SERVICE LIFE MODELING OF VIRGINIA BRIDGE DECKS

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ABSTRACT

A model to determine the time to the End of Functional Service Life (EFSL) for concrete bridge decks in Virginia was developed. The service life of Virginia bridge decks is controlled by chloride-induced corrosion of the reinforcing steel. Monte Carlo resampling techniques were used to integrate the statistical nature of the input variables into the model. This is an improvement on previous deterministic models in that the effect of highly variable input parameters is reflected in the service life estimations. The model predicts the time required for corrosion to initiate on 2% of the reinforcing steel in a bridge deck and then a corrosion propagation time period, determined from empirical data, is added to estimate the EFSL for a given bridge deck or set of bridge decks.

Data from 36 Virginia bridge decks was collected in order to validate the service life model as well as to investigate the effect of bridge deck construction specification changes. The bridge decks were separated into three distinct groups: 10 bare steel reinforcement decks – 0.47 $^{water}/_{cement}$ ($^{w}/_{c}$), 16 Epoxy-Coated Reinforcement (ECR) decks – 0.45 $^{w}/_{c}$, and 10 ECR decks – 0.45 $^{w}/_{(c+pozzolan)}$. Using chloride titration data and cover depth measurements from the sampled bridge decks and chloride corrosion initiation values determined from the literature for bare steel, service life estimates were made for the three sets of bridge decks. The influence of the epoxy coating on corrosion initiation was disregarded in order to allow direct comparisons between the three sets as well as to provide conservative service life estimates.

The model was validated by comparing measured deterioration values for the bare steel decks to the estimated values from the model. A comparison was then made between the three bridge deck sets and it was determined that bridge decks constructed with a 0.45 $^{w}/_{(c+p)}$ will provide the longest service life followed by the 0.47 $^{w}/_{c}$ decks and the 0.45 $^{w}/_{c}$ decks, respectively. From this it can be inferred that the addition of pozzolan to the concrete mix will improve the long-term durability of a bridge deck while a reduction in $^{w}/_{c}$ appears to be of no benefit.

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INTRODUCTION

Bridge deck rehabilitation and replacement costs contribute significantly to the overall state transportation budget in Virginia. Federal Highway Administration (FHWA) report # RD-01-156 estimates the total direct costs of corrosion for highway bridges in the United States to be $8.3 billion dollars per year, approximately half of which can be attributed to superstructure deterioration. (FHWA, 2002) In addition to the direct costs associated with maintaining the integrity of Virginia's bridge system there are also immeasurable impacts on the local economy due to interruptions in the flow of traffic related to bridge closures. According to the 2004 National Bridge Inventory (NBI) compiled by FHWA, 1,186 of Virginia's 13,160 bridges (9%) are currently considered structurally deficient. (FHWA, 2004) Bridges may be considered structurally deficient for several reasons, for example excessive settlement or scour. However, the principal cause of structural deterioration is related to the corrosion of the concrete reinforcing steel. For this reason, being able to accurately model and predict the corrosion of the reinforcement in bridge decks is of considerable interest to Virginia Department of Transportation (VDOT) bridge engineers.

BACKGROUND

Corrosion of reinforcing steel in bridge decks can be initiated by one of two modes, chloride-induced corrosion or carbonation-induced corrosion. To understand the different modes of corrosion and the reasons for corrosion initiation one must first understand the nature of steel in concrete. Steel when placed in an alkaline environment such as concrete pore water, will develop a passive layer of corrosion products. This passive layer is composed of iron oxides/hydroxides and is very dense. Over time this passive layer will reach a thickness sufficient to protect the base metal and the corrosion rate will for all practical purposes be equal to zero. As long as the passive layer remains intact active corrosion will not initiate, however, if the reinforcing steel is depassivated active corrosion can initiate and continue to propagate.

LITERATURE REVIEW

Corrosion of Steel in Concrete

The corrosion of reinforcing steel in concrete is a complex electrochemical process. In order for a corrosion cell to form there must be an anode, a cathode, an electrical connection between the two, and an electrolyte. For reinforcing steel in concrete both the anodic and cathodic areas may be present on the surface of an individual bar or they may be on different bars. The electrical connection is provided by the reinforcement itself and the saturated/partially-saturated concrete completes the corrosion cell by serving as an electrolyte. Corrosion cells where the anode and cathode are directly adjacent to one another are referred to as micro-cells whereas cells with the anode and cathode separated by some distance are considered macro-cells.

The anode is the site of active corrosion where the dissolution of iron and formation of corrosion products occurs and the cathode is the site where oxygen is reduced to form hydroxyl ions. The reactions that take place at the anode and cathode are presented below (Broomfield, 1997):

Anode

1. $Fe \rightarrow Fe^{++} + 2e-$ (Dissolution of iron)
2. $Fe^{++} + 2OH^{-} \rightarrow Fe(OH)_2$ (Ferrous hydroxide)
3. $4Fe(OH)_2 + O_2 + 2H_2O \rightarrow 4Fe(OH)_3$ (Ferric hydroxide)
4. $2Fe(OH)_3 \rightarrow Fe_2O_3 \cdot H_2O + 2H_2O$ (Hydrated ferric oxide – rust)

Cathode

1. $2e - + H_2O + \frac{1}{2}O_2 \rightarrow 2OH -$

The primary concern with corrosion of reinforcing steel in bridge decks is not the loss of cross-sectional area because bridge decks are designed in such a way that the amount of steel used typically far exceeds that which is required for strength. The problem most associated with the corrosion of reinforcement in bridge decks is the durability of the surrounding concrete. As corrosion progresses and iron reacts to form ferric oxides a significant volume change takes place. The volume change (200 – 1000%) associated with the corrosion process results in a buildup of pressure at the concrete/reinforcement interface, which will ultimately result in the cracking, spalling, and delaminating of the concrete cover. (Broomfield, 1997)

Chloride-Induced Corrosion

Corrosion of reinforcing steel in bridge decks is primarily associated with the diffusion of chlorides into the concrete. Chloride ingress into concrete is the result of the application of chloride bearing deicing salts, exposure to sea or brackish water, and to a lesser degree the wind transport of salt water in coastal regions. Chlorides that are deposited on the surface of the bridge deck will diffuse through the porous concrete and in time reach the depth of the reinforcing steel. It should be noted, however, that while the ingress of chlorides is typically modeled as strictly a diffusion process it is in actuality a much more complex process. The initial movement of chlorides at the surface of the concrete can be affected by capillary suction depending upon the percent saturation of the concrete. Additionally, the chlorides once present within the concrete have the potential to bind with aluminates and can also be affected by the fluxes of other ionic species. Due to the complexity of these processes and their strong dependence upon the chemical composition of the cement paste and pore water, their effects are generally ignored. It is also important to note that by fitting a diffusion curve to chloride profiles all contributing factors are accounted for whether they are diffusion related or not.

When the chlorides diffuse to the depth of the reinforcing steel they begin to attack the passive layer of corrosion products present on the surface of the steel. The chloride attack on the passive layer will not result in a decrease in pH, therefore, the passive layer will continually reestablish itself. For low chloride concentrations the passive layer is able to sustain itself and prevent active corrosion. However, when the concentration of the chlorides at the reinforcement reaches a critical level the passive layer on the steel reinforcement surface will break down and active corrosion will initiate.

Chloride-induced corrosion manifests itself through the formation of micro-cells (pits) rather than general uniform corrosion. Pitting corrosion is a localized mode of corrosion where the cathodic sites are directly adjacent to the anodic sites. Chloride-induced corrosion tends to develop pitting corrosion because the concentration of chlorides is typically higher in some areas and lower in others. The areas of high chloride concentration will develop pits while the areas with low chloride concentrations will remain passive. It is possible for macro-cells to be developed between the passive areas of steel, which will serve as the cathode, and the actively corroding anodic areas. If this occurs the corrosion rate of the already actively corroding areas may be accelerated. The process of chloride-induced corrosion is illustrated in Figure 1.

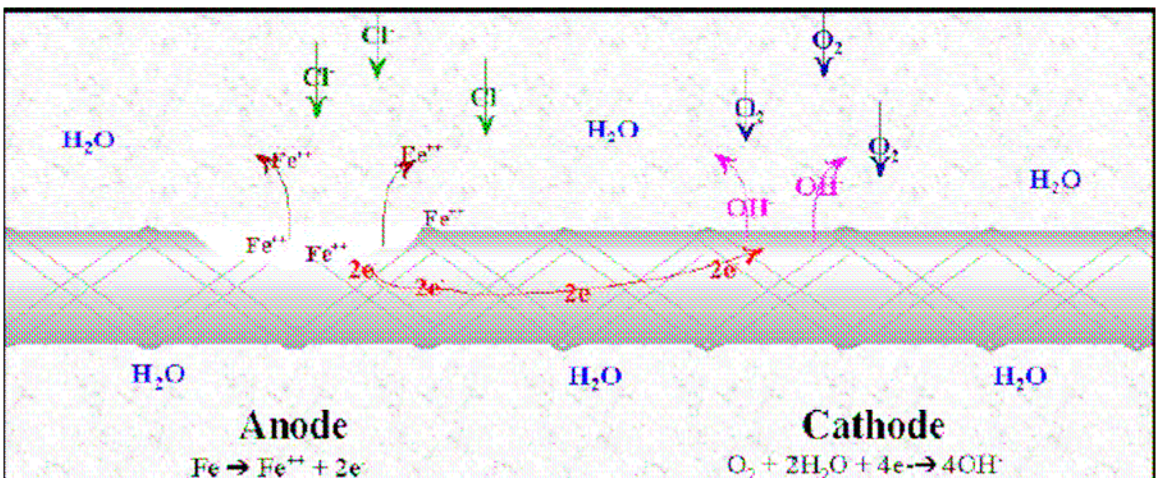


Figure 1 – Chloride-Induced Corrosion Process (Brown, 2002)

Carbonation-Induced Corrosion

The other corrosion mode of reinforcing steel in concrete, although much less common in the United States, is carbonation-induced corrosion. Carbonation-induced corrosion is associated with a decrease in the alkalinity of the concrete pore water due to the ingress of carbon dioxide into the concrete over time. This is more evident in structures where vehicle emissions cannot easily escape, such as tunnels and parking garages. When the pH of the concrete pore water decreases below approximately 9.0 the passive layer of corrosion will no longer be stable and active corrosion can initiate. (Bertolini et al, 2004)

The pH of the concrete pore water is lowered by the reaction of carbon dioxide with water and calcium hydroxide. The carbon dioxide initially reacts with water to form carbonic acid, which then reacts with calcium hydroxide to form calcium carbonate and water. This reaction process is presented below.

$$CO_2 + H_2O \rightarrow H_2CO_3$$
$$H_2CO_3 + Ca(OH)_2 \rightarrow CaCO_3 + 2H_2O$$

As more and more of the calcium hydroxide, which is highly alkaline, is consumed by the reaction, the pH of the concrete pore water becomes lower resulting in the destabilization of the passive layer. It has been shown that the progression of carbonation can be modeled as a diffusion process with the rate of diffusion being dependent upon concrete properties. The concrete factors that affect the carbonation corrosion process to the greatest degree are: concrete cover depth, cement alkali content, and concrete density. (Broomfield, 1997)

Carbonation-induced corrosion will typically be reflected as general corrosion over larger areas of steel as compared to chloride-induced corrosion, which tends to be localized.

Corrosion of Bare Steel in Concrete

The corrosion of bare steel in concrete as related to chloride ingress initiates as described previously. Corrosion will initiate at a location where the chloride concentration has reached a critical concentration. Chloride concentrations can vary widely across a deck, but it is generally observed that the areas of the bridge deck with the lowest cover will initiate corrosion first. There are many other factors that play a role in where corrosion will actually intiate and some of those will be discussed in further detail later in this chapter.

Corrosion of Epoxy-Coated Steel (ECR) in Concrete

Contrary to bare steel ECR will not necessarily initiate corrosion in the areas of the lowest cover depth. During the fabrication process the steel reinforcing bar is not uniformly coated and small pinholes in the coating, referred to as holidays, may be present in the finished product. In addition to this, ECR bars are often damaged while in transit or during construction. These exposed areas are susceptible to chloride-induced corrosion that will initiate in the same fashion as does bare steel. Because of this the initiation of corrosion for ECR is less dependent upon cover depth and more dependent upon the condition of the coating.

Once corrosion has initiated on an ECR bar the corrosion rate will be affected by the coating as well. The coating acts as a physical barrier to chlorides, oxygen, and water as well as an electrical barrier. The coating is intended to electrically isolate anodic areas from cathodic areas. The distance between the anodic areas and cathodic areas is dependent upon the distance between damaged coating areas on the bar. As this distance increases the resistance evident in the corrosion cell also increases, which results in a decreased corrosion rate.

The effectiveness of the epoxy coating in reducing the corrosion rate has been a source of debate. One area of controversy is whether or not corrosion is able to propagate underneath the coating. This is largely dependent upon the strength of the bond between the coating and the bar. One study reports that adhesion loss was evident in bridge decks as young as 4 years and suggests that "ECR will not provide any or little additional service life for concrete bridge decks in comparison to black steel". (Pyc, 1998)

Chloride Threshold Concentrations

As has been established that the concentration of chlorides at the depth of the reinforcement must reach a threshold value in order for corrosion to initiate and sustain a significant corrosion rate. It is also known that there is no single value that is appropriate for use as the threshold concentration. Threshold values can vary widely between bridge decks as well as within a single bridge deck. This is due to the many factors that affect corrosion initiation. Several major factors that have been identified are "the concentration of hydroxyl ions in the pore solution, the potential of the steel, and the presence of voids at the steel/concrete interface." (Bertolini et al., 2004) Other factors that are known to affect corrosion threshold concentrations are: cement composition, moisture content, $^{w}/_{c}$ ratio, and temperature. These factors influence the chloride binding capacity of the cement paste, the capability for corrosion to initiate based upon the electrochemical potential of the steel, the ability for localized corrosion (pitting) to form, and the passive layers' capacity to re-establish itself.

The proper way to report chloride threshold levels and chloride concentrations has been a point of contention among researchers for many years. The most common methods used to specify concentrations are: % chloride by weight of cement, weight per unit concrete, and chloride to hydroxyl ratio. It has been reported that the chloride/hydroxyl ratio must reach a specified level in order for corrosion to initiate. (Page et al., 1986; Lambert et al., 1991) This may be true, however, there is still a wide range of reported values and hydroxyl concentrations are difficult to obtain. A review of the reported values concluded that "chloride threshold levels are best presented as total chloride contents expressed relative to the weight of cement." (Glass and Buenfeld, 1997)

The decision to report the free chloride concentration or the total chloride concentration is another area of debate. The free chloride concentration is taken to be the water-soluble concentration and the total concentration is taken to be the acid-soluble concentration. The difference between the two is the bound chloride concentration. It has been argued that bound chlorides are not available to initiate or propagate corrosion, however, it has been shown that as corrosion propagates bound chlorides can be liberated and may become available to continue the corrosion reaction. Due to the fact that the extent and mechanism of the liberation of bound chlorides is not fully understood the acid-soluble chloride concentration less the background chloride concentration will be utilized in this study. Additionally, the concentration of chlorides per unit of concrete will be the convention used in this study, as actual cement contents for the bridge decks sampled are not known and do not vary significantly in Virginia where a minimum cement content and maximum $^{w}/_{c}$ are specified.

Chloride threshold levels also vary depending upon the type of reinforcement present within the concrete. Alternative reinforcements such as galvanized steel and stainless steel have much higher threshold levels. Specific ranges and statistical distributions for chloride threshold values for reinforcement types will be presented in the Method of Analysis section.

Time to Corrosion-Induced Cracking of Cover Concrete

The corrosion rate of steel in concrete is affected by several factors among which are: the permeability of the concrete, moisture content of the concrete, temperature, and the availability of oxygen. The time required for corrosion to propagate to a level where the concrete will be damaged will vary depending upon the local climate. For example a region with a dry climate will have a significantly longer propagation time than a region with a tropical climate due to the lack of moisture in the concrete. For bridge decks in Virginia it is reasonable to assume a single value for the propagation time, as the climate does not vary significantly within the state and also because the propagation time is small relative to the time required to initiate corrosion. A study on the corrosion of reinforcing steel in the U.S. estimates the propagation time to be between 2 and 5 years for bare steel. (Weyers et al, 1992) Some increase in propagation time is expected for alternative reinforcements such as epoxy-coated reinforcement (ECR) and stainless steel reinforcement.

ECR, the most common alternative reinforcement, provides additional propagation time by inhibiting the formation of corrosion macrocells. The additional propagation time for ECR has been estimated to be between 2.7 and 5.7 years. (Weyers et al., 2006) Several corrosion cracking models will be presented later in the literature review and time to cracking estimates will be made based upon current construction specifications and expected corrosion rates for Virginia bridge decks.

Deck Protection, Repair, and Rehabilitation Methods

It has long been known that chlorides from deicing salts are the primary cause of reinforcement corrosion in bridge decks in the Northern climates of the United States.

VDOT, just as nearly all other state agencies, has taken steps to prevent or delay the onset of chloride-induced corrosion. VDOT has increased cover-depth requirements and implemented the use of low permeable concretes as well as specified the use of ECR. It is known that by increasing the concrete cover-depths and using low permeable concretes the time required for chlorides to penetrate to the depth of reinforcing steel will be significantly increased. This in turn will delay the onset of corrosion and prolong the service life of Virginia bridge decks. This section addresses the deck protection, repair, and rehabilitation methods that are currently available and in use in the U.S. Additional methods that are under investigation will also be discussed, but not addressed in the service life and life cycle cost determinations.

Deck Protection Methods

Deck protection methods include those methods that are used to reduce the rate of the diffusion of chlorides into the concrete, extend the corrosion propagation period, or prevent active corrosion from initiating.

Deck Sealers

The first protection method that will be discussed is the use of deck sealers. Deck sealers can either be solvent or water based liquids that are applied to the deck surface creating a finite impermeable layer, which prevents chloride-laden water from penetrating the concrete structure. Currently, "only penetrating sealers, silanes and siloxanes (or combinations), are recommended for deck surfaces. Other sealer types have an inadequate depth of penetration and quickly wear when exposed to traffic abrasion." (Weyers, R.E. et al, 1993) Deck sealers while preventing water from entering the concrete structure will allow water vapor to escape. This allows for the concrete deck to dry out, which increases the resistance in the corrosion cell by limiting the electrolyte (the concrete pore water).

There are, however, limitations to the use of deck sealers. The effectiveness of a sealer is dependent on the permeability of the original concrete, which cannot always be estimated due to variability in construction practices. Additionally, it is possible to achieve similar reductions in concrete permeability by reducing the $^{w}/_{c}$ ratio of the concrete and through the addition of pozzolanic materials to the concrete mixture.

Cathodic Prevention

Cathodic prevention is a technique used to prevent active corrosion from initiating. Cathodic prevention is the process by which passive reinforcement is cathodically polarized to increase its potential by applying a low current. Increasing the electrical potential of the steel will elevate the critical chloride threshold to a point that will in theory never be reached during the service life of the bridge deck. Another type of cathodic prevention is the introduction of a sacrificial anode into the deck, typically zinc. The zinc being a less noble metal will act as the anode, therefore making the steel reinforcement the cathode. The benefit to having the zinc corrode in place of the steel is that the zinc corrosion process is less expansive and the corrosion products are able to diffuse into the surrounding concrete matrix without causing damage to the structure. The problems with cathodic prevention techniques are that they are expensive, require constant maintenance, and if the current is interrupted active corrosion can initiate immediately.

Currently, cathodic prevention systems are only being used for rare cases in the United States. They are primarily used in coastal structures that are exposed to very high chloride concentrations or in other specialized cases where corrosion is expected to be severe or structurally problematic. Implementing a cathodic prevention system is essentially an admission that the structure is not designed for long-term durability. Perhaps other corrosion prevention methods should be considered to ensure that corrosion will not initiate under field conditions.

Stainless Steel (SS) Reinforcement

SS reinforcement is a possible alternative to standard carbon bare steel because it has a much higher chloride threshold than carbon steel reflecting corrosion rates “at least 50 times lower than that of black steel in chloride contaminated concrete.” (Gu, P. et al, 1996) Clemena reports threshold values for stainless steel of at least 10.4 times greater than carbon steel. (2003) This elevated chloride threshold will never be reached for nearly all bridge deck applications. To be considered stainless steel the steel must be composed of at least 10.5% chromium. Additional elements such as nickel and molybdenum are also added to improve the corrosion resistance and mechanical properties of the steel. SS is divided into four categories based upon its microstructure:

-- Ferritic

-- Austenitic

-- Martensitic and

-- Austenitic – Ferritic (Duplex)

Currently only austenitic and duplex SS are used as concrete reinforcement as the mechanical properties of the other types are not comparable to carbon steel. However, a 12% ferritic SS is being considered for use. (Bertolini et al, 2004)

The additional corrosion resistance of SS is due to the formation of a passive layer of chromium oxide at the surface of the reinforcing bar. This passive layer has a critical chloride concentration that is much higher than that of carbon steel.

The benefits of using SS over other alternative reinforcements are that the corrosion resistance is substantially greater and the additional corrosion resistance is a bulk material property. This means that if the reinforcing bars are damaged during construction, the corrosion resistance of the reinforcement will remain constant unlike epoxy-coated and galvanized reinforcement where if the coating is damaged the additional corrosion resistance may be compromised.

Based on the substantial increase in corrosion resistance for SS reinforcement it would appear that it would be in wide use, however, using SS may be cost prohibitive for some bridge deck applications. The initial cost of SS is 6-10 times that of carbon steel, so it may be cost effective only for bridges where very long service lives must be guaranteed due to the adverse affects associated with bridge closures or in marine environments where chloride concentrations can be very high. For example, a bridge with very high traffic flow may require a service life of 100+ years because a closure could result in substantial economic loss to the surrounding area. As a means to reduce the costs of incorporating SS into bridge decks, the SS may be used only in the top mat of reinforcement, where chloride contents are higher. Using SS in the top mat and carbon steel in the bottom mat, there is a potential for galvanic corrosion to take place. However, a recent study has shown that "no significant risk of galvanic corrosion exists when carbon steel and stainless steel are electrically coupled in reinforced concrete structures". (Abreu, C.M., et al, 2002)

Another method to reduce the costs associated with using SS was introduced in the U.S. in 1998. That method was the use of stainless steel clad (SSC) reinforcing bars. SSC bars are manufactured by forming a tube of stainless steel and then packing it with carbon steel shavings. The tube is then heated to form a composite material. The purpose of using SSC bars is to reduce the cost of the reinforcement by using carbon steel cores while maintaining the excellent corrosion properties of the SS bars. The estimated cost for SSC bars is approximately $1.00/lb ($2.21/kg) compared to $3.00/lb ($6.62/kg) for solid SS reinforcing bars. It is hoped that SSC bars will provide the desired service life of 100 years and be cost effective. A recent study conducted by the Virginia Transportation Research Council (VTRC) concluded that "stainless steel-clad bars can be used as direct substitutes for either uncoated black steel or epoxy-coated bars for effective, corrosion-resistant reinforcement of concrete bridge decks" and recommends that "stainless steel-clad bars be considered an option for reinforcing new concrete bridges". (Clemena, Kukreja, and Napier, 2003) However, it has also been reported that corrosion at cladding defects and cut ends may be a problem. (Cui and Sagues, 2001 and 2003) Currently there is insufficient data to make any final conclusions on the corrosion resistance of SSC reinforcement. It is clear that further research is required for this reinforcement alternative, but initial results are promising. Additionally, the mechanical properties of one SSC reinforcement have been shown to meet ASTM standards for both tensile and bond strength. (Stainless UK)

Galvanized Steel (GS) Reinforcement

GS reinforcement is another alternative reinforcement that has been employed to increase the service life of bridge decks. Steel is galvanized using the hot-dip-galvanizing process in which steel is dipped into molten zinc at a temperature between 435 – 454^{o}C. The zinc reacts with the steel and oxygen forming a protective layer of zinc oxide. (American Galvanizers Association, 2006) The thickness of the zinc coating is generally between 100 and 150 μm (.0039 – 0059 in.). The threshold chloride concentration values for GS reinforcement are at least 2.5 times greater than that of carbon steel, which results in a time to corrosion initiation 4-5 times longer. The increase in threshold chloride concentration can be attributed to the fact that galvanized steel will stay passivated in concrete to pHs as low as 9.5 in comparison to bare steel, which depassivates at a pH of approximately 11.5. (Yeomans, 2004)

The mechanical properties of GS are comparable to carbon steel and the bond strength between the GS and concrete is also sufficient.

There are several limitations of GS reinforcement associated with its implementation. The first limitation is that if the zinc oxide coating is damaged during construction exposing the carbon steel, the exposed steel may corrode just as normal carbon steel would. However, it has been shown that the zinc layer surrounding damaged areas will corrode sacrificially to protect the base carbon steel. The cost of GS is also approximately 2.5 times that of carbon steel. (Bertolini et al, 2004) It is also important to note that in concrete with low chloride concentrations carbon steel will have a lower corrosion rate than GS, but in concrete with moderate chloride concentrations GS will increase the service life of the bridge deck. Additionally, GS reinforcement is not a viable option for bridge decks that will have high chloride concentrations being that if the chloride threshold of the GS is surpassed quickly, the sacrificial benefits of the zinc coating may only extend the corrosion propagation period of the reinforcing steel by approximately 5 years. (Bautista and Gonzalez, 1996) It should be noted, however, that the corrosion products of galvanized rebar are much less voluminous than those resulting from the corrosion of carbon steel (increase in volume of 150% as compared to 250%).

Additionally, it has been shown that the GS corrosion products are friable and that they "migrate away from reinforcement surface and fill cracks and voids in the cover concrete". (Yeomans, 1998) This not only relieves stresses at the reinforcement, but also densifies the concrete. The effects are that the concrete is much less likely to crack and the diffusion of chlorides into the concrete is inhibited, both of which will increase the service life of a bridge deck.

Epoxy-Coated Reinforcement (ECR)

ECR became widely used in the 1980's as a corrosion prevention method due to the relatively small additional expense in comparison with a substantial expected increase in service life. Many state agencies have reported good performance of ECR in bridge decks, but there have also been instances where substantial corrosion has been evident in as little as 5 years after construction. (Sagüés and Powers, 1997) It is believed that ECR protects against corrosion by inhibiting the penetration of oxygen, water, and chlorides from contacting the steel surface. The epoxy coating may also restrict the ability for a corroding bar to develop macro-cells by limiting the amount of exposed bare steel that can act as a cathode. ECR is fabricated using a fusion-bonding method consisting of four primary steps: 1) Bar surface preparation, 2) Heating of the bar, 3) Powder coating the bar, and 4) The curing and quenching of the bar. As the epoxy powder comes into contact with the heated bar it melts and covers the surface. The heat also initiates a cross-linking reaction that provides the epoxy coating with its' necessary physical and mechanical properties. (CRSI, 2006)

The epoxy coating adds approximately \$0.15/lb (\$0.33/kg) to the cost of the reinforcement, which is a minute amount of the total cost of the structure. However, a study of ECR bridge decks in Virginia concluded that the increase in service life for ECR bridge decks over bare steel decks may only be 5 years and that the "service life extension provided by ECR is not a cost-effective corrosion protection method for bridge decks in Virginia". (Brown and Weyers, 2003) The inability of ECR to provide a substantial increase in service life can be attributed to several problems. The problems associated with ECR include coating holidays developed during the fabrication process, potential for coating damage during transportation and construction, and debonding of the coating from the steel surface in moist concrete.

MMFX-2 Reinforcement

MMFX-2 is a relatively new reinforcement alternative. It is a proprietary micro-composite steel that contains chromium, which provides similar corrosion resistance to stainless steel. The resistance to corrosion of MMFX-2 is not as great as that of stainless steel, however the mechanical properties are far superior and the cost is significantly less. The improved mechanical properties of MMFX-2 steel may not significantly reduce the total amount of reinforcement required for bridge decks as the required amount of reinforcing steel is often controlled by temperature and shrinkage spacing requirements.

A recent study by VTRC recommends that MMFX-2 be considered for use in bridge decks as it "has shown promise as a cost-effective countermeasure for corrosion." (Clemena, 2003) Clemena reports that the chloride threshold value for MMFX-2 is in the range of 4.7-5.9 times that of A615 carbon steel. Another study conducted by the South Dakota Department of Transportation suggests that the threshold chloride concentration for MMFX-2 is 3.5 times higher than that of carbon steel and that the corrosion rate once corrosion has initiated can be as much as 2/3 lower. (Darwin et al, 2003) The increased chloride threshold for MMFX-2 has also been confirmed by research conducted by Trejo at the University of Texas A & M who reports threshold values of 7.7 pcy (4.6 kg/m^3) for MMFX-2 as compared to 0.9 pcy (0.5 kg/m^3) for carbon steel. (Trejo, 2002)

Corrosion Inhibitors (CI)

Various chemical admixtures referred to as corrosion inhibitors have been used to delay the onset of corrosion or to slow the corrosion rate of steel in concrete. CIs associated with preventing pitting corrosion can act: "a) by a competitive surface adsorption process of inhibitor and chloride ions b) by increasing and buffering of the pH in the local (pit) environment, and c) by competitive migration of inhibitor and chloride ions into the pit (so that the low pH and high chloride contents necessary to sustain pit growth cannot develop).” (Bertolini et al, 2004) These different CIs can target the anodic reaction site, the cathodic reaction site, or both the anodic and cathodic sites.

The effectiveness of CI’s that target the anodic reaction site depends largely upon using the correct dosage. If the CI concentration is too low the corrosion rate of the steel may actually be increased. This is because anodic CI’s are designed to reduce the area of the anodic reaction. Therefore, if some of the anodic area is removed the remaining anodic area will have a much higher cathode to anode ratio, which will in turn increase the rate of corrosion for that area. To determine the correct dosage it is necessary to accurately predict the level of chlorides that will be present within the concrete in the future. Based on the estimated chloride content, the dosage level can be calculated with a recommended CI to chloride ratio of 0.6. (Elsener, 2001)

Cathodic site CI’s will reduce the corrosion rate of steel regardless of the dosage level. This is because the anodic area will remain the same, so any reduction in the cathodic area will reduce the cathode to anode ratio, which will result in a lower corrosion rate.

There are two methods of application for CI’s: addition to the concrete mixture and surface application. CI’s are most commonly added to the concrete mixture during construction to ensure that there will be a sufficient concentration of the inhibitor at the depth of the reinforcement. Surface-applied CI’s are generally used to slow the corrosion rate of reinforced concrete structures that are already reflecting some corrosion. However, the effectiveness of surface applied CI’s is largely dependent upon the properties of the concrete. If the concrete has a low permeability the CI will not diffuse through the concrete and reach the depth of the reinforcement and no benefit may be realized. A report conducted by VTRC found that two surface applied corrosion inhibitors did not penetrate to the depth of the rebar. (Sharp, 2004) For this reason surface applied CI’s are rarely used.

The use of CI's in bridge deck patches and overlays has also been investigated by VTRC, but it was concluded that no "benefit from the use of corrosion inhibiting admixtures" was evident. (Sprinkel, 2003)

Problems associated with the use of CI's are: accuracy in determining appropriate dosage rates, leaching out of CI early on in the life of the structure, and the inability of surface applied inhibitors to penetrate to the reinforcement depth.

Improved Concrete Properties

In addition to the methods listed above one of the relatively easiest and effective methods used to delay the onset or prevent the corrosion of reinforcement in concrete is to improve the properties of the concrete. It is known that the characteristic of concrete that affects the chloride concentration at the reinforcement depth the most is the permeability of the concrete. The most common means of reducing the permeability of concrete is by reducing the $^{w}/_{c}$ ratio of the concrete mixture. In Virginia the $^{w}/_{c}$ ratio for concrete used in bridge decks has been reduced to 0.45 from 0.47. Additionally, mineral admixtures, commonly referred to as pozzolans, can be introduced into the concrete mixture to further reduce the permeability. The most frequently used pozzolans are fly ash (FA), and silica fume (SF). Pozzolans reduce the permeability of concrete by reacting with free calcium hydroxides present in the pore water to form cement products, which fill the concrete pore space. Ground granulated blast furnace slag (GGBFS), used in conjunction with Portland cement will also reduce the permeability of concrete.

