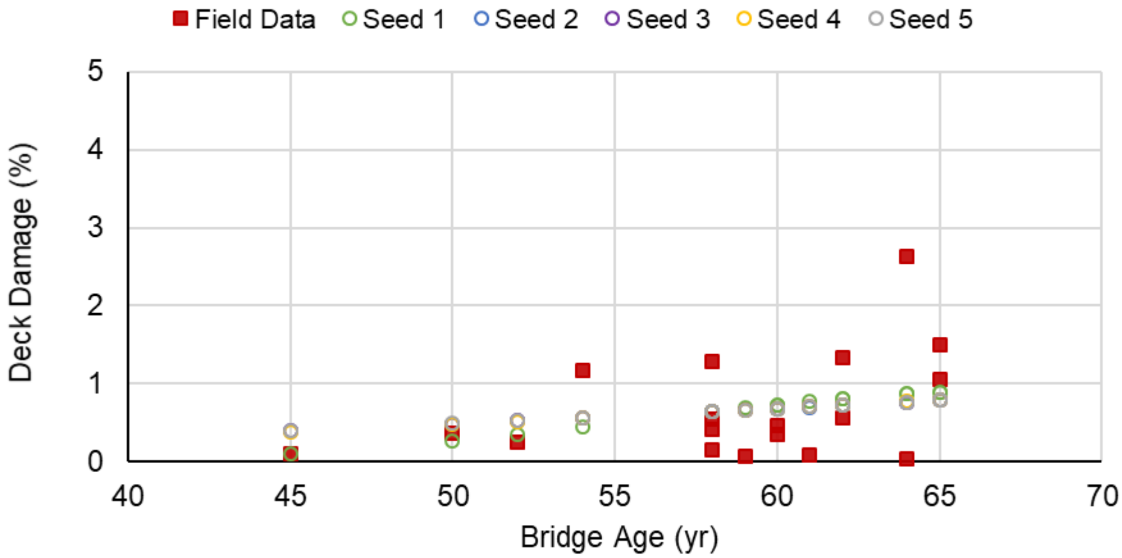

Figure 26. Graphical representation of the results in Table 20, showing the fitting of the model for each seed (colored circles) against the field data (red squares) in the Southern region.

Figure 26 shows significant scatter in the data, which led to a poorer fit. Despite the advanced age of some of the decks in this population there was little corresponding damage. Seed 1 has the thickest cover, an amount of cover that was not specified in this geographic region even in modern times. Seed 3 and 5 represent very similar decks, with ordinary concrete cover thickness, low chloride exposure, normal to elevated permeability concrete, normal to elevated rebar corrosion resistance, average spall resistance, and substantial cracking. Seeds 2 and 4 represent the two likeliest parameter sets for this region, with the main difference being a thicker cover and less permeable concrete with significant cracking in the former and thinner cover with more permeable concrete and virtually no cracking in the latter. Between these two, Seed 2 was selected to represent the Southern baseline set of parameters, because a 1.46 inch cover is likely too low for this region. Therefore the parameters for Seed were and the corresponding damage function representing the baseline deck performance in the Southern region.

6.3.4 The Baseline Values

Table 26 through Table 28 presents all the selected baseline values for the key model parameters for each of the three models. Having a dataset with more decks and information on the ranges for the modeling parameters would generate better fits and more confidence in the baselines for the models. However, as the work in the next sections is concerned primarily with relative changes to the predicted service life and damage curves, more so then forecasting with certainty the specific service life of the decks, the inaccuracy in the baseline values should be less consequential.

Table 26. Baseline parameters for the diffusion model.

Cover	Cs mean	D	C_t	t_p
2.75	8.31	0.05	1.97	5

Table 27. Baseline parameters for the probabilistic simulation.

Case	Cover mean	Cover S.D.	C_s mean	C_s S.D.	D mean	D S.D.	C_t	t_p
1	2.75	0.14	8.31	0.42	0.05	0.0025	1.97	5
2	2.75	0.14	8.31	0.42	0.05	0.0025	3.60	5
3	2.75	0.14	21.8	1.09	0.05	0.0025	1.97	5
4	2.75	0.14	21.8	1.09	0.05	0.0025	3.60	5

Table 28. Baseline parameters for the full probabilistic model.

Region	Cover mean	Cover S.D.	C_s mean	C_s S.D	D mean	D S.D.	C_t	t_p	C_F
Mountain	1.60	0.12	2.30	0.41	0.53	1.00	2.17	3.02	20.97
Central	2.39	0.06	3.32	0.03	0.06	0.02	1.72	7.06	0.65
Southern	2.10	0.59	2.10	0.24	0.14	0.08	2.09	6.19	11.24

For the work in the following sections, the diffusion and full probabilistic model were used to forecast the expected changes to service life from the changed construction practices and materials. The probabilistic simulation was used in the cover evaluations solely, where its use of two-dimensional cover surveys is advantageous.

6.4 Novel and Alternative Construction Materials

The literature review in Chapter 2 identified three broad categories of novel and alternative construction materials that may provide improvements to the service life of decks: improved mix designs, alternative reinforcement, and surface coatings. Improved mix designs and surface coatings result in lower diffusivity concrete and less chloride ingress. Alternative reinforcement, such as epoxy-coated rebar, increases the chloride threshold to prolong the service life of the decks. In general, improved mix designs and alternative rebar selection are strategies for new construction, whereas surface coatings are often applied to extend the lives of existing decks.

6.4.1 Modeling the Effects of Improved Mix Designs

As identified in the literature review, SCM use has the potential to prolong corrosion initiation by reducing the permeability and diffusivity of concrete [27]. SCM use is prescribed in modern GDOT bridge deck mix guidelines. The mix design guidelines for incorporating FA and BFS in bridge deck concrete are provided in Table 29 [98]. GDOT specifications state that FA may replace cement in the mix with a maximum 1.5:1 replacement by weight, and cannot exceed 15 percent total replacement [98]. Similarly, BFS may replace cement in the mix with a 1:1 replacement by weight, up to a limit of 50 percent replacement. To model the effect of improved-SCM based concrete mix designs may have, the pore-refining properties of the mixes were represented by a decrease in diffusivity for the concrete.

Table 29. GDOT guidelines for bridge deck mix proportions

Mix	**Min. Cement Factor (lb/yd^3)**	**SCM (lb/yd^3)**	**Stone Volume (ft^3/yd^3)**	**Max W/C Ratio**	**Design Slump (in)**	**Stone Size**	**Compressive Strength (lb/in^2)**
PC	611	0	11.2	0.490	3	56, 57, 67	3000
PC+FA	520	125	11.2	0.490	3	56, 57, 67	3000
PC+BFS	306	305	11.2	0.490	3	56, 57, 67	3000

It is important to note that the decrease in diffusivity represents an improvement in cover quality, because the modeling is solely describing chloride ingress through the cover concrete. That is to say, that while a low and uniform diffusivity of the concrete throughout the deck is desirable, the diffusivity of the cover concrete is of paramount importance in this modeling. The selected values for *D* for the SCM concretes were taken from the literature, and are presented in Table 26 [99].

Table 30. Selected D values for SCM concretes

Mix Designation	**FA Cement Replacement (%)**	**BFS Cement Replacement (%)**	**D (in^2/yr)**
15% FA	15	-	0.072
30% FA	30	-	0.058
50% FA	50	-	0.083
15% BFS	-	15	0.095
30% BFS	-	30	0.074
50% BFS	-	50	0.054

The results of the substitution of these values for *D* into the baseline parameters for each region using the full probabilistic model are given graphically in Figure 27, and numerically in Table 31 and Table 32. As the damage never exceeded the five percent damage threshold in the Southern region, the data for that region is presented graphically solely without an accompanying table. When analyzing Figure 27, attention should be directed to the points where each curve intersects with the five and ten percent damage limits. In general, a shift of a curve down or to the right represents a longer lifespan.

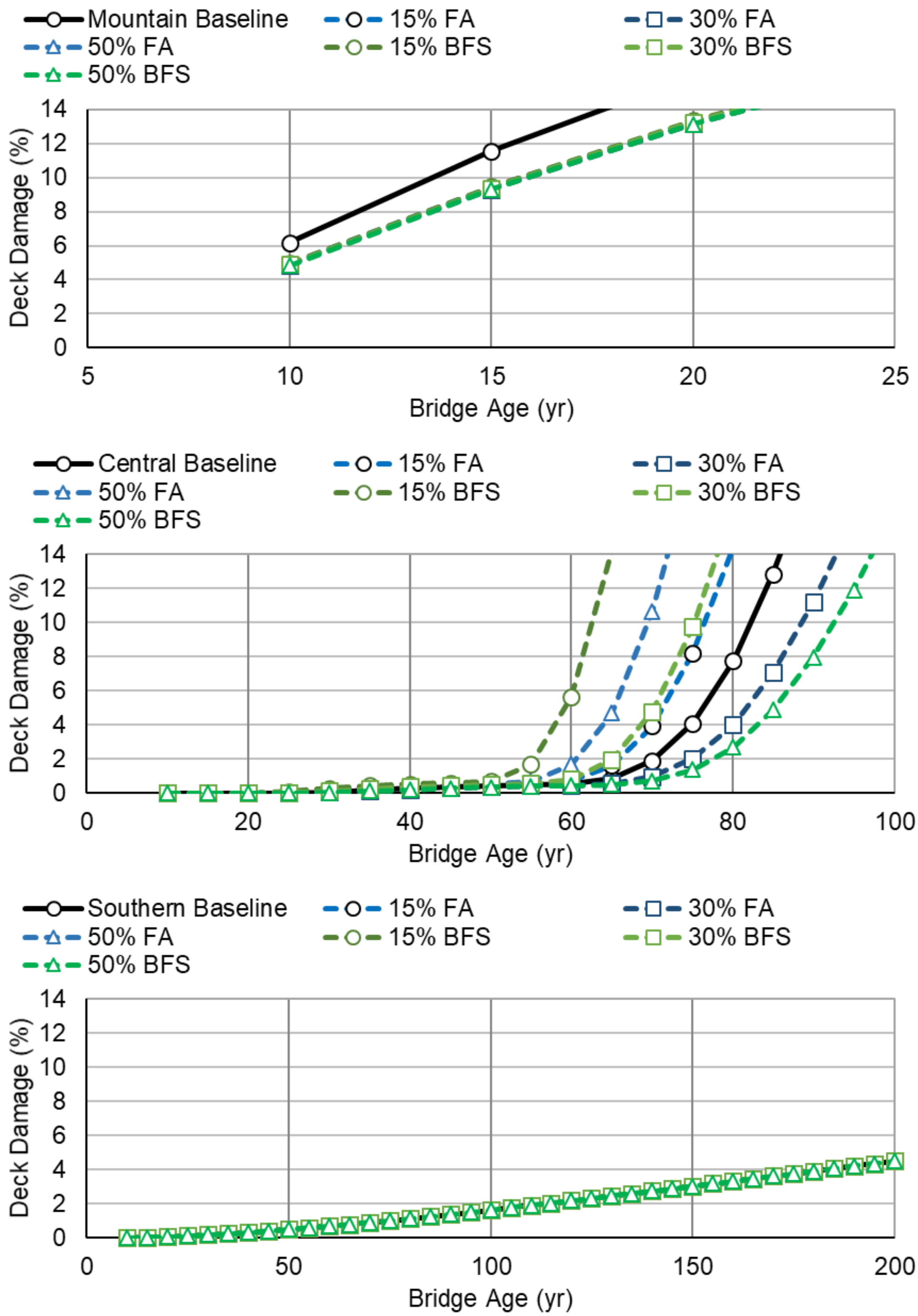

Figure 27. The damaged functions for the mix designs incorporating SCM for the Mountain (top), Central (middle), and Southern (bottom) regions.

Table 31. Predicted service life for each mix for decks in the Mountain region.

	Mix Composition						
Limit State	**Baseline**	**15% FA**	**30% FA**	**50% FA**	**15% BFS**	**30% BFS**	**50% BFS**
5% Damage	9.1	10.1	10.2	10.1	10.1	10.1	10.2
10% Damage	13.6	15.8	15.9	15.8	15.7	15.8	15.9

Table 32. Predicted service life for each mix for decks in the Central region.

	Mix Composition						
Limit State	**Baseline**	**15% FA**	**30% FA**	**50% FA**	**15% BFS**	**30% BFS**	**50% BFS**
5% Damage	76.2	71.2	81.6	65.3	59.2	70.2	85.2
10% Damage	82.2	76.4	88.6	69.4	62.5	75.2	92.6

The Mountain region has the most cracking (21 percent) of all regions, which negated most of the benefit from the less permeable concrete, in line with the assertion in [27]. In general, all the alternatives mix designs outperformed the baseline for the region. The largest increases to predicted service life were about two years for the range of mix composition evaluated. For bridges with similarly thin cover and significant cracking analogous to those of this region, marginal benefit is expected from the reduced permeability concrete of the SCM mixes. However, since SCMs mixes are often more economical than pure PC mixes and present better (even if only marginally so) service lives, their use may still be recommended even in these conditions.

The Central region presented the most variable performance among the mixes (1/3 of mixes outperformed baseline), due in part to its low C_f, higher C_s, and initial low D value (0.06 in^2/yr). This resulted in smaller magnitude changes to the service life of decks on the order of +/- ten years, with the 50 percent BFS mix projecting the largest change in service life. Using a large percentage replacement with slag results in slower early strength gain, which may be problematic in some deck construction. The mix with 30 percent FA had a longer expected life than the current 15 percent FA limit by about a decade, which suggests that elevating the current limit may be beneficial. The 50 percent FA mix projecting as the lowest service life for those containing FA suggests that the benefits from the pore-refining and filler properties of FA that remain when there is insufficient CH are maximized at lower replacement percentages. Were the cracking to be reduced, a large benefit would be expected from the order of magnitude decrease in the diffusivity.

The Southern region presents adequate performance regardless of the mix design used, despite moderate cracking, with all alternatives presenting near identical performance to the baseline. This can be attributed to the low C_s and high C_t, a consequence of the field data from this region showing little damage despite a long period in service. For bridges with similar characteristics, there is little predicted benefit to adding SCM concrete, though as acknowledged previously the may still be recommended due to their lower cost.

The diffusion model provides similar results when replacing the D value with those from Table 30, with the projected service lives near the baseline value. This was due to the low initial D value, 0.05 in^2/yr, which was not considerably reduced by the SCM mixes (min 0.054, max 0.095 in^2/yr).

6.5 Improved Construction Practices

All major corrosion parameters can be affected by construction practices. For this section, cover and cracking control will be the focus. Improved cover control manifests in an average cover nearer the design cover for the deck (i.e., cover accuracy), and a lower cover standard deviation (i.e., cover variability). Improved crack control results in less cracking and can be accomplished in a variety of ways (e.g., better curing practices) [59]. Both of these improvements will be modeled in the following sections.

6.5.1 *Improved Cover Accuracy*

The two aspects of cover control (cover accuracy and cover variability) were examined separately, to gauge their effects on expected service life. For cover accuracy, both the diffusion modeling and the probabilistic simulation showed a change of approximately two year service life increase per every 0.1 inch of additional cover (with 2.75 inch design cover as the reference). For the diffusion model this amount was calculated by averaging the predicted service life at each cover thickness in Figure 15, and then calculating the relative difference along the cover range (average: 2.4 years, min: 1.6, max: 3.3 years).

For the probabilistic simulation, the fitted equations in Figure 21 were used to determine the average expected change in service life by cover thickness. The results align with the diffusion model results, with an average of approximately two years of service life extension per 0.1 inch cover deviation from the design cover. Furthermore, with thicker covers the slope of the curves in Figure 21 approach grade, suggesting that the gains of additional cover diminish more readily than the losses expected from thinner cover. While minor variations of a couple years on the service life may not be significant for any one bridge, over the entire inventory small losses may aggregate to a significant effect.

The full probabilistic model was also used to model the effects of improved cover accuracy using a similar method as the previous section. While all other parameters remained constant in each region, the cover mean was set between 2.25 and 2.85 inch in 0.2 inch increments, and the corresponding damage functions were generated. This range was selected as it envelopes the modern 2.25 and 2.75 inch Georgia top mat cover specifications, as well as the other common cover thickness: 2.5 inch. The results from this modeling are presented in Figure 28 and Table 33. In Figure 28, the baseline for each region was designated by a black line, while the others were represented by colored dashed lines. Each curve represents the expected damage over time for the specified covers. As expected, the service life increased with thicker covers (seen through transitions to the lower right in the curves), but the magnitude of the effect was dependent on which region was modeled. In Table 33, the Southern region was omitted since the damage remains under the limit states under all conditions.

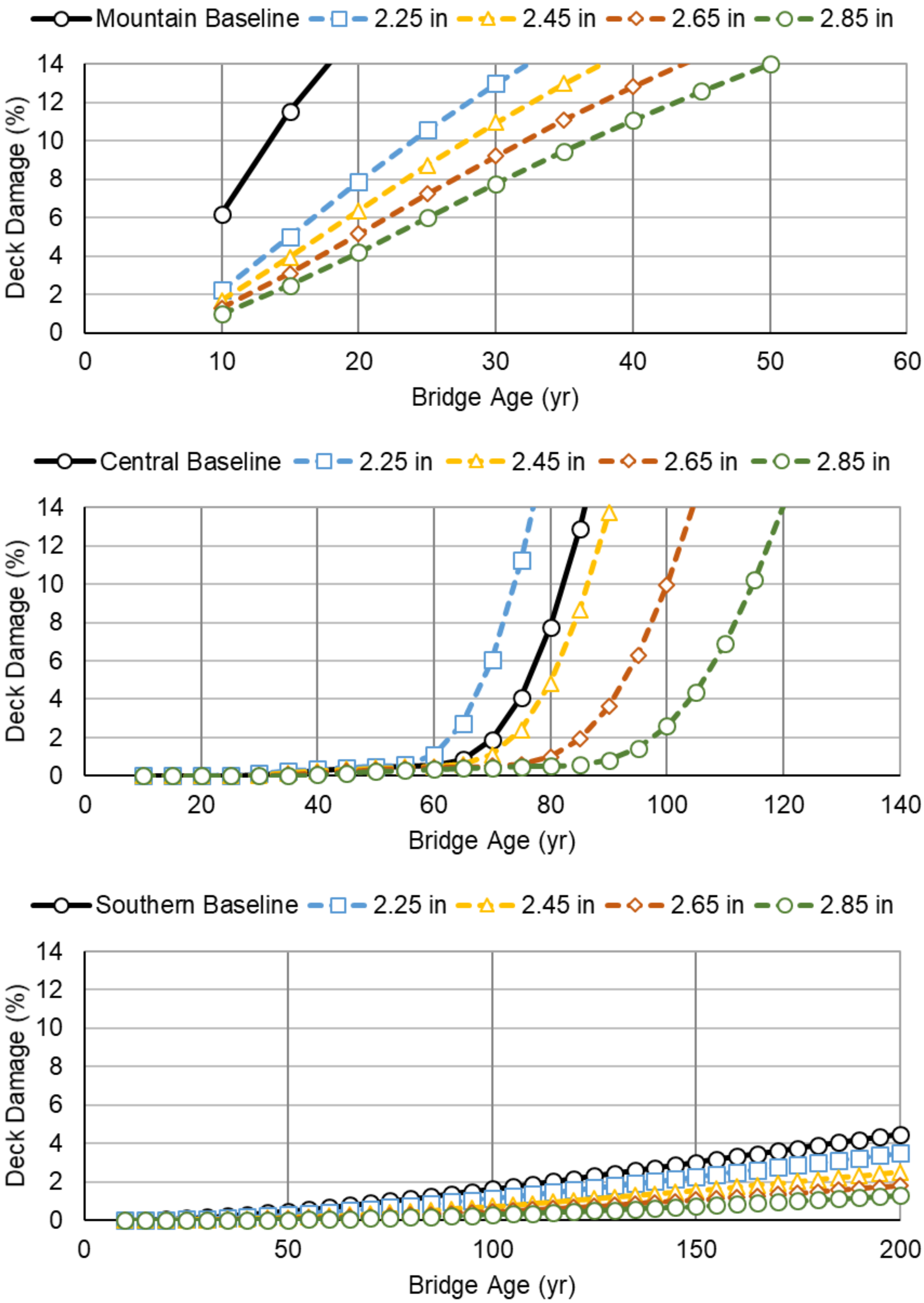

Figure 28. Effects of varying the average cover thickness on the expected service life. Mountain region (top) Central region (middle) Southern region (bottom).