VDOT specifies that low permeable concrete (LPC) be used for bridge deck applications. However, the effect of LPC on the service life of bridge decks is unclear. The reason for this is that LPC and ECR as well as an increase in the concrete clear cover depth were all implemented at approximately the same time. Therefore, the resulting increase in service life cannot be attributed to any one specification change. It is necessary to investigate the effect of LPC and an increased cover depth on bridge deck service life in order to determine if ECR is a major contributing factor.

Deck Repair Methods

Repair methods are used to temporarily extend the service life of a bridge deck without the expense of a major rehabilitation. Deck repairs can restore the deck to an acceptable level of functionality, but they do nothing with regards to addressing the cause of deterioration. At some point in the future a repaired deck will require major rehabilitation or replacement and the cause of deterioration must be dealt with at that time. The repair methods that will be discussed are patching and bridge deck overlays.

Patching

Deck patching is the primary method that is used to repair areas of deck deterioration. Deterioration is typically due to corrosion induced spalling and delaminations. Any degradation caused by the corrosion of the reinforcing steel will extend at least to the top mat of reinforcement and possibly to the bottom mat as well. Patching can either be partial-depth or full-depth depending upon the extent of the deterioration. Partial-depth repairs require that a minimum clearance of 0.75 in (19.05 mm). below the top mat of reinforcing steel be provided, but the total repair depth should not exceed half the thickness of the deck. If full-depth patches are required, the concrete will be removed in its entirety for the area of the deck that is to be patched. (Weyers, R.E. et al, 1993)

The materials that are used for deck patching are Portland Cement Concrete (PCC), hydraulic concretes and mortars, and polymer modified concretes and mortars.

Deck patches provide only a short-term solution to deterioration problems within a bridge deck. For deterioration related to corrosion, the underlying problem is not addressed and the reinforcement will continue to corrode in the surrounding deck area. The corrosion rate of the surrounding steel may actually increase as the patched area will contain very low chloride concentrations and will therefore become cathodic, which will in turn drive the adjacent actively corroding anodic sites. For this reason current research is investigating the possibility of introducing small cubes of zinc into patched areas that will act as sacrificial anodes, which will prevent the surrounding steel from continuing to corrode for a short period of time. However, after the zinc is consumed the reinforcement in the deck will once more become anodic and corrosion will progress just as before.

Deck Overlays

Deck overlays can be used either as a repair method or a rehabilitation method. When used as a repair method, overlays are placed on the surface of sound concrete that has not yet begun to deteriorate as a consequence of reinforcement corrosion. The reason overlays are placed is to decrease the permeability of the concrete and increase the cover-depth of the concrete, which in turn increases the distance that chloride ions must diffuse to reach the depth of the reinforcement. Overlays can be either latex modified concrete (LMC), low-slump dense concrete (LSDC), polymer concrete (epoxy), micro-silica concrete (MSC), or hot-mix asphalt concrete and membrane (HMAM) systems. Depending on the overlay type the service life of a bridge deck may be extended by over 25 years. (Weyers, R.E. et al, 1993) However, the increase in the dead load on the bridge will decrease the live load carrying capacity of the bridge. Additionally, if an overlay is placed on a bridge deck that already contains a critical level of chlorides present in the concrete there will only be a small extension to the service life of the deck due to a reduction in the moisture content of the underlying concrete deck.

Problems associated with bridge deck overlays are: debonding of the overlay from the deck, shrinkage cracking of the overlay, and determining the appropriate time to install an overlay on a deck. When specified as a repair or corrosion prevention method, the most commonly used overlay type in Virginia is multiple layer polymer concrete, which will significantly reduce the permeability of the concrete and improve skid resistance and surface apperance. Additionally, the time required for installation is significantly less than for other overlay alternatives.

Deck Rehabilitation Methods

Deck rehabilitation methods are used to restore a bridge deck to an acceptable level of performance and to address the cause of deterioration. Deck overlays and cathodic protection will be discussed in this section.

Deck Overlays

Deck overlays when used as a repair method are placed over sound concrete. When a deck is already reflecting significant damage, those areas must be repaired before the deck is overlaid and this is considered to be rehabilitation. Those deck areas that are experiencing active corrosion, are deteriorated, or have a critical concentration of chlorides must be removed and replaced prior to the installation of the overlay. This addresses the cause of the deterioration, that being corrosion of the reinforcement. Future corrosion will be delayed due to the increase in the cover depth and the decrease in permeability of the deck. It has been estimated that deck overlays that are used to rehabilitate decks may provide up to 70 years of additional service life. (Weyers, R.E. et al, 1993)

Cathodic Protection

Cathodic protection differs from cathodic prevention in that it is used on structures that are already experiencing corrosion. For cathodic protection, titanium anodes are placed into the concrete and a current is introduced to reduce the corrosion potential to a level where corrosion is slowed or stopped. If the current is strong enough, the reinforcing steel will repassivate and corrosion will stop. Cathodic protection has proven to be an effective means of rehabilitating concrete structures with high chloride concentrations without having to conduct major rehabilitation. The problems associated with using cathodic protection are that incorporating the systems tends to be costly and constant monitoring of the systems is required.

Chloride Diffusion Models

As mentioned previously, the corrosion of bridge deck reinforcing steel is primarily associated with the diffusion of chlorides into the concrete. For that reason the service life estimation methods discussed will only address chloride-induced corrosion and not carbonation-induced corrosion.

Chloride diffusion models can be divided into two categories: Fickian and Finite Element. Fickian diffusion models, the most commonly used models, assume that chlorides diffuse into the concrete according to Fick's Second Law of Diffusion. Finite element diffusion models are based upon averaging the microscopic transport equations for moisture and ionic diffusion over a representative volume element. Finite element models are also capable of accounting for the effects of localized chemical reactions and chemical damage.

Fickian Diffusion

Fickian service life models are based upon the chloride transport properties of the concrete, surface chloride concentration, concrete clear cover, and chloride concentration thresholds. Service life is estimated using two distinct time periods, time to corrosion initiation and time for corrosion propagation. The time to corrosion initiation is determined using Fick's second law of diffusion, which estimates the time required for a specified threshold chloride concentration to be reached based upon the parameters listed above. For a slab with an infinite depth, constant chloride surface concentration, and constant diffusion coefficient, Fick's law will reduce to the following form (Crank, 1975):

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

Where: $C_{(x,t)}$ = chloride concentration at depth and time,
C_o= surface chloride concentration,
D_c= apparent diffusion coefficient,
t = time for diffusion,
x = depth, and
erf = statistical error function.

Fick's law is used to estimate the chloride concentration at a specified depth and time given the surface chloride concentration and concrete diffusion coefficient.

The assumption that bridge decks are of infinite depth is valid for a typical 8 in. (200 mm) deep deck because the chlorides will never diffuse through the entire depth of the deck over the life span of the structure. To use Equation 1 shown above the diffusion coefficient (D_c) and the surface chloride concentration (C_o) must be taken as constants over time, however, it is known that both D_c and C_o are time-dependent parameters. The effects of their time-dependent nature and the methods that are used to account for them are discussed in the following two sections.

Effect of Time-Dependent Diffusion Coefficients

As mentioned previously, the diffusion coefficient used to estimate the service life of a bridge deck is a time-dependent parameter. Work by Bamforth (1999) and Mangat and Molloy (1994) clearly indicate that the rate of chloride ingress into concrete reduces with time. The rate of reduction will differ between bridge decks depending upon the composition of the concrete as well as environmental conditions.

The most commonly cited reason for a reduction in D_c is continued hydration of the cement. Pozzolanic reactions within the concrete m atrix will also result in a reduction in D_c. Additionally, it has been speculated that the presence of insoluble salts at the surface of the concrete will slow the ingress of chlorides by acting as a physical barrier. All of the mechanisms listed manifest themselves within concrete differently, however, they all result in a decrease in D_c by constricting the capillary pore system. While the exact chemical and physical phenomena that precipitate changes in D_c are not fully understood, they can be reasonably modeled from empirical data.

It has been shown that the initial D_c for a concrete mixture is relatively high, but will reduce rapidly reaching a near steady state in 5-10 years. The magnitude and rate of reduction are dependent primarily upon the $^w/_c$ ratio of the concrete, amount and type of pozzolan present, and the rate of hydration. The most effective method available to reduce D_c is the addition of pozzolan to the concrete mixture. Pozzolans act as secondary cementitious materials, which react to form hydration products after the initial hydration of the cement. The pozzolanic hydration products fill the capillary pore space resulting in a less permeable concrete.

The effect of the time-dependency of D_c will vary subject to the magnitude of the reduction and the time of first exposure to chlorides. Clearly if a bridge deck is exposed to deicing salts at a very young age (<3 months), the effect on service life estimations will be much greater than if the deck is first exposed at an age greater than 6 months. In order to accurately model the effective diffusion coefficient it is necessary to know the age at which the deck will be first exposed. This, of course, is difficult to predict and impossible to know for bridges that are already in place. A more reasonable approach to estimating the service life for bridges that are in place is to use the apparent diffusion coefficient (D_{ca}) at some specified time, for example, at 10 or 20 years. Several models are available that can be used to estimate D_{ca} for a given bridge deck and they will be presented in the Fickian Diffusion Models section.

Effect of Time-Dependent Surface Chloride Concentrations

It is known that the chloride concentration at the surface (C_o) of a bridge deck will increase over a period of time and eventually become relatively constant at some maximum value. This is evident in data collected by Weyers et al. for 15 bridge decks in the snow-belt region over a 15-year period. (1994) However, most researchers use a constant C_o to predict the service lives of bridge decks. Kassir and Ghosn completed a research project that investigated the effect of a time-varying C_o and found that in some cases the difference in predicted service lives can be as much as 100%. (2002) The influence was modeled by incorporating an exponentially time-varying C_o term into the standard Fickian equation.

While it is true that some cases had differences in predicted services lives of up to 100%, for a typical Virginia bridge deck the differences are expected to be in the range of 5-15%. These differences are relatively small when taken in context of a 100-year service life. Additionally, these calculations do not take into account the effect of a time-dependent diffusion coefficient, which will counteract the effects of time-dependent C_o values. In order to accurately predict bridge deck service lives using these parameters it is imperative that the age of the deck be known at the time of first salt application and also the nature of the time-dependency of the specific concrete mixture. For that reason this research will disregard the effect of time-varying C_o values and will only investigate the effects of time-varying D_c values to select an appropriate time for the measuring of apparent D_c.

Effect of Pozzolans

A pozzolan is defined as a "siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitous properties." (ASTM) Pozzolans are used as a partial replacement for cement in a concrete mixture to increase strength and decrease the permeability of the resultant concrete. The improved properties of concrete containing pozzolans are attributable to the mechanism in which pozzolans react within the concrete mixture. Pozzolans react with calcium hydroxide, a by-product of the hydration of cement present within the concrete pore water, to form additional cementitous materials. These materials further densify the concrete matrix resulting in higher compressive strengths and lower permeabilities. Additionally, the reaction consumes much of the available alkali, which reduces the potential for alkali-aggregate reactivity.

Pozzolans that are currently in use can be either naturally occurring or manufactured. Some examples of natural pozzolan are volcanic ash, tuff, clays and shales. Typical manufactured pozzolans are fly ash, microsilica (silica fume), and ground-granulated-blast-furnace slag (GGBFS). These materials are by-products from coal burning power plants, silicon metal production, and iron ore reduction, respectively.

The use of pozzolans in concrete not only provides higher quality concrete, but also has economic and environmental benefits as well. However, care should be taken when introducing a pozzolan into a concrete mixture as water demand, setting time, and early age strength can be adversely affected. The effects are due primarily to differences between the fineness of cement and pozzolan.

Pozzolans are currently used in the construction of bridge decks to provide a low permeable concrete. The reduced permeability of the concrete will slow the ingress of chlorides into the concrete resulting in the delayed onset of reinforcement corrosion. The permeability of concrete containing pozzolan will continue to decrease with time just as does Ordinary Portland Cement concrete, however, it will do so at a faster rate beyond early ages. It should be noted that in order to obtain the beneficial effects of using pozzolans, it is necessary to maintain high levels of quality control as increases in the $^{w}/_{c}$ ratio can negate the positive effects of the pozzolans.

Effect of Chloride Binding

It has been well documented that chlorides can be bound either physically or chemically to hydration products. (Tang and Nilsson, 1993; Martín-Pérez et al., 2000) As free chlorides are bound to hydration products the amount of chlorides available to induce corrosion of the reinforcement is reduced. This last point is somewhat controversial; as it has been argued that as the free chloride concentration is reduced by the corrosion reaction the bound chlorides will be liberated and become available to continue the reaction. Tang and Nilsson (1993) proposed that some chlorides are "irreversibly combined into hydrated products by chemical reaction, and others can unbind as the free chloride concentration decreases." This phenomenon is still not entirely understood and alterations to available chloride diffusion models based upon chloride binding should be done so with caution.

Chloride binding is typically modeled by using one of three isotherms (Martín-Pérez et al., 2000): Linear, Langmuir, or Freundlich. These isotherms are presented below.

Linear isotherm: $C_b = \alpha C_f$ Equation 2

where: C_b = Concentration of bound chlorides (kg/m^3 of concrete)

α = Slope of the line

C_f = Concentration of free chlorides (kg/m^3 of pore solution)

Langmuir isotherm: $C_b = \dfrac{\alpha C_f}{1 + \beta C_f}$ Equation 3

where: α, β = Binding constants that vary depending upon the binder composition

Freundlich isotherm: $C_b = \alpha C_f{}^{\beta}$ Equation 4

The linear isotherm is the most simplistic and most convenient to use, but does not reflect the non-linear binding properties of the cement paste. It will overestimate the bound chlorides at high free concentrations and underestimate them at low concentrations. (Martín-Pérez et al., 2000) Realizing the non-linear nature of the problem some still believe that a linear isotherm is an adequate representation of the phenomenon. (Sergi et al., 1992)

Tang and Nilsson (1993) have shown that the Langmuir isotherm best describes the relationship between the bound chloride and the free chloride for low free-chloride concentrations (< 0.05 mol/l) and the Freundlich isotherm is better suited for high free-chloride concentrations (> 0.01 mol/l)

The problem associated with incorporating chloride binding into diffusion models is that it is necessary to experimentally determine the isotherm chloride binding constants for every concrete mixture that is to be investigated. This can be a difficult and time-consuming process and values can vary greatly depending upon the $^{w}/_{c}$ ratio, pozzolan content, and C_3A content of the concrete.

For this project the effects of chloride binding will be neglected. The chloride initiation concentrations from the literature are based upon total chloride concentrations less a background concentration. Therefore, in order to relate chloride measurements from this project to reported values the total chloride concentrations must be used. The background chloride concentrations to be subtracted out will be determined by analyzing the chloride concentrations in the concrete below the reinforcing steel. The chlorides found at this depth will be taken to be irreversibly bound either within the cement paste or the aggregate.

Fickian Diffusion Models

Cady and Weyers (1983)

Cady and Weyers developed a service life model for bridge decks based upon chloride-induced corrosion of the reinforcing steel. The ingress of chlorides into the concrete is taken to be primarily the result of Fickian diffusion and moisture flow through subsidence cracks. For areas of the deck where subsidence cracking is present, corrosion of the reinforcement is taken to be instantaneous. Chloride ingress in uncracked areas is estimated using Fick's 2nd Law of Diffusion as presented earlier.

Empirical data for C_o, $C_{x,t}$, D_c, and x were used as input to develop service life estimates for a typical 0.45 $^w/_c$, OPC bridge deck. (Page et al, 1981; Goto and Roy, 1981; Clear, 1975; Clear, 1976) Subsidence cracking is dependent upon the cover depth with smaller cover depths resulting in more cracking. The extent of cracking was estimated using cover depth data gathered in a previous study and an assumed 2 in. (50 mm) slump. (Dakhil et al., 1975) Using that data, the initial subsidence cracking for a typical bridge deck was estimated to be 4.07%. Therefore, 4.07% of a bridge deck is expected to initiate corrosion upon the first exposure of the deck to chlorides and subsequent corrosion initiation is estimated by Fickian diffusion.

The modeling was an early attempt at estimating the service lives of bridge decks and is limited in scope. Service life estimates made using this model assume that subsidence cracking will occur on all bridge decks to some degree regardless of concrete quality and that concrete diffusion properties are constant. Additionally, the model is deterministic and therefore is not capable of modeling the effects of input data variability. Since its' conception many modifications to this model have been proposed to better estimate bridge deck service life and are presented below.

Mangat and Molloy

Mangat and Molloy (1994) developed a diffusion model that accounts for time-dependent diffusion. The model approximates the relationship between D_c and time with the following empirical relationship:

$$D_c = D_i t^{-m} \qquad \text{Equation 5}$$

where:

D_c = Effective diffusion coefficient at time t (cm^2/s)

D_i = Effective diffusion coefficient at time t equal to 1 second

t = Time (s)

m = Empirical coefficient that varies with mixture proportions

In order to estimate D_i for a particular concrete mixture a regression analysis must be performed on empirical data fitted to a log-log plot. D_i is then projected based upon that analysis and m is taken to be equal to the slope of the regression line. It was also found that m can be estimated if the $^w/_c$ ratio of the mix is known using the following equation:

$$m = 2.5\left(\frac{w}{c}\right) - 0.6 \qquad \text{Equation 6}$$

The corresponding closed form solution to the diffusion problem is then:

$$C_{(x,t)} = C_o\left[1 - erf\left(\frac{x}{2\sqrt{\frac{D_i}{1-m}t^{(1-m)}}}\right)\right] \qquad \text{Equation 7}$$

This equation is valid only when the initial diffusion coefficient is taken at t = 1 second and the age at the time of first exposure to chlorides is also taken to be at t = 1 second.

Maheshwaran and Sanjayan

A more general form of Mangat and Molloy's solution was presented by Maheswaran and Sanjayan (2004) to allow for the use of different times of chloride application and D_c measurement. Their solution is presented below:

$$C_{(x,t)} = C_o \left[1 - erf\left(\frac{x}{2\sqrt{\frac{D_{ref}(t_{ref})^m}{1-m}\left[(t)^{1-m} - (t_i)^{1-m}\right]}} \right) \right] \qquad \text{Equation 8}$$

where:

D_{ref} = Diffusion coefficient measured at time t_{ref}

t_i = Age at first exposure to chloride

The solution presented in Equation 8 uses a general form of the D_c equation developed by Mangat and Molloy, which was presented by Nokken et al (2006). That equation is as follows:

$$D(t) = D_{ref}\left(\frac{t_{ref}}{t}\right)^m \qquad \text{Equation 9}$$

The empirical coefficient, m, depends to a great extent upon the composition of the concrete mixture as well as upon the age at which the concrete is first exposed to chlorides. Values have been determined for m for specific concrete mixtures at specific exposure times, but their applicability to other mixtures and exposure conditions is questionable. More research is necessary in order to specify appropriate values for m.

Bamforth

Bamforth (1999) accounted for a time-dependent D_c in a similar fashion basing the D_c predictions on the value at 1 year using the following equation:

$$D_{ca} = at^n \qquad \text{Equation 10}$$

where:

D_{ca} = Apparent diffusion coefficient

a = D_{ca} at t = 1 year

n = Empirical constant

The calculated apparent diffusion coefficient was then implemented into the closed form solution as presented below:

$$C_{(x,t)} = C_o\left(1 - erf\left(\frac{x}{2\sqrt{\left[D_{ca(t_m)}\left(\frac{t}{t_m}\right)^n t\right]}}\right)\right) \qquad \text{Equation 11}$$

where:

$D_{ca(tm)}$ = Apparent diffusion coefficient measured at time t_m

This approach is similar to that of Mangat and Molloy, however, Bamforth presents values for n as they relate specifically to the diffusion coefficient at an age of 1 year.

Limitations of Proposed Models

Mangat and Molloy's model is limited in that it requires the diffusion coefficient at time equal to 1 second as an input. To estimate D_i, multiple measurements of D_c must be made on a given mixture over a period of time and a regression analysis performed, or m must be estimated and D_i projected based upon one D_c measurement. Making numerous D_c measurements is not feasible and estimates of m may not be accurate. Additionally, the model assumes that the first exposure to chlorides occurs at t = 1 second, which will result in the overestimation of chloride concentrations.

Maheswaran and Sanjayan addressed several of the issues with Mangat and Molloy's model. They adapted the model to allow for a reference D_c to be used in place of D_i and also accounted for the possibility of first chloride exposure occurring at times other than t = 1 second. However, estimating an appropriate value for m still remains difficult.

Bamforth's model uses an apparent D_c (D_{ca}) value, which is an approximation of D_c, a time-dependent variable. Maheswaran and Sanjayan (2004) demonstrated numerically that the empirical constant n used in Bamforth's model is a function of time and cannot be used to accurately estimate a D_{ca} value.

It is also important to note that the rate at which D_c will reduce is highly dependent upon mixture proportions. The rate is affected by $^{w}/_{c}$ ratio, pozzolan content, and curing conditions. It is clear that due to the high variability in mixture proportioning and curing conditions that estimating appropriate parameters for calculating a time-dependent D_c remains a difficult task. Therefore, this project will first attempt to model the diffusion of chlorides using Fick's second law of diffusion with a constant apparent D_c value determined from field measurements. The apparent D_c value that is used will be the value determined at the time of sampling, the apparent D_c's are to be considered constant values normalized over the lifespan of the structure.

Finite Element Method (FEM) Modeling of Diffusion

The Finite Element Method is a numerical analysis technique used to approximate the solution to a set of partial differential equations. It has many applications in engineering including the analysis of structural, thermal, and fluid systems. In recent years FEM has also been applied to diffusion problems, specifically the diffusion of chlorides into concrete.

FE modeling of chloride diffusion is a formidable task due to the inherent complexities of the diffusion process. The parameters that affect chloride diffusion that can be incorporated into a FE model include: the ionic fluxes of the various ionic species present within the concrete pore water, moisture flux, heat transfer, chemical reactions, and chemical damage.

The diffusion of chlorides has been described using simple Fickian behaviour, but it is known that the rate of chloride diffusion is affected by the prescence of other ionic species within the pore water. To model the diffusion of chlorides, the fluxes of each ionic species must be accounted for, as chlorides are charged particles that are affected by electrical forces including those created by other ions. These forces can result in either the increasing or decreasing of chloride diffusion.

The exposure conditions of a bridge deck require that changes in moisture and heat transfer be accounted for, as the moisture content of the concrete and ambient temperature will change over the course of a year. To model the time-dependent nature of these parameters, it is necessary to use the Finite Difference Method (FDM). The FDM is a numerical analysis technique used to approximate the differential equation rather than its' solution as FEM would. FDM is less robust and is not capable of handling complex geometries. However, due to the simple geometry of the assumed one-dimensional diffusion problem and the ease of FDM incorporation, it is commonly used to model time-dependent parameters.

The use of FEM has also made it possible to model the effects of chemical reactions and chemical damage. The chemical reactions of concern are those relating to the binding of chlorides and those that affect the pore structure of the concrete.

While the use of FEM to estimate the diffusion of chlorides into concrete may provide much more detail and may account for many more parameters than those investigated in Fickian models, being able to account for the statistical nature of the input parameters remains a problem. Additionally, the required input data for a FEM model demands a much higher level of testing on the concrete, which is time consuming and costly.

FEM Diffusion Models

Saetta, Scotta, and Vitaliani (1993)

Saetta et al. developed a numerical procedure based upon the FEM to solve a set of nonlinear equations that describe the movement of chlorides through partially saturated concrete. The model takes into account the effects of a time-dependent chloride diffusion coefficient, variable temperature, variable humidity, degree of cement hydration, and moisture flux. The results from the numerical model were validated based upon laboratory testing results from Collepardi and Biagini (1989), which simulated concrete in a tidal splash zone. The contribution of the moisture flux to the apparent chloride diffusion coefficient is significant for the case of concrete in tidal zones where the concrete is subjected to frequent wetting and drying cycles. The applicability of the model to bridge decks has yet to be verified.

Boddy, Bentz, Thomas, and Hooton (1999)

Boddy et al. developed a PC-based chloride transport model that accounts for the following parameters:

- "initial chloride profile due to sorption or prior history,
- initial diffusion value (D) for concrete,
- time-dependent reduction of D,
- nonlinear chloride binding isotherms,
- superposition of a hydraulic head on an external salt water environment,
- varying surface concentration with time
- monthly variations in temperature during a year, and
- chloride build up due to wicking (evaporation from inside face)."

Chloride transport by diffusion and convection was modeled using the following equation:

$$\frac{\partial C}{\partial t} = D \cdot \frac{\partial^2 C}{\partial x^2} - \bar{v} \cdot \frac{\partial C}{\partial x} + \frac{\rho}{n} \cdot \frac{\partial S}{\partial t} \qquad \text{Equation 12}$$

where:

C = "free" chloride in solution at depth x after time t

S = "bound" chloride

D = diffusion coefficient

ρ = concrete density

n = porosity

v = average linear velocity = $\dfrac{Q}{nA} = -\dfrac{k}{n} \cdot \dfrac{dh}{dx}$

Q = flow rate

A = cross-sectional area

k = hydraulic conductivity

h = hydraulic head

The above equation can be explained simply as the rate of change in chloride concentration with respect to time is equal to the summation of the effects of chloride diffusion, convective flow, and chloride binding.

The input parameters that are required by the program are as follows:

- "m, constant dependent on mix proportions, which controls the rate of reduction of diffusivity,
- T, absolute temperature of exposure for structure,
- C_{crit}, threshold concentration or critical chloride level required to initiate corrosion of steel,
- D_{ref}, diffusion coefficient at some reference time t_{ref} and temerature T_{ref},
- k_{ref}, permeability coefficient at some reference time t_{ref},
- C_o, surface concentration, and
- depth of cover"

The proposed diffusion model relies on several basic assumptions used to simplify the diffusion process. The assumption that the concrete is fully saturated is the most notable. By making this assumption, the model does not incorporate the effect that capillary absorption has on the transport of chlorides. In addition to this D_c is calculated using Fick's law instead of taking into account the influences of opposing ionic fluxes.

Another important aspect to note is that the model is highly sensitive to the input parameter m. The value for m that is used is selected based upon the concrete mixture composition. To date, there is not adequate research to determine appropriate values of m for specific mixture proportions with confidence. A sensitivity analysis was conducted and predicted changes in service lives of up to 550% with only moderate variations in m were observed. Because of this there is certainly some question as to the ability of this model to accurately estimate the service lives of concrete structures. Additional testing to determine the effective diffusion rate and concrete permeability is also required in order to use this model.

Martín-Pérez, Pantazopoulou, and Thomas (2001)

Their model is an improvement upon the Boddy et al. model presented above in that the effects of moisture flux are included. Additionally, the model includes the diffusion of oxygen as a supplementary controlling factor once corrosion has initiated. However, as was the case in the Boddy model, the influence of ionic electrical coupling is not accounted for and the model remains deterministic in nature.

The model uses a "numerical formulation of the associated mass transport partial differential equations" that is "solved in space using a finite element formulation and in time using a finite difference marching scheme."

This model has addressed several limitations that were evident in the Boddy model. However, given the case that a FE model is capable of accurately predicting the time to diffusion and corrosion for one set of input variables, it is still necessary to account for the variability in the input data. This is a problem that is not easily handled by FEM due to the complexity of the calculations required.

Commercially Available Models

STADIUM

STADIUM is a computer-based "numerical model that predicts the mechanisms of ionic transport in unsaturated cement systems." (Marchand, 2000) The model incorporates the effects of ionic diffusion, moisture transport, chemical reactions, and chemical damage. STADIUM is, to the author's knowledge, the only diffusion model currently available that is capable of modeling the electrical coupling between various ionic species as well as the chemical equilibrium reactions between the solid and liquid phases of the concrete matrix. STADIUM is primarily designed to model ionic flow within cement-based systems; however, there are other potential applications such as modeling ionic flow through soils and porous membranes.

The model is based upon determining the transport mechanisms at the microscopic scale and then averaging them over a Representative Elementary Volume (REV) using the homogenization technique. The REV must be of sufficient size to adequately model the pore space as well as the surrounding cement paste and is typically on the order of a few cubic centimeters in size. (Samson et al, 2005)

STADIUM is currently capable of estimating the electrical coupling between six ionic species those being: OH^-, Na^+, K^+, SO_4^{2-}, CA^{2+}, and Cl^-. In addition to modeling ionic electrical coupling, STADIUM incorporates the chemical equilibrium reaction equations for five different solid phases. Those phases, considered to be the most important, are Portlandite, Ettringite, Hydrogarnet, Friedel's salt, and Gypsum.

As might be expected, in order to model these complex equations extensive user input is required. A list of the necessary input data is presented below:

- Material density
- Paste content
- Diffusion coefficients
- Water diffusivity
- Total porosity
- Capillary porosity
- Initial values of ion concentration, volumetric water content in the pores, and electrical potential
- Initial amount of solid phases
- Equilibrium constants
- Boundary conditions for ion concentration, volumetric water content in the pores, and electrical potential

- Temperature

To acquire the input data a number of tests and calculations must be conducted including: determination of the initial composition of the material by evaluating the chemical make-up of the binder, degree of hydration, and mixture characteristics; determination of the initial composition of the pore water; determination of the initial total porosity using standard procedures such as ASTM C 642; calculation of the capillary porosity; migration tests to determine the diffusion coefficient of the material; and determination of the water diffusion coefficient by nuclear resonance imaging. (Marchand, 2000)

STADIUM is a robust diffusion model that accounts for many material and environmental factors that effect the diffusion of various ionic species within a cement based system. It has potential applications in many fields, however, due to the complexity of the model and the required user input, implentation for general engineering applications may pose some difficulty. When evaluating the feasibility of applying this model to the corrosion of reinforcing steel within a bridge deck the extensive testing required must be considered. It is also noted that incorporating the statistical nature of the input variables into this model may prove to be a difficult task.

Life-365 (Thomas and Bentz, 2000)

Life-365 is a computer-based model that was developed for the American Concrete Institue (ACI). The models' intended use is as a service life estimation and Life Cycle Cost Analysis (LCCA) tool for engineers for a wide variety of applications. Life-365 estimates the service life for a reinforced concrete structure using a four-step approach. The four steps are:

1) "Predicting the time to the onset of corrosion, commonly called the initiation period, t_i
2) Predicting the time for corrosion to reach an unacceptable level, commonly called the propagation time, t_p
3) Determining the repair schedule after first repair

4) Estimating life-cycle costs based on the initial concrete (and other protection) costs and future repair costs"

The time to the onset of corrosion is estimated using basic Fickian behaviour as described previously using a finite difference approach to account for the time-dependency of D_c and C_o. The propagation time is taken as a set value that is selected by the user.

To provide a user-friendly tool Life-365 uses a simplified approach to estimating the diffusion process. The model assumes that "ionic diffusion is the sole mechanism of chloride transport" and that the concrete is "completely saturated." By doing so, the amount of required input is significantly reduced. The input parameters that are required by the model are as follows:

- C_o – Surface concentration
- D – Diffusion coefficient
- m – Diffusion decay index
- C_t – Chloride threshold
- t_p – Propagation period
- Temperature

All of the input parameters can either be user defined or determined by Life-365 based upon concrete mixture proportions and geographic location. The model is not currently capable of integrating the variability of the input parameters and can be used only as a deterministic model. In order to make the program easier to use many assumptions and simplifications were made. It is recommended that the user take caution when accepting the default values provided by the program and that actual field data be used whenever available.

The authors proposed future improvements to the model and they are listed below:

- "Upgrade the propagation period portion of the model, including life-cycle costing analysis.

- Include probabilistic considerations in the model to account for the variability in input parameters and field construction conditions.
- Include additional corrosion protection products.
- Refine the model as appropriate as additional field verification data becomes available.” (Thomas and Bentz, 2000)

Corrosion-Induced Cracking of Concrete Cover

As described previously, the dissolution of iron due to corrosion and subsequent formation of corrosion products (hydrated ferric oxide) results in a volumetric expansion. As corrosion progresses and more and more corrosion products are formed, stresses are developed within the concrete structure. Once the tensile strength of the concrete that surrounds the reinforcement is surpassed it will crack as illustrated in Figure 2. Cracking of the cover concrete will ultimately result in the spalling or delamination of the deck surface.