Table 33. Predicted service lives with changing deck cover thickness.

	Mountain Region					**Central Region**				
Limit State	**Base -line**	**2.25 in**	**2.45 in**	**2.65 in**	**2.85 in**	**Base-line**	**2.25 in**	**2.45 in**	**2.65 in**	**2.85 in**
5% Damage	9.1	15.0	17.2	19.6	22.3	76.2	68.4	80.2	92.6	106.2
10% Damage	13.6	23.9	27.9	27.9	36.7	82.2	73.8	86.4	100.1	114.7

From Table 33, the average change in expected service life per 0.1 inch is 1.2 and 2.1 years for the five and ten percent damage states in the Mountain region, and 6.3 and 6.8 years for the five and ten percent damages in the Central region. As noted earlier, the Southern region remained below the damage limit states along the entire 200 year modeling period irrespective of the cover. The results from the Mountain region are thus in line with those from the probabilistic simulation modeling, while the Central region experienced a more pronounced effect from altering the cover. This is likely the result of the lower cracking fraction in the Central region, coupled with more chloride exposure and lower diffusivity concrete. While increasing the cover in the lower region did result in a longer service life, the effects are inconsequential in the critical 50-100 year timescale.

The next section will model the effects of improving the cover variability, by altering the cover standard deviation and measuring the expected changes in service life.

6.5.2 Decreased Cover Variability

This section will model the effects of changes to the cover variability. The expectation is that reducing the cover variability will result in a corresponding increase in service life. For this investigation, the full probabilistic model will be utilized solely, as the diffusional model does not feature the cover variability, and the probabilistic simulation is expected to provide similar results (in terms of trends) to the full probabilistic model, but without the important consideration of cracking.

To evaluate the effect of changing cover variability the cover standard deviation (Cover S.D.) parameter will be varied between 0.05 and 0.65 inch in 0.2 inch increments. These values represent a suitable range of cover variation based on the cover survey results in Chapter 5, where the average standard deviations of the deck were around 0.15 inch. The results from this modeling for the full probabilistic model is presented in Table 34 and Figure 29 below. Figure 29 shows a clear transition of the curves to the upper left, representing less expected service life, for decks with more cover variability. The Southern region was omitted from Table 34 as it remained below the five and ten percent limits.

Table 34. Predicted service lives with changing deck cover variability.

	Mountain Region					**Central Region**				
Limit State	**Base-line**	**0.05 in**	**0.25 in**	**0.45 in**	**0.65 in**	**Base-line**	**0.05 in**	**0.25 in**	**0.45 in**	**0.65 in**
5% Damage	9.1	9.0	8.6	7.1	4.6	76.2	76.4	66.7	51.6	36.8
10% Damage	13.6	13.7	13.2	11.9	9.9	82.2	82.3	74.8	62.2	49.0

From the results, the cover variability has a significant effect on the service lives of the decks. As expected, as cover variability decreases, service life increase. For the Mountain and Central regions, the baseline outperformed the service lives for the alternatives due to their low initial cover variability. In the Southern region, the complement is true. For the Mountain region, for each 0.1 inch increase in variability, the corresponding average decrease in expected service life was 0.7 and 0.6 years for five and ten percent limit states respectively. For the Central region, for each 0.1 inch increase in variability, the corresponding average decrease in expected service life was 6.6 and 5.6 years for five and ten percent limit states respectively. Comparing these results with those from the cover accuracy section, it appears that both the cover accuracy and cover variability had very similar effects on the expected service life irrespective of region. This suggests that

DOTs should consider both the cover accuracy and cover variability in about equal measure, and amend codes and contracts to control the cover variability as well. Chapter 7 provides an example of a provision that incentivizes both improved cover accuracy and variability.

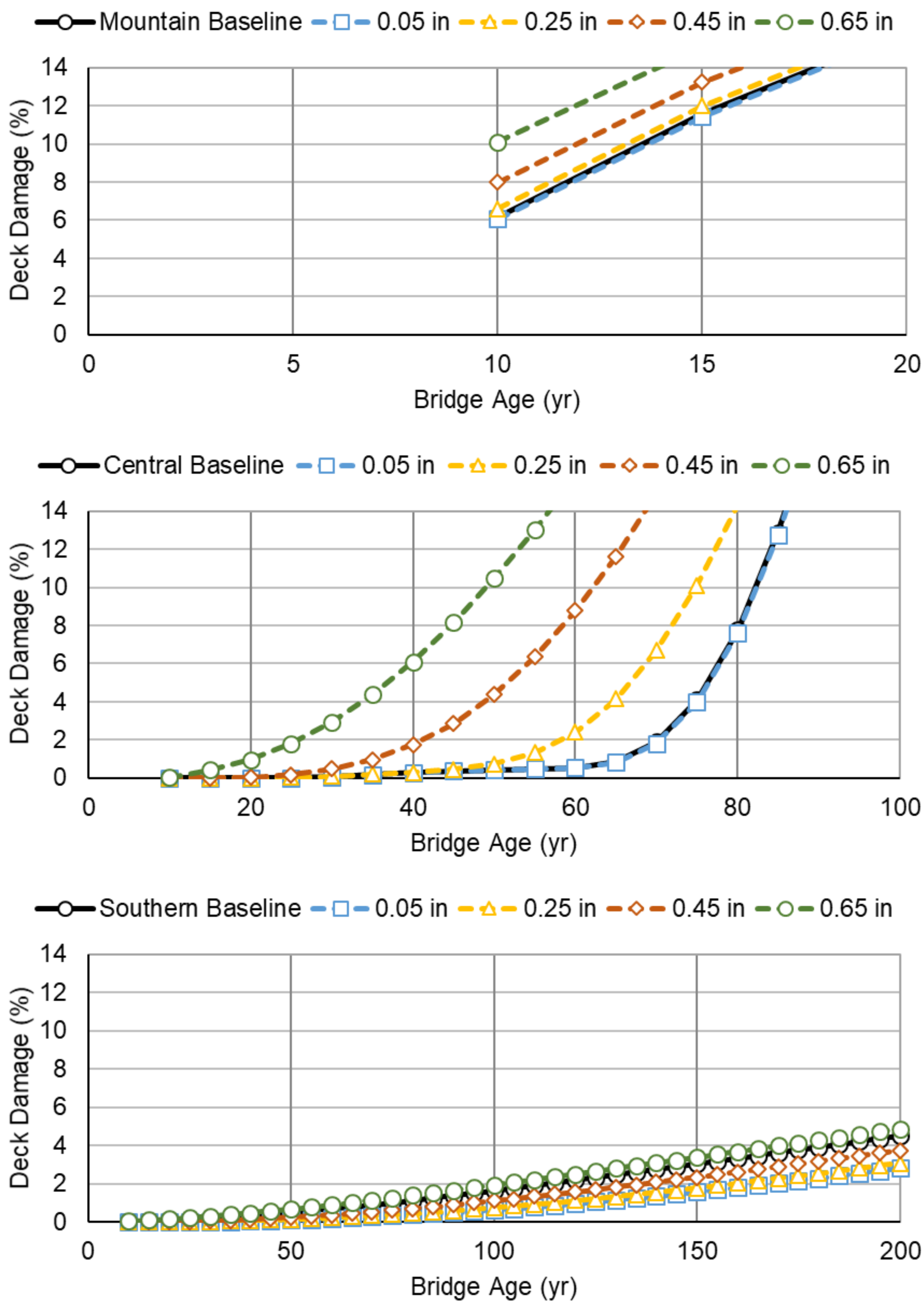

Figure 29. Effects of varying the average cover thickness on the expected service life. Mountain region (top) Central region (middle) Southern region (bottom).

6.5.3 *Modeling the Effects of Cracking*

The last major construction practice that is modeled is the effects of crack control on deck service life. From the modeling in previous sections it is clear that the cracking along the deck, represented by the crack fraction, can be responsible for a large deviation in performance between decks. In extreme cases, cracking may completely negate the expected benefits from thicker cover or less permeable concrete. To simulate the effects of cracking on the expected service life of decks, the cracking factor was varied between zero percent and 18 percent in six percent intervals for all three regions for the probabilistic model, which is the only model capable of modeling cracking. The results are presented in Table 35 and Figure 30. Since the Southern region remained below the five percent and ten percent limit states, it was omitted from Table 35.

Table 35. Predicted service lives with changing deck cracking.

	Mountain Region					**Central Region**				
Limit State	**Base line**	**0%**	**6%**	**12%**	**18%**	**Base line**	**0%**	**6%**	**12%**	**18%**
5% Damage	9.1	12.3	11.2	10.2	9.6	76.2	77.0	62.1	39.1	35.0
10% Damage	13.6	17.8	16.5	15.2	14.1	82.2	82.7	76.9	63.2	44.2

The expected service life for the Mountain region does not appear to be affected significant by the crack fraction (in magnitude not percentage, all alternatives outperform the baseline), likely due to its low D and t_p, which limit the improvements from decreasing the cracking fraction. In contrast the performance of the Central region was significantly affected. The Central region experienced an average drop of about 2 years per every 1% increase in the crack fraction for both the five and ten percent limit states. Considering the range of the crack fraction (zero to 21 percent), this corresponds to a maximum loss of 42 years of expected service life.