Corrosion-cracking models have been developed to attempt to estimate the time required for corroding bare steel reinforcement to crack the cover concrete.

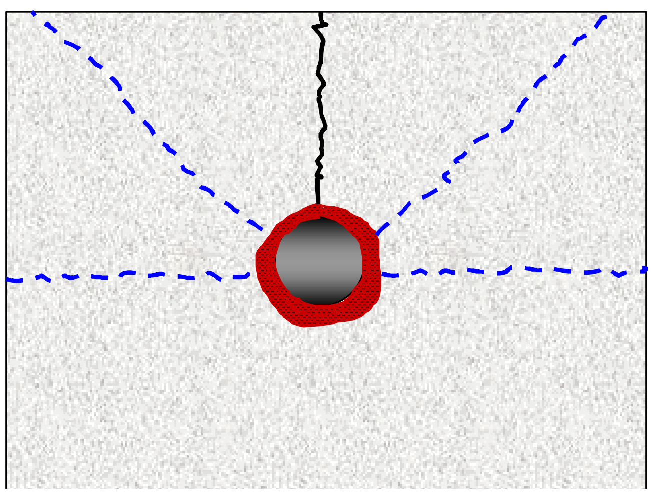


Figure 2 – Corrosion-Induced Cracking

Corrosion Cracking Models

Bažant's Model (Bažant, 1979)

Bažant developed a theoretical physical model for the corrosion of steel in concrete exposed to seawater based upon mathematical electrochemical relationships. In order to derive an appropriate model several assumptions were made and they are as follows:

- "Oxygen and chloride ion transport through the concrete cover (is) quasi-stationary and one-dimensional"
- Corrosion progresses in the steady-state
- The production of $Fe(OH)_3$ produces a volumetric expansion, but $Fe(OH)_2$ does not

After the theoretical model was developed a set of simplified equations was proposed to estimate the time to corrosion cracking for steel in concrete. These equations are presented below.

$$t_{cor} = \rho_{cor} \frac{D \Delta D}{s j_r} \qquad \text{Equation 13}$$

where: t_{cor} = Duration of steady-state corrosion

ρ_{cor} = Function of the densities of steel and rust ≈ 3.6 g/cm^2

D = Original bar diameter

ΔD = Change in bar diameter

s = Spacing of the bars

j_r = Rate of rust production per unit area of plane x = L

The cracking of the concrete cover can present itself in one of two primary modes, inclined cracks or cover peeling (delaminations). For structures where s > 6D the failure mode is assumed to be due to inclined cracks and where s < 6D the failure mode will be delamination. The critical value of ΔD for inclined cracks can be estimated using Equation 14.

$$\Delta D = 2 f'_t \frac{L}{D} \delta_{pp}$$ Equation 14

where: f'$_t$ = Tensile strength of concrete

L = Concrete cover thickness

δ_{pp} = Bar hole flexibility

The bar hole flexibility is taken to be the average of the calculated flexibility for a bar in a thick-wall cylinder and the flexibility for a bar in infinite space. The equations for both cases are presented below.

Thick-wall cylinder

$$\delta^o{}_{pp} = \frac{D}{E_{ef}}(1+\nu) + \frac{2D^3}{s^2 E_{ef}}$$ Equation 15

where: ν = Poisson's ratio

E_{ef} = Effective elsastic modulus = $\dfrac{E_c}{1+\Phi_{cr}}$

Φ_{cr} = Creep coefficient of the concrete

Infinite Space

$$\delta^1{}_{pp} = \frac{D}{E_{ef}}\left[1+\nu+\frac{D^2}{2L(L+D)}\right] + \frac{2D^3}{s^2 E_{ef}}$$ Equation 16

The equation for the alternative failure mode of delamination is:

$$\Delta D = f'_t \left(\frac{s}{D} - 1\right)\delta_{pp}$$ Equation 17

Bažant's model provides a useful theoretical basis for the understanding of corrosion in concrete, but it has never been validated experimentally. Additionally, research attempting to relate experimental data to Bažants's model has been unsuccessful. (Newhouse, 1993)

Liu/Weyers Model

The time to the cracking of concrete is modeled by estimating the amount of corrosion products necessary to produce stresses capable of fracturing the cement paste. The propagation time is dependent upon the corrosion rate, cover depth, bar spacing, bar size, and the mechanical properties of the concrete.

The corrosion-cracking model is composed of a four-step process:

1) Corrosion initiation,
2) Free expansion,
3) Stress initiation, and
4) Concrete cracking.

Once corrosion has initiated the corrosion products will begin to fill the porous zone present at the concrete/steel interface. The volume of the porous zone is dependent upon the "surface area of the reinforcement, $^{w}/_{c}$ ratio, degree of hydration, and degree of consolidation." (Liu and Weyers, 1998) The corrosion process will continue to progress until the porous zone has filled with corrosion products. At that time stresses will begin to increase within the concrete. When the amount of corrosion products increases to the point where the expansive forces created exceed the tensile strength of the concrete the concrete will crack.

The critical amount of corrosion products required to crack the concrete is estimated using Equation 18.

$$W_{crit} = \rho_{rust}\left(\pi\left[\frac{Cf'_t}{E_{ef}}\left(\frac{a^2+b^2}{b^2-a^2}+\nu_c\right)+d_0\right]D+\frac{W_{st}}{\rho_{st}}\right) \qquad \text{Equation 18}$$

where: W_{crit} = Critical amount of corrosion products (mg/mm)

ρ_{rust} = Density of the corrosion products (mg/mm^3)

C = Concrete cover (mm)

f'_t = Tensile strength of the concrete (Mpa)

E_{ef} = Effective elastic modulus of the concrete = $Ec/1+\varphi_{cr}$ (Mpa)

ν_c = Poisson's ratio of the concrete

d_o = Thickness of the pore band around the steel/concrete interface (mm)

D = Diameter of the reinforcement (mm)

W_{st} = Mass of steel corroded = αW_{crit}, α = 0.523-0.622

ρ_{st} = Density of steel (mg/mm^3)

W_{crit} is then used in conjunction with the rate of rust production (k_p) to estimate the time to cracking using Equations 19 and 20.

$$k_p = 0.098\left(\frac{1}{\alpha}\right)\pi D i_{corr} \qquad \text{Equation 19}$$

where: i_{corr} = Annual mean corrosion rate (mA/ft^2)

$$t_{cr} = \frac{W_{crit}^{\ \ 2}}{2k_p} \qquad \text{Equation 20}$$

where: t_{cr} = Time to cracking (years)

The model was validated experimentally by comparing observed times to cracking for concrete slabs subjected to outdoor exposure to the predicted times to cracking generated by the model.

Alonso, Andrade, Rodriguez, and Diez Model (Alonso et al, 1998)

Alonso et al. undertook a project with the aim of identifying the key parameters that control chloride-induced cracking in reinforced concrete structures. The parameters that were considered were the cover/reinforcement diameter ratio, proportions of the cement, $^{w}/_{c}$ ratio, cast position of the bar, and corrosion rate. The empirical model that was developed is based on Faraday's Law, which relates the depth of corrosion penetration to the corrosion rate as is presented in the following equation:

$$x = 0.0116 I_{corr} t \quad \text{Equation 21}$$

where: x = Depth of corrosion penetration (mm)

I_{corr} = Corrosion current density ($\mu A/cm^2$)

t = Time (years)

It was determined that the depth of corrosion penetration required to induce cracking in the cover concrete was related to the cover/reinforcement diameter ratio in the following way:

$$x_o = a + b\frac{C}{\phi} \quad \text{Equation 22}$$

where: x_o = Depth of corrosion penetration (μm)

a = Corrosion constant = 7.53

b = Corrosion constant = 9.32

C = Cover depth

ϕ = Diameter of the reinforcement

It was found that for C/ϕ ratios ≥ 2 a corrosion penetration depth of 50 μm (0.002 in.) was required to produce a crack with a width of 0.05 mm (0.002 in.) while a penetration depth of only 15-30 μm (0.0006 – 0.0012 in.) was required for C/ϕ ratios < 2. Additionally, for $C/\phi \geq 2$, 100-200 μm (0.0039 - 0.0079 in.) was required to produce a crack width of 0.3 mm (0.012 in.). Other crack widths can be interpolated.

It was also determined that higher corrosion rates will require greater depths of corrosion penetration to produce cracks than lower corrosion rates. This is important mainly in cases where accelerated experimental results are used to develop cracking models for actual structures. Another conclusion that was made was that concrete with higher $^{w}/_{c}$ ratios will propagate cracks slower than concrete with low $^{w}/_{c}$ ratios. This is related to the ability of corrosion products to diffuse into the concrete. Additionally, top bars tend to propagate cracks slower than bottom bars. The phenomena were observed and explained, however, no relationships were developed to quantify their affects.

Torres-Acosta/Sagüés Propagation Model (Torres-Acosta and Sagüés, 2004)

Most corrosion cracking models define the build-up of corrosion products as a uniform process over the length of the reinforcing bar. However, it is known that concrete reinforcement in bridge decks typically undergoes local (pitting) corrosion at isolated locations along the length of the bar. Torres-Acosta and Sagüés investigated the effect that the size of the anodic region had on the amount of corrosion necessary to crack the surrounding concrete. It was found that corrosion penetration depths of 0.03 – 0.272 mm (0.0012 – 0.011 in.) were required to crack the concrete for localized corrosion as compared to depths of 0.003 – 0.074 mm (0.00012 – 0.0029 in.) for uniform corrosion. From the experimental data, an empirical equation was developed to estimate the amount of corrosion required to induce cracking in the concrete, based upon the specimens' geometry. The developed equation is presented in Equation 23.

$$x_{crit} \approx 0.011\left(\frac{C}{\phi}\right)\left(\frac{C}{L}+1\right)^2 \qquad \text{Equation 23}$$

where: x_{crit} = Critical corrosion penetration (mm)

C = Cover depth (mm)

ϕ = Reinforcing bar diameter (mm)

L = Length of the anodic region (mm)

Vu, Stewart, and Mullard Model (Vu et al, 2005)

The corrosion propagation models discussed thus far are designed to estimate the time to crack initiation. Vu et al. realized that after cracking has initiated in a concrete structure, there is some additional service life prior to reaching a predetermined cracking limit state. The total service life would then be equal to the time to crack initiation plus the time to excessive cracking. For this model the time to crack initiation is estimated using the Liu and Weyers model presented previously. (Liu and Weyers, 1998) The time to excessive cracking is dependent upon the corrosion rate, concrete cover, $^{w}/_{c}$ ratio, and the maximum acceptable crack width. The model has currently been fitted to experimental data for three different crack width limit states: 0.3 mm (0.012 in.), 0.5 mm (0.02 in.), and 1.0 mm (0.039 in.). The model is also capable of estimating the time to excessive cracking from experimental specimens subjected to accelerated corrosion rates by applying a rate of loading factor (k_R). The empirical equations and coefficients that were developed are presented below:

$$t_{sp} \approx t_{1st} + k_R \times 0.0114 i_{corr} \left[A \left(\frac{C}{wc} \right)^B \right] \qquad \text{Equation 24}$$

where: t_{sp} = Time to excessive cracking for reinforced concrete structures (years)

t_{1st} = Time to crack initiation

k_R = Rate of loading correction factor

i_{corr} = Corrosion current density ($\mu A/cm^2$)

A, B = Constants based upon the crack width limit state selected

C = Cover depth (mm)

wc = Water to cement ratio

A and B are related to the limit crack width

Limit crack width	A	B
w_{lim} = 0.3 mm	65	0.45
w_{lim} = 0.5 mm	225	0.29
w_{lim} = 1.0 mm	700	0.23

$$k_R \approx 0.95\left[\exp\left(-\frac{0.3 i_{corr(\exp)}}{i_{corr(real)}}\right) - \frac{i_{corr(\exp)}}{2500 i_{corr(real)}} + 0.3\right]$$ Equation 25

where: $i_{corr(exp)}$ = The applied corrosion rate used during the experiment ($\mu A/cm^2$)

$i_{corr(real)}$ = The corrosion rate observed in the structure being analyzed

Equations 24 - 25 presented above are based upon the assumption that the corrosion rate is time-invariant, meaning that the corrosion rate will remain constant with time. However, the model can also be adjusted to incorporate a corrosion rate that is expected to be time-variant (increase or decrease with time). Vu and Stewart have suggested that the corrosion products that are formed during the corrosion process may result in the slowing of the corrosion rate. (Vu and Stewart, 2000) Therefore, it has been proposed that the estimated time to excessive cracking be modified by Equation 26 to account for a time-variant corrosion rate.

$$T_{sp} = \left[\frac{\beta+1}{\alpha} \times \left(t_{sp} - 1 + \frac{\alpha}{\beta+1}\right)\right]^{\frac{1}{\beta+1}}$$ Equation 26

(Valid for $t_{sp} > 1$ year and $w_{lim} \leq 1.0$ mm)

where: t_{sp} = Time to excessive cracking from the time-invariant corrosion rate model

α and β = Corrosion rate constants, recommended by Vu and Stewart to be 0.85 and –0.3, respectively

It should be noted that this model takes a general approach to determine the time to excessive cracking for reinforced concrete structures. This model does not account for the affect of repetitive loading on the crack propagation time period, as would be the case for a bridge deck. It may be difficult to directly correlate propagation times estimated from this model to propagation times observed for concrete bridge decks because of this.

Finite Element Analysis of Corrosion Induced Crack Propagation

Several attempts have been made to use commercially available finite element codes to model corrosion cracking in concrete structures. (Hansen and Saouma, 1999; Molina et al., 1993; Du et al., 2006) Either a radial expansion of the bar or an internal pressure is used to model the pressure on the surrounding concrete that is created by the formation of corrosion products. While the results are promising, a standard, quantitative, and efficient method has yet to be developed.

OBJECTIVE

The objectives of this project are as follows:

- Estimate the service life of bare steel bridge decks with a $^{w}/_{c}$ ratio of 0.45 or a $^{w}/_{(c+p)}$ of 0.45
- Estimate the service life of ECR and alternative reinforcement bridge decks with a $^{w}/_{c}$ ratio of 0.45 or a $^{w}/_{(c+p)}$ of 0.45
- Estimate the cost of maintaining these bridge decks for a service life of 100 years
- Evaluate the affect of using alternative reinforcements with a $^{w}/_{(c+p)}$ of 0.45 on service life cost

It is important for VDOT to be able to reasonably estimate the service lives of bridge decks containing bare, epoxy-coated, and alternative reinforcements in order to calculate the life cycle costs for the different alternatives. Currently, VDOT has a reasonable estimate of the service life that is obtainable for bare steel bridge decks with $^{w}/_{c}$ ratios of 0.47, but the affect of reducing the $^{w}/_{c}$ ratio to 0.45 on service life is not known. Additionally, bridge decks containing ECR have not been in service for a long enough period of time to know what their long-term performance will be. It is the objective of this project to develop estimates that will enable different corrosion prevention alternatives to be compared with regards to their ability to extend the service life of bridge decks and reduce total life cycle costs.

SCOPE

The scope of this project is limited to those corrosion prevention systems that are currently in place or being investigated in Virginia. These systems include bridge decks containing bare steel reinforcement and ECR with ${}^{w}/_{c}$ or ${}^{w}/_{(c+p)}$ ratios of 0.45 and 0.47. Additionally, the use of alternative reinforcements that are under consideration, such as MMFX-2, stainless steel, and galvanized steel, will be investigated. The practice of extending the service life of bridge decks needing repair and rehabilitation using Latex Modified Concrete (LMC), Microsilica Concrete (MSC), and Polymer Concrete (PC) overlays will also be addressed and incorporated into the life cycle cost analysis.

Bridge deck service lives will be estimated using a service life model based upon Fick's second law of diffusion described by Equation 1. To simplify the complex diffusion process and to provide conservative estimations, the surface chloride concentration and diffusion coefficient values will be determined from samples taken from in service decks, generally greater than 15 years of age. The model will take advantage of Monte Carlo simulation to provide more realistic results by taking into account the variability of surface and corrosion initiation chloride concentration values, cover depths, and diffusion coefficients.

METHODS AND MATERIALS

Bridge Deck Selection

The original research plan called for the survey of 40 bridge decks. The distribution of bridge deck types was to be as follows: 10 bare steel with $^{w}/_{c} = 0.47$, 15 ECR with $^{w}/_{c} =$ 0.45, and 15 ECR with $^{w}/_{(c+p)} = 0.45$. VDOT officials compiled a list of potential bridge decks that indicated whether or not pozzolans were used in the deck, the type of reinforcement present, the age of the structure, and if the bridge had been overlaid. It was important to select bridge decks that had not received overlays because an overlay would affect the chloride concentrations in the deck and skew the calculated diffusion coefficients. 40 bridges that met the research plan criteria were randomly selected from the VDOT prepared list. Additionally, the bridges selected for surveying were evenly dispersed across the 6 climatic zones in Virginia. The survey locations are presented in Figure 3.

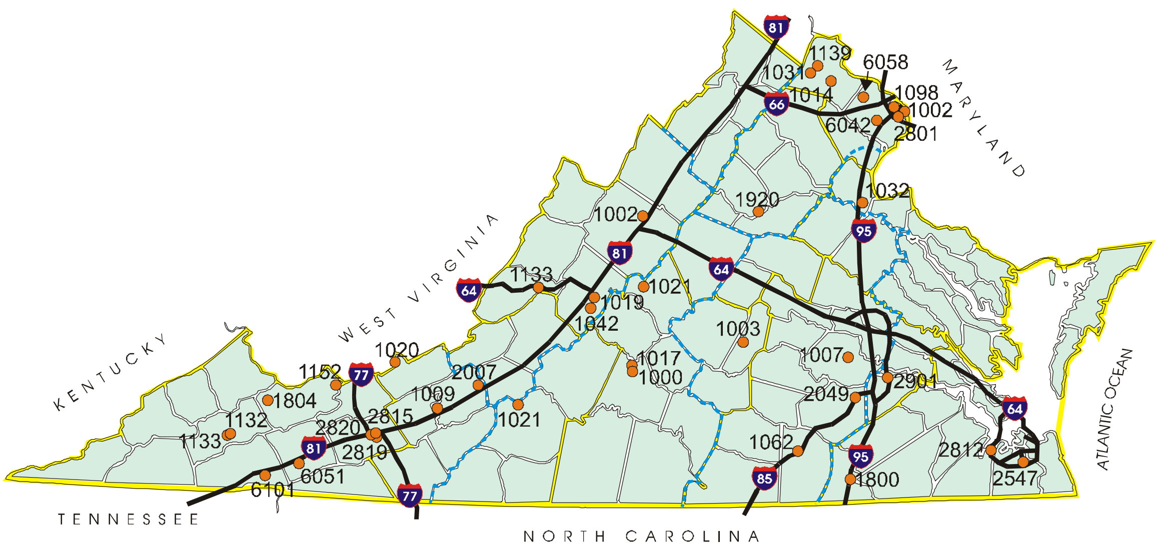


Figure 3 – Surveyed Bridge Locations

It was necessary to survey bridge decks from all six climatic zones because research conducted over a three-year period from 2001 to 2003 indicated that the chloride exposure for decks in different zones could vary substantially.[1] Results of that research are presented in Table 1. The corresponding climatic zones are illustrated in Figure 4.

Table 1 – Chloride Exposure by Climatic Zone

Zone #	Climatic Zone	Kg-Cl$^-$/lane-km (lb-Cl$^-$/lane-mile)
1	Southwestern Mountain (SM)	688 (2441)
2	Central Mountain (CM)	671 (2381)
3	Western Piedmont (WP)	220 (781)
4	Northern (N)	4369 (15501)
5	Eastern Piedmont (EP)	530 (1880)
6	Tidewater (TW)	225 (798)

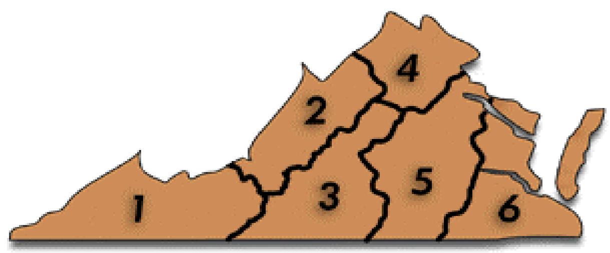


Figure 4 – Virginia Climatic Zones

Of the 40 bridge decks originally selected 37 were actually sampled. Three decks were not sampled due to research and traffic constraints. Core samples from the decks were used to confirm the composition of the concrete and it was found that several of the decks purported to contain pozzolan in actuality did not, and some of the decks that were not reported to contain pozzolans, did. The final distribution of deck types sampled was as follows: 10 bare steel with $^{w}/_{c} = 0.47$, 16 ECR with $^{w}/_{c} = 0.45$, and 11 ECR with $^{w}/_{(c+p)} = 0.45$. A list of bridge decks surveyed is presented in Table 2.

[1] Information obtained through personal communication with Dr. Richard E. Weyers (Professor of structural engineering materials – Virginia Polytechnic Institute) in January of 2004.

Table 2 – Bridges Surveyed

District	County	Structure Number	Year Built	Age at Survey (years)	Climatic Zone	Reinforcement Type	Specified Concrete
1	Tazewell	1804	1969	34	1 (SM)	Bare	0.47 w/c
1	Washington	6101	1969	34	1 (SM)	Bare	0.47 w/c
2	Montgomery	2007	1970	33	1 (SM)	Bare	0.47 w/c
3	Nelson	1021	1971	32	3 (WP)	Bare	0.47 w/c
4	Brunswick	1062	1969	34	5 (EP)	Bare	0.47 w/c
4	Dinwiddie	2049	1968	35	5 (EP)	Bare	0.47 w/c
5	Emporia	1800	1970	33	6 (TW)	Bare	0.47 w/c
6	Stafford	1032	1971	32	6 (TW)	Bare	0.47 w/c
9	Alexandria	2801	1970	33	4 (N)	Bare	0.47 w/c
9	Fairfax	6042	1969	34	4 (N)	Bare	0.47 w/c
1	Russell	1132	1988	15	1 (SM)	ECR	0.45 w/c
1	Russell	1133	1988	15	1 (SM)	ECR	0.45 w/c
1	Wytheville	2820	1986	17	1 (SM)	ECR	0.45 w/c
1	Smyth	6051	1990	13	1 (SM)	ECR	0.45 w/c
2	Giles	1020	1986	17	1 (SM)	ECR	0.45 w/c
3	Cumberland	1003	1988	15	5 (EP)	ECR	0.45 w/c
4	Chesterfield	1007	1990	13	5 (EP)	ECR	0.45 w/c
4	Prince George	2901	1991	12	5 (EP)	ECR	0.45 w/c
5	Cheasapeake	2547	1984	19	5 (EP)	ECR	0.45 w/c
7	Orange	1920	1991	12	4 (N)	ECR	0.45 w/c
8	Rockbridge	1019	1984	19	2 (CM)	ECR	0.45 w/c
8	Alleghany	1133	1987	16	2 (CM)	ECR	0.45 w/c
9	Loudoun	1014	1987	16	4 (N)	ECR	0.45 w/c
9	Loudoun	1031	1990	13	4 (N)	ECR	0.45 w/c
9	Arlington	1098	1988	15	4 (N)	ECR	0.45 w/c
9	Loudoun	1139	1987	16	4 (N)	ECR	0.45 w/c
1	Tazewell	1152	1987	16	1 (SM)	ECR	0.45 w/(c+p)
1	Wytheville	2815	1986	17	1 (SM)	ECR	0.45 w/(c+p)
1	Wytheville	2819	1986	17	1 (SM)	ECR	0.45 w/(c+p)
2	Franklin	1021	1988	15	3 (WP)	ECR	0.45 w/(c+p)
3	Campbell	1000	1991	12	3 (WP)	ECR	0.45 w/(c+p)
3	Campbell	1017	1990	13	3 (WP)	ECR	0.45 w/(c+p)
5	Suffolk	2812	1991	12	6 (TW)	ECR	0.45 w/(c+p)
8	Augusta	1002	1988	15	2 (CM)	ECR	0.45 w/(c+p)
8	Rockbridge	1042	1990	13	2 (CM)	ECR	0.45 w/(c+p)
9	Arlington	1002	1987	16	4 (N)	ECR	0.45 w/(c+p)
9	Fairfax	6058	1991	12	4 (N)	ECR	0.45 w/(c+p)

$^{w}/_{c}$ – water/cement ratio

$^{w}/_{(c+p)}$ – water/(cement + pozzolan) ratio

Deck Survey

Bridge deck rehabilitation decisions are based upon the deterioration of the worst-span-lane of the deck. The right-hand lane normally receives more traffic and therefore deteriorates at a faster rate. For that reason, and due to safety and traffic control issues, only the right-hand lanes were surveyed. The deck survey included a visual survey, non-destructive testing, and the collection of 15 - 102 mm (4 in.) concrete cores per deck.

Visual Survey

The following data was gathered for each bridge deck during the visual survey:

- The length and width of the right traffic lane were measured.
- Patched areas within the right-hand lane were measured and recorded.
- A visual determination of the wheel path locations was made.
- A crack survey was performed consisting of recording the number, length, width and orientation of the cracks in each span.
- Other structural data including the type of superstructure (i.e. steel, prestressed, and cast-in-place concrete), structure type (continuous or simply supported), and the girder spacing was recorded. Additionally, the presence of stay-in-place forms, the superelevation condition, and the skew angle of the deck were recorded.

Non-destructive Field Testing

The following non-destructive tests were conducted during the field survey:

- Cover depth determinations for the top mat of reinforcing steel. 40-80 measurements were taken per span at 4-foot (1.2 m) intervals in the wheel paths using a Profometer 3 cover depth meter. If the span length did not allow for 40 measurements to be taken at 4-foot (1.2 m) intervals the interval was reduced to 2 feet (0.6 m).
- The right-hand lane was sounded using the chain drag method to determine delaminated areas.
- Copper-copper half-cell (CSE) measurements were taken at the same locations as the cover depth measurements.
- Corrosion current densities were determined at 4-6 locations using an unguarded three-electrode linear polarization (3LP) instrument.
- Concrete resistivity measurements were taken at 9 locations using a four-probe Wenner apparatus.

Deck Cores and Laboratory Testing

The concrete cores were taken within the wheel paths on the deck as that is the critical deterioration area. Three of the cores were taken for petrographic analysis purposes and did not contain reinforcing steel. Of the remaining 12, all contained reinforcing steel and 3 were taken directly over cracks in the deck. After the cores were removed the water on the surface resulting from the coring process was allowed to dry off and then the cores were wrapped in two layers of 102 µm (4-mil) polyethylene and one layer of aluminum foil. The cores were then wrapped in a protective layer of duct tape. This was done to preserve the in-place moisture content of the concrete samples until they could be analyzed in the lab.

The following tests were conducted in the laboratory on the cores taken from the bridge decks:

- Chloride concentrations were determined in accordance to ASTM C 1152-97 at the following seven depths for each concrete core sample: 0.5 in. (12.7 mm), 0.75 in. (19.05 mm), 1.0 in. (25.4 mm), 1.25 in. (31.75 mm), 1.5 in. (38.1 mm), at the depth of the reinforcement, and below the reinforcement.
- Concrete resistivity measurements were taken at depths of 0.25 in. (6.35 mm), 0.75 in. (19.05 mm), 1.25 in. (31.75 mm), and below the reinforcement. They were taken both parallel and perpendicular to the reinforcing bar.
- Concrete permeability data were obtained in accordance to ASTM C 1202.
- Concrete density/absorption data were determined in accordance to ASTM C 642.
- The condition of the reinforcement was recorded including: the number of holidays (ECR only), the number of damaged areas (ECR only), coating thickness measurements (ECR only), peel tests (ECR only), and estimated % rusted surface area.
- Concrete moisture contents were determined for each core.

METHOD OF ANALYSIS

Chloride titration data for D_c's and C_o values, cover depth measurements, and the deck damage survey will be the data used in this study. The electrochemical, concrete permeability, and concrete moisture content measurements are not input data for the current service life model and therefore will not be considered. Their influence on the corrosion of reinforcing steel in concrete is presently being investigated in a separate study.

Service Life Model

This project estimates the service life of a bridge deck using three distinct time periods (see Figure 5).

1) Time to corrosion initiation,
2) Time from initiation to cracking, and
3) Time for corrosion damage to propagate to a limit state.

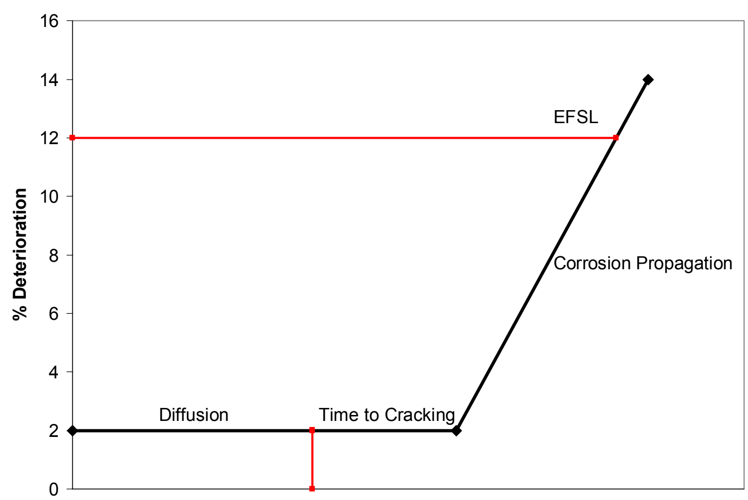


Figure 5 – Service Life Model

The time period for corrosion initiation in this study is the time required for the chloride initiation concentration to be reached at 2% of the bridge deck reinforcing steel. The initiation of corrosion on up to 2% of the bridge deck can be modeled using basic Fickian diffusion behaviour. Making predictions beyond the 2% initiation level is potentially misleading due to the way that corrosion propagates throughout a bridge deck. As areas of reinforcement begin to actively corrode, other areas of reinforcement will become cathodic. Cathodic areas may remain cathodic even at high chloride concentrations. Additionally, corrosion at the anodic sites will eventually cause the concrete cover to crack, which will allow for the rapid ingress of chlorides. This increase in chlorides at the corroding site will accelerate corrosion in that area and in adjacent areas.

Once corrosion has initiated on 2% of the reinforcing steel corrosion will progress until the cover concrete cracks and the deterioration becomes visually evident. The time to cracking will be estimated using the corrosion cracking models presented previously.

The total corrosion propagation time is the time required for corrosion deterioration to advance from a level of 2% to a level of 12%. A deterioration level of 12% connotes the End of Functional Service Life (EFSL) for a bridge deck as demonstrated by a survey conducted by Fitch et al. of DOT officials which concluded that bridge decks are most commonly rehabilitated at the 12% damage level of the worst span lane. (1995) Damage is defined as the sum of the patched, spalled, and delamninated areas.

Modeling the propagation of corrosion after initiation is a difficult task. As mentioned previously, once corrosion has initiated to a level of 2% subsequent initiation will no longer be dependent upon the Fickian diffusion of chlorides, but rather the complex electrochemical corrosion reactions and ion transport mechanisms. To estimate the corrosion propagation rate for Virginia bridge decks field data will be analyzed and an empirical equation developed.

The service life software that was used for this project, Bridge Corrosion Analysis (BCA), was previously developed by Liang et al. at Virginia Tech. (2001) BCA models the diffusion of chlorides using simple Fickian behaviour with time-independent input parameters. By doing so, the model simplifies the diffusion process to the extent that it can be easily used as a DOT bridge engineering and management tool. A Fickian based diffusion model was used rather than a more complex FEM model to allow for the easy incorporation of the stochastic nature of the input variables. BCA was developed as an Excel module, which was added to the standard Microsoft Office package.

Deterministic vs. Probabilistic Service Life Models

Service life models can be either deterministic or probabilistic. Deterministic service life models estimate the EFSL for a bridge deck using an average value for the various input parameters. Conversely, probabilistic models incorporate the stochastic nature of the variables into service life estimates. Previous research concerning the service life modeling of bridge decks has shown that a deterministic approach to the problem will result in a significant overestimation of service life. (Kirkpatrick, 2001) Thus, the software used in this project will make use of Monte Carlo statistical resampling techniques to allow for the integration of input parameter variability into the model.

Monte Carlo Simulation

Monte Carlo Simulation (MCS) is defined as "any method, which solves a problem by generating suitable random numbers and observing that fraction of the numbers obeying some property or properties." (Weisstein, 2006) MCS is used for complex problems that cannot be easily solved using standard deterministic models. It takes into account the statistical nature of the input parameters by randomly selecting numerical values from a provided data set (simple bootstrapping) or based upon a known distribution for each data set (parametric bootstrapping). As mentioned previously, the range of the input variables is defined by the data gathered for each individual bridge deck. It is also important to ensure that the number of sampling iterations used is adequate. For a small number of iterations (<20) the range of the expected service life for a bridge deck is quite large, but as the number of iterations increases the range will narrow to an acceptable level. It has been shown that for 10,000 iterations the service life prediction will converge to a near constant value when the distributions of the input variables are adequately defined. (Kirkpatrick, 2001)

Parametric Bootstrapping

Parametric bootstrapping is the process by which numerical values are randomly generated for a given variable based upon a provided range of values and distribution. For example, cover depth measurements have been shown to be normally distributed. Therefore, the range of possible values for cover depths can be defined by the mean and standard deviation of the cover depth data set. Once the distribution has been identified the service life model will randomly generate values for the cover depth based on the developed variable parameters.

Simple Bootstrapping

In contrast to parametric bootstrapping, simple bootstrapping will only sample values from a provided data set and will not incorporate any known or assumed distribution. Simple bootstrapping is useful in situations where large quantities of data (> 15 data points) are available, but if the data set is fairly limited simple bootstrapping may not accurately reflect the variability actually present.

Modeling the Time to Corrosion Initiation

The first step in estimating the time to corrosion initiation is calculating D_{ca} from the chloride profiles developed from the bridge deck samples.

Apparent Diffusion Coefficient (D_{ca}) Calculation

D_{ca} is calculated using Fick's second law of diffusion as described by Equation 1. The diffusion portion of the BCA service life model was validated by comparing the results with two other diffusion programs. (Zemajtis, 1998 and Sagues[2])

The diffusion model back calculates D_{ca} using chloride concentration data obtained from the concrete samples taken from the bridge decks. The surface chloride concentration is one of the most important factors in calculating D_{ca}. It has been shown that the chloride concentration profiles for bridge deck cores reach a maximum at a depth of approximately ½ in. (12.7 mm) during the summer after 5 – 10 years of exposure to

[2] Information obtained through personal communication with Dr. Alberto A. Sagues (Professor of Civil Engineering - University of South Florida) in August of 2005.

deicing salts, which was the time of the sampling period for this project. (Weyers et al., 1992), due to the propensity for chlorides to be washed out of the surface of the deck resulting in lower concentrations at the surface than at a depth just below the surface. Thus, it is necessary to use the chloride concentration at a depth of ½ in. (12.7 mm) as the driving surface chloride concentration (C_o). The variability of chloride profiles throughout the course of a year is illustrated in Figure 6 below.

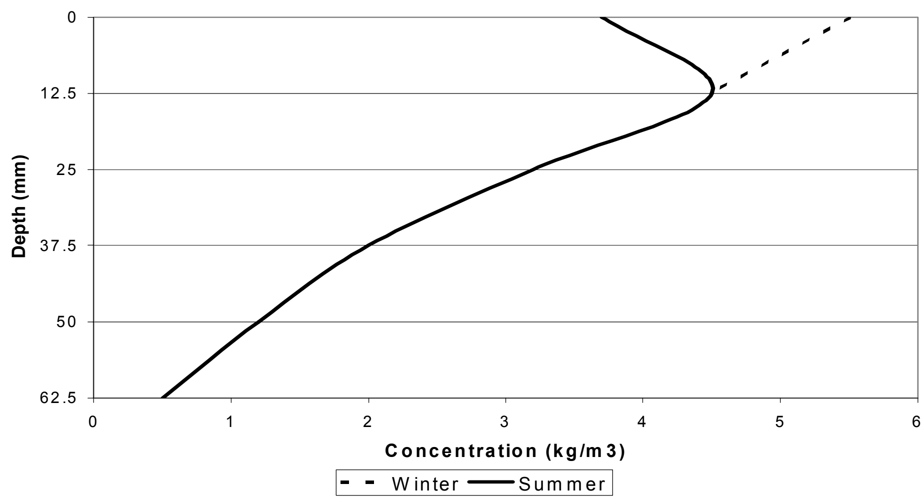


Figure 6 – Seasonal Fluctuations in Chloride Profiles

Using the chloride profiles, concentrations at various depths determined from the laboratory data, D_{ca} can be calculated using a least sum of the squared error curve fitting analysis of Equation 1 as illustrated in Figure 7 below. The chloride profiles represent the measured chloride concentrations less a predetermined background chloride concentration. The process of determining the background chloride concentration is discussed in the following section. The calculated apparent diffusion coefficients are used as input to the service life model. The measured surface concentrations and cover depths are also used to define the sample set of their corresponding model variables.

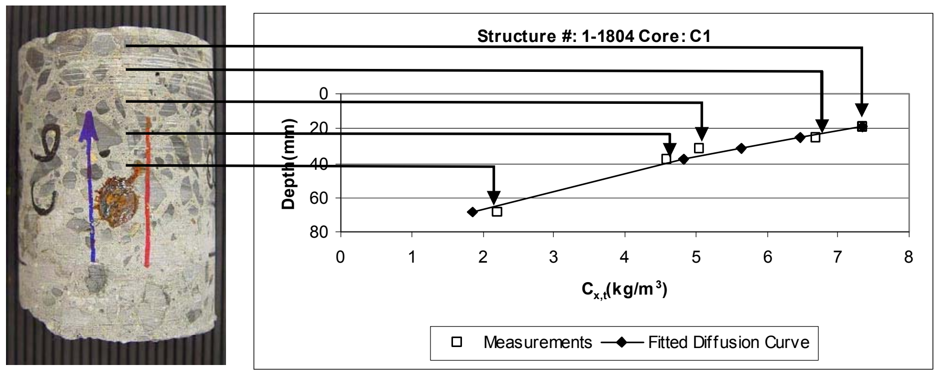


Figure 7 – Determination of Apparent Diffusion Coefficient

The Graphical User Interface (GUI) used by BCA to calculate D_{ca} is presented below.

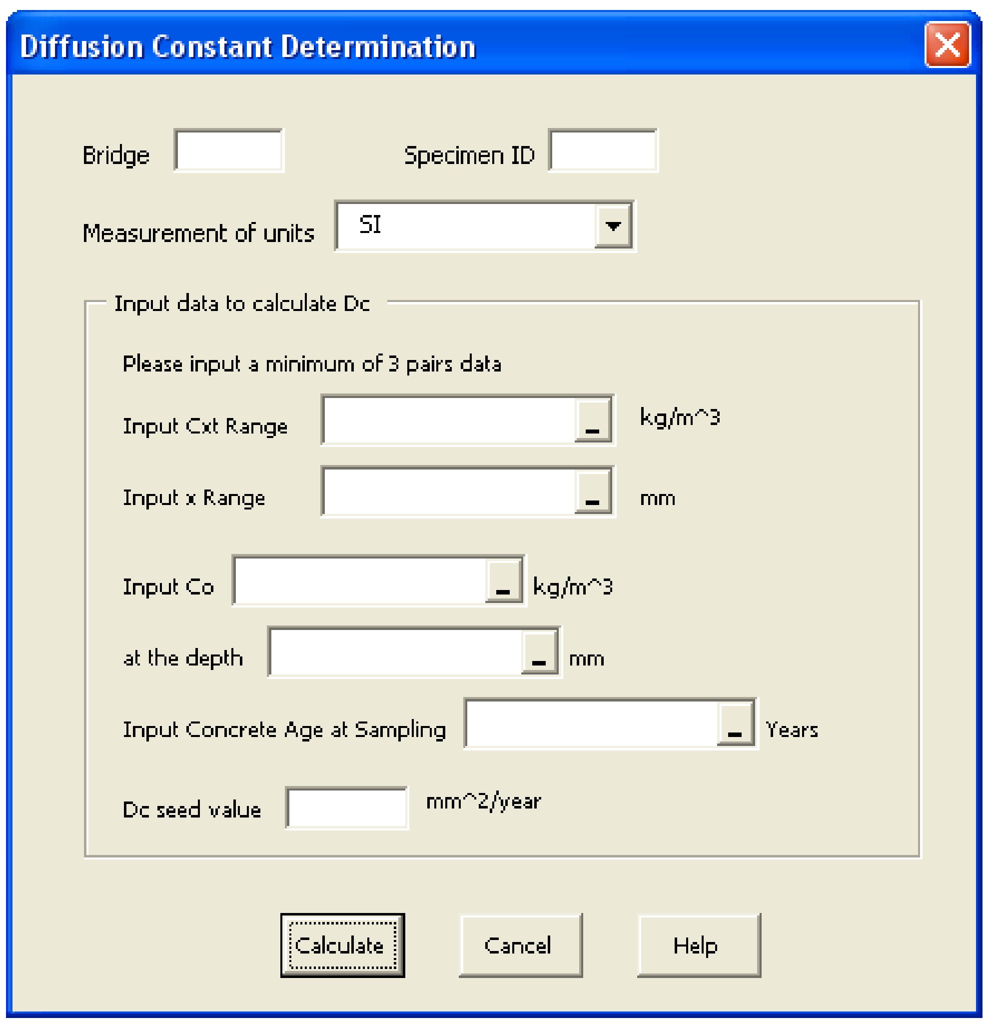


Figure 8 – BCA GUI - D_{ca} Input Form

The input data can be in either SI or U.S. Customary units. The D_c seed value is the initial value that is used to estimate the diffusion curve. The value is user defined and for most cases it does not matter what the value is. However, for bridge decks with low diffusion rates it is necessary to use a low D_c seed value (i.e. < 1). Using the input data BCA estimates the diffusion coefficient and generates a chloride profile along with a fitted diffusion curve as illustrated in Figure 9.

	A	B	C	D	E	F
1	Bridge		Concrete Age(years)			Difussion Constant Least Square Estimate(mm^2/year)
2	1-1804-C1		34			27.12432
3	Depth(mm	Cx,t meas	Cx,t predic	Difference square		
4	19.05	7.334012	7.334012	0		
5	19.05	7.334012	7.334012	0		
6	25.4	6.698929	6.471944	0.051522		
7	31.75	5.051397	5.628484	0.33303		
8	38.1	4.59613	4.821044	0.050586		
9	68.2625	2.218152	1.847015	0.137743		
10				0.572881		

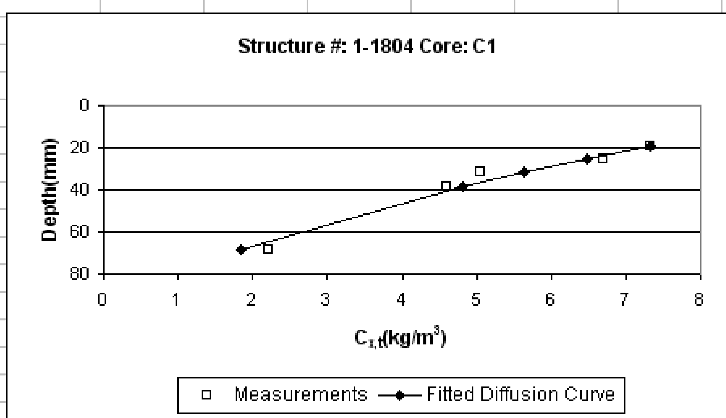


Figure 9 – BCA Output - D_{ca}

Background Chloride Concentration Determination

Chlorides initially present within a concrete mixture either contained within the cement paste or the aggregate are known as bound chlorides. The concentration of bound chlorides is relatively uniform and varies between bridge decks, but are in general relatively low. It has been argued that bound chlorides may in some cases enter the concrete pore water over the life of a structure due to corrosion-related equilibrium requirements, but for this study the bound chlorides have been taken to remain bound due to the nature of the concrete, cement, and aggregates.

The background chloride concentration for the individual bridge decks was determined by investigating the chloride titrations that were conducted on specimens taken from below the reinforcement. These specimens were generally at depths greater than 75 mm (3in.). For the quality of the concrete specified and the average age of the bridge decks investigated, it is unlikely that chlorides would have diffused to this depth; therefore, any chlorides present are expected to be background (bound) chlorides. It should also be noted that if the chlorides present below the reinforcement were the result of diffusion then the reinforcement should reflect very high levels of corrosion, which was not the case for any of the decks investigated.

The background chloride concentration was calculated to be the average of the chloride concentrations taken from below the reinforcement. The background chloride concentration was then subtracted from the chloride profile prior to the determination of D_{ca}. Background concentrations were determined for each bridge deck independently, as it is not uncommon for different concrete mixtures to have significantly different background chloride concentrations.

The calculated background concentration was also verified by investigating the chloride profiles. The chloride profiles will become asymptotic at the background chloride concentration value as illustrated in Figure 10. For the case presented below the background chloride concentration would be taken as 0.2 kg/m^3 (.38 lbs/cy).

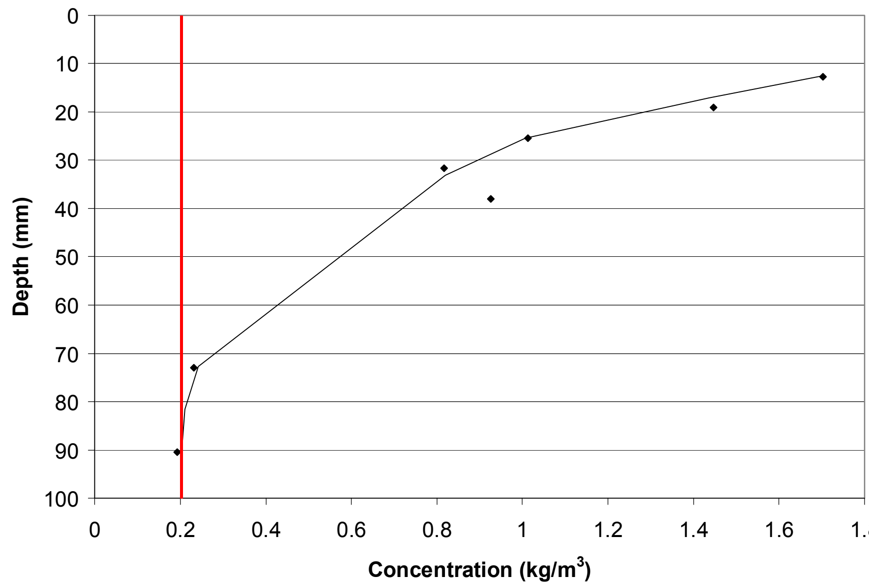


Figure 10 – Background Chloride Concentration Determination Example

Adjusting Diffusion Coefficients for Time Dependency

As described previously, the diffusion properties of concrete are time-dependent. To account for time effects in the measured D_{ca} values three diffusion models were used to estimate the decay of D_c over time. The time-dependency equations for those models are presented below. Pozzolan containing bridge decks were taken to have a 25% replacement rate of cement with fly ash (FA).

Mangat and Molloy (1994)

$$D_c = D_i t^{-m} \qquad \text{Equation 27}$$

where: $$m = 2.5\left(\frac{w}{c}\right) - 0.6 \qquad \text{Equation 28}$$

Using the above equation the calculated values of m for 0.45 $^w/_c$ and 0.47 $^w/_c$ OPC bridge decks were 0.525 and 0.575, respectively. To calculate a value of m for a bridge deck containing FA the FA is ignored in this equation. A typical concrete mixture with a 0.45 $^w/_c$ contains approximately 375 kg/m^3 (632 lbs/cy) of cementitious material and 169 kg/m^3 (285 lbs/cy) of water. Twenty-five percent replacement of the cement with FA is typical for VDOT bridge deck concrete mixtures yielding 169 kg (373 lbs) of water to 281 kg (620 lbs) of cement or a $^{w}/_{c}$ of 0.6. Using that ratio a value of 0.9 was calculated for m.

Bamforth (1999)

$$D_{ca} = at^{n} \qquad \text{Equation 29}$$

Bamforth estimated values of n for a 0.45 $^{w}/_{c}$ concrete mixtures. The values for OPC and FA concrete are –0.264 and –0.70, respectively.

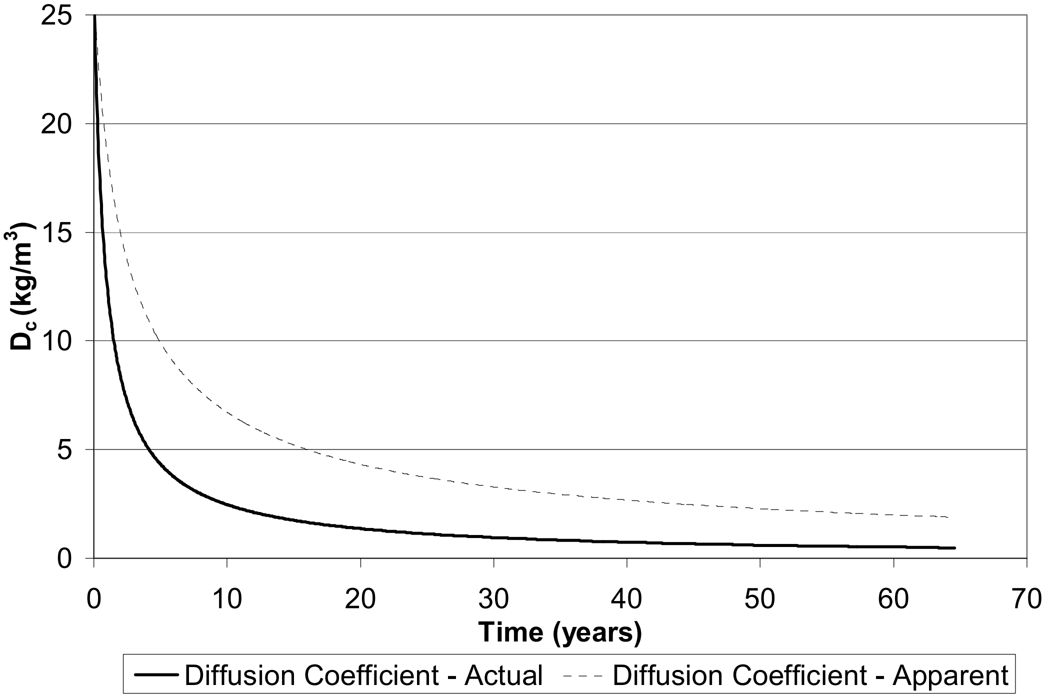
Life – 365

$$D(t) = D_{ref}\left(\frac{t_{ref}}{t}\right)^{m} \qquad \text{Equation 30}$$

where: $$m = 0.2 + 0.4\left(\frac{\%FA}{50} + \frac{\%Slag}{50}\right) \qquad \text{Equation 31}$$

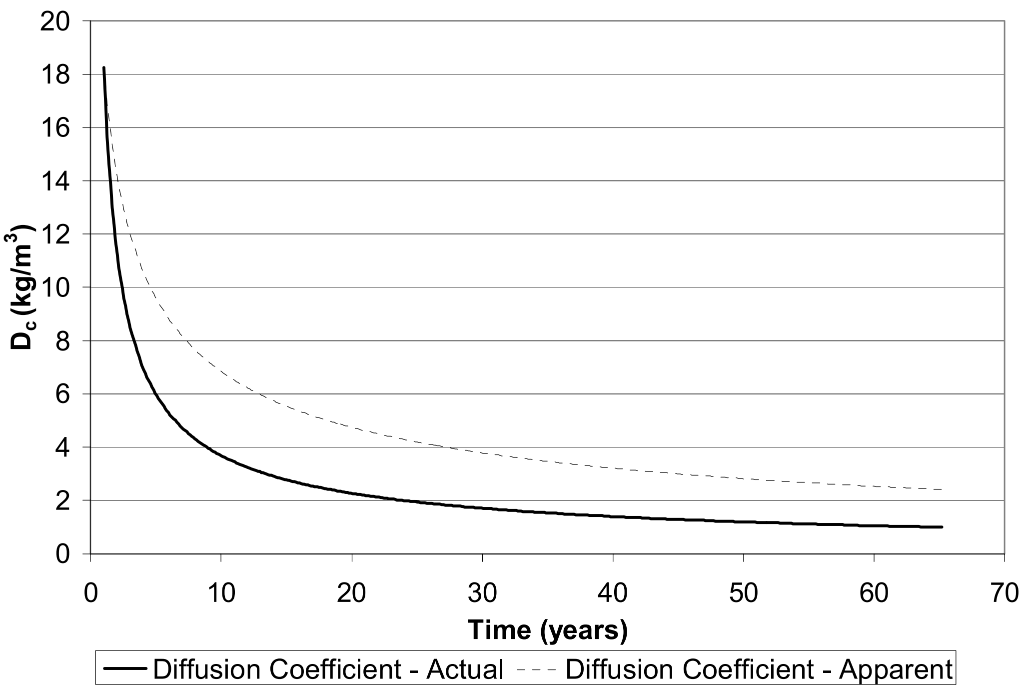
Using the above equation values of m for OPC and FA concrete were calculated to be 0.2 and 0.4, respectively.

The equations presented above were used to model the decay of D_c over time. However, the diffusion coefficients that are calculated from the field data are the apparent diffusion coefficients (D_{ca}) not the actual diffusion coefficients for the concrete at the time of sampling. D_{ca} can be defined as the average D_c value over the life of a structure. Therefore, D_{ca} will have a higher value than D_c at any given point in time with the margin of difference between D_{ca} and D_c narrowing with time. This relationship is presented for each of the three models in Figures 11 – 13 for 0.45 $^{w}/_{c}$ FA concrete.



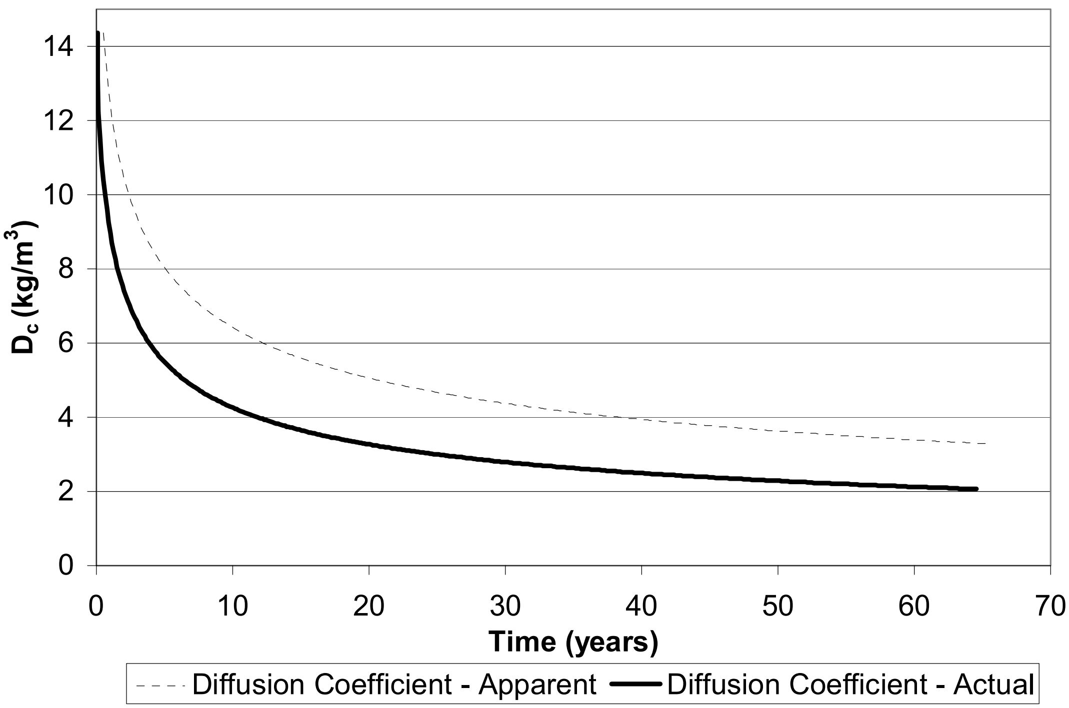
*1 kg/m^3 = 1.686 pcy

Figure 11 – Diffusion Decay – Mangat and Molloy



*1 kg/m^3 = 1.686 pcy

Figure 12 – Diffusion Decay – Bamforth



*1 kg/m^3 = 1.686 pcy

Figure 13 – Diffusion Decay – Life – 365

Using the three diffusion decay models presented above the D_{ca} curve can be forced to fit through a known measured value at a specified time and then the actual D_c can be calculated for that particular sample at any time in the past or a prediction can be made for the future value of D_c. For example the diffusion comparison presented in Figure 13 above for the Life – 365 model was determined using a known D_{ca} value of 6.0 mm^2/yr (0.009 in^2/yr) at an age of 12 years. The steps that were used to generate the diffusion decay plot were as follows:

1) A random initial D_c value was selected and the diffusion curve was generated based upon that value and the value for m that was selected based upon the concrete mixture proportions.
2) The D_{ca} diffusion curve was then generated by averaging the D_c values over the life of the structure.
3) The D_{ca} curve was then checked to ensure that it passed through the known point of 6.0 mm^2/yr (0.009 in^2/yr) at an age of 12 years.

4) If the D_{ca} curve does not pass through the known point a new value for the initial D_c would be selected and the process repeated. Figure 13 represents the final iteration.

The process was completed for each individual bridge deck and also for each of the three sets of bridge decks. The slopes of the apparent D_c curves have been shown to decrease linearly at a relatively slow rate after approximately 35 years. Thus, it was deemed appropriate to use the 35-year values as input values for the service life model.

BCA Service Life Model

The BCA service life model uses Monte Carlo Simulation to calculate the time required for a given bridge deck to reach a 2% level of corrosion initiation. The input form for the service life calculation is presented in Figure 14.

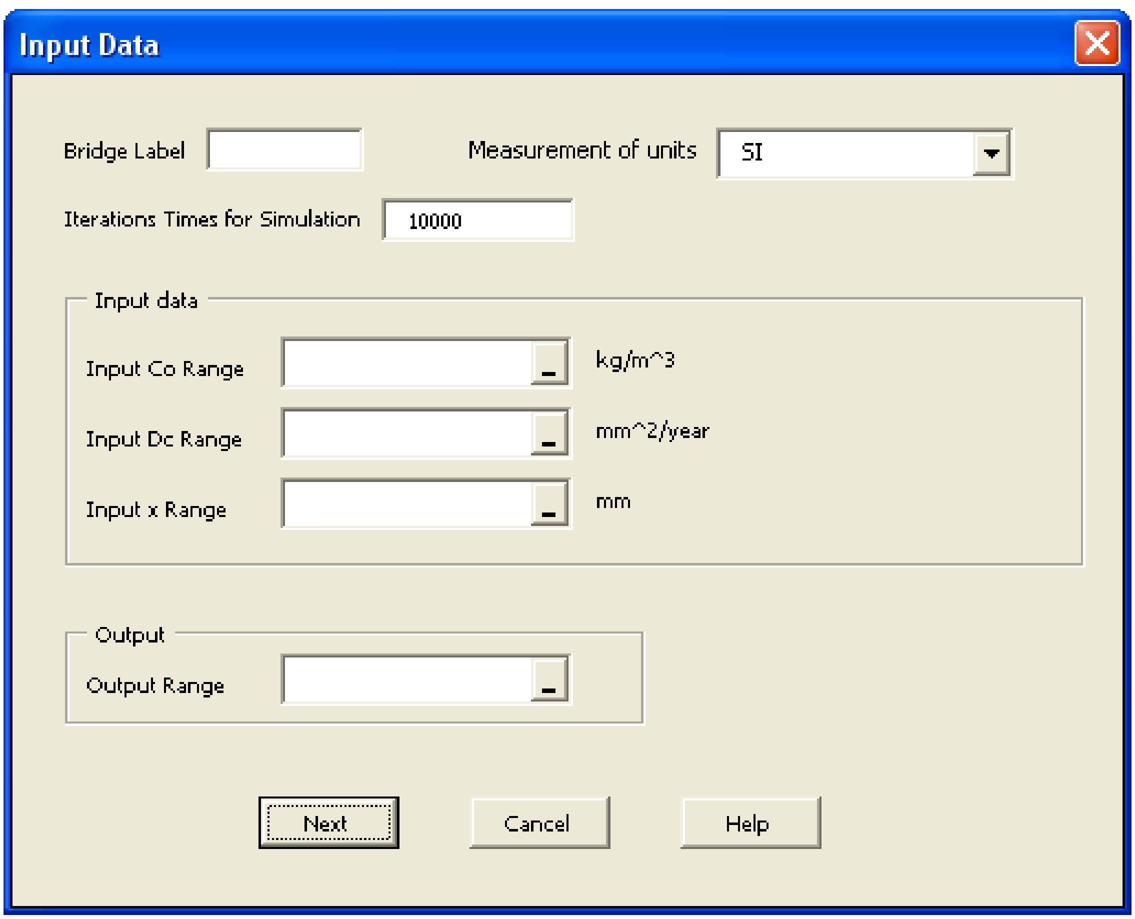


Figure 14 – BCA GUI – Service Life Input Form

BCA accepts a set of input values from which it will randomly select a value for each variable using the simple bootstrapping method. Once the necessary input data has been provided the program will prompt the user for information regarding the chloride threshold concentrations. The chloride threshold input form is presented in Figure 15.

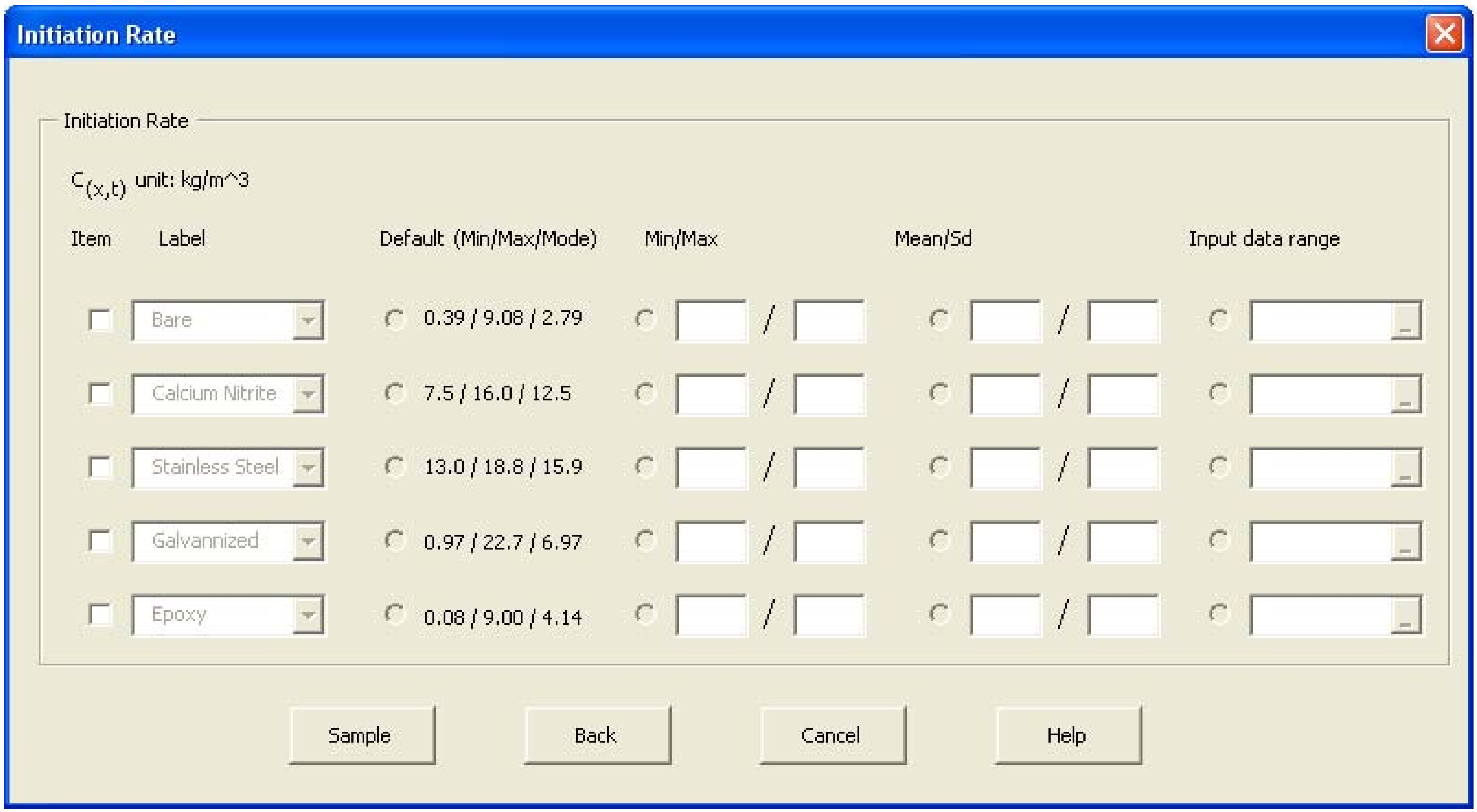


Figure 15 – BCA Chloride Threshold Input Form

The chloride threshold values that will be used in the service life calculation can be specified in one of four ways.