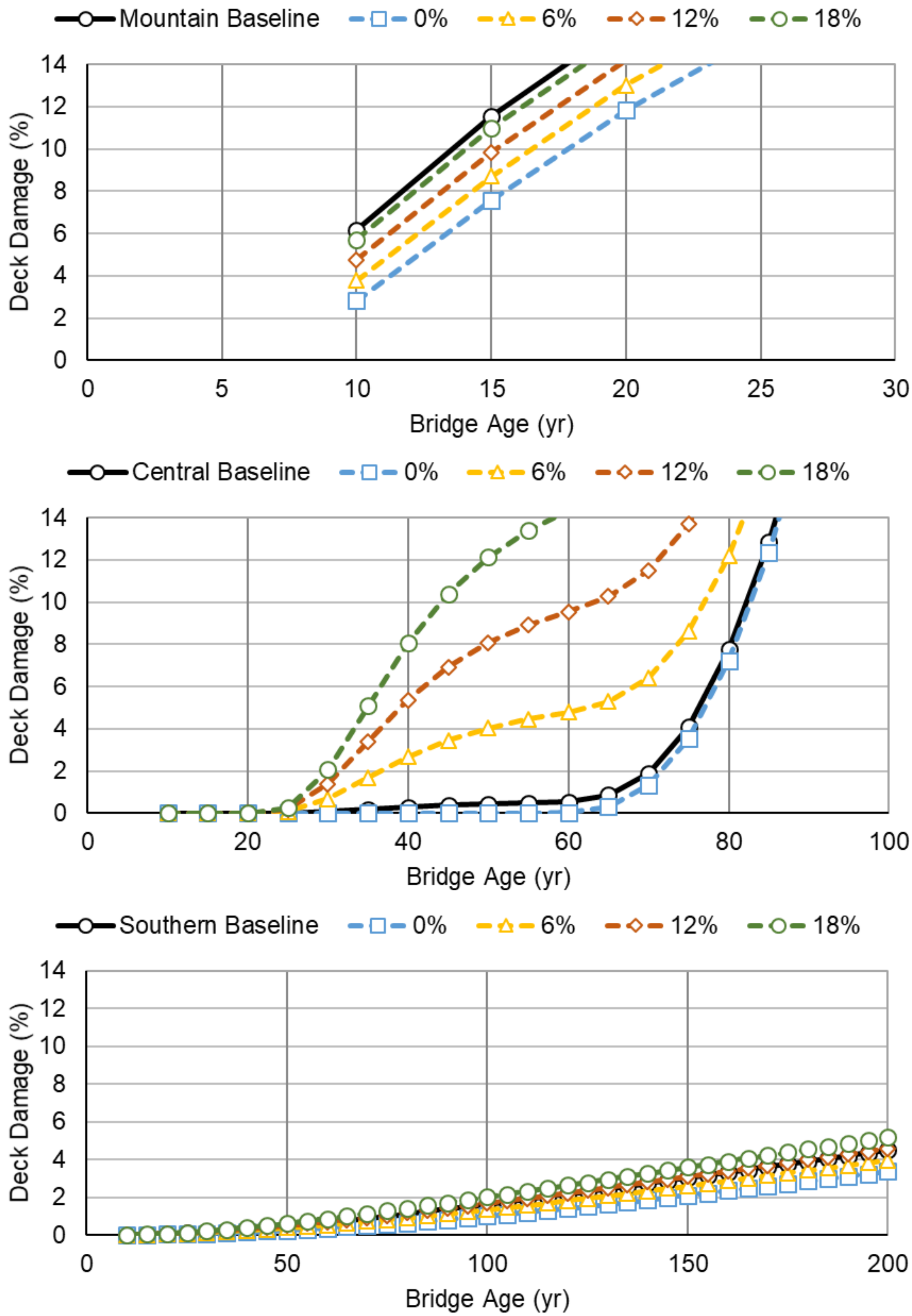

Figure 30. Effects of varying the crack fraction on the expected service life. Mountain region (top) Central region (middle) Southern region (bottom).

Unlike cover control, where the cover depth and variability can be directly controlled, ensuring a specific range for the crack fraction is much more complex. As noted prior, the crack fraction can be influenced by many factors, such as the ambient temperature during the deck pour, the subsequent curing regime, the structural loading, etc. Therefore, while reducing the crack fraction is desirable, it may prove difficult in practice to achieve.

6.6 Novel and Alternative Materials

6.6.1 Modeling the Use of Alternative Reinforcement

As noted in Chapter 2, there are a variety of alternative reinforcements available for use in bridge decks, such as epoxy-coated rebar, multiple forms of metallic clad rebar, metallic composite rebar, and polymer based rebar. For this work, four rebar types will be evaluated: epoxy-coated rebar (ECR), dual phase ferritic-martensic steel (DP), stainless steel rebar (SS), and stainless clad rebar (SCR). GDOT currently utilizes both plain low-carbon steel rebar and ECR for their decks.

The expected service lives of a decks constructed with ECR, DP, SS, and SCR were simulated using the diffusion model. The diffusion model was used because the extremely long service lives projected for decks built with these rebar types remained below the damage limit states for both the probabilistic simulation as well as the full probabilistic model. The use of these rebars was simulated by adjusting the chloride threshold from the 1.97 lb/yd^3 baseline to the literature values provided in Table 36 below. When generating Table 36, in some cases a reference provided the expected C_t as a multiple of that of plain low-carbon steel rebar, which were then converted based on the baseline C_t.

Table 36. Chloride threshold and modeling results for the use of alternative rebar.

Rebar Type	C_t Range	C_t Selected	Predicted Service Life	References
Low-carbon steel	1.27-1.97	1.97	59	[20, 40, 42]
Epoxy-coated	1.97-3.94	2.96	94	[21, 29]
Dual-phase	3.5-9.2	6.34	836	[29, 32]
Stainless and stainless clad	12-33	19.7	1000+	[21, 30]

From Table 36 it is apparent that the use of any alternative reinforcement which achieves the corresponding chloride threshold would in essence inhibit chloride-induced corrosion degradation for 100 years or more. In the extreme cases such as the 1000+ year projected service life for stainless steel rebar construction, the service life of the deck would likely be controlled by other mechanisms (e.g., abrasion) well before that time. Therefore, any service life substantially above 100 years should be considered with skepticism.

6.6.2 Surface Coatings

As described in the literature review, the application of a coating to the deck surface is a frequent practice to extend the life of the deck. The coating is either applied shortly after construction, or when the deck has minor damage. It prevents further ingress of chloride from the environment, in particular through cracks, by forming an impermeable (or less permeable) barrier.

Current GDOT Specifications allow for multiple bridge deck surface coatings. The first, described in Section 533, is a bridge deck waterproofing membrane overlaid with asphaltic concrete [100]. This process is used on new construction to prevent chloride and water penetration into the deck, with the asphaltic concrete serving the additional function of a protective wearing surface. In a survey of DOTs in the U.S. and Canada, asphalt overlays with membranes were described as providing an average of 17 years of life [4].

The second surface coating, described in Section 519, is a PC overlay applied after hydrodemolishing the existing deck. The hydrodemolition removes the deck concrete to a depth of at least 0.5 inch below the top mat reinforcement. Next, the deteriorated concrete is removed and the reinforcement is cleaned and repaired. Finally, a PC overlay is poured to a minimum thickness of 3.875 inch, resulting in at least 2.25 inch of cover [100]. This process, in essence, creates a new deck from the top mat of rebar to the surface, which would in theory result in a doubling (in total) of the service life of the deck.

The final coating, also described in Section 519, is the application of a two-part polymer bridge deck overlay [100]. The purpose of the overlay is to provide complete deck waterproofing (inhibiting chloride ingress) and a non-skid surface wear-resistant surface. It appears that this overlay can be used to address low cover in new decks or to repair damage in existing decks. The two-part polymer is applied in layers intermixed with aggregate, until a minimum thickness of 0.375 inch. Unlike the other two surface coatings, the specification require that the contractor guarantee the wearing surface against all defects for a period of ten years. This warranty is around 60 percent of the average lifespan of 17.5 years for polymer overlays [4].

A concern with modeling the use of surface coatings is that their composition and thickness is not specified, which can significantly affect performance. GDOT provides tolerances, performance targets, and oversight, but the material is left to the contractor. Furthermore, as discussed in Chapter 2, the reliability of the coating governs the performance as well. If cracks form, the benefits of the surface coating may be entirely moot.

The use of surface coatings could be modeled by decreasing the diffusivity of the concrete, reducing the cracking fraction, or extending the propagation period. Due to the concerns identified prior, and since these effects, with the exception of extending the propagation time have already been modeled, they will not be replicated here. Instead, the surface coatings will be assumed to extend the life of the deck by 17 years, which would result in an expected service life of 76 years (based on the 59 year initial life from the diffusion model).

6.7 Chapter Summary

The expected service life for decks in Georgia was modeled under a variety of conditions based on both literature values, and the damage observed from decks in service. These data sources were used to establishing a baseline set of key corrosion parameters and performance for the three models developed. The changes to expected service life from improved mix designs, cover control, crack control, and surface coatings was modeled. The improvement to service life is from using alternative reinforcement, but utilizing SCM concrete, reducing the crack fraction, improving cover control, and applying surface coatings all positively affected the projected service lives as well. These results may be implemented in codes and contractual mechanisms, which promote these practices and materials.

CHAPTER 7

CONTRACTUAL RECOMMENDATIONS

7.1 Chapter Overview

This chapter presents a sample contractual implementation of the service life modeling of Chapter 6 in the context of top mat cover investigations of Chapter 5. The approach presented in this chapter, namely the creation and evaluation of an adjustable payment provision for cover, can be emulated for many other similar quality assurance problems (e.g., ensuring compressive strength of concrete). As discussed in Chapter 2, the adjustable payment plan approach detailed here has been successfully applied by DOTs in multiple states and, at first glance, appears to pass legal muster in Georgia.

7.2 Adjustable Payment Plan Draft Specification

This section demonstrates the process for creating an adjustable payment plan. The adjustable payment plan will then be explored as a means to motivate and ensure bridge deck top mat cover control in Georgia. An adjustable payment plan was selected over an acceptance plan because of the incremental response to deficiency versus a binary decision (i.e., pass/fail), which is advantageous in the case of cover control because the cover deficiencies vary in severity. In cases where an acceptance plan is better suited, the reader is referred to the acceptance plan approach in AASHTO R9-05 [101], as a reference.