1) Default values for 5 different corrosion protection methods can be used that are specified by BCA,
2) User can specify minimum, maximum, and mode values for the chloride threshold and BCA will generate values according to a triangular distribution,
3) User can specify a mean and standard deviation of the threshold values and BCA will generate values according to a normal distribution, or
4) User can specify a set of threshold values and BCA will sample directly from that set.

The first three options use parametric bootstrapping to generate and select chloride threshold values while the fourth method uses simple bootstrapping to sample directly from a provided data set.

After the chloride threshold information has been provided BCA will calculate the time required for corrosion to initiate for that set of input values and chloride thresholds. The process is repeated for the number of specified iterations up to a maximum of 65000 iterations. The computer routine that is used is illustrated in Figure 16.

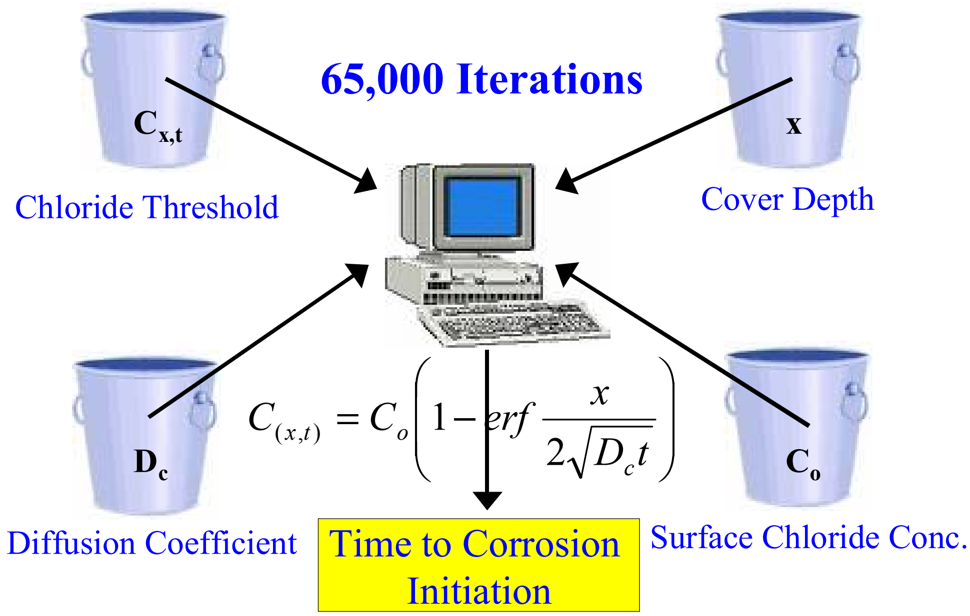
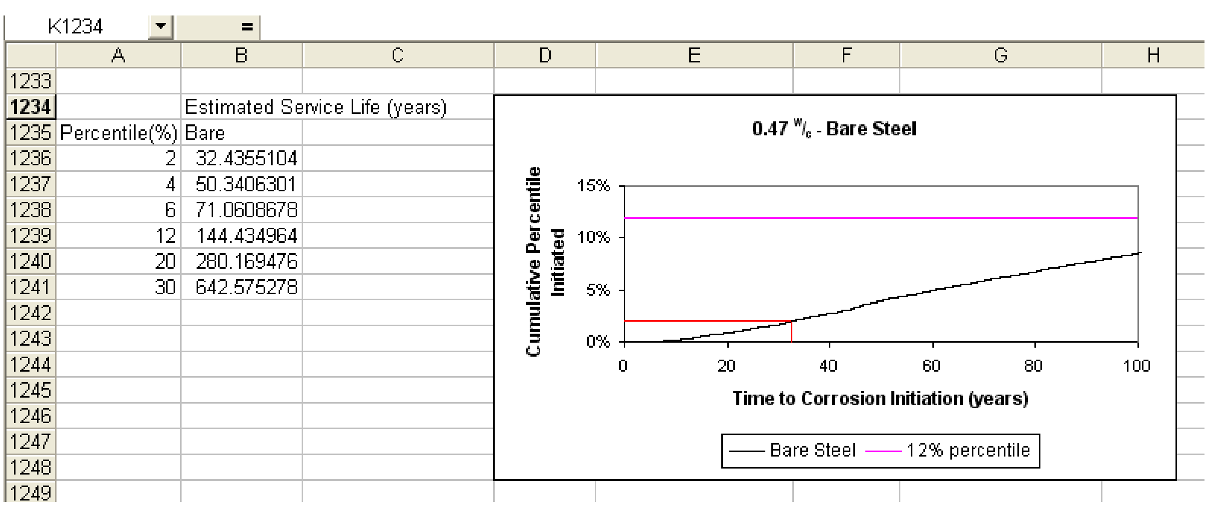


Figure 16 – BCA Service Life Estimation Routine

A Cumulative Distribution Function (CDF) is then generated using the calculated corrosion initiation times and the time required for 2% of the reinforcing steel to initiate corrosion can then be estimated. An example of the output generated by BCA is presented in Figure 17.



	A	B
1233		
1234		Estimated Service Life (years)
1235	Percentile(%)	Bare
1236	2	32.4355104
1237	4	50.3406301
1238	6	71.0608678
1239	12	144.434964
1240	20	280.169476
1241	30	642.575278

Figure 17 – BCA Service Life CDF

Estimating the Time to Corrosion Cracking for Bare Steel

After the time required for corrosion to initiate has been calculated the time to corrosion cracking must be added. The time for cracking to initiate has been calculated for each of the models presented previously. The values for the parameters used were selected to represent the characteristics of a typical bridge deck in Virginia.

Bažant Model

Parameters:

$D = 15.875$ mm;	$s = 203.4$ mm	$\rho_{cor} = 3.6$ g/cm^2
$j_r = 5.3797\text{x}10^{-5}$ g/cm^{2}days	$L = 50.8$ mm	$f'_t = 3.3$ Mpa
$E_{ef} = 9000$ Mpa	$\nu = 0.18$	

Calculated time to crack initiation: **24 days**

Liu/Weyers Model

Parameters:

$C = 52.2$ mm	$D = 15.875$ mm	$d_o = .0125$ mm
$f'_t = 3.3$ Mpa	$E_{ef} = 9000$ Mpa	$i_{corr} = 2$ mA/ft^2
$\nu_c = 0.18$	$\alpha = 0.523 – 0.622$	$\rho_{st} = 7.86$ mg/mm^3
$\rho_{rust} = 3.6$ mg/mm^3		

Calculated time to crack initiation: **2.08 – 2.80 years**

Alonso, Andrade, Rodriguez, and Diez Model

Penetration depth of 50 – 200 µm

Calculated time to crack initiation: **2.0 – 8.5 years**

Vu, Stewart, and Mullard Model

Parameters:

A = 62, 225, 700	B = 0.45, 0.29, 0.23	$C = 52.2$ mm
$^{w}/_{c} = 0.45$	$i_{corr(exp)} = 100$ µA/cm^2	$i_{corr(real)} = 2.14$ µA/cm^2

$\alpha = 0.85$ $\beta = -0.3$ Crack width = 0.3 – 1.0 mm

Calculated time to crack initiation: **2.08 – 2.80 years (from Liu/Weyers Model)**

Calculated time to excessive cracking (Vu et al. Model):

3.42 – 13.56 years (time-invariant corrosion rate)

9.3 – 38.9 years (time-variant corrosion rate)

Total calculated time for cracking to progress to a limit state:

Time-invariant corrosion rate – 5.5 – 16.36 years

Time-variant corrosion rate – 11.38 – 41.7 years

Torres-Acosta/Sagüés Propagation Model

Parameters:

x_{crit} = 0.03 mm -.272 mm i_{corr} = 2.14 $\mu A/cm^2$

Calculated time to crack initiation: **1.17 – 10.59 years**

Table 3 -Time to Crack Initiation Summary

Model	Time to Crack Initiation (years)
Bazant	0.07
Liu/Weyers	2.08 - 2.80
Alonso et al.	2.0 - 8.5
Vu et al.*	3.42 - 13.56 (invariant corrosion rate) 9.3 - 38.9 (variant corrosion rate)
Torres-Acosta/Sagues	1.17 - 10.59

*Time to excessive cracking

For purposes of estimating the corrosion cracking propagation time for this project the Vu, Stewart, and Mullard model was used in conjunction with the Liu/Weyers model. The Liu/Weyers model was used to predict the time to crack initiation while the Vu, Stewart, and Mullard model was used to predict the time required for the crack to propagate to a limit state. The Liu/Weyers model was selected because Vu concluded that "the Liu and Weyers model seemed the most realistic because it included: a) the amount of corrosion needed to fill the porous zone between the steel/concrete interface (free expansion); and b) the reduction with time of the rate of rust production when determining the critical tensile stress needed to cause cover cracking." (Vu, 2003)

The corrosion products were taken to be a composition of $Fe(OH)_3$ and $Fe(OH)_2$ and the crack width limit state was taken to be 0.3 mm (0.012 in.). A crack width limit state of 0.3 mm (0.012 in.) was selected because it has been suggested that crack widths of less than 0.3 mm (0.012 in.) have little to no effect on the ingress of chlorides into the concrete. (Atimay and Ferguson, 1974) Using these parameters the time to crack initiation was calculated to be 2.40 years and the time for crack propagation was calculated to be 3.42 years yielding a total corrosion cracking propagation time of 5.82 years. For simplicity the total crack propagation time that will be used to estimate bridge deck service life is 6 years.

Estimating the Time to Corrosion Cracking for ECR

It is known that the epoxy coating on a reinforcing bar will slow the corrosion rate for that bar. Several research projects indicate that the additional propagation time provided by the epoxy coating is approximately 5 years. (Brown, 2002; McDonald, 1998) Therefore, the crack propagation time for ECR bridge decks will be taken as 11 years as compared to the 6 years provided by bare steel.

Estimating the Propagation Time of Corrosion for Bare Steel Bridge Decks

The propagation of corrosion is a complex process that cannot be described by simple Fickian behaviour. BCA is not currently capable of determining the propagation period. Therefore, the corrosion propagation time has been estimated using empirical data.

To estimate the corrosion propagation time for bare steel bridge decks in Virginia those bridge decks that reflected the most damage during the intial survey were resurveyed three years later. A total of 7 bridge decks were resurveyed and their effective deterioration rates calculated. It was found that the deterioration rate of a bridge deck is related to the amount of initial damage on that deck. The relationship is presented in Figure 18.

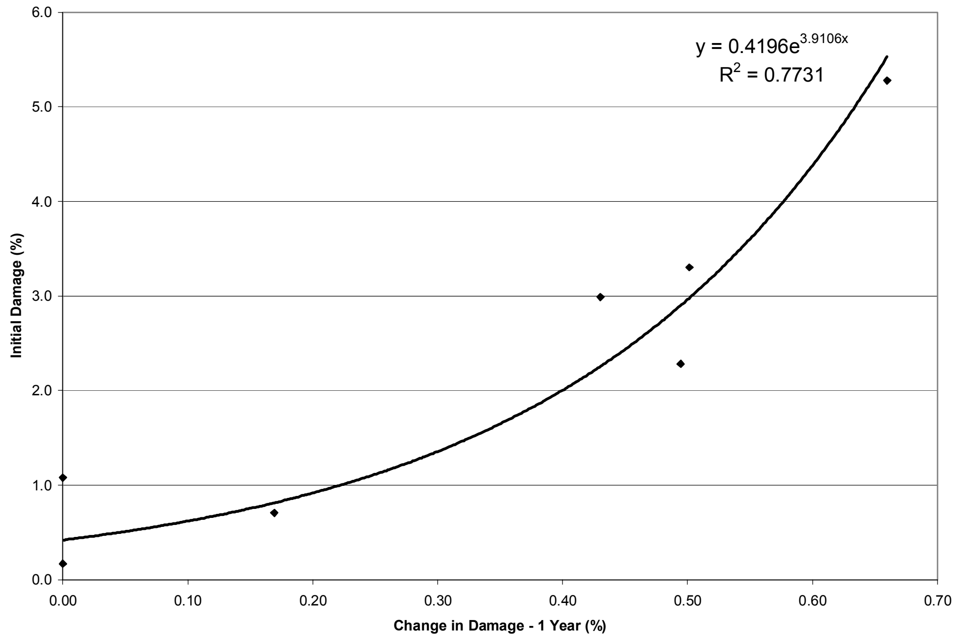


Figure 18 – Corrosion Propagation Rates

The regression analysis demonstrated that the corrosion propagation rate increases exponentially with the amount of initial damage present on the deck. The regression analysis was then used to estimate the required amount of time for corrosion to propagate from a level of 2% to a level of 12%. The resulting plot is presented in Figure 19 below reflecting an estimated total propagation time of approximately 16 years, for damage to propagate from 2% to 12% damage.

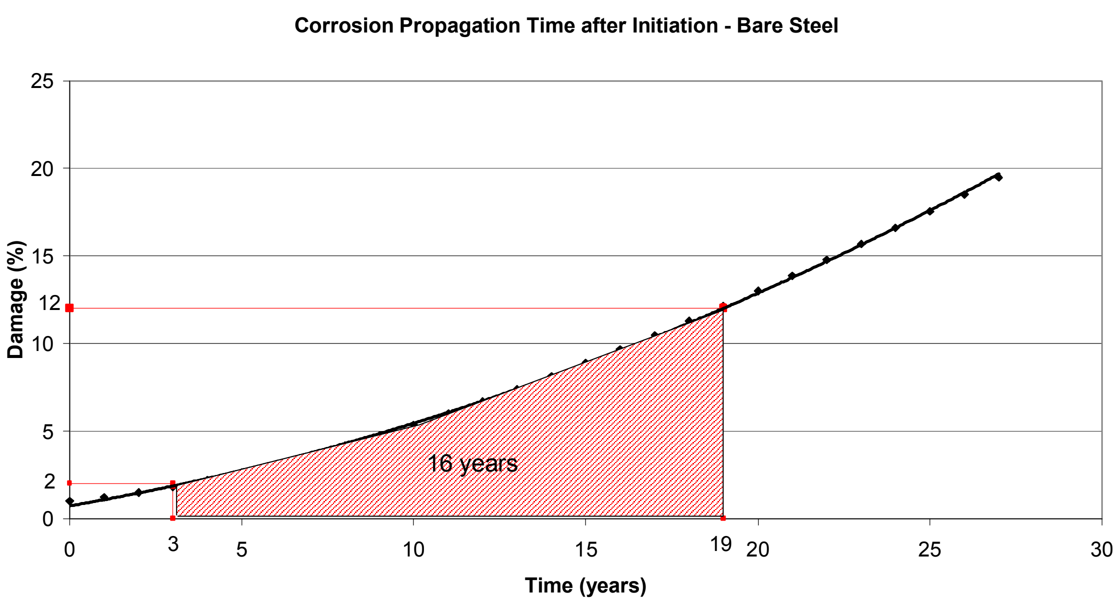


Figure 19 – Total Propagation Time

The time required to reach a specific level of deterioration can also be calculated using the following equation:

$$TimeToDeterioration = 8.61\left(\sqrt{\%Deterioration + 1.38} - 1.45\right) - 3.34 \qquad \text{Equation 32}$$

The time calculated will be the time for corrosion to progress from a deterioration level of 2% to the specified level.

Estimating the Propagation Time of Corrosion for ECR Bridge Decks

Estimating the corrosion propagation time for ECR bridge decks using empirical data is not possible due to their relatively young age and low levels of deterioration. The effect that the epoxy coating will have on the rate of corrosion is not fully understood. It is suspected that the corrosion rate will be lower due to the electrical isolation provided by the coating. However, if the coating debonds from the steel substrate it is possible for corrosion to progress rapidly underneath the coating. To estimate the propagation rate for ECR the deterioration rate for one bridge deck located in Blacksburg was analyzed.

A visual survey[3] of structure number 8003 (Prices Fork Road of Rt. 460) was conducted during the summer of 1996. At that time the right hand traffic lane reflected a deterioration level of 0.5%. A VDOT bridge inspection conducted in August of 2001 determined that the deterioration had progressed to a level of 8.2%. The bridge deck was rehabilitated during the summer of 2003 due to excessive damage. VDOT work orders report that a total of 105 sq yds (87.8 m^2) were patched prior to overlaying the deck. It is common construction practice to remove some additional concrete surrounding deteriorated areas to ensure that all of the damaged area is removed. This additional patched area is estimated to be 20% of the total patched area yielding a total deteriorated area of 84 sq yds or 756 sq ft (70.2 m^2).

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[3] Information obtained through personal communication with Professor Richard Weyers of the Virginia Polytechnic Institute and State University, November 2006.

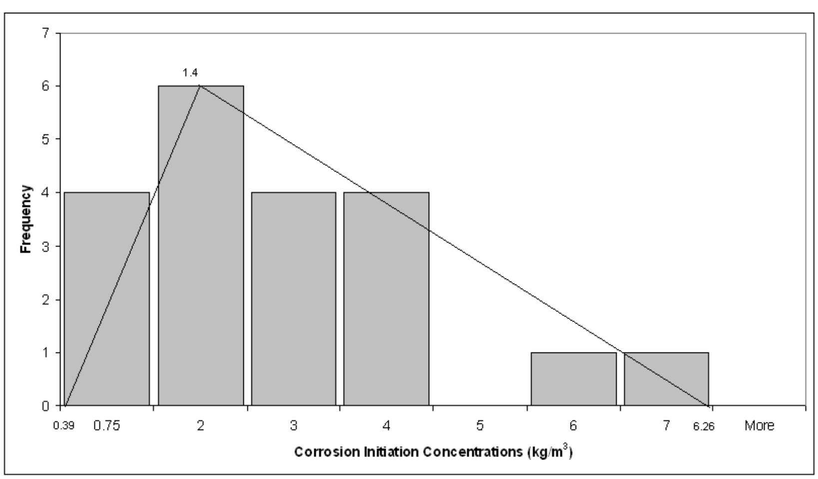
Considering the deterioration levels for both traffic lanes to be reasonably equivalent it was estimated that 378 sq ft (35.1 m^2) of the damage was present in the right-hand traffic lane. The right-hand traffic lane represents a total deck area of 2400 sq ft (223 m^2). Thus, the deterioration level was 15.8% at the time of rehabilitation, which is within reasonable agreement of the 12% EFSL value.

This recorded deterioration data indicates that the corrosion propagated from a level of 0.5% to a level of approximately 15.8% in 7 years. This ECR propagation rate of 2.2%/yr far exceeds those values evident in the bare steel structures of 0.62%/yr presented previously. Realizing that this bridge deck example may be a worst case scenario, but also that it is possible for ECR bridge decks to propagate corrosion at a rate equal to or higher than bare steel bridge decks, the propagation rate for ECR has been considered to be equivalent to that of bare steel for this study.

Simulation Input Data

Corrosion Initiation Concentrations

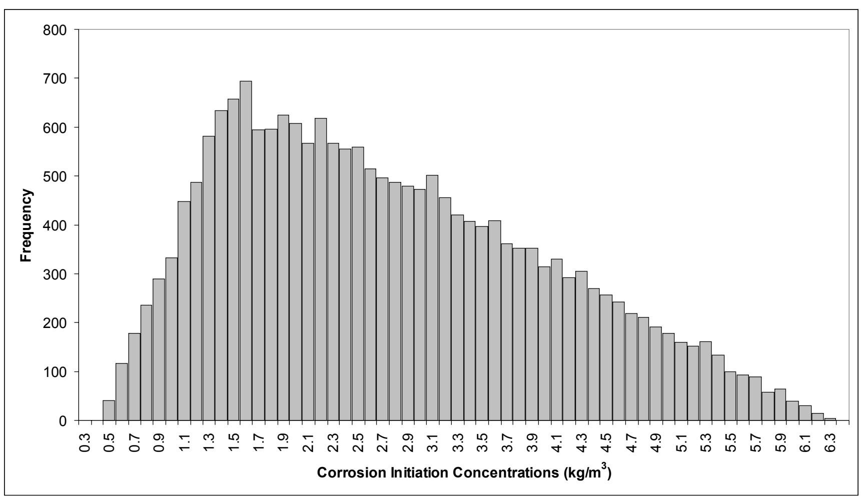
The corrosion initiation concentration values (C_{init}) that were used to estimate the service life of the bridge decks were developed from experimental results that were obtained from corrosion testing carried out on bridge deck cores. (Brown, 2002) The estimated corrosion initiation concentrations are presented in Figure 20.



*1 kg/m^3 = 1.686 pcy

Figure 20 – Corrosion Initiation Concentrations – Bare Steel

The initiation values reasonably agree with a triangular distribution with a minimum of 0.39 kg/m^3 (0.66 pcy) and a maximum of 6.26 kg/m^3 (10.55 pcy). The distribution is skewed to the left with an estimated mode of 1.4 kg/m^3 (2.36 pcy). The range of initiation values is in general agreement with values reported in the literature. (Stratfull et al., 1975; Vassie, 1984; Matsushima et al., 1998; Henriksen, 1993) Glass and Buenfeld completed a comprehensive review of corrosion initiation concentrations and report values ranging from 0.59 – 8.75 kg/m^3 (0.99 – 14.75 pcy) (1997). Using the parameters obtained from Brown's research a set of 20000 initiation values was randomly generated for use as input to the service life model. Twenty thousand values were generated in order to completely define the distribution. It was found that the actual number of values generated will not significantly affect the time to corrosion initiation estimations as long as a minimum of 2000 values are used. The generated distribution is presented in Figure 21.



*1 kg/m^3 = 1.686 pcy

Figure 21 – Corrosion Initiation Concentrations (Bare Steel - Randomly Generated)

Apparent Diffusion Coefficients (D_{ca})

The D_{ca} values that were determined from the sampled bridge deck cores were normalized to an age of 35 years using the methods described previously. These values were then used in the service life estimations. The actual values used will be presented in the Results section.

Surface Chloride Concentrations (C_o)

The C_o values used in the service life model were the measured values taken from the deck cores at a depth of ½ in. (12.7 mm) minus the estimated background chloride concentration. In instances where the chloride concentration reached a maximum at a depth greater than ½ in. (12.7 mm) the higher concentration was used for C_o.

Cover Depths (x)

The cover depths determined from pachometer measurements were used as the input to the service life model. The pachometer measurements were validated by comparing the field measurements at the core locations to the actual cover depths that were measured in the laboratory for the 10 - 0.45 $^{w}/_{c}$ bridge decks. The field measurements differed from the actual values by as little as 1% to a maximum of 13%. The average difference was 5%, which is an insignificant amount when used in the service life model.

RESULTS

Bridge Deck Damage Surveys

The results of the deck damage surveys are presented in Tables 4-5. The damage levels presented are the total of both the measured visual damage and the damage determined through sounding for the right-hand traffic lane (critical traffic lane). The 0.47 $^{w}/_{c}$ bridge decks were resurveyed 3 years after the initial damage survey to provide data on corrosion propagation rates.

Table 4 – 0.47 $^{w}/_{c}$ Bridge Deck Damage Surveys

0.47 $^{w}/_{c}$ Bridge Decks				
Structure #	**Age at Survey**	**% Damaged 2003**	**Age at Survey**	**% Damaged 2006**
1-1804	34	3.3	37	5.3
1-6101	34	0.0	37	0.0
2-2007	33	0.8	36	2.3
3-1021	32	0.2	35	0.2
4-1062	34	0.0	37	0.0
4-2049	35	0.2	38	0.7
5-1800	33	1.8	36	3.3
6-1032	32	0.0	35	0.0
9-2801	33	1.7	36	3.0
9-6042	34	1.1	37	1.1

Table 5 – 0.45 $^{w}/_{c}$ and 0.45 $^{w}/_{c+p}$ Bridge Deck Damage Surveys

0.45 $^{w}/_{c}$ Bridge Decks			0.45 $^{w}/_{(c+p)}$ Bridge Decks		
Structure #	**Age at Survey**	**% Damaged 2003**	**Structure #**	**Age at Survey**	**% Damaged 2003**
1-1132	15	0.0	1-1152	16	0.0
1-1133	15	0.0	1-2815	17	0.0
1-2820	17	0.0	1-2819	17	0.0
1-6051	13	0.0	2-1021	15	0.0
2-1020	17	0.0	3-1000	12	0.0
3-1003	15	0.0	5-2812	12	0.0
4-1007	13	0.0	8-1002	15	0.0
4-2901	12	0.0	8-1042	13	0.3
5-2547	19	0.0	9-1002	16	0.0
7-1920	12	0.3	9-6058	12	0.0
8-1019	19	0.0			
8-1133	16	0.0			
9-1014	15	0.0			
9-1031	13	0.1			
9-1098	15	0.0			
9-1139	16	0.0			

The 0.45 $^w/_c$ bridge decks reflected little to no damage due to their young age relative to the older 0.47 $^w/_c$ bridge decks.

Surface Chloride Concentrations

The C_o values determined from the chloride profiles of the bridge deck cores are presented in Table 6. The values represent the chloride concentrations at a depth of 12.7 mm (½ in.) below the surface of the deck.

Table 6 – Average Surface Chloride Concentrations

0.47 $^w/_c$ Bridge Decks		0.45 $^w/_c$ Bridge Decks		0.45 $^w/_{(c+p)}$ Bridge Decks	
Structure #	Chloride Conc. (kg/m^3)	Structure #	Chloride Conc. (kg/m^3)	Structure #	Chloride Conc. (kg/m^3)
1-1804	6.52	1-1132	5.74	1-1152	6.54
1-6101	1.03	1-1133	6.45	1-2815	3.59
2-2007	3.85	1-2820	5.07	1-2819	6.94
3-1021	5.85	1-6051	2.42	2-1021	4.38
4-1062	1.74	2-1020	5.57	3-1000	1.91
4-2049	2.79	3-1003	2.86	5-2812	2.67
5-1800	0.62	4-1007	1.73	8-1002	2.74
6-1032	2.59	4-2901	1.72	8-1042	4.82
9-2801	2.80	5-2547	1.16	9-1002	2.66
9-6042	6.67	7-1920	3.33	9-6058	3.96
		8-1019	5.77		
		8-1133	3.59		
		9-1014	2.29		
		9-1031	2.83		
		9-1098	2.12		
		9-1139	2.83		

*1 kg/m^3 = 1.686 pcy

Cover Depths

The average of 40 – 114 cover depth measurements for the individual bridge decks are presented in Tables 7 - 9. The cover depth measurements were taken in the wheel paths of the critical deterioration area (right-hand traffic lane). As shown the average cover depths are generally higher for the 0.45 $^w/_c$ bridge deck sets. This is due to an increase in the specified cover depth that coincided with the decrease in the $^w/_c$.

Table 7 – Average Cover Depths for 0.47 $^w/_c$ Bridge Decks

0.47 $^w/_c$ Bridge Decks			
Structure #	**Avg. Cover Depth (mm)**	**n**	**Standard Dev. (mm)**
1-1804	57.4	102	6.97
1-6101	53.2	92	7.32
2-2007	38.5	80	2.70
3-1021	52.1	48	6.63
4-1062	58.5	80	9.98
4-2049	52.5	60	6.91
5-1800	44.8	40	10.25
6-1032	62.2	80	7.81
9-2801	40.8	80	8.47
9-6042	60.7	42	6.18

*1 mm = 0.0394 in.

Table 8 – Average Cover Depths for 0.45 $^w/_c$ Bridge Decks

0.45 $^w/_c$ Bridge Decks			
Structure #	**Avg. Cover Depth (mm)**	**n**	**Standard Dev. (mm)**
1-1132	55.4	84	6.95
1-1133	62.1	78	4.51
1-2820	72.7	80	5.73
1-6051	55.6	103	6.81
2-1020	55.4	114	6.50
3-1003	58.4	68	5.73
4-1007	61.1	68	3.74
4-2901	62.4	80	7.45
5-2547	65.0	80	6.03
7-1920	51.1	80	7.32
8-1019	63.5	102	5.83
8-1133	68.6	76	4.18
9-1014	50.1	40	3.63
9-1031	48.6	80	6.05
9-1098	56.7	80	5.53
9-1139	61.5	79	8.41

*1 mm = 0.0394 in.

Table 9 – Average Cover Depths for 0.45 $^{w}/_{c+p}$ Bridge Decks

0.45 $^{w}/_{(c+p)}$ Bridge Decks			
Structure #	Avg. Cover Depth (mm)	n	Standard Dev. (mm)
1-1152	63.1	60	14.25
1-2815	63.9	103	5.50
1-2819	67.3	114	6.17
2-1021	61.8	48	4.15
3-1000	52.3	50	5.07
5-2812	63.8	40	4.12
8-1002	57.7	68	6.49
8-1042	58.7	56	11.12
9-1002	66.2	82	8.69
9-6058	55.9	40	4.34

*1 mm = 0.0394 in.

Diffusion Coefficients

Apparent diffusion coefficients were calculated for the sampled bridge decks using Fick's second law of diffusion. The D_{ca} values were then normalized to an age of 35 years using the methods described previously. The average calculated D_{ca} values and the average D_{ca} values projected at an age of 35 years are presented in Table 10.