For the remainder of this chapter, an adjustable payment plan will be considered and analyzed for a hypothetical reinforced concrete Georgia bridge deck with a design cover of 2.75 inch. The bridge deck is assumed to utilize the current construction practices [88]. These two constraints serve important functions. The design cover selected will serve as the target value, which the specification will incentivize adherence. Consequently, the adjustable payment plan needs to be modified for each design cover specified, which for Georgia is limited to 2.75 and 2.25 inch. The second constraint, the utilization of current construction practices, is important because the adjustable payment plan requires an understanding of the natural variability of the process being controlled. For this case, that natural variability can be thought of as the ordinary spread in cover achieved by contractors using current techniques. With more precise technology or processes, this natural variability would, in turn, decrease. To assess this natural variability, the historical cover surveys in Chapter 5 are utilized.

The process that is followed for creating the adjustable payment plan is outlined in AAHSTO Specification R9-05 [101]. The specification requires the designation of a few key parameters. The first parameter that was defined is the acceptable quality limit (AQL). It is recommended to set the AQL equal to the target value in processes with a stipulated target value (e.g., cover) [101]. For this example, the AQL is set to the design cover of 2.75 inch. From examining the data from the 36 spans in Chapter 5, the average difference between the average cover of the span and the design cover was -0.07 inch, which indicates that the contractors were able to target the design cover, in most cases.

The next parameter defined is the percent within limits (PWL), which refers to the percentage of the true cover distribution of the lot (i.e., area being evaluated for payment) that is within the specifications. The specification recommends that 90 PWL be used. Depending on the severity of the impacts of deviations from the limits, the PWL can be more or less restrictive. For cover control, based on the modeling in Chapter 6, 90 PWL should be sufficient for this process. For clarity, a 90 PWL means that if the sample were to have the target value as the mean and a given specified standard deviation for the process, then 90 percent of the true cover distribution, not necessarily the sampled distribution, would be within the specified limits.

The next parameter that was determined is the lot size, which is the area being evaluated for compliance. The lot size should balance both the accuracy and labor required for compliance testing. For this case, options for lot size include the whole deck, each span independently, or portions of a span (e.g., half or quarter spans). A good balance between accuracy and labor requirements is achieved when selecting the lot size as a bridge deck span. Each span serves as a clear unit of construction (likely with consistent quality), which can be evaluated independently with less labor than the deck as a whole.

The combined standard deviation is selected next. The combined standard deviation considers both the natural variability in cover control (termed process standard deviation, $\sigma_{process}$) as well as how accurately the contractors can match the mean of the delivered cover to the design cover (called center standard deviation, σ_{center}). The combination of the process and center standard deviations is done in accordance with Equation 10.

$$\sigma_{\mathrm{combined}} = \sqrt{\sigma_{\mathrm{center}}{}^2 + \sigma_{\mathrm{process}}{}^2} \qquad \text{Eq. 10}$$

Both the center and the process standard deviations were determined using the 36 spans from Chapter 5. The center standard deviation was calculated by computing the standard deviation of the average minus the design cover (2.75 inch) for all 36 spans. The value was found to be 0.143 inch. To estimate the process standard deviation, the standard deviations from all 36 spans were averaged, which resulted in a value of 0.041 inch. Using Equation 10, the combined standard deviation was found to be 0.15 inch.

The combined standard deviation and the PWL allow for calculating the limits for the specification. For this example, the mean is set as the design cover and the standard deviation is 0.15 inch. For a PWL of 90, the lower specification limit (LSL) is 2.51 inch and the upper specification limit (USL) is 3.00 inch.

Next, the rejection quality level (RQL) is specified. The RQL represents the percentage of samples that would be within the specification limits if the mean of the sampling was the rejection threshold and the standard deviation was 0.15 inch. The specification states that RQLs are generally between 70 and 30 PWL. The GDOT Standard Specifications [100] allow for an up to 0.25 inch deviation from the plans in cover pre-grinding and 0.5 inch deviation post-grinding. It has been noted previously that the cover measurements are taken pre-grinding when confirming concrete cover, so the 0.25 inch is considered the RQL threshold. Therefore, the percent of the distribution if the means are 2.50 and 3.00 inch was determined. Figure 31 below represents these limits graphically. The area of interest is the area of each outside distributions falling within the LSL and USL.

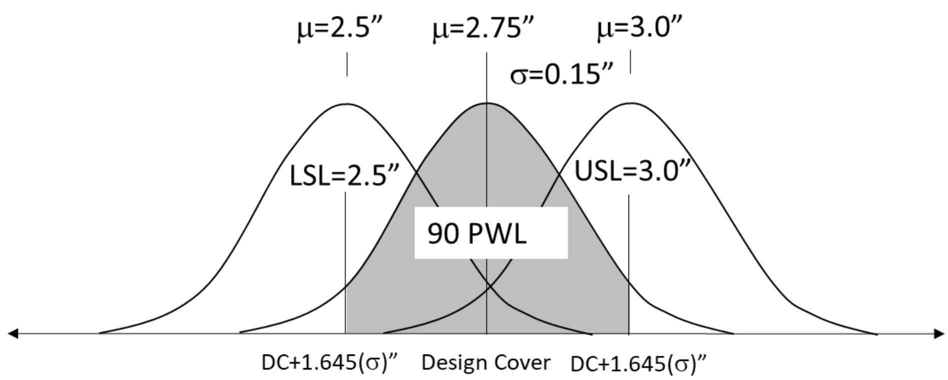

Figure 31. Specification limits for cover.

To calculate the area, a Z table is employed using the Z value for the LSL and USL in relation to the two exterior distributions. The area of each exterior distribution within the LSL and USL is 50%, therefore the RQL is 50. The specification states that RQLs are generally between 70 and 30 PWL, so the chosen RQL is within the normal range.

Next, two important parameters were established: the number of random samples per lot, *n*, and the form of the pay equation. When selecting *n*, the primary concern is the risk that the sampling of the lot fails to approximate the true cover distribution, and by extension, the payment for the work will not reflect the delivered quality. As the number of samples increases, the corresponding risk of having an unrepresentative sample decreases, but concurrently the time and expense for the sampling increases. In practice, a balance between the two interests is struck, with a normal random sampling range in highway construction and materials acceptance plans being between three and seven samples [102].

ASTM E112-17 prescribes a means of calculating the sample size needed to estimate the average of a parameter within a desired precision [103]. The relevant formulation is given in Equation 11. It is important to note that the equation assumes the true distribution is normal, or approximately normal, which was demonstrated in Chapter 5.

$$\mathrm{n} = \left(\frac{3\sigma_{\mathrm{o}}}{\mathrm{E}}\right)^2 \qquad \text{Eq. 11}$$

In Equation 11, n is the sample size, σ_o is an advance estimate of the standard deviation, and E is the maximum acceptable deviation between the true average and the sample average. If σ_o is assumed to be the 0.15 inch, as determined through the process outlined previously in this section, and E is selected as 0.1 inch, then n is approximately 20. It is important to note that E is the maximum acceptable deviation, and that even if 0.1 inch is selected, the observed deviations will generally be some value smaller. Interestingly, if the value for E is 0.2 inch instead, then n drops to approximately 5, which is within the ordinary range of 3 to 7. For this application, considering the narrow range of +/- 0.25 inch average deviation for the vast majority of decks, n will be set at 20 to conserve the granularity of the testing.

The next consideration is the manner in which the 20 random samples are selected. For this topic, the reader is referred to GDT 73 Method C “Random Selection of

Roadway Concrete Samples." The document outlines a procedure by which random samples may be selected, with the modification that the lot boundaries are the spans and that each lot is evaluated independently [84]. There are many alternative ways in which to randomly sample the 20 points on each span.

The next matter is selecting the form of the pay equation, which converts the results of the testing to the value of the pay for the work. One of the first concerns is whether or not there should be a "bonus" incorporated in the pay equation, in addition to penalties. As discussed in Chapter 2, there appears to be a legal requirement for such a combination of both incentives and penalties. Aside from the legal argument, Burati et al provides a strong argument for the need of a positive incentive as well as a penalty [104]. Namely that the positive incentive is required to ensure that, on average, 100 percent is paid for the AQL work and thus payment is not unfairly biased downward. The bonus provision promotes adoption by the construction industry and adds economic value from the improved quality assurance and control [104]. Novak et al. [36] notes in their work developing a performance-related specification for controlling the compressive strength of deck concrete for the Vermont Agency of Transportation (VTrans) that a net overpayment of 3 percent was budgeted and implemented in their system. This net overpayment was intended to incentivize industry partners to improve their technology and practices and reduce the likelihood of the industry simply raising bid prices to offset expected losses from delivering consistent with their historical norms [36]. While the case has been made for a reward as well as a penalty, some of the equations evaluated in this research will be devoid of the bonus provision, for purposes of insight as well as for implementation in cases where net overpayments have not been budgeted.

Multiple pay factor equations were considered in order to evaluate how the formulation of the pay factor can influence the payment received. The first pay factor equation evaluated is shown in Equation 12, which is the default provided in the specification [101, 104]. The equation assumes a linear form, with an apparent bonus that cannot exceed 5 percent. The expected payment just above the RQL is 80 percent, which represents the floor for the pay factor (PF).

$$\mathrm{PF}(\%) = 55 + 0.50(\mathrm{PWL}) \qquad \text{Eq. 12}$$

The second pay factor equation evaluated is a simple modification of Equation 12, where the intercept is reduced from 55 to 50, thereby removing any possibility of a bonus as shown in Equation 13.

$$\mathrm{PF}(\%) = 50 + 0.50(\mathrm{PWL}) \qquad \text{Eq. 13}$$

The third and fourth pay factor equations are based on the comparable SOP 46, “Procedure for Calculating Pay Reduction for Failing Roadway and Bridge Approach Smoothness” [85]. These pay factor equations consider are some modifications to the SOP 46, including the average cover and design cover as a replacement for correction smoothness and actual smoothness, and the intended result when the ratio of the average to design cover exceeds one. It is on this point that the equation is split into two, with the third pay equation (Equation 14) having no restrictions on overpayment. For the fourth (Equation 15), a stepwise formulation is adopted to prevent overpayment when the ratio is greater than 1.

$$\mathrm{PF}(\%) = \left(\frac{\mathrm{AC}}{\mathrm{DC}}\right) * 100 \qquad \text{Eq. 14}$$

$$\mathrm{PF}(\%) = \begin{cases} \left(\frac{\mathrm{AC}}{\mathrm{DC}}\right) * 100, & \left(\frac{\mathrm{AC}}{\mathrm{DC}}\right) \leq 1 \\ 100, & \left(\frac{\mathrm{AC}}{\mathrm{DC}}\right) > 1 \end{cases} \qquad \text{Eq. 15}$$

In the equations, AC is the average cover and DC is the design cover. It should be apparent that this formulation lacks many of the benefits provided by equations based on

PWL, which incorporate the variability of the cover, and seek to better estimate the true impact of the cover distribution.