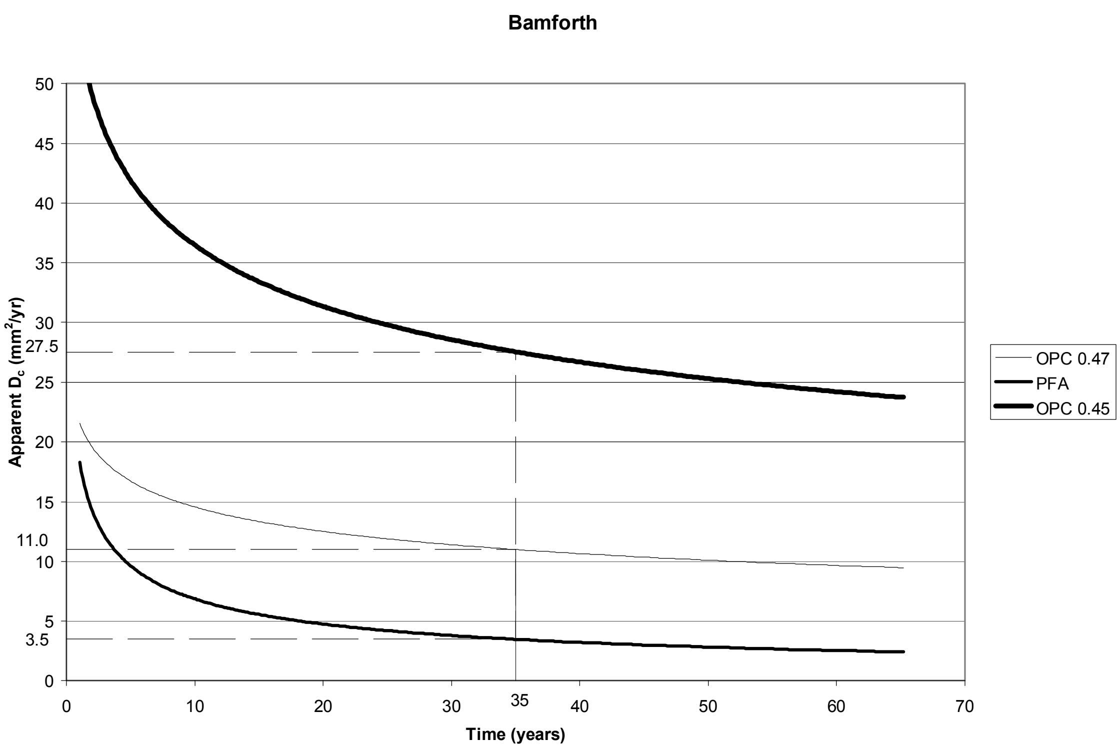
Table 10 – Average Diffusion Coefficients

0.47 $^{w}/_{c}$ Bridge Decks			0.45 $^{w}/_{c}$ Bridge Decks			0.45 $^{w}/_{(c+p)}$ Bridge Decks		
Structure #	Apparent D_c (mm^2/yr)	D_{c35} (mm^2/yr)	Structure #	Apparent D_c (mm^2/yr)	D_{c35} (mm^2/yr)	Structure #	Apparent D_c (mm^2/yr)	D_{c35} (mm^2/yr)
1-1804	35.74	36.35	1-1132	38.26	30.09	1-1152	2.3	1.52
1-6101	4.55	4.63	1-1133	82.64	64.98	1-2815	3.73	2.54
2-2007	6.30	6.35	1-2820	14.85	12.08	1-2819	5.35	3.63
3-1021	20.68	20.68	1-6051	51.72	39.19	2-1021	3.49	2.22
4-1062	3.63	3.69	2-1020	17.49	14.23	3-1000	1.96	1.12
4-2049	6.15	6.30	3-1003	46.02	36.19	5-2812	6.39	3.65
5-1800	7.97	8.04	4-1007	26.47	20.06	8-1002	5.09	3.25
6-1032	4.39	4.39	4-2901	26.18	19.43	8-1042	5.77	3.43
9-2801	4.72	4.76	5-2547	5.53	4.64	9-1002	11.82	7.79
9-6042	13.05	13.28	7-1920	56.44	41.88	9-6058	7.09	4.05
			8-1019	30.69	25.76			
			8-1133	24.06	19.26			
			9-1014	19.74	15.52			
			9-1031	36.82	27.90			
			9-1098	9.28	7.29			
			9-1139	40.98	32.80			

*1 mm^2/yr = 0.0015 in^2/yr

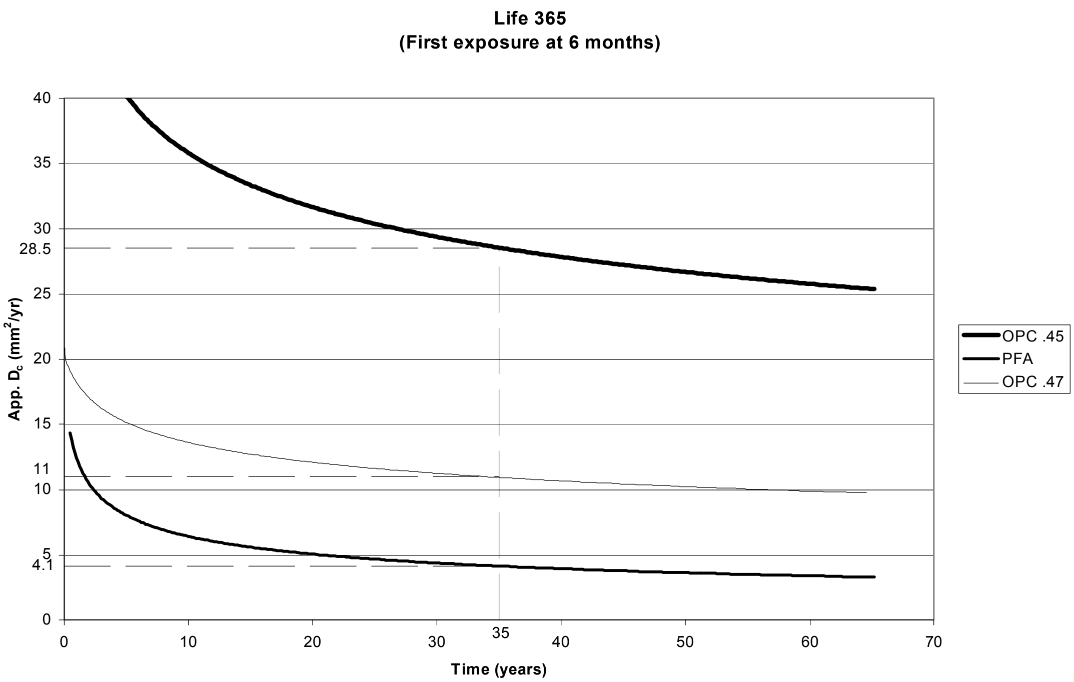
Time Adjusted Diffusion Coefficients

The average D_{ca} values for the individual bridge decks were presented in Table 10 above. However, the primary objective of this project is to analyze the bridge decks as three distinct sets. D_{ca} values at an age of 35 years were projected using the average ages and average D_{ca} values for the three bridge deck sets. The projections were completed using the curve fitting method presented previously. The resulting plots are presented in Figures 22 - 24. (Bamforth, 1999; Mangat and Molloy, 1994; Thomas and Bentz, 2000) A summary of the projected D_{ca} values is presented in Table 11.



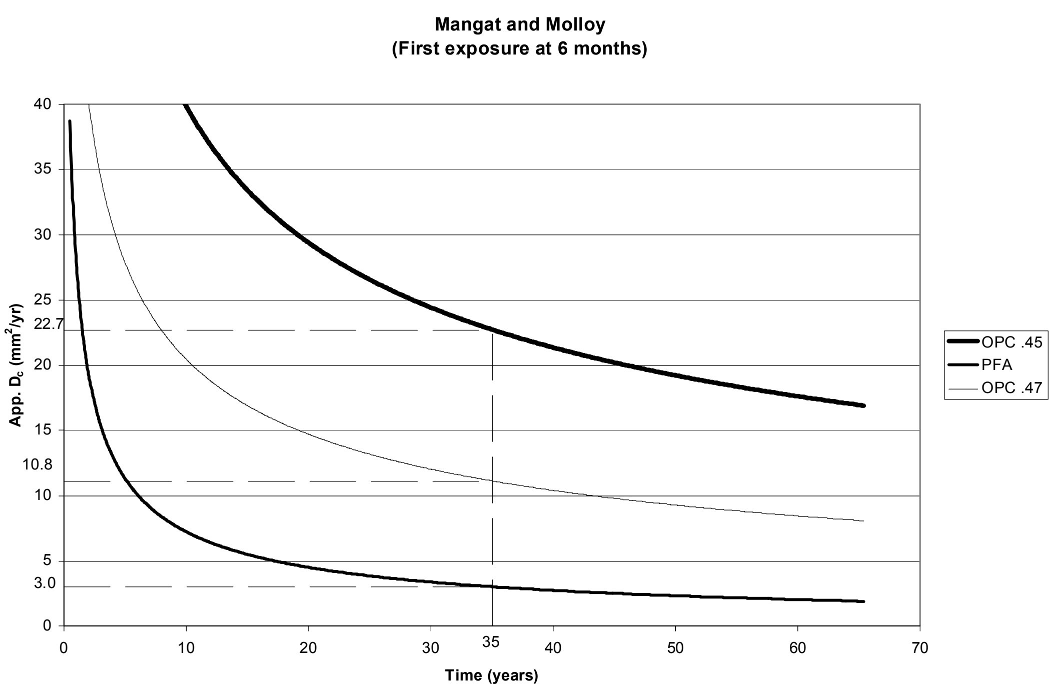
*1 mm^2/yr = 0.0015 in^2/yr

Figure 22 – Apparent D_c Time-Dependency Model – Bamforth



*1 mm^2/yr = 0.0015 in^2/yr

Figure 23 – Apparent D_c Time-Dependency Model – Life 365



*1 mm^2/yr = 0.0015 in^2/yr

Figure 24 – Apparent D_c Time-Dependency Model – Mangat and Molloy

Table 11 – Apparent D_c Time-Dependent Model Data

			Predicted Apparent D_c at 35 years of age			
Concrete Type	Average Age (years)	Apparent D_c (mm^2/yr)	Mangat and Molloy	Bamforth	Life 365	Average
OPC 0.47	36.4	10.9	11.1	11.0	11.0	11.0
OPC 0.45	15.1	33.3	22.7	27.5	28.5	26.2
PFA 25%	14.4	5.7	3.0	3.5	4.1	3.5

*1 $mm^2/yr = 0.0015\ in^2/yr$

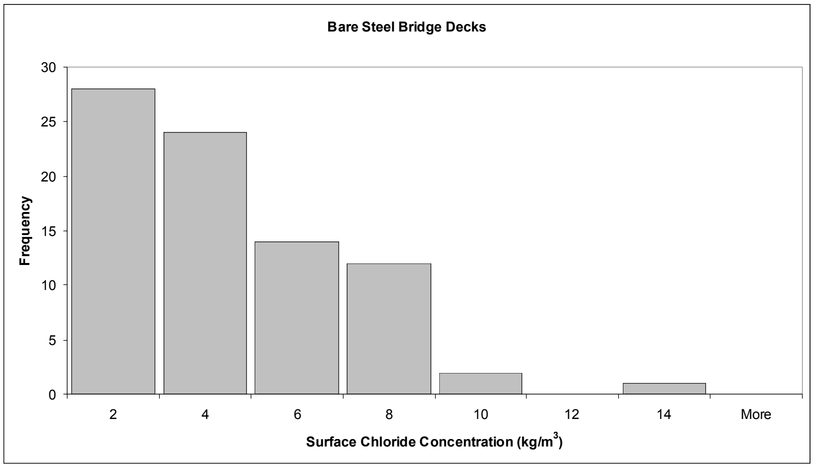
By projecting the D_{ca} values for the three sets to an age of 35 years the influence of the three types of concrete may be compared at an equivalent age. As shown in Table 11, at an age of 35 years the decks containing a pozzolan with a $^w/_c$ ratio of 0.45 have the lowest D_{ca} values and the decks with no pozzolan and a $^w/_c$ of 0.45 have the highest D_{ca} value. It is interesting that the 0.45 $^w/_c$ OPC decks have significantly higher D_{ca} values than the 0.47 $^w/_c$ OPC decks. The implication is that the newer 0.45 $^w/_c$ decks may not have been constructed as specified or the older decks were constructed with properties in excess of their specifications. In either case it is clear that the older decks are reflecting better diffusion properties than the newer decks. It is not possible to draw any conclusions concerning the construction quality of the bridge decks containing pozzolan as the decreased D_{ca} values are most likely attributable to the addition of pozzolan. In all cases the D_{ca} will continue to decrease with time to a limited extent and the pozzolan concrete decks will decrease at a faster rate than both of the OPC bridge deck sets.

Using the projected D_{ca} values of the three bridge deck sets it is estimated that the bridge decks with 0.45 $^w/_c$ concrete will have a reduction in D_{ca} of 21%, and the pozzolan decks will have a reduction of 39% at an age of 35 years.

Service Life Model Input Data

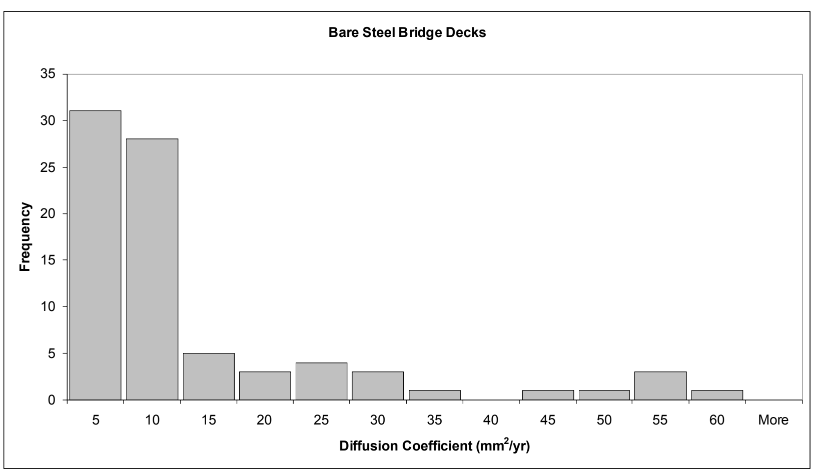
Histograms of the input data were developed for each of the three bridge deck sets to investigate the distributions of the input parameters. The histograms for the three bridge deck sets are presented in Figures 25 - 33.

0.47 $^{w}/_{c}$ Bridge Decks



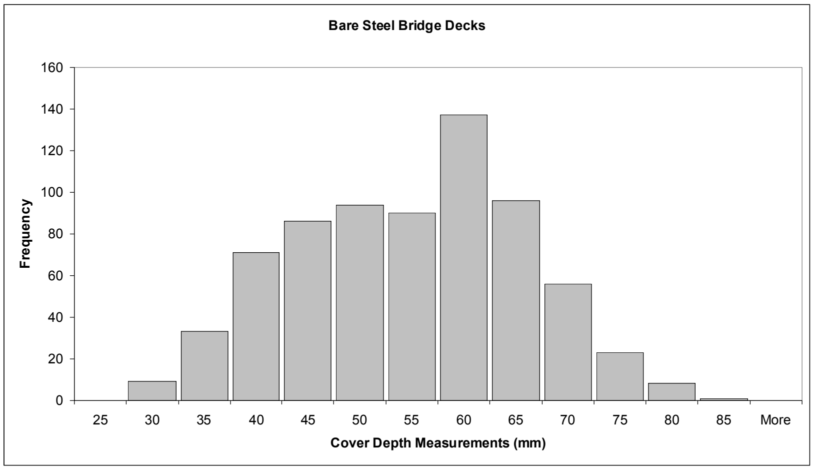
*1 kg/m^3 = 1.686 pcy

Figure 25 – C_o Measurements – 0.47 $^{w}/_{c}$



*1 mm^2/yr = 0.0015 in^2/yr

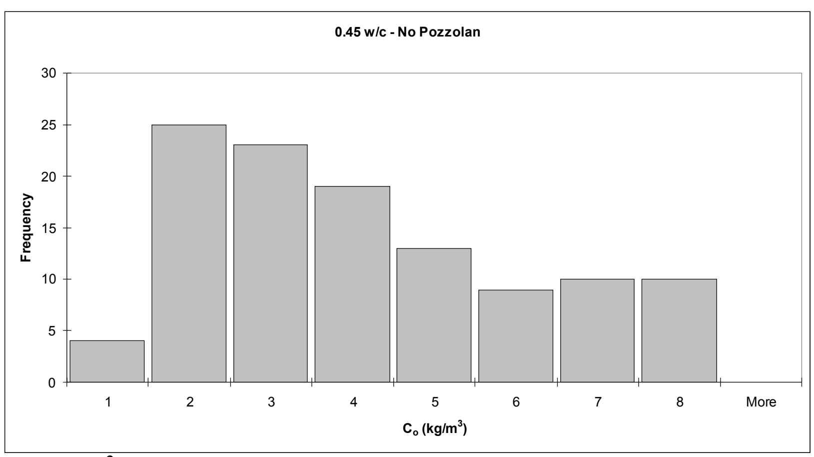
Figure 26 – Diffusion Coefficients – 0.47 $^{w}/_{c}$



* 1 mm = 0.039 in.

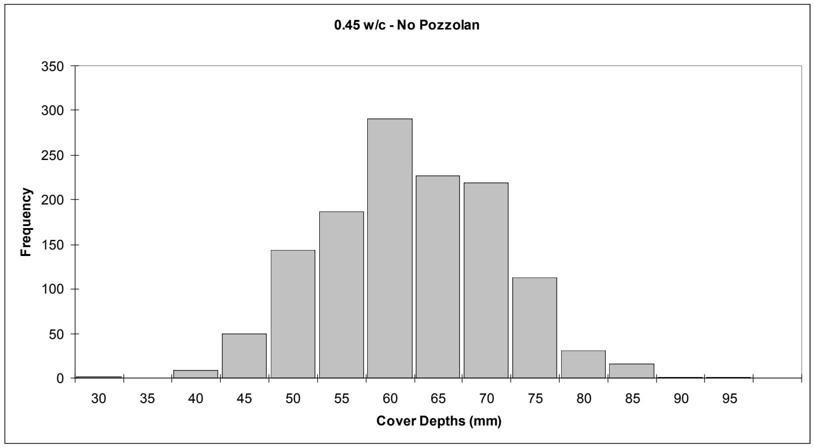
Figure 27 –Cover Depth Measurements – 0.47 $^{w}/_{c}$

0.45 $^{w}/_{c}$ – No Pozzolan Bridge Decks



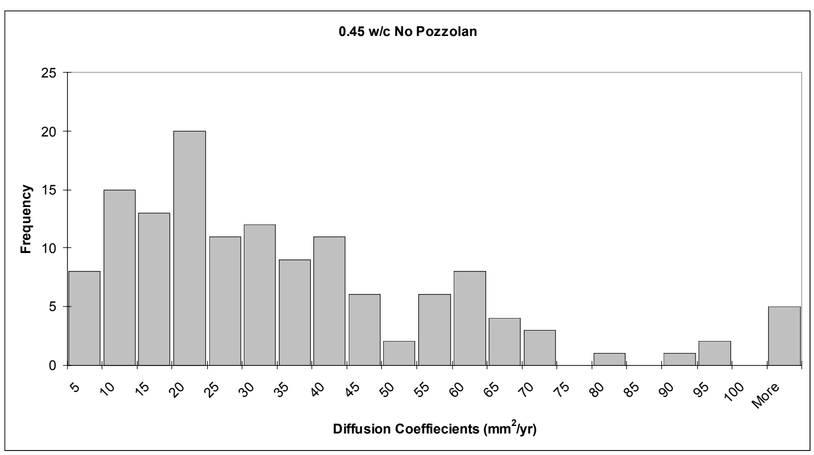
*1 kg/m^3 = 1.686 pcy

Figure 28 – Surface Chloride Concentrations – 0.45 $^{w}/_{c}$ No Pozzolan



* 1 mm = 0.039 in.

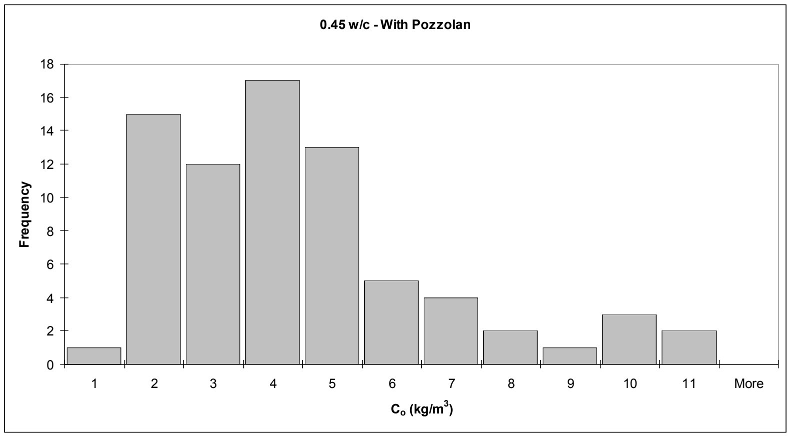
Figure 29 – Cover Depth Measurements – 0.45 $^{w}/_{c}$ No Pozzolan



*1 mm^2/yr = 0.0015 in^2/yr

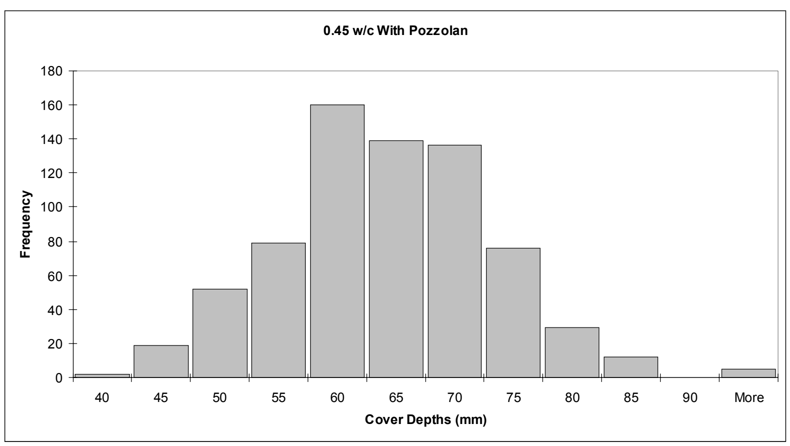
Figure 30 – Diffusion Coefficients – 0.45 $^{w}/_{c}$ No Pozzolan

0.45 $^{w}/_{(c+p)}$ Bridge Decks



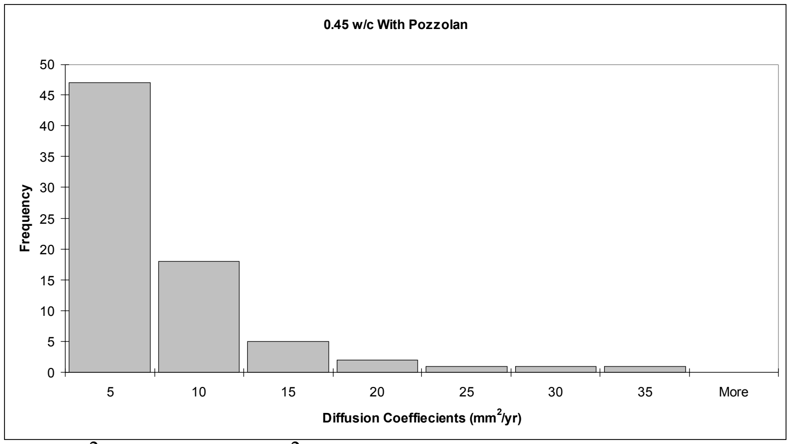
*1 kg/m^3 = 1.686 pcy

Figure 31 – Surface Chloride Concentrations – 0.45 $^{w}/_{c}$ With Pozzolan



* 1 mm = 0.039 in.

Figure 32 – Cover Depth Measurements – 0.45 $^{w}/_{c}$ With Pozzolan



*1 $mm^2/yr = 0.0015\ in^2/yr$

Figure 33 – Diffusion Coefficients – 0.45 $^{w}/_{c}$ With Pozzolan

The histograms presented above indicate that C_o, and D_c may be reasonably described by the gamma distribution while cover depths follow a normal distribution. For service life modeling purposes it may be possible to assign a distribution to the input parameters in cases where sufficient data is not available to permit the use of simple bootstrapping. For the case of modeling the bridge deck sets this is not necessary due to the large quantity of data available.

ANALYSIS AND DISCUSSION

Validation of Service Life Model

The service life model that will be used for this project has been previously validated by comparing the predicted service lives for a set of 129 bridge decks in Virginia to their actual recorded service lives. (Kirkpatrick, 2001) To further validate the model the predicted service lives for the 0.47 $^{w}/_{c}$ bridge decks were compared to actual measured deterioration levels. The 0.47 $^{w}/_{c}$ bridge decks were investigated because they are the oldest and reflect the most damage. The younger 0.45 $^{w}/_{c}$ bridge decks have insufficient damage to correlate the service life model to actual field conditions. Ten estimations of the time to corrosion initiation were conducted using the actual field determined input data presented in the previous section and the average time is presented in Table 12. The individual data measurements were used in the corrosion initiation estimations as opposed to the average values in order to incorporate the effects of input data variability.

Table 12 – Average Time to Corrosion Initiation (0.47 $^{w}/_{c}$ Bare Steel Bridge Decks)

% Corroded	Average Time to Corrosion Initiation (years)	Standard Deviation (years)	C.V. (%)
2	30.7	1.5	5.0

As shown the standard deviation is 1.5 years with a C.V. of 5.0 %. This level of variability has been deemed acceptable for this application.

As determined previously using the Liu/Weyers and Vu, Stewart, and Mullard corrosion propagation models the time to excessive concrete cracking is estimated to be 6 years for bare steel bridge decks. Thus, the total time period for the 0.47 $^{w}/_{c}$ set to reach a deterioration level of 2% is 37 years.

From the damage surveys that were conducted on the 0.47 $^{w}/_{c}$ bridge decks, approximately 2% of the total area of the bridge deck set is currently showing signs of corrosion damage (spalling, patches, or delaminations). The average age of the decks is 36.4 years with a range of 3 years and the predicted age of the set is 37 years. Thus, for this set of bridge decks the service life model appears to be an accurate predictor of service life for at least low levels of damage.

Comparison of Key Parameters

The key service life parameters (D_c, C_o, and x) from the three bridge deck sets were investigated to determine if there were significant differences between the sets. Single factor Analysis of Variance (ANOVA) comparisons were conducted for each combination of sets at the 95% confidence limit. The average values for the key parameters are presented in Table 13.

Table 13 – Key Parameter Summary

Parameter	0.45 $^w/_c$ no Pozzolan	0.45 $^w/_c$ with Pozzolan	0.47 $^w/_c$ no Pozzolan
Diffusion Coeff. (mm²/yr)	26.3	3.45	10.93
Surface Chloride Conc. (kg/m³)	3.47	3.95	3.53
Cover Depths (mm)	59.4	61.5	52.18

*1 mm^2/yr = 0.0015 in^2/yr; 1 kg/m^3 = 1.686 pcy; 1 mm = 0.039 in.

Differences in D_{ca} at a projected age of 35 years were investigated first. It was determined that there are significant differences between all sets. The ANOVA's for the D_{ca} comparisons are presented in Figures 34 - 36. The differences between projected D_{ca} values indicate that the quality of the concrete varies significantly between sets. This was expected due to the changes in construction specifications.

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
0.45 - No Pozz.	137	3603.2	26.3	567.4
0.45 - With Pozz.	75	258.6	3.4	12.2

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	25311.2	1	25311.2	68.1	1.7E-14	3.9
Within Groups	78065.1	210	371.7			
Total	103376.2	211				

Figure 34 – D_{ca} Comparison – 0.45 No Pozz. to 0.45 With Pozz.

SUMMARY

Groups	Count	Sum	Average	Variance
0.45 - With Pozz.	75	258.6	3.4	12.2
0.47 - No Pozz.	81	885.3	10.9	180.1

ANOVA

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	2179.8	1	2179.8	21.9	6.2E-06	3.9
Within Groups	15310.9	154	99.4			
Total	17490.7	155				

Figure 35 – D_{ca} Comparison – 0.45 With Pozz. to 0.47 No Pozz.

SUMMARY

Groups	Count	Sum	Average	Variance
0.47 - No Pozz.	81	885.3	10.9	180.1
0.45 - No Pozz.	137	3603.2	26.3	567.4

ANOVA

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	12026.6	1	12026.6	28.4	2.51E-07	3.9
Within Groups	91575.6	216	424.0			
Total	103602.3	217				

Figure 36 – D_{ca} Comparison – 0.47 No Pozz. to 0.45 No Pozz.

The ANOVA's presented above can be interpreted by comparing the calculated F value to the F_{crit} value as well as by comparing the p-value to the 95% confidence level. The groups being compared are said to be statistically different if the calculated F value is greater than the F_{crit} value. The greater the difference between the two values, the greater the statistical difference. Additionally, if the calculated p-value is less than 1 minus the confidence limit, 1- 0.95 = 0.05 for this case, the groups being compared are said to be statistically different. The lower the p-value, the greater the confidence in the decision will be. As presented above, the confidence in the statistical difference between the calculated D_c values of the three data sets is high due to the large differences between F and F_{crit}. The calculated p-values are also well below the 0.05 value associated with a confidence limit of 95%.

The second parameter to be investigated was C_o. It was determined that the differences between the average surface chloride concentrations are not significant with the calculated F values being lower than F_{crit} and the p-values being greater than 0.05 in all cases. This indicates that C_o is not related to length of exposure (beyond 10-15 years of age) or concrete type for the study groups. Additionally, this confirms that all three sets have similar exposure conditions. The ANOVA's for C_o are presented in Figures 37 - 39.

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
0.45 - No Pozz.	137	476.0	3.5	3.7
0.45 - With Pozz.	75	296.1	3.9	5.4

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	10.9	1	10.9	2.5	0.11	3.9
Within Groups	895.5	210	4.3			
Total	906.4	211				

Figure 37 – C_o Comparison – 0.45 No Pozz. to 0.45 With Pozz.

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
0.45 - With Pozz.	75	296.1	3.9	5.4
0.47 - No Pozz.	81	285.9	3.5	6.1

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	6.8	1	6.8	1.2	0.3	3.9
Within Groups	884.5	154	5.7			
Total	891.3	155				

Figure 38 – C_o Comparison – 0.45 With Pozz. to 0.47 No Pozz.

SUMMARY

Groups	Count	Sum	Average	Variance
0.47 - No Pozz.	81	285.9	3.5	6.1
0.45 - No Pozz.	137	476.0	3.5	3.7

ANOVA

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	0.2	1	0.2	0.0	0.9	3.9
Within Groups	985.4	216	4.6			
Total	985.6	217				

Figure 39 – C_o Comparison – 0.47 No Pozz. to 0.45 No Pozz.

The last input parameter to be investigated was cover depth. The cover depth comparison ANOVA's are presented in Figures 40 - 42. It was determined that there were significant differences between all three bridge deck sets. The F values were much greater than the F_{crit} values for comparisons between .45 $^{w}/_{c}$ bridge decks and .47 $^{w}/_{c}$ bridge decks. This was expected because the 0.45 $^{w}/_{c}$ bridge decks were constructed under specifications that required larger cover depths than the 0.47 $^{w}/_{c}$ bridge decks. The cover depth requirement was increased at the same time that the $^{w}/_{c}$ specifications were decreased. However, it is interesting that there is a significant difference between those decks containing pozzolan and those that do not although the average difference is to be considered small, 2.1 mm (0.083 in.). Whereas, the other differences are considerably larger, 9 mm (0.354 in.) and 7.2 mm (0.283 in.) for $^{w}/_{c}$ of 0.47 and 0.45 with pozzolan and 0.47 and 0.45 without pozzolan, respectively.

SUMMARY

Groups	Count	Sum	Average	Variance
0.47 - No Pozz.	704	36732.7	52.2	119.2
0.45 - With Pozz.	709	43633.9	61.5	79.0

ANOVA

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	30985.5	1	30985.5	312.9	2.14E-63	3.8
Within Groups	139733.5	1411	99.0			
Total	170719.0	1412				

Figure 40 – Cover Depth Comparison – 0.47 No Pozz. to 0.45 With Pozz.

SUMMARY

Groups	Count	Sum	Average	Variance
0.45 - With Pozz.	709	43633.9	61.5	79.0
0.45 - No Pozz.	1291	76688.4	59.4	76.4

ANOVA

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	2096.9	1	2096.9	27.1	2.11E-07	3.8
Within Groups	154489.9	1998	77.3			
Total	156586.8	1999				

Figure 41 – Cover Depth Comparison – 0.45 With Pozz. to 0.45 No Pozz.

SUMMARY

Groups	Count	Sum	Average	Variance
0.45 - No Pozz.	1291	76688.4	59.4	76.4
0.47 - No Pozz.	704	36732.7	52.2	119.2

ANOVA

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	23782.4	1	23782.4	259.9	4.63E-55	3.85
Within Groups	182352.7	1993	91.5			
Total	206135.1	1994				

Figure 42 – Cover Depth Comparison – 0.45 No Pozz. to 0.47 No Pozz.

Global Analysis

As stated previously the primary objective of this project is to model the service life of the investigated bridge decks as three distinct sets: 0.47 $^{w}/_{c}$, 0.45 $^{w}/_{c}$, and 0.45 $^{w}/_{(c+p)}$. It is important to note that the 0.45 $^{w}/_{c}$ bridge decks contain ECR, however, the effect of the epoxy coating will be disregarded initially. This was done in order to investigate the influence of the specification changes relating to increased cover depths, lowered $^{w}/_{c}$ ratios, and the addition of pozzolans. Additionally, although a statistical analysis of the C_o values indicated that there was no significant difference between sets, the C_o values from the 0.47 $^{w}/_{c}$ bridge deck set were used during the analysis of all three sets to remove any variability related chloride exposure.

0.47 $^{w}/_{c}$ Bridge Decks

The 0.47 $^{w}/_{c}$ set was the first set to be analyzed and the results were used to validate the service life model. The input data that was used for the service life model included 84 C_o measurements, 704 cover depth measurements, and 84 D_{ca} values adjusted to an age of 35 years. The 22 years added to the time to 2% corrosion initiation is the result of 6 years to corrosion cracking of 2% of the bridge deck plus 16 years for the damage to propogate from 2% to 12%, the time at which an overlay is needed. The results of the service life model are presented in Table 14.