7.3 Evaluating the Pay Factor Equations

There are many ways that the proposed pay factors could be evaluated, though they all should rely on the stated objectives of the plan and the feedback of the stakeholders. For this research, the input of the stakeholders have not been thoroughly investigated, which is a task for future work. Rather, comparable payment plans that have been implemented in other states were examined to gain insight. Novak et al. states that VTrans decided that there should be a broad and conservative peak (centered on the target value) in their pay structure, with a linear transition as the observed distribution deviates from the target [36]. A very similar broad payment structure (a conservative peak with linear transitions) can also be seen in the work of Buddhavarapu et al. and their refinement of an existing adjustable payment plan for hot mixed asphalt for the Texas Department of Transportation (TxDOT) [35]. Therefore, the pay factor equations should be mild and gradual in its pay adjustments and not overly sensitive or harsh.

With the above in mind, a series of 40 bridge spans, from bridges constructed from 1978 to 2019 were randomly selected to serve as case studies by which to evaluate the pay equations. As discussed in Chapter 5, there appears to be spatial autocorrelation between the cover measurements along the deck, which may have been subject to bias. Sampling bias can result in an incorrect assessment of the cover control on the deck, which may lead to falsely overpaying or underpaying for the quality received. In addition, the dimensions of the spans vary considerably. The first limitation may be overlooked for the purposed of making initial economic comparisons by assuming that the current systematic cover sampling sufficiently captures the true cover distribution. In order to address the second limitation it was assumed that a square foot of deck has the same cost in each bridge, and that the number of data points per cover survey is a reasonable estimator of the size, and thus cost of the bridge. To put some reasonable numbers to the cost, the values from a Florida Department of Transportation (FDOT) document are used [105]. From FDOT, a medium and long simple span new construction bridge with a concrete deck and steel girders costs between \$125- \$142 per square foot (midpoint of \$133). For a similar bridge with pre-stressed girders, the cost per square foot is estimated between \$90 - \$145 (midpoint \$118). The estimates are the total cost and, therefore, the deck itself is some fraction. For the purposes here a square foot of deck will be assumed to cost \$100, a conservative value slightly below the midpoint for both cost figures.

Of the 40 randomly sampled spans, seven (17.5 percent) failed to meet the acceptance threshold (i.e., the average cover being within 0.25 inch pre-grinding) and would thus be outright rejected and likely corrective action would be called for. Consequently, all the subsequent analysis was performed on the remaining 33 spans.

The cover and corresponding pay factors (calculated through Equations 12-15) for all spans is given in Table 37. For this group of spans, the average and standard deviation of the pay factors (in percentage) for Equations 12-15 were 94.6 (S.D. 7.44), 89.6 (S.D. 7.44), 98.7 (S.D. 5.07), 97.2 (S.D. 2.98) respectively. Crucially, the averages were less than 100 percent for all equations, which provides an incentive for improvement, rather than maintaining the status quo. The pay factor equation should exert influence over the full range of expected span covers, with a gradual change in pay. These characteristics would result in a larger standard deviation for the pay factor. From Table 37, Equations 12 and 13 have larger standard deviations than those of Equations 14 and 15 (with the smallest standard deviation), and are therefore better suited for this application. Figure 32 represents the results from Table 37 in a graphical manner. A large dispersion in the pay factor for the spans is desirable.

Table 37. Summary of results of the 33 spans

Span	Design Cover (in)	Avg. Cover (in)	S.D. (in)	n	Avg. Cover - Design Cover	PWL	PF Eq. 12	PF Eq. 13	PF Eq.14	PF Eq. 15
1	2.25	2.23	0.17	16	-0.02	87.0	98.5	93.5	99.1	99.1
2	2.5	2.54	0.35	12	0.04	51.0	80.5	75.5	101.6	100.0
3	2	2.00	0.16	36	0.00	90.0	100.0	95.0	100.0	100.0
4	2.5	2.38	0.32	44	-0.13	53.0	81.5	76.5	95.0	95.0
5	2.75	2.56	0.09	84	-0.19	74.0	92.0	87.0	93.1	93.1
6	2	2.06	0.27	67	0.06	64.0	87.0	82.0	103.0	100.0
7	2.5	2.41	0.15	16	-0.09	85.0	97.5	92.5	96.4	96.4
8	2.25	2.16	0.18	36	-0.09	79.0	94.5	89.5	96.0	96.0
9	2.25	2.10	0.12	10	-0.15	79.0	94.5	89.5	93.3	93.3
10	2.25	2.12	0.11	48	-0.13	86.0	98.0	93.0	94.2	94.2
11	2.25	2.47	0.18	20	0.22	56.0	83.0	78.0	109.8	100.0
12	2.25	2.43	0.13	12	0.18	69.0	89.5	84.5	108.0	100.0
13	2.75	2.60	0.07	12	-0.15	93.0	101.5	96.5	94.5	94.5
14	2.75	2.65	0.19	96	-0.10	75.0	92.5	87.5	96.4	96.4
15	2.5	2.35	0.07	35	-0.15	92.0	101.0	96.0	94.0	94.0
16	2.75	2.78	0.12	42	0.03	97.0	103.5	98.5	101.1	100.0
17	2.5	2.47	0.17	45	-0.03	85.0	97.5	92.5	98.8	98.8
18	2.25	2.43	0.15	12	0.18	67.0	88.5	83.5	108.0	100.0
19	2.5	2.67	0.11	18	0.17	76.0	93.0	88.0	106.8	100.0
20	2.25	2.43	0.07	24	0.18	83.0	96.5	91.5	108.0	100.0
21	2.75	2.63	0.07	45	-0.12	97.0	103.5	98.5	95.6	95.6
22	2.75	2.76	0.09	50	0.01	100.0	105.0	100.0	100.4	100.0
23	2.75	2.74	0.03	75	-0.01	100.0	105.0	100.0	99.6	99.6
24	2.75	2.72	0.16	50	-0.03	88.0	99.0	94.0	98.9	98.9
25	2.75	2.55	0.12	28	-0.20	65.0	87.5	82.5	92.7	92.7
26	2.75	2.56	0.11	8	-0.19	69.0	89.5	84.5	93.1	93.1
27	2.75	2.82	0.13	5	0.07	94.0	102.0	97.0	102.5	100.0
28	2.25	2.26	0.10	100	0.01	100.0	105.0	100.0	100.4	100.0
29	2.25	2.25	0.22	60	0.00	74.0	92.0	87.0	100.0	100.0
30	2.5	2.29	0.17	35	-0.21	59.0	84.5	79.5	91.6	91.6
31	2.75	2.71	0.11	52	-0.04	98.0	104.0	99.0	98.5	98.5
32	2.25	2.15	0.19	131	-0.10	74.0	92.0	87.0	95.6	95.6
33	2.75	2.51	0.16	20	0.24	52.0	81.0	76.0	91.3	91.3
						AVG:	**94.6**	**89.6**	**98.7**	**97.2**
						S.D.:	**7.44**	**7.44**	**5.07**	**2.98**

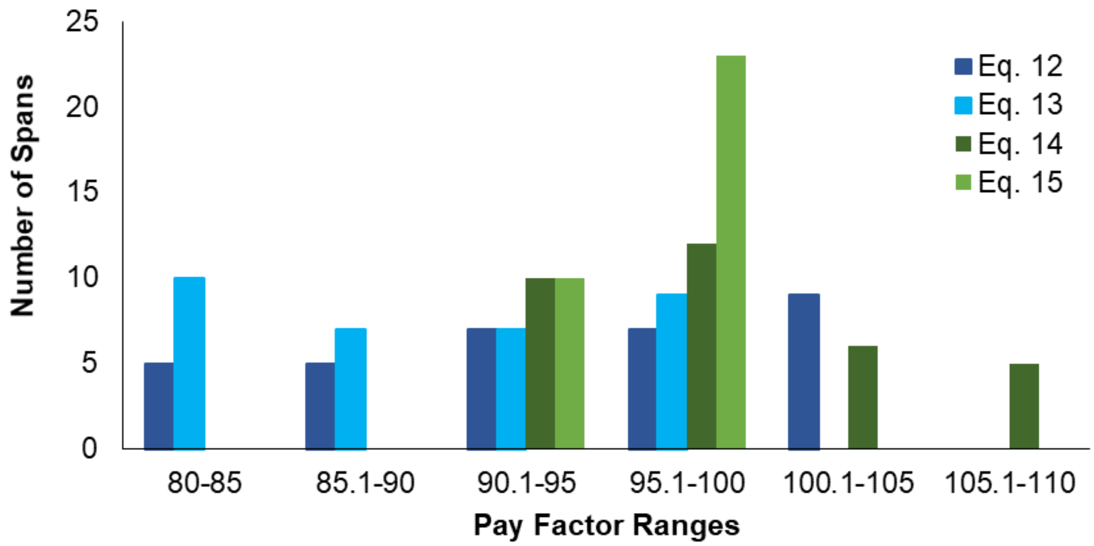

Figure 32. Histogram showing the proportion of spans within each pay factor range.

From Figure 32, Equations 14 and 15 result in an undesirable sharp peak and narrow payment band. Amongst Equations 12 and 13, the lack of the bonus in Equation 13 results in a shift in the prevalence of pay factors to the left, toward lower average payments, which has the negative effect of doubling the number of spans paid at the 80-85 percent ranges compared to Equation 12. Therefore, amongst these equations evaluated for the desired broad payment ranges, Equation 12 appears best suited, though an economic analysis is needed to further understand the impacts of these equations.

Considering pay factor equations from a cost perspective, with the assumption that each cover measurement was approximately 10 ft from its nearest neighbor (in a square grid), each data point has a tributary area of approximately 100 square feet (half the distance to neighbor squared). Therefore, the preadjustment cost for a bridge deck is estimated by Equation 16, where *n* is the number of sampled points in the cover survey. The results from applying the payment factors to the estimated deck costs can be seen in Table 38.

$$\text{Preadjustment Deck Cost} = \text{n} * 100\text{ft}^2 * \$100/\text{ft}^2 \qquad \text{Eq. 16}$$

Table 38 provides significant insights as overall trends, without focusing on the specific values. Without exception, all the proposed pay equations result in a net reduction in the overall payment for this set of 33 spans, which is expected given the sub 100 percent average pay factors in Table 37. That trend may not be desirable because the overall industry would expect lower prices, which may just result in an industry wide increase in bid prices as opposed to improvements in their construction practices.

Table 38. Estimates of the economic consequences of the proposed pay equations

	Difference in Pay			
Estimated Deck Cost	**Eq. 12**	**Eq. 13**	**Eq. 14**	**Eq. 15**
$160,000	-$2,400	-$10,400	-$1,422	-$1,422
$120,000	-$23,400	-$29,400	$1,920	$0
$360,000	$0	-$18,000	$0	$0
$440,000	-$81,400	-$103,400	-$22,000	-$22,000
$840,000	-$67,200	-$109,200	-$58,036	-$58,036
$670,000	-$87,100	-$120,600	$20,100	$0
$160,000	-$4,000	-$12,000	-$5,760	-$5,760
$360,000	-$19,800	-$37,800	-$14,400	-$14,400
$100,000	-$5,500	-$10,500	-$6,667	-$6,667
$480,000	-$9,600	-$33,600	-$27,733	-$27,733
$200,000	-$34,000	-$44,000	$19,556	$0
$120,000	-$12,600	-$18,600	$9,600	$0
$120,000	$1,800	-$4,200	-$6,545	-$6,545
$960,000	-$72,000	-$120,000	-$34,909	-$34,909
$350,000	$3,500	-$14,000	-$21,000	-$21,000
$420,000	$14,700	-$6,300	$4,582	$0
$450,000	-$11,250	-$33,750	-$5,400	-$5,400
$120,000	-$13,800	-$19,800	$9,600	$0
$180,000	-$12,600	-$21,600	$12,240	$0
$240,000	-$8,400	-$20,400	$19,200	$0
$450,000	$15,750	-$6,750	-$19,636	-$19,636
$500,000	$25,000	$0	$1,818	$0
$750,000	$37,500	$0	-$2,727	-$2,727
$500,000	-$5,000	-$30,000	-$5,455	-$5,455
$280,000	-$35,000	-$49,000	-$20,364	-$20,364
$80,000	-$8,400	-$12,400	-$5,527	-$5,527
$50,000	$1,000	-$1,500	$1,273	$0
$1,000,000	$50,000	$0	$4,444	$0
$600,000	-$48,000	-$78,000	$0	$0
$350,000	-$54,250	-$71,750	-$29,400	-$29,400
$520,000	$20,800	-$5,200	-$7,564	-$7,564
$1,310,000	-$104,800	-$170,300	-$58,222	-$58,222
$200,000	-$38,000	-$48,000	-$17,455	-$17,455
Average:	-$17,832	-$38,195	-$8,057	-$11,219
Net:	-$588,450	-$1,260,450	-$265,890	-$370,223
Max:	$50,000	$0	$20,100	$0
Min:	-$104,800	-$170,300	-$58,222	-$58,222

As noted in Novak et al., to combat this possibility the pay equation was altered to result in a net overpayment of 3 percent [36]. The negative trend is more pronounced in the pay equations without the ability to exceed 100 percent for the pay, which as mentioned previously, helps to offset the risk that the sampling causes a bridge to receive less payment than the work may be entitled to, as well as providing an economic incentive to improve. Between Equations 12 and13, the lack of a bonus causes the average and net losses over the entire series of spans to double. Interestingly, between Equations 14 and 15, there is not a doubling of the average and net losses, but rather an approximately 39 percent increase. Equation 12 provides the most diverse spread between the best and worst spans evaluated, providing the largest reward (~$50k) and the second largest penalty (~$105k), which would be ideal for incentivizing improvements.

Considering the results from Table 37 and Table 38 as well as Figure 32, it is in the opinion of the author that Equation 12 is best suited for purposes of improving the cover construction practices. However, there still exists uncertainty with the discrepancies between the sampled cover distribution and the true cover distribution. To investigate these risks, the software OCPLOT was utilized to examine Equation 12 [106]. As part of the analysis, OCPLOT creates 500 random sampling trials (of the specified sampling number, in this case n=20) on a simulated bridge deck with a specified “true” PWL and standard deviation. The software determines the sampled PWL and the corresponding pay factor for each trial. The software then uses the results from the aggregate of the trials to determine the expected PWL and the expected pay factor according to Equation 12. This allows the user to assess the risks associated with the pay equation, which can then be altered if needed. The results from that evaluation are given in the curve in Figure 33.

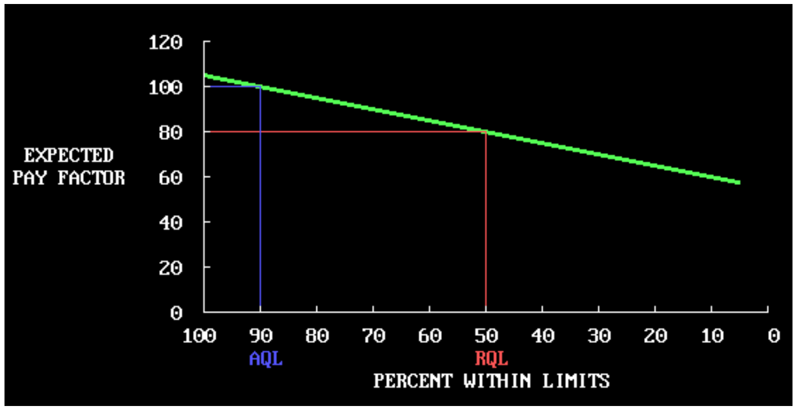

Figure 33. Output from OCPLOT showing the expected pay given the percent within the specification limits.

Figure 33 graphically displays the results expected for determining the payment when using Equation 12. The figure supports the notion that if the work submitted is exactly at the adequate quality level, then it is expected that the contractor will receive 100 percent of the pay even though, for any one span, the sampling will cause the contractor to receive too much or too little pay in any given trial. To demonstrate how each sampling affects the pay factor, Figure 34 was examined. It shows that if the work was truly 50 PWL, the variability in the estimated *PWL* from the 20 sampled points (min/max of ~25 PWL and 78 PWL) and how that affects the corresponding pay factor.

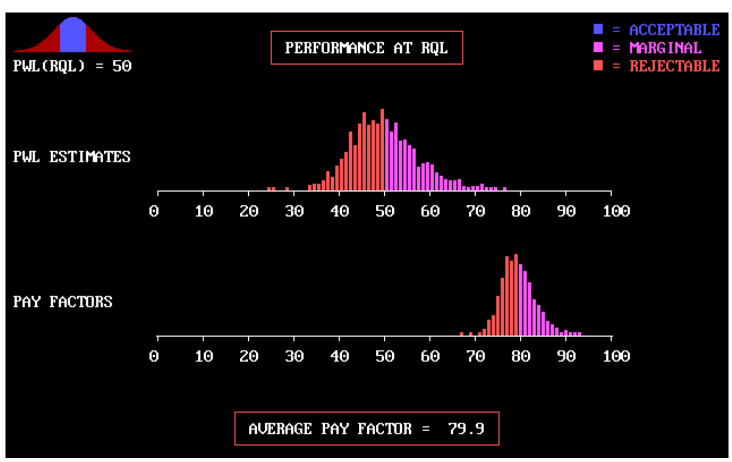

Figure 34. OCPLOT output demonstrated the range in PWL estimated from sampling and corresponding range in payment.

From Figure 34, on average, a contractor could expect a pay factor of 80 percent given work that is 50 PWL with the proposed sampling scheme. With fewer random samples the spread in *PWL* estimates is expected to increase, which may increase the risk that on any one span evaluation that the work receives an inaccurate pay factor. Therefore the proposed specification, as provided in Appendix B, incorporates Equation 12 as well as the randomly sampling procedure (n=20).

7.4 Chapter Conclusions

An adjustable payment plan for top mat cover control was created and evaluated in this chapter, considering multiple pay factor equations. The selected pay factor equation, Equation 12, provides a broad payment scheme to incentivize proper cover control. The equation balances the risk to the contractor of receiving less pay for work that is truly acceptable by the incorporation of a 3 percent overpay. The process outlined in this chapter can be applied in other areas identified in Chapter 5, such as ensuring a low diffusivity concrete, in an effort to extend the anticipated lifespan of new construction.

CHAPTER 8

CONCLUSIONS

8.1 Contributions to State of the Art

The aging of the U.S. bridge inventory presents a significant challenge to the safe and economic conveyance of people and goods, particularly as bridges are required to remain in service past their designed service lives. Therefore, means to extend the service life of both existing bridges and new construction are needed. The research addresses this need and was narrowed to focus on extending the service lives of reinforced concrete bridged decks and included the development of science-based contractual mechanisms for implementation.

A sampling of deck cover surveys from Georgian bridges built in the late 1970s to recent years was analyzed to determine the current cover control delivered. Greater than 90 percent of the sampled bridge decks had an average cover within 0.25 inch of their design cover. This tolerance appears constant over the last 40 years. Furthermore, the cover data was best approximated by a normal distribution or lognormal distribution, and exhibits spatial interdependence. These cover findings were used in modeling the durability of bridges, which incorporate cover thickness as a key parameter.