Table 14 – Service Life Estimates – 0.47 $^{w}/_{c}$ Bare Steel Bridge Decks

	0.47 $^{w}/_{c}$ no Pozzolan		
	Time (years)	St. Dev. (years)	C. V. %
Time to 2 % Initiation	31	1.5	5.0
Estimated Service Life	53	-----	-----

0.45 $^{w}/_{c}$ and 0.45 $^{w}/_{(c+p)}$ Bridge Decks

The same methodology was used to estimate the influence of the concrete quality used to construct the 0.45 $^{w}/_{c}$ bridge decks. By doing so, a direct comparison of concrete quality between sets is possible. The data used in the service life model for concrete with and without pozzolan was presented in Figures 28 - 30 previously. The total number of data points used was as follows: No Pozzolan; D_{ca} at 35 years – 114 values, Cover Depth – 1105 values; With Pozzolan; D_{ca} at 35 years – 98 values, Cover Depth – 868 values.

The 84 surface chloride concentrations from the 0.47 $^{w}/_{c}$ bridge decks were used as the input for the 0.45 $^{w}/_{c}$ decks in order to remove any environmental influence from the analysis. Additionally, the D_{ca} values that were calculated for the 0.45 $^{w}/_{c}$ decks were adjusted to reflect the time-dependency of the diffusion values as described previously. The results of the service life model are presented in Table 15 for the 0.45 $^{w}/_{c}$ bridge decks.

Table 15 – Service Life Estimates - 0.45 $^{w}/_{c}$ Bridge Decks

	0.45 $^{w}/_{c}$ no Pozzolan			0.45 $^{w}/_{c}$ with Pozzolan		
	Time (years)	St. Dev. (years)	C. V. %	Time (years)	St. Dev. (years)	C. V. %
Time to 2 % Initiation	21	0.4	1.9	169	4.8	2.8
Estimated Service Life	43	-----	-----	191	-----	-----

Comparison of 0.47 $^{w}/_{c}$ and 0.45 $^{w}/_{c}$ Bridge Decks

Analysis of the service life estimates at the 2% corrosion initiation level for the three bridge deck sets demonstrated that the 0.45 $^{w}/_{c}$ bridge decks with pozzolan were predicted to be the most durable. The results demonstrate the influence of larger cover depths and the lower D_{ca} values associated with the 0.45 $^{w}/_{(c+p)}$ set of bridge decks. More importantly, however, is the lower predicted service life of the 0.45 $^{w}/_{c}$ OPC bridge decks as compared to the 0.47 $^{w}/_{c}$ OPC bridge decks, 21 years and 31 years, respectively. Although the predictions take into account the larger cover depths of the 0.45 $^{w}/_{c}$ decks, the larger D_{ca} values have negated the influence of the larger cover depths. The higher D_{ca} values for the 0.45 OPC decks may be due to quality control/construction issues with the newer 0.45 $^{w}/_{c}$ OPC decks. It should be noted that the 10 - 0.47 $^{w}/_{c}$ bridge decks used in this study were selected from a previous study sample of which only 68% survived. (Kirkpatrick, 2001) The bridges that had already been rehabilitated were removed from the data set, as an accurate chloride analysis and damage survey would be impossible. Therefore, the 10 decks selected were from the better performing 68%, which may place some bias on the sample set.

The corrosion initiation period for 2% damage estimates illustrates that the addition of pozzolan improves the long-term durability of bridge decks by decreasing the concrete diffusivity. It appears that reducing the $^{w}/_{c}$ of the concrete had a negligible effect. It can be inferred that the study sample of the older 0.47 $^{w}/_{c}$ concrete is of a higher quality than the newer 0.45 $^{w}/_{c}$ study sample.

Alternative Reinforcements

As discussed previously, alternative reinforcements can potentially increase the service life of bridge decks substantially. In order to analyze the various alternatives, D_c, C_o, and cover depth values from the concrete containing pozzolans were used as input to the service life model. These values were selected for use because they best represent the current state of bridge deck construction in Virginia.

Epoxy-Coated Reinforcement

The corrosion initiation concentrations used for ECR were the same values that were used for bare steel because research conducted by the FHWA concluded that "the chloride threshold for damaged epoxy-coated bars is similar to that of black bars." (McDonald et al., 1998) Thus, areas of the epoxy coating that are damaged during construction will initiate at the same chloride levels as bare steel and those areas will then continue to propagate along the length of the reinforcing bar. However, there is evidence that the corrosion propagation rate for ECR is lower than that of bare steel, which will result in some additional service life for bridge decks containing ECR. (McDonald et al., 1998; Brown, 2002)

Galvanized Steel

The chloride threshold values for galvanized steel are estimated to be 2.5 times greater than carbon steel. (Yeomans, 2004) Corrosion initiation values were generated by simply multiplying the bare steel initiation values by 2.5.

Stainless Steel

Chloride threshold values for stainless steel have been reported to be at least 10.4 times greater than carbon steel. (Clemeña, 2003)

MMFX-2 Steel

Several research projects have attempted to estimate the chloride threshold value for MMFX-2. The reported range of initiation values is 3.5 – 9.2 times that of carbon steel.

(Darwin et al, 2003; Trejo, 2002) A conservative initiation value of 3.5 times that of carbon steel was used.

Service-Life Estimates

Using the adjusted chloride threshold values where appropriate, service-life estimates were performed for the above reinforcement alternatives. The estimates should provide a reasonable expectation of the time required for corrosion to initiate. It should also be noted that alternative reinforcements will have longer corrosion propagation times, which are not included in the service-life estimates presented below. (See Table 16)

Table 16 – Service Life Predictions (Alternative Reinforcements) – A4 Concrete - $^{w}/_{c}$ = 0.45 with pozzolan

	Time to Initiation (years)				
Reinforcement Type	**Bare**	**Epoxy-Coated**	**Galvanized**	**Stainless**	**MMFX-2**
Time to 2 % Initiation	160	160	602	9999	1312

As shown, bridge decks constructed using current bridge deck specifications (0.45 $^{w}/_{c}$ with pozzolan and 2 in. minimum cover depths) will in most cases never initiate corrosion within the desired service life of 100 years regardless of reinforcement type. Galvanized, stainless steel, and MMFX-2 reinforcement will provide a reliable second, redundant corrosion protection system where such systems are required.

Individual Deck Analysis

After validating the service life model and comparing the service life estimates of the three bridge deck sets the potential for the model to estimate the service life of an individual deck was investigated. The data collected for each individual bridge deck was used as input to the service life model and the results compared to actual field conditions. The input generally consisted of 9 C_o values, 9 D_{ca} values, and 40-80 cover depth, x values. The number of values for C_o and D_{ca} was lower in some instances due to missing or inconsistent data and the number of cover depth measurements was dependent upon the length of the deck.

The ability of the service life model to accurately estimate the service lives of individual bridge decks was first investigated using the 0.47 $^{w}/_{c}$ bridge deck data. The service life estimates for those ten bridge decks are presented in Table 17.

Table 17 – 0.47 $^{w}/_{c}$ Bridge Deck Service Life Estimates - Individual

Structure #	% Damage	Time to Initiation 2% or less (years)	Time to Corrosion Cracking	Propagation Time (years)	Estimated Age (years)	Actual Age (years)	Difference (years)
1-1804	5.3	13	6	6	25	37	12
1-6101	0.0	> 100	6	---	---	37	---
2-2007	2.3	18	6	1	25	36	11
3-1021	0.2	11	6	---	17	35	18
4-1062	0.0	> 100	6	---	---	37	---
4-2049	0.7	> 100	6	---	100	38	-62
5-1800	3.3	> 100	6	3	100	36	-64
6-1032	0.0	> 100	6	---	---	35	---
9-2801	3.0	72	6	2	80	36	-44
9-6042	1.1	32	6	---	38	37	-1

As is evident in Table 17, the model is not capable of accurately estimating the service life for individual bridge decks on a consistent basis with the amount of data available. The model overestimates the service life in some cases and underestimates in others. It is believed that the models' inability to accurately predict the service lives of individual bridge decks is related to the quantity of input data. Low quantities of input data may not accurately represent the characteristics of the bridge decks. The parameter where this is most evident is D_{ca}. It is possible for an individual bridge deck to have a coefficient of variation in excess of 100% for D_{ca}. Therefore, the probability of 9 values being able to accurately define the distribution is low. Due to the observed inability of the model to predict the service lives of the 10 bridge decks investigated, any analysis of individual bridge decks should be viewed with caution. The required amount of input data is currently unclear. It may be possible, however, to determine the probability of corrosion initiating based upon a limited number of diffusion parameters measured in the field.

The estimated times for 2% corrosion initiation for all of the bridge decks are presented in Table 18. As mentioned previously, due to the limited amount of input data available, these estimations should be viewed with caution.

Table 18 – Corrosion Initiation Estimations for Individual Bridge Decks

0.47 $^{w}/_{c}$				0.45 $^{w}/_{c}$				0.45 $^{w}/_{(c+p)}$			
Structure #	Avg. Time to 2% Initiation (years)	St. Dev. (years)	C.V. %	Structure #	Avg. Time to 2% Initiation (years)	St. Dev. (years)	C.V. %	Structure #	Avg. Time to 2% Initiation (years)	St. Dev. (years)	C.V. %
1-1804	13	0.2	1.8	**1-1132**	17	0.6	3.7	**1-1152**	333	1.8	0.5
1-6101	2027	64.7	3.2	**1-1133**	7	0.1	2.0	**1-2815**	252	7.7	3.0
2-2007	18	0.3	1.5	**1-2820**	66	0.8	1.2	**1-2819**	63	4.9	7.8
3-1021	16	0.5	3.2	**1-6051**	28	1.0	3.5	**2-1021**	286	1.0	0.3
4-1062	325	14.5	4.5	**2-1020**	32	0.7	2.3	**3-1000**	1181	97.5	8.3
4-2049	111	2.1	1.9	**3-1003**	28	0.8	2.7	**3-1017**	---	---	---
5-1800	9999	0.0	0.0	**4-1007**	123	2.4	2.0	**5-2812**	399	4.4	1.1
6-1032	260	9.4	3.6	**4-2901**	138	5.1	3.7	**8-1002**	259	3.9	1.5
9-2801	72	3.4	4.7	**5-2547**	847	76.1	9.0	**8-1042**	144	4.0	2.8
9-6042	36	0.7	1.8	**7-1920**	14	0.2	1.4	**9-1002**	161	10.4	6.4
				8-1019	26	0.7	2.8	**9-6058**	191	1.4	0.7
				8-1133	47	1.4	3.1				
				9-1014	61	2.3	3.7				
				9-1031	27	0.6	2.4				
				9-1098	154	6.6	4.3				
				9-1139	28	0.5	1.8				

Although analysis of individual service life estimates may not be possible general observations can be made. Of the bridge decks containing pozzolan only one of 10 is expected to initiate corrosion within its' 100-year design life as compared to 12 out of 16 and 5 out of 10 for the 0.45 $^{w}/_{c}$ and 0.47 $^{w}/_{c}$ sets, respectfully.

Effects of Variability and the Number of Iterations

The number of iterations required for the service life estimates to converge to a near constant value was investigated. Five successive runs of the service life model were completed for 8 iteration levels. The means and 95% confidence limits were calculated for each of the eight levels and are presented in Figure 43.

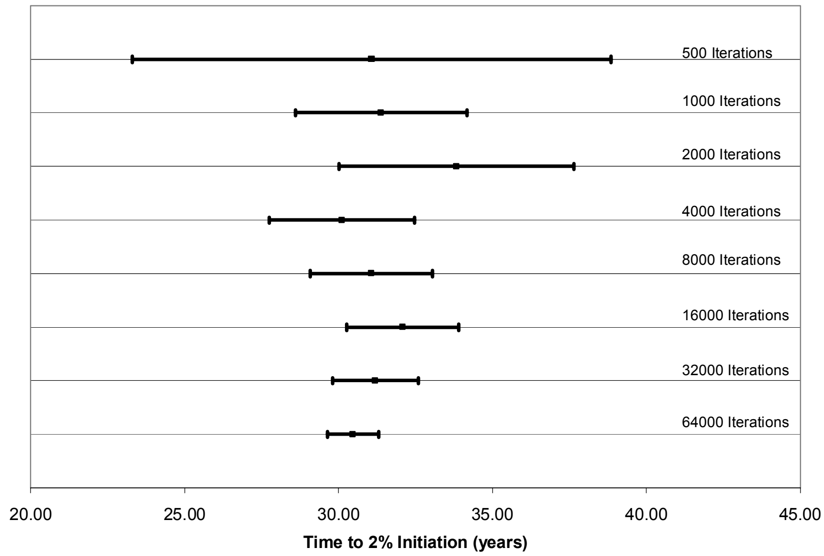


Figure 43 – Iteration Analysis for 0.47 $^{w}/_{c}$ Bridge Deck Set

As shown, 95% confidence intervals range from a maximum of +/- 7.8 years for 500 iterations to a minimum of +/- 0.8 years for 64000 iterations. The service life model is currently capable of 65000 iterations, which yields a 95% confidence interval of less than +/- 0.8 years. That level of variability has been deemed acceptable for bridge deck service life estimates and was the number of iterations used in all of the previously presented analyses.

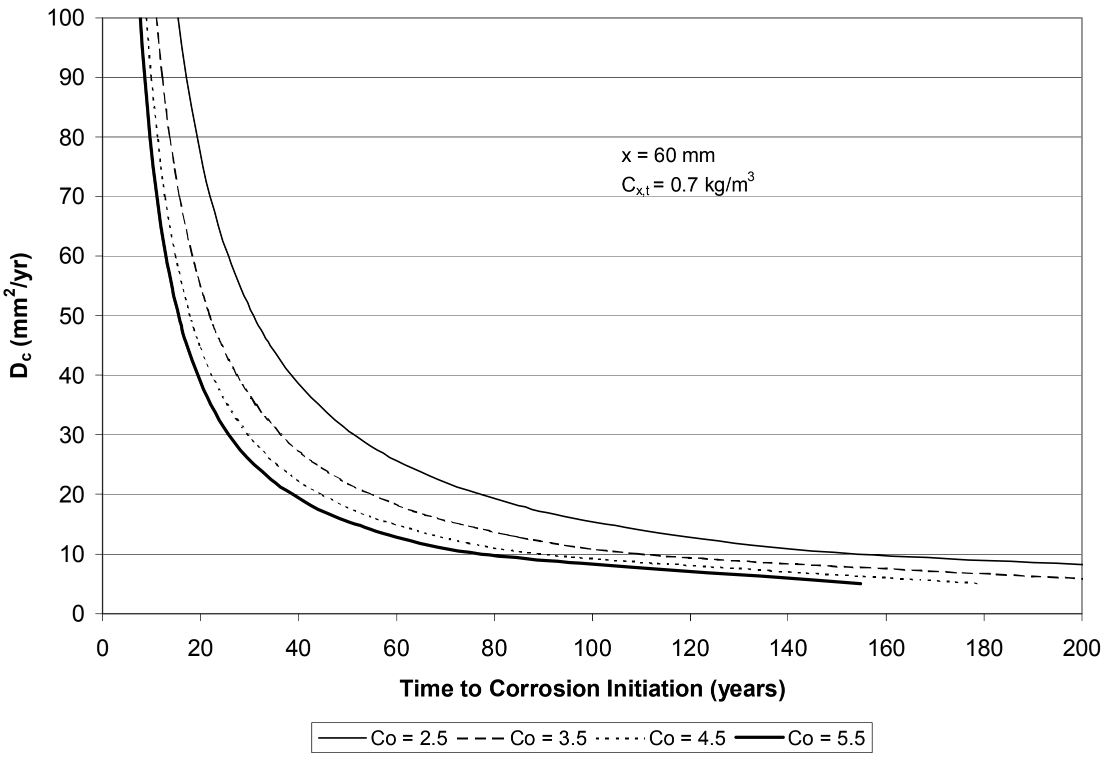
It is important to note that the precision of the model is not at all related to the accuracy of the service life estimations. While the model has a 95% confidence interval of +/- 0.8 years the accuracy of service life estimations remains dependent upon the accuracy and quantity of the input data as well as the models' ability to accurately estimate the mechanisms of deterioration. Service life estimations are sensitive to variations in the input data, so it is pertinent that great care be taken with field and laboratory data collection. The sensitivity of each individual input parameter is investigated in the following section.

Sensitivity Analysis

The effect of the sensitivity of the input variables on the estimated time to corrosion initiation was investigated. Understanding the sensitivity of the input parameters is important for identifying possible sources of error and variability in the service life estimations. The more sensitive that an input parameter is to the estimation the more important it is to determine accurate values for that parameter.

Apparent Diffusion Coefficient (D_{ca})

The relationship between D_{ca} and time to corrosion initiation is presented in Figure 44.



*1 mm^2/yr = 0.0015 in^2/yr

Figure 44 – Apparent Diffusion Coefficient vs. Time to Corrosion Initiation

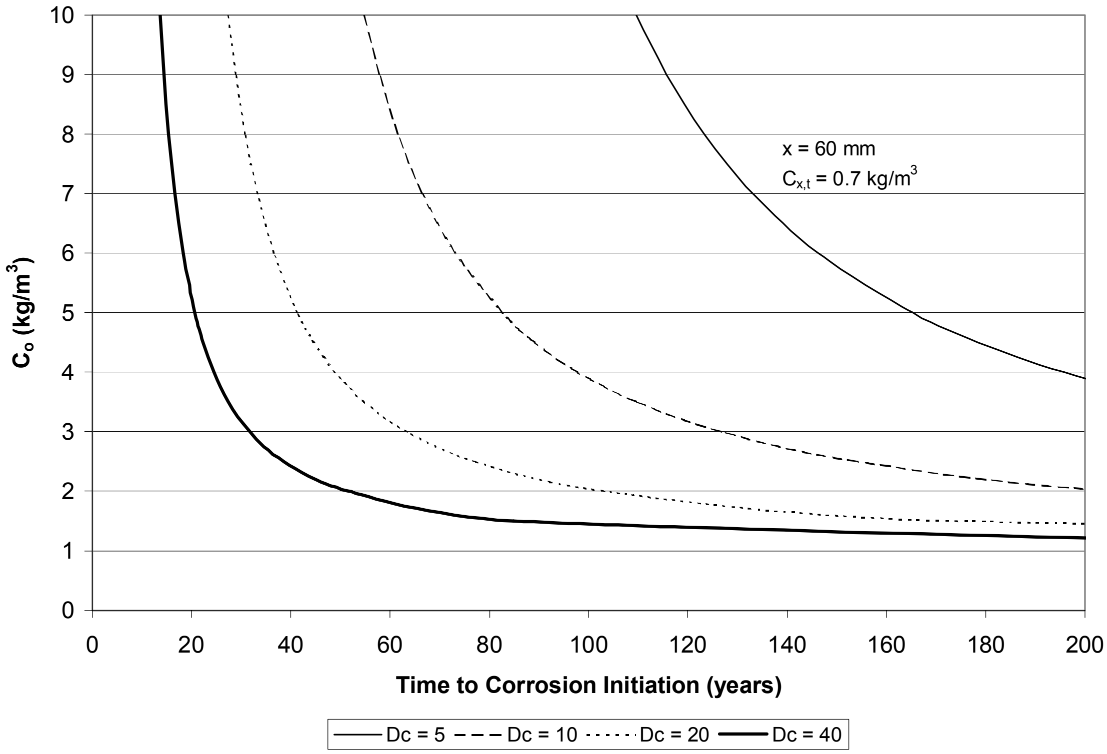
The relationship becomes asymptotic for both high and low values of D_{ca}. As shown, the changes in the time to corrosion initiation are relatively small for D_{ca} values greater than 40 mm^2/yr (0.06 in^2/yr). However, for small changes in D_{ca} less than 10, the increase in time to initiation can be large, doubling from 80 to 160 years from 10 to 5 mm^2/yr (0.0075 in^2/yr) for example. Variations in D_{ca} values from 10 and 40 mm^2/yr (0.015 and

0.06 in^2/yr) will yield relatively large changes in the estimated time to corrosion initiation, generally about 60 years. These relationships are generally valid for all four concentrations of C_o presented and as expected lower levels of C_o will result in longer times to corrosion initiation. However, the additional time to corrosion initiation that can be attributed to lower levels of C_o is relatively small in comparison to the effect of D_{ca}.

As presented in the previous Diffusion Coefficient section the average D_{ca} values for the sampled bridge decks are generally in the range of 1 – 40 mm^2/yr (0.0015 – 0.06 in^2/yr). The sensitivity analysis indicates that D_{ca} values within this range are highly sensitive, which gives reason for some concern. D_{ca} values are the most difficult input parameter to obtain and are generally the most variable. For that reason it is recommended that D_{ca} be determined from field data whenever possible as opposed to using experimental results to set values for D_{ca}. The sensitivity analysis also reinforces the importance of using a range of D_{ca} values rather an average value, as D_{ca} can vary significantly within an individual bridge deck.

Surface Chloride Concentration (C_o)

The relationship between C_o and the time to corrosion initiation is presented in Figure 45.



*1 kg/m^3 = 1.686 pcy

Figure 45 – Surface Chloride Concentration vs. Time to Corrosion Initiation

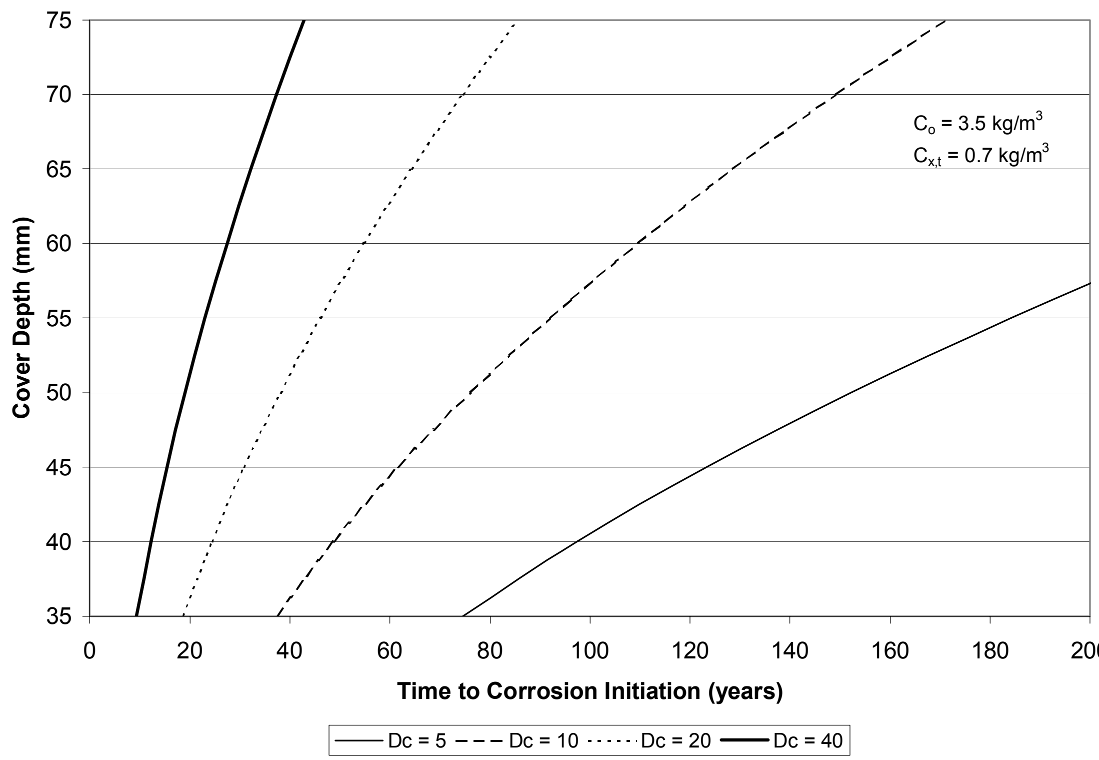
The relationship is presented for a $C_{x,t}$ value of 0.7 kg/m^3 (1.18 pcy) and a cover depth of 60 mm (2.36 in.). The time to corrosion initiation becomes asymptotic for high and low values of C_o regardless of D_{ca}. However, the trend is less pronounced for low values of D_{ca}.

As shown, bridge decks with surface chloride concentrations of less than 1.5 kg/m^3 (2.53 pcy) have a very low probability of initiating corrosion within their desired service life. Whereas, bridge decks with surface chloride concentrations greater than 4 kg/m^3 (6.74 pcy) are much more likely to initiate corrosion depending upon the diffusion properties of the concrete.

The average C_o values for the sampled bridge decks presented in the results section were generally in the range of 1 – 6.5 kg/m^3 (1.69 – 10.96 pcy). The service life estimates are fairly sensitive to C_o values within this range for bridge decks with D_{ca} values greater than 20 mm^2/year (0.03 in^2/yr). However, for bridge decks with D_{ca} values less than 10 mm^2/yr (0.015 in^2/yr) the magnitude of C_o is irrelevant because of the large estimated time to initiation values.

Cover Depth (x)

The relationship between cover depth and time to corrosion initiation is presented in Figure 46.



*1 mm = 0.039 in.

Figure 46 – Cover Depth vs. Time to Corrosion Initiation

For cover depths greater than 35 mm (1.38 in.) it is reasonable to model the relationship between cover depth and time to corrosion initation using a linear function. The slope of the line is dependent upon the associated diffusion coefficient. As shown, the cover depth is the least sensitive input parameter of the three. The cover depth is also the input parameter with the least variability.

The effect of cover depth variability on the service life estimations of bridge decks constructed with low permeable concrete (D_{ca} < 10 mm^2/yr [0.015in^2/yr]) is irrelevant because in most instances corrosion will not initiate within a 100 year design life. However, for bridge decks with highly permeable concrete (D_{ca} > 20 mm^2/yr [0.03 in^2/yr]) the effect is more significant.

LIFE CYCLE COST ANALYSIS (LCCA)

The total cost of a bridge deck cannot be described by a one-time construction expenditure. There are always agency costs related to the operation, maintenance, repair and rehabilitation of a bridge deck throughout its' service life. Often times there are many alternatives available for maintaining a bridge deck, with different costs and timings associated with each. Due to the variety of bridge management options available it has become necessary to analyze the alternatives and select the most economical option. This, however, is not as simple as summing the expected costs for each alternative. The time-value of money and the affect on the user must also be considered. To assist in the comparison of alternatives an approach referred to as Life-Cycle Cost Analysis (LCCA) is commonly used. LCCA is a methodology that has received much attention in recent years as a means of comparing available management alternatives. LCCA as related to bridges can be defined as "a set of economic principles and computational procedures for comparing initial and future costs to arrive at the most economical strategy for ensuring that a bridge will provide the services for which it was intended." (Hawk, 2003)

LCCA may be separated into the five basic steps listed below:

1. Establish design alternatives
2. Determine activity timing
3. Estimate costs (agency and user)
4. Compute life-cycle costs
5. Analyze the results (FHWA, 2002)

Step 1: Establishing Design Alternatives

The first step in conducting a LCCA is to define the possible design/maintenance alternatives. To begin with, there is the possibility of having multiple initial construction options. Each construction option will have an associated maintenance expectation. Secondly, the different maintenance strategies must be identified. When establishing the design alternatives to be compared it is best to vary only one parameter at a time. For example if different initial construction options are being compared, the maintenance strategy selected should be the same for both. However, the implementation times of the individual maintenance activities will most likely be different for comparison design alternatives. Otherwise, it may not be possible to identify what parameter is contributing to the lower cost, initial construction or maintenance strategy. After a construction option is selected the possible maintenance alternatives can then be investigated.

Step 2: Determine Activity Timing

Once the possible design alternatives have been identified the timing of the maintenance/repair activities must be determined. Different maintenance/repair activities will extend the service life of a bridge deck by varying degrees. Therefore, the timing of subsequent repairs will be affected by the initial maintenance strategy selected. For example, a latex concrete overlay may extend the service life of a bridge deck by 25 years whereas a polymer overlay may only extend the service life by 15 years after which time additional repair/rehabilitation measures must be taken. Determining the appropriate timing of the design alternatives is important in that it will affect the total amount of maintenance that will be projected for an alternative within the analysis time period. Additionally, the time value of money plays a large role in LCCA, so if the selected timings are not accurate the resulting life cycle cost estimations may not be reliable.

One important aspect of LCCA is that all design/maintenance alternatives must be compared using a common service life period. Thus, the desired service life of a structure must be predetermined and any remaining service life provided by a maintenance alternative after the end of the analysis period will not be considered. The remaining service life may be accounted for by assigning a positive salvage value at the end of the analysis. However, this may not be desirable as a predetermined service life may indicate that the structure is expected to be functionally obsolete at that time and would require replacement regardless of the state of deterioration. Therefore, any remaining service life would be irrelevant.

Step 3: Estimate Costs (Agency and User)

The costs that are necessary to conduct a LCCA are all the costs associated with each design alternative that are not common to all alternatives. For example the costs of different overlays must be provided, but the costs of biannual inspections do not because they are required for all alternatives regardless of overlay type. The costs that are required include not only those related to the actual construction/repair (Agency Costs), but also the costs related to the impact on the user (User Costs).

Agency Costs

Agency costs are those costs associated with the design, initial construction, maintenance (preventative and corrective), rehabilitation, and replacement of a structure. They will also include costs incurred that are related to traffic control and mobilization. Agency costs are in general much easier to estimate than user costs. Estimates can be made for a particular activity based upon historical bid records and engineering judgement.

The salvage value of a structure is another agency cost that may be considered. It is defined as the value of any alternative at the end of the analyis period. The salvage value may be positive for alternatives with a remaining service life or if the structure/element has some residual value or it may be negative in instances where there is a cost associated with the removal and disposal of material.

User Costs

For a LCCA to be complete the indirect costs incurred by the public (users) for each alternative must be considered. These costs "typically arise from the timing, duration, scope, and number of construction and rehabilitation work zones characterizing each project alternative." (FHWA, 2002) The user costs associated with transportation projects can generally be separated into three categories: Vehicle Delay Costs, Vehicle Operating Costs, and Accident Costs. The three user cost categories can be defined as follows:

1. Vehicle Delay Costs - the personal cost to drivers delayed by roadwork;

2. Vehicle Operating Costs - the costs associated with the operation of vehicles delayed by roadwork; and
3. Accident Costs - the cost of damage to vehicles and injury to people due to roadwork.

The following formulas have been developed to estimate the user costs associated with a given construction alternative.

"Equation 33 can be used to compute the cost to drivers of roadwork-related traffic delays.

$$VehicleDelayCosts = \left(\frac{L}{S_a} - \frac{L}{S_n} \right) \times ADT \times N \times w \qquad \text{Equation 33}$$

where:
L is the length of affected roadway or which cars drive,
S_a is the traffic speed during bridge work activity,
S_n is the normal traffic speed,
ADT is the average daily traffic, measured in number of cars per day,
N is the number of days of road work, and
w is the dollar value of each hour of a driver's time.

The hourly value w is a weighted average of commercial vehicle drivers' and personal automobile drivers' time.

Vehicle operating costs can be calculated using Equation 34.

$$VehicleOperatingCosts = \left(\frac{L}{S_a} - \frac{L}{S_n} \right) \times ADT \times N \times r \qquad \text{Equation 34}$$

where r is a weighted-average vehicle cost similar to the weighted cost in Equation 33 and the remaining parameters are the same as those in Equation 33.

Accident costs can be calculated using Equation 35.

$$AccidentCosts = L \times ADT \times N \times (A_a - A_n) \times c_a \qquad \text{Equation 35}$$

where c_a is the cost per accident, A_a and A_n are the during-construction and normal accident rates per vehicle-kilometer, and the remaining parameters are the same as those listed in Equations 33 and 34." (Ehlen, 2003)

Step 4: Compute Life-Cycle Costs

Once the design alternatives have been defined and their associated activity timings and costs determined their LCC's may be calculated. LCC comparisons are based upon the economic principle that there is a time-value of money. Thus, depending upon the discount rates and inflation rates the value of one dollar now will be higher or lower in the future. The concept of the time-value of money is discussed in more detail in the following section. The importance of this concept is to realize that the anticipated cost of future repairs cannot be simply added together to determine the LCC for a specific design alternative.