Bi-annual inspection reports from the state of Georgia were also used to quantify the prevalence of various degradation mechanisms of the bridges. These reports showed a significant amount of spalling, which indicated that a corrosion-based service life model is needed to predict service life. Therefore, chloride-induced corrosion models were selected for forecasting the degradation of bridge decks in Georgia. Limit states for the model were selected as the periods where five percent and ten percent of the deck surface spalled, delaminated, or patched. These limit states were selected because they are in alignment with both literature sources and the damage noted in inspection reports from recently decommissioned decks.

The service life of decks was modeled using three primary corrosion models, which demonstrated that improved construction practices (e.g., better cover control) and the utilization of novel and alternative materials (e.g., SCM concretes) can substantially prolong the service life of decks. These models relied on literature values for key parameters, as well as historical data from the bi-annual inspection reports. Among the alternatives modeled, using alternative rebar had the greatest impact on expected service life, with service life projections well past the critical timeframe for 50 to 100 years.

The results from the models created in this research were used to inform contracting practices for new construction and maintenance practices for existing decks. This research explored the use of various contracting mechanics (e.g., warranties, acceptance plans) to achieve extensions of service life through cover control. In the exploration, legal, technical, and economic considerations were taken into account. The adjustable payment plan was found to be best suited for this application because they are often used in similar construction quality applications and are likely enforceable. To demonstrate the adjustable payment plan approach, a sample plan for improved cover control was created. The approach provides the framework for uses contracting mechanisms for other construction quality applications.

8.2 Recommendation for Practice

DOT should consider the following recommendations: The use of SCMs should be expanded in bridge deck concrete mixes, as they are often more economical and may extend the service life of the decks. In particular 20 percent FA replacement and 50 percent BFS performed the best in the service life modeling among the mixes evaluated.

As described earlier, a contractual mechanism such as the one presented in this research should be adopted to improve cover control. From the modeling, the impacts of cover accuracy and cover variability should be considered in about equal measure.

As identified in the literature review and Chapter 4, DOTs should consider transitioning from the visual-based inspections that are currently practice toward testing-based evaluations. A number of promising technologies which could be used for inspections, such as resistivity measurement, half-cell potential mapping, and impact-based evaluations were identified in the literature review.

APPENDIX A: NBI RATINGS

The selection process for the decks was described in (Chapter 4). The NBI ratings for each bridge are provided in Figure 35, and for the purposes of comparison, the superstructure ratings and substructure ratings are plotted as well.

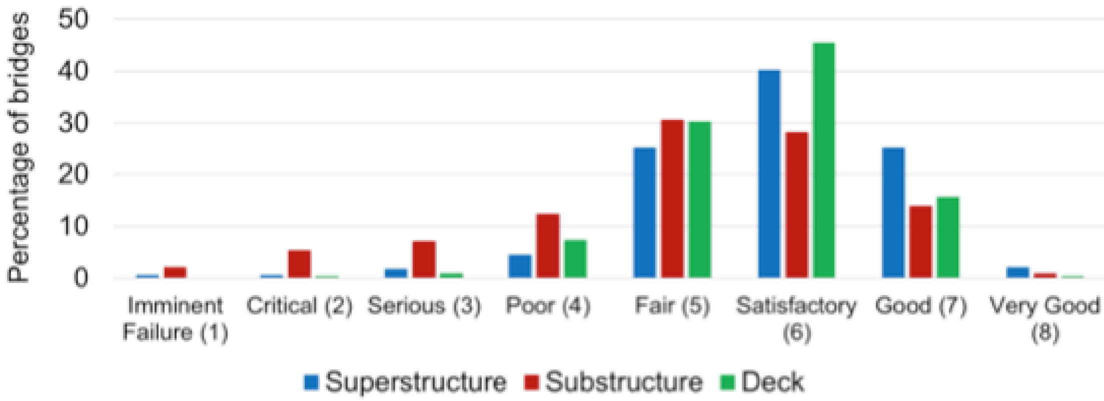

Figure 35. NBI condition ratings for the various components of the bridges at the time of decommissioning.

From the data, it appears that the most common rating for a bridge component was a six, a “satisfactory” rating. A satisfactory rating is given when “structural elements show some minor deterioration” [37]. On average, for a given bridge, the substructure had the lowest rating while the deck had the highest. It is important to note, however, that the relative ratings are in regards to the average, with individual bridges deviating from the trend. The overall conclusion is that degradation has been observed in the decks, as seen by the reduction in NBI rating to a 6 by decommission on average.

APPENDIX B: SAMPLE ADJUSTABLE PAYMENT PLAN

A sample provision for incentivizing cover control is provided below. This provision is not in effect, and is for academic purposes only.

Standard Operating Procedure (SOP)

Procedure for Calculating Pay Adjustments for Failing Concrete Cover Control of Reinforced Concrete Bridge Decks

I. General

If the concrete cover control is inadequate and cannot be remediated, the project engineer is empowered to reject the work. The purpose of this SOP is to provide a means of calculating pay reductions for failing concrete cover control of reinforce concrete bridge decks that in the project engineer's judgement do not warrant outright rejection of the work.

To inform the project engineer as to the cover control of the reinforced concrete bridge deck, a series of no fewer than 20 cover measurements will be taken per span. The method of sampling may either be a systematic sampling on a grid with approximately 10 ft separations, or any manner of random sampling. The method of executing the cover measurements is set forth in British Standard 1881-204, and is to be observed. The results from these measurements will be used to calculate the pay reduction factor for each span as outlined in the method below.

A. Method of Calculating Pay Reduction For Failing Cover Control

The pay reduction will be determined by the specified pay factor equation below.

$$PF=55+0.5(PWL) \qquad \text{Eq.1}$$

Where PF is the pay factor for the span, and PWL is the estimated percent of the true cover distribution that is within the limits in Table 1, based on the specified design cover.

Table 1- Control Limits for Common Design Covers

Design Cover (in)	Lower Cover Limit (in)	Upper Cover Limit (in)
2	1.75	2.25
2.25	2.00	2.50
2.5	2.25	2.75
2.75	2.5	3.00

To calculate the PWL, the following procedure is followed. First, the sample mean and standard deviation of the 20 randomly sampled cover measurements is calculated. If the sample mean is outside the limits as provided in Table 1, the work should be rejected and remediated. Provided that the sample mean is within the limits from Table 1, the upper and lower Q indices are to be calculated according to these equations:

$$Q_U = \frac{UCL - Sample\ mean}{Standard\ Deviation} \qquad \text{Eq.2}$$

$$Q_L = \frac{Sample\ mean - LCL}{Standard\ Deviation} \qquad \text{Eq.3}$$

Where the UCL and LCL are taken from Table 1. Next, the PWL must be determined for both the upper and lower Q indices. To determine the PWL, the value of the Q index is matched to Table 2 below (for n=20), noting that if the Q value falls between table values, to round up to the next PWL.

Table 2- PWL Reference

PWL	Q	PWL	Q	PWL	Q	PWL	Q	PWL	Q
100	3.20	89	1.22	78	0.78	67	0.45	56	0.15
99	2.18	88	1.17	77	0.75	66	0.42	55	0.13
98	1.96	87	1.12	76	0.71	65	0.39	54	0.10
97	1.81	86	1.08	75	0.68	64	0.36	53	0.08
96	1.70	85	1.04	74	0.65	63	0.34	52	0.05
95	1.61	84	1.00	73	0.62	62	0.31	51	0.03
94	1.52	83	0.96	72	0.59	61	0.28	50	0.00
93	1.45	82	0.92	71	0.56	60	0.26		
92	1.39	81	0.88	70	0.53	59	0.23		
91	1.33	80	0.85	69	0.50	58	0.20		
90	1.27	79	0.81	68	0.47	57	0.18		

Next, the PWL for the upper and lower bound are combined into a single PWL according to the equation below:

$$PWL = (PWL\ upper + PWL\ lower) - 100 \qquad \text{Eq.4}$$

Next, the PF is calculated according to Eq. 1, using the PWL from Eq. 4. Finally, the value of the pay factor will be used to calculate the payment for the span according to the equation below:

Adjusted Payment=(PF/100)*(Original Payment) Eq.5

B. Example Pay Reduction Calculation

Suppose a bridge contains only one span, with a design cover of 2.25", and an original payment of $100,000. In accordance with the method in section A, 20 cover measurements are randomly taken which are found to be as follows:

Table 3- Results of the Randomly Selected Cover Points

Location	Cover Value (in)	Location	Cover Value (in)
1	2.10	11	2.00
2	2.30	12	2.20
3	2.20	13	2.30
4	1.90	14	2.50
5	1.80	15	2.90
6	2.10	16	2.10
7	2.50	17	2.60
8	2.60	18	2.30
9	2.00	19	2.20
10	2.30	20	2.30

From Table 3, the lower cover limit is 2" and the upper cover limit 2.5" (inclusive). The sample mean and standard deviation are found to be 2.26" and 0.258". Using Eqs. 2 and 3, the Q indices are found to be 0.93 and 1.01 for the upper and lower bounds respectively. Using Table 2, and selecting the next largest PWL when the Q index falls between table values, the PWLs are found to be 83 and 85. The combined PWL is then found to be 68 (100-83+85). Using Eq.1, the pay factor is then found to be 89%. Therefore, the adjusted payment is thus found to be (89/100)*$100,000, which equates to $89,000. For this example which only consists of one span, the adjusted payment is $89,000 for the bridge deck.

Suppose that instead of a standard deviation of 0.258", the standard deviation was calculated as 0.129" instead, with the same sample mean of 2.26". In that case, the Q indices are 1.86 and 2.02 for the upper and lower indices respectively. Using

Table 2, those Q indices would yield PWLs of 98 and 99. The combine PWL would then be calculated as 97. Using Eq.1, the pay factor is then found to be 103.5%. Therefore, the adjusted payment is thus found to be (103.5/100)*$100,000, which equates to $103,500. In this case, since the mean was almost exactly the design value, and the standard deviation was reduced by half, the bridge contractor received a reward for the additional performance expected out of this bridge span.

II. Report

The technical group will provide a letter of recommendation to the District Engineer to include a pay factor reduction or a waiver for all failing cover control projects. The Director of Construction, State Construction Engineer, Area Engineer, and OMAT's Material Audits Unit will be copied on all letters of recommendation.

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State Materials Engineer

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Director of Construction

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