In order to conduct a LCCA a base year for comparison must be selected. Generally, the current year is selected to allow for the use of simple present worth computations and current cost data. The equation that is used to calculate the present worth of a single future expenditure is presented below.

$$PW = \frac{FV}{(1+r)^n} \qquad \text{Equation 36}$$

where:

PW = Present Worth

FV = Future Value

r = Real discount rate (Nominal discount rate – Inflation)
n = Number of years in the future that the cost will be incurred

In some cases it may be desirable to model future expenditures as either a uniform cost over a given number of years or as a gradient series. A gradient series would be used for expenditures that increase year-over-year. For example, as a bridge deck begins to deteriorate it will require more patching in year 5 than in year one. Therefore, a gradient series may be used to model that increase in maintenance costs. A uniform series would be used in cases where the expenditure is expected to remain constant for a set time period. For example, a bridge deck may require annual maintenance throughout its service life. Equations 37 and 38 are used to convert these expenditures to a present-worth value.

$$PWUS = \frac{(1+r)^n - 1}{r(1+r)^n} \qquad \text{Equation 37}$$

$$PWGS = \frac{1}{r(1+r)^n} \times \left[\frac{(1+r)^n - 1}{r} - n \right] \qquad \text{Equation 38}$$

where:
PWUS = Present Worth of a Uniform Series
PWGS = Present Worth of a Gradient Series

Typically, a single cost, uniform series, or a gradient series will occur at some specified point in the future. To convert the present worth of a single expenditure calculated for a uniform or gradient series to the present worth in the base year Equation 36 is used.

Time-value of Money

As mentioned previously, LCCA is based upon the concept that money has a time-value. The basis of time-value lies in the idea that money not spent today can be placed into an investment vehicle, which will provide some return over time. For example, if $100 were placed into an investment account that provides an interest rate of 5% per year, one year later that account would be worth $105. Therefore, $105 one year from now would have an equivalent value of $100 today, in this example. To account for the time-value of money discount rates and inflation rates are used.

Discount Rate

The discount rate "is a parameter used to calculate the equivalent worth of economic resources used or received now or in the future." (Hawk, 2003) The discount rate can be given as the nominal discount rate (the discount rate including inflation), or the real discount rate (the discount rate less inflation). LCC comparisons use real discount rates. By doing so, the opportunity value of money is incorporated into the analysis.

The applicability of discount rates to public agency cost analyses is debatable. The problem with applying discount rates to transportation projects arises in the fact that transportation officials do not typically have the option of investing money saved in the present for use in the future. Therefore, there would be no potential return on investment. However, it is possible that money saved initially may be invested in other projects that will yield some tangible benefit, as in extended service life of a structure or beneficial research. Discount rates as high as those evident in the private sector are not expected, however, some opportunity value of money should be considered. Currently, government agencies are instructed to use guidelines provided by the Office of Management and Budget (OMB). The discount rate that is specified for use in cost analyses is the real discount rate that corresponds to the interest rate on the treasury note or bond that has the same maturity time as the analysis time period being investigated. For example, a project that has an analysis time period of 10 years should be investigated using the real discount rate that corresponds to the 10-year treasury note. Interest rates for treasury notes and bonds are updated annually and can be found in Appendix C of Circular No. A-94. This information can be easily located at the OMB website www.whitehouse.gov/omb/. The following rates are current as of January 2007: (OMB, 2007)

Table 19 – Nominal and Real Interest Rates on Treasury Notes and Bonds

Nominal Interest Rates on Treasury Notes and Bonds of Specified Maturities (in percent)

3-Year	5-Year	7-Year	10-Year	20-Year	30-Year
4.9	4.9	4.9	5.0	5.1	5.1

Real Interest Rates on Treasury Notes and Bonds of Specified Maturities (in percent)

3-Year	5-Year	7-Year	10-Year	20-Year	30-Year
2.5	2.6	2.7	2.8	3.0	3.0

For the analysis of bridge decks the specified service life is generally either 75 or 100 years. Therefore, the real interest rate of the 30-year treasury note should be used in their LCCA's. It should be noted that predicting interest rates and inflation rates at distant points in the future should be done with caution. There is a high level of uncertainty associated with these estimations.

Inflation/Deflation Rate

Inflation can be defined as a rise in the general level of prices, as measured against some baseline of purchasing power. Inflation affects LCCA in that the cost of materials and construction will increase over time. Conversely, if deflation occurs costs will be lower in the future. Historically, the United States economy has been subject to inflation with only several short periods of price deflation. The inflation rate can be estimated using many different indexes ranging from those that track broad-based consumerism to those that monitor only a niche market. The index that is most commonly used to determine inflation rates in the United States is the Consumer Price Index (CPI). It may be reasonable to apply inflation rates determined from the CPI to the inflation rate of construction materials; however, given that there are indexes available that specifically track the inflation of construction materials and labor the CPI may not be the most applicable for LCCA of bridge decks. Three indexes that relate specifically to construction were investigated in this study: Engineering News Record Construction Cost Index, FHWA Composite Construction Cost Index, and FHWA Structures Construction Cost Index. These cost indexes and and their associated inflation trends will be discussed in detail in the 'LCCA's of Virginia Bridge Decks' section.

Deterministic vs. Probabilistic

LCCA's can be conducted using deterministic or probabilistic input parameters. A deterministic analysis requires a specific application time for each expenditure. Conversely, a probabilistic analysis enables the user to assign a probability distribution to input parameters. An example of a deterministic analysis would be a bridge deck requiring an overlay at 50 years of age. In reality there is some uncertainty associated with the timing of specific maintenance events. Therefore, it is more appropriate to specify an overlay installation at times normally distributed around a mean of 50 years. This procedure may be completed for each maintenance, repair, and rehabilitation event for each alternative. The LCCA program will then randomly select timings for each event according to their distributions and compute an associated LCC. The selection process is repeated many times and a distribution of present values as well as a mean value will be generated. The generated present value distributions can then be compared for each alternative.

The problem that arises when incorporating the probabilistic nature of input parameters is being able to accurately define the distributions for those parameters. Large amounts of data are typically required to define distributions for a specific event. Therefore, a probabilistic analysis may not be appropriate in all cases.

Step 5: Analyze the Results

Analyzing the results of a LCCA can be straightforward or quite complex depending upon the method of analysis used. If a deterministic approach is used, analyzing the results may be as simple as comparing the computed present values of each alternative. However, if a probabilistic approach is used additional consideration must be given to the results. In a probabilistic analysis the selection of multiple alternatives is possible depending upon the level of risk that the user is willing to accept.

Often times it is also desirable to perform a sensistivity analysis on the LCCA. Sensitivity analyses are performed by varying the timings of expenditures and discount rates within a LCCA to determine how sensitive the analysis is to specific parameters. If an analysis is found to be highly sensitive to an individual parameter it may be necessary to better define the input data for that variable.

The user may also choose not to consider user costs in the analysis, as they may not be confident in their validity or may simply not be concerned with their effects. Finally, the user should reevaluate the possible design alternatives to determine if modifications can be made to provide a more cost effective approach. Reevaluations, commonly known as concurrent engineering, are necessary if either the agency costs or user costs are found to be disproportionately high for a specific alternative.

LCCA's of Virginia Bridge Decks

LCCA's for bridge decks constructed under current design specifications are unnecessary because as demonstrated previously the estimated service life of 0.45 $^{w}/_{c+p}$ bridge decks will exceed a specified design life of 100 years regardless of reinforcement type. Therefore, reinforcement type should be selected solely on a first-cost basis except in cases where extreme chloride exposure is expected.

However, the question remains as to what the best maintenance strategy is for bridge decks constructed under previous specifications. Therefore, multiple maintenance alternatives will be analyzed to determine the optimum maintenance strategy for in-place bridge decks. The LCCA's will be conducted according to the methodology presented previously for a design life of 100 years and a base year of 2008. A base year of 2008 was selected because it is anticipated that 2008 will be the earliest possible time of implemenation for this project. LCC comparisons in the examples presented will be based upon computed unit costs ($/SY; $/m^2). Using the calculated unit costs, LCC's for bridge decks of any size can be easily computed relative to their surface areas.

Step 1: Establish Design Alternatives

Bridge deck LCCA's will be conducted for two possible repair/rehabilitation alternatives for preventing/repairing corrosion related deterioration. The initial condition of the bridge decks to be compared will be considered equivalent. Therefore, the only differences in the LCCA alternatives are related to the maintenance procedures. The two deck maintenance alternatives that will be investigated are polymer overlays and concrete overlays (microsilica/latex-modified).

Maintenance Alternative 1: Polymer Overlays

The first maintenance alternative to be investigatied is the use of polymer overlays as a preventative maintenance technique. Polymer overlays are used primarily to reduce the rate of ingress of water and chlorides into the concrete as well as to improve skid resistance, ride quality, and surface appearance. (Krauss and Ferroni, 1986; Sprinkel, 1989) They are to be installed prior to extensive corrosion related deterioration. Polymer overlays are suitable for bridge decks that reflect between 0-1% corrosion related deterioration in the worst span lane. Bridge decks that have corrosion-deteriorated areas in excess of 1% should be repaired/rehabilitated using alternative maintenance strategies. Polymer overlays are not recommended for use on bridge decks that have any of the following characteristics:

1. "Corrosion-induced delamination and spalls
2. Cover concrete that is critically chloride-contaminated
3. Half-cell potentials more negative than –250 mV CSE
4. Unsound concrete (tensile rupture strength less than 150 psi)
5. Poor drainage
6. Poor ride quality" (Weyers et al, 1993)

If the bridge deck being investigated reflects any of the above characteristics the use of a concrete overlay may be more appropriate.

The estimated service life of a polymer overlay ranges from 10 years for a bridge with a very high ADT (> 50,000) to 25 years for a bridge with a low to moderate ADT (< 5000). (Weyers et al, 1993) For purposes of this analysis polymer overlays will be investigated for service lives of 10, 15, and 25 years. In addition to the cost of the overlay the cost associated with 1% patching of the bridge deck at the time of the overlay will be taken into account. For an individual bridge deck it may be necessary to place multiple overlays in order to reach the desired service life. When that is the case the installation of the polymer overlay will become a recurring cost as well as the removal of preceding overlays.

Maintenance Alternative 2: Concrete Overlays

The second maintenance alternative that will be investigated is the use of concrete overlays as a deck rehabilitation measure. They are used as a repair/rehabilitation method for decks reflecting significant levels of corrosion related deterioration. The installation of concrete overlays requires that a specified depth of the cover concrete be removed by milling and the underlying concrete removed and patched as necessary. In order to provide the desired service life "all spalled, delaminated, actively corroding, and critically chloride-contaminated concrete" must be removed and repaired prior to placement of the overlay. (Weyers et al, 1993)

Concrete overlays are typically used as a rehabilitation method for bridge decks that have reached their EFSL. As mentioned previously the EFSL for a bridge deck is defined in this project as the time at which 12% of the bridge deck has deteriorated in the worst span lane. Therefore, a maintenance strategy that specifies the use of a concrete overlay must consider the costs associated with 12% patching of the bridge deck prior to placement of the overlay. As demonstrated by this project the required patching will occur on average during the 16 years preceding the placement of the overlay.

The service life of a concrete overlay is estimated to be between 22 and 26 years and is dependent upon the severity of chloride exposure and condition of the underlying concrete, but is independent of ADT. (Weyers et al, 1993) The overlay service life used in the following analyses is 25 years. The cost for 12% patching, milling, and groving of the bridge deck for each required overlay is also included.

Step 2: Determine Activity Timing

With the two maintenance alternatives identified the next step is to determine the timings of the associated maintenance activities. The activity timings presented in Table 20 represent those timings associated with a bridge deck that has a current deterioration level between 0 and 1%. Year zero does not represent the the time at which the bridge deck is put in place but rather the time of analysis.

Table 20 – LCCA Maintenance Activity Timing

Year	Polymer Overlays			Concrete Overlays
	ADT			
	VH	H	M	
0	PO & 1% P	PO & 1% P	PO & 1% P	
5				6% P
10	OR, PO, & 1% P			6% P
15		OR, PO, & 1% P		M, CO, & G
20	OR, PO, and 1%P			
25			OR, PO, & 1% P	
30	OR, PO, and 1%P	OR, PO, & 1% P		6% P
35				6% P
40	OR, PO, and 1%P			M, CO, & G
45		OR, PO, & 1% P		
50	OR, PO, and 1%P		OR, PO, & 1% P	
55				6% P
60	OR, PO, and 1%P	OR, PO, & 1% P		6% P
65				M, CO, & G
70	OR, PO, and 1%P			
75		OR, PO, & 1% P	OR, PO, & 1% P	
80	OR, PO, and 1%P			6% P
85				6% P
90	OR, PO, and 1%P	OR, PO, & 1% P		M, CO, & G
95				
100				

* PO = Polymer Overlay; P = Patching; OR = Overlay Removal; M = Milling; CO = Concrete Overlay; G = Bridge Deck Grooving

As shown, a bridge deck with 0 - 1% corrosion related damage would either require a polymer overlay in the current year or a concrete overlay approximately 15 years in the future. For the case of a polymer overlay, the 0 – 1% of deteriorated deck would be patched at the time of the overlay. Additionally, for subsequent polymer overlays there is a cost associated with the removal of the previous overlay. Concrete overlays are not placed until substantial deterioration is evident (12% of the worst span lane). The maintenance activity timings presented reflect the 12% of required patching prior to placement of the concrete overlay. For simplicity the patching has been estimated to occur in equal amounts of 6% at 5 and 10 years prior to the placement of the concrete overlay. The required milling and bridge deck grooving associated with a concrete overlay are also considered.

It should be noted that the activity timings presented in Table 20 are only valid for bridge deck maintenance decisions made in the current year (2008). For bridge decks that do not currently require maintenance, activity-timing tables must be developed for future years. LCC comparison tables will be developed and presented for the maintenance alternatives where strategy decisions are to be made in the year 2008, 2018, 2028, 2038, 2048, and 2058.

It is important to note that the maintenance timings presented may be altered at the discretion of the engineer. The estimated service lives of the overlays are intended to reflect the average expected service life for a given set of circumstances. The actual service life of an overlay is dependent upon the severity of the environment to which it is exposed, the condition of the underlying deck, the quality of construction, and the level of traffic on the bridge. The purpose of the examples provided in this report is to present the concepts of LCCA rather than to make determinations for the maintenance strategies of individual bridge decks.

Step 3: Estimate Costs

The next step in conducting the LCCA is to determine the costs associated with each maintenance alternative. As mentioned previously, the base year for comparison is 2008. Therefore, all maintenance expenditures presented in this section will be adjusted to reflect their estimated costs in 2008. The costs will be estimated using tabulated VDOT bid data from 1997, 1999, and 2004. The average inflation rate will be determined for each bid item for the time period of 1997 – 2004. Using the 2004 cost data and the average inflation rate the costs will then be projected for the year 2008. The bid items associated with the two maintenance alternatives are presented below:

Alternative 1 – Polymer Overlays

- Patching – Type B – The removal and patching of deteriorated concrete to a depth below the first mat of reinforcement.
- Polymer Overlay – The placement of a multi-layer polymer overlay on the entire bridge deck.
- Polymer Overlay Removal – The removal of a previous polymer overlay prior to the placement of a subsequent overlay.
- Traffic Control – The cost associated with controlling the traffic over the lifetime of the project.
- User Costs – Costs incurred by the traveling public attributable to the construction project. Varies between projects and therefore will not be considered in the presented examples.

Alternative 2 – MSC/LMC Overlays

- Patching – Type B
- Milling Type A – The removal of the top ½ in. of concrete prior to placement of a concrete overlay.
- Concrete Overlay – The placement of a concrete overlay with a specified depth of 1 ¼ in. – 1 ¾ in.
- Bridge Deck Grooving – The grooving of the concrete overlay after placement and curing.
- Traffic Control
- User Costs – Again, because user costs are site specific, they are not included in the presented examples.

The average contract bid prices for the required construction items are presented in Table 21 and are illustrated in graphical form in Figures 47 - 51 for the time period of 1997 – 2004.

Table 21 – VDOT Bid Data

VDOT Bid Data		
Bid Item	Contract Price*	
Bridge Deck Grooving (SY)	$5.14	1997
	$3.56	1999
	$3.98	2004
Deck Patch Type B (SY)	$145.70	1997
	$169.04	1999
	$314.82	2004
Milling Type A (SY)	$9.83	1997
	$8.62	1999
	$9.24	2004
Overlay - Latex or Silica (CY)	$839.03	1997
	$584.61	1999
	$1,085.83	2004
Overlay - Polymer (SY)	$19.19	1997
	$20.57	1999
	$23.40	2004
Remove Polymer Overlay (SY)	$15.25	1999

* Weighted Average

*$1/SY = $1.19/m^2

The weighted average contract price was calculated using Equation 39.

$$WA = \frac{\sum C_i \times Q_i}{Q_T} \qquad \text{Equation 39}$$

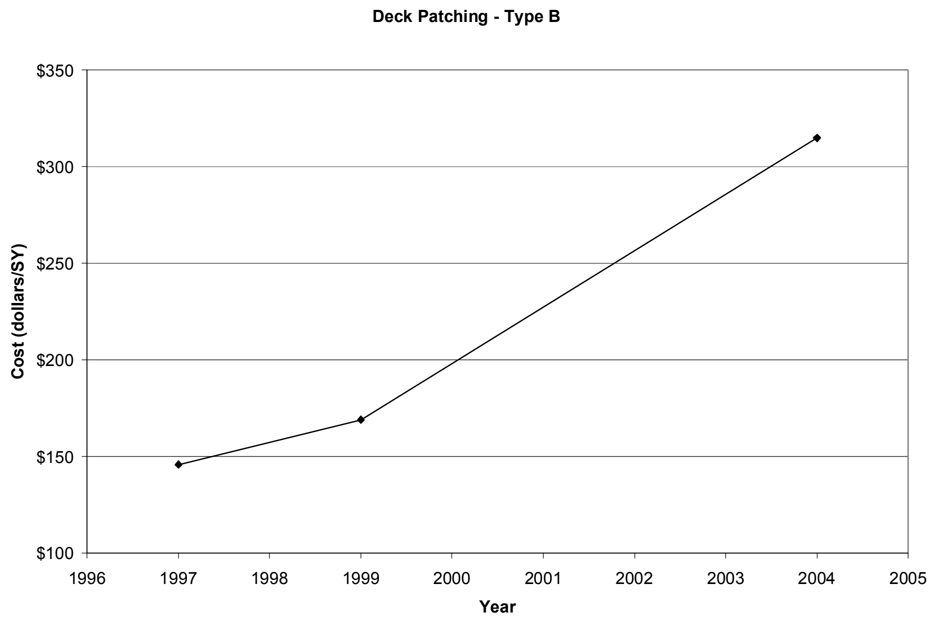
where:

WA = Weighted average contract price

C_i = Contract price for an individual project

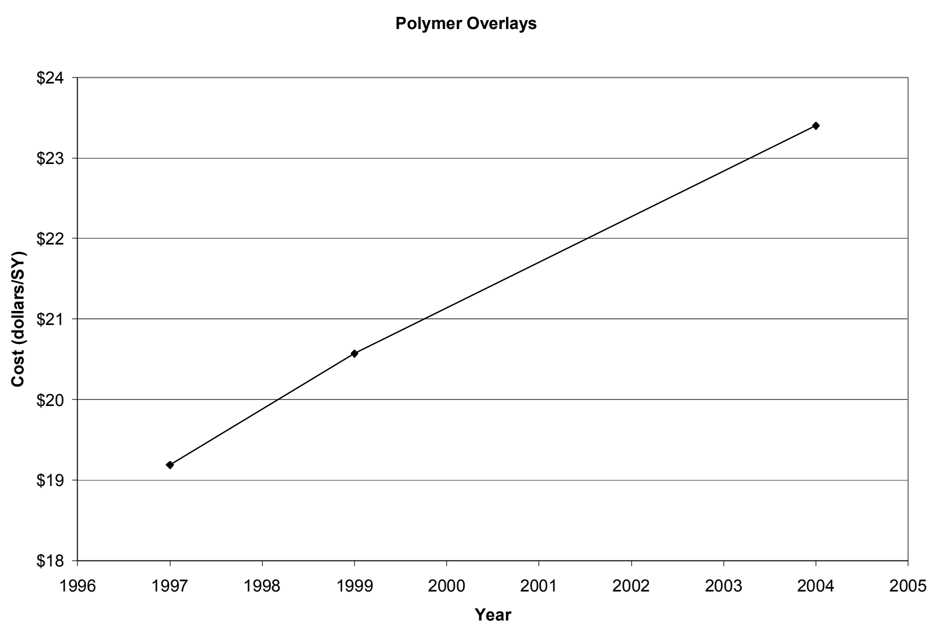
Q_i = Quantity of work for an individual project

Q_T = Total quantity of work for all projects



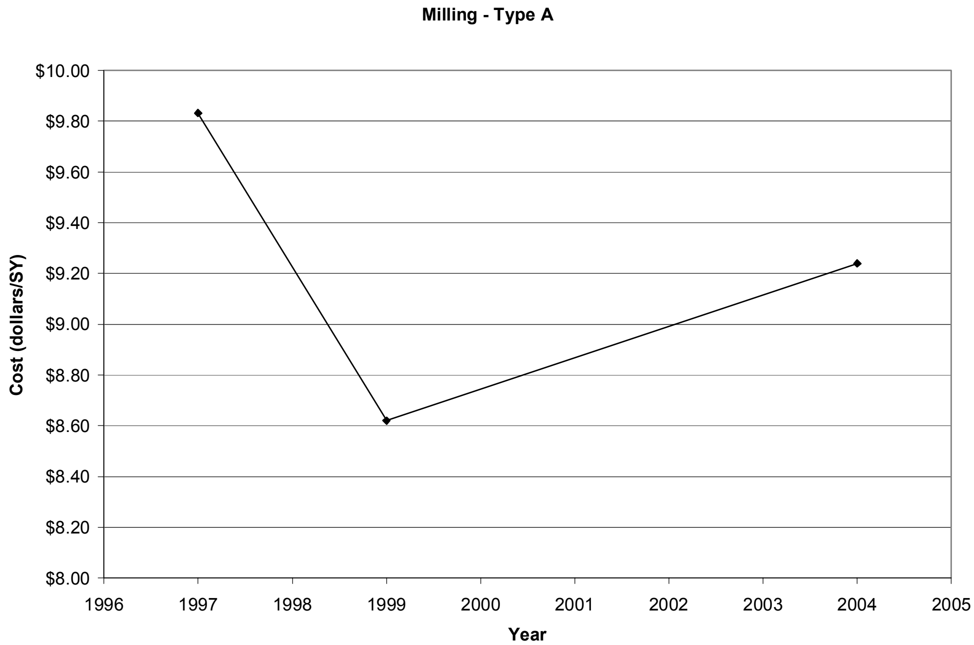
*$1/SY = $1.19/m^2

Figure 47 – VDOT Bid Data - Deck Patching – Type B



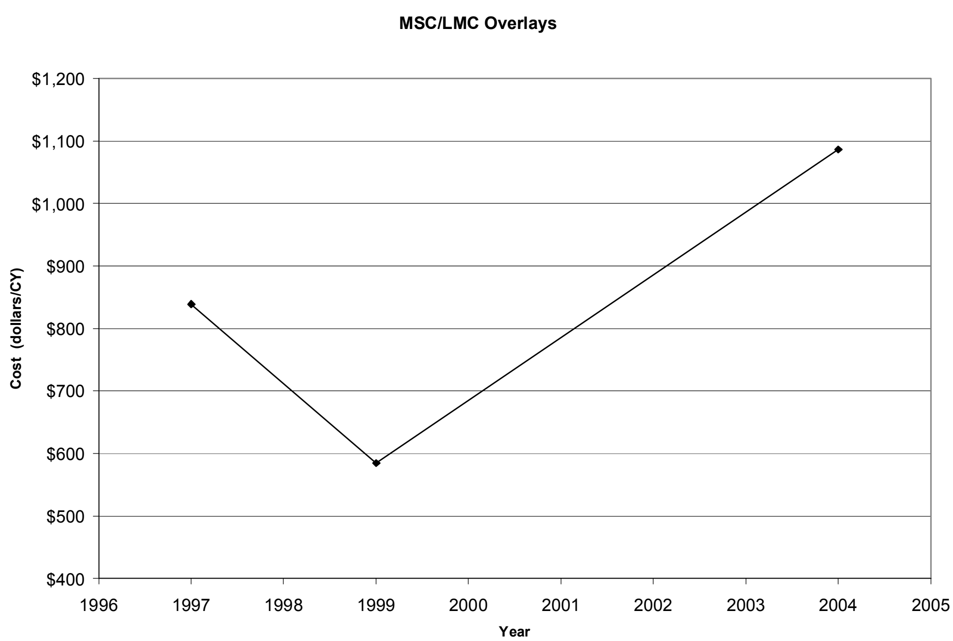
*$1/SY = $1.19/m^2

Figure 48 – VDOT Bid Data – Polymer Overlays



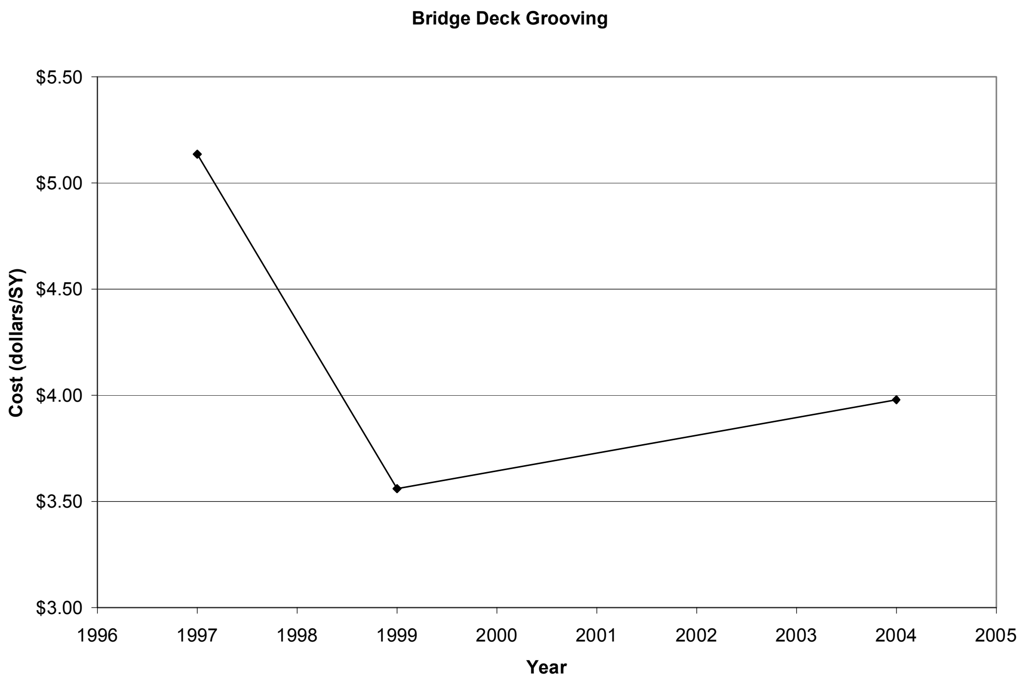
*$1/SY = $1.19/m^2

Figure 49 – VDOT Bid Data – Milling – Type A



*$1/SY = $1.19/m^2

Figure 50 – VDOT Bid Data – MSC/LMC Overlays



*$1/SY = $1.19/m^2

Figure 51 – VDOT Bid Data – Bridge Deck Grooving

As shown, several bid items reflect relatively steady inflation rates while others have substantial price fluctuations. The fluctuations may be due to variations in average project size, availability of materials, or bid competition. For those bid items that appear to have a continuous increase in cost over the 7-year period being investigated, the inflation rate will be determined and used to project prices for the year 2008. For bid items where significant price fluctuations are evident the average value will be used. The cost data used in the following analyses are presented in Table 22.

Table 22 – Bridge Deck LCCA Cost Data ($/SY)

Bridge Deck LCCA Cost Data			
Item	**Avg. Inflation Rate (%)**	**Average Cost ($)**	**2008 Projected Cost ($)**
Bridge Deck Grooving (SY)	----	$4.23	$4.23
Deck Patch Type B (SY)	11.6	----	$487.27
Milling Type A (SY)	----	$9.23	$9.23
Overlay - Latex or Silica (CY)/(SY)*	3.8	----	$1264.54/$70.25
Overlay - Polymer (SY)	2.9	----	$26.28
Remove Polymer Overlay (SY)	----	$15.25	$15.25

* Cost calculated based upon an overlay depth of 2"

*$1/SY = $1.19/m^2

The cost data presented above are estimated average values and should not be used for the LCCA of an individual bridge deck. Construction costs vary widely due to the effects of bid quantity, bid competion, and geographic location.

Bid Quantity Effects

For a given project, the mobilization, overhead, and profit costs will be factored into a bid price. For projects with large quantities of work these costs are distributed over the entire project and will have little influence on the unit cost of a particular bid item. However, for small quantities of work, these costs may increase the unit cost substantially. For example, take two projects that require bridge deck patching. The first project is 10 SY (8.4 m^2) and the second project is 100 SY (84 m^2). The mobilization cost associated with a patching project is taken to be $100. Therefore, the mobilization cost included in the first project is $10/SY ($11.90/m^2), but is only $1/SY ($1.19/m^2) for the larger project. Thus, the engineer should take the quantity of required work into account when estimating costs for a particular project and make use of economies of scale.

Bid Competition Effects

The amount of transportation-related construction occurring within the state of Virginia at any given time is constantly changing. The same laws of supply and demand that govern the cost or value of any item also apply to the construction industry. At times when there is an oversupply of projects, contractors are not available to take on additional work and the average bid for a particular item will increase. Likewise, when there is a limited supply of projects and multiple contractors are competing for the same work the cost will be driven down. The effects that bid competition will have on an individual project are difficult to estimate, however, attempts by maintenance engineers to maintain steady work orders may help to negate these effects.

Geographic Location Effects

The effect that the location of a project has on cost can be substantial. Within Virginia, labor and material costs can vary significantly. For example, the hourly wage of a construction crewman in Northern Virginia is expected to be greater than for a crewman in Roanoke. The distance of the project from the necessary resources is also an important factor as the transportation costs for materials is often higher than the cost of the materials themselves. For most cases district engineers will be able to accurately estimate costs for their specific locality, but special cases may arise where the location of the project warrants additional consideration. An example of this would be a bridge that is located a great distance from a ready-mix plant. In that case it may be necessary to use precast bridge panels rather than a cast-in-place bridge deck. This would affect the initial costs of the project as well as the maintenance costs over the lifetime of the structure.

Traffic Control Costs

The traffic control (TC) costs related to a bridge deck maintenance project will vary by location, route type, ADT, and duration. Therefore, there is no single value that can be used for all LCCA's. In the following examples an average TC cost of $5.88/ft^2 ($52.92/SY or $62.97/m^2) will be used for concrete overlays. This value was recommended in a previous study relating to bridge maintenance in Virginia. (Pyc et al, 2000) To estimate the TC costs associated with a polymer overlay, the duration of the projects was considered. It is estimated that the application of a polymer overlay will take approximately 1/3 of the time that is required for a concrete overlay. Therefore, the reduction in TC costs should correspond with the reduction in construction time. Thus, TC costs for polymer overlays will be taken to be 1/3 of concrete overlays or $1.96/ft^2 ($17.64/SY or $20.99/m^2).

User Costs

User costs will not be considered initially. The primary focus of this LCCA is to estimate the direct Agency costs that will be incurred for the two sets of bridge decks. A more robust LCCA modeling program will be presented that is capable of estimating user costs based upon the parameters of a given project. A detailed analysis of an individual bridge deck will be presented to illustrate the capabilities of the available software.

Step 4: Compute Life-Cycle Costs

To compute the LCC's for this project a deterministic approach has been taken. A database of the distribution of times for specific maintenance events is not available. Therefore, a probabilistic approach is not possible.

As mentioned previously, the discount rate that is specified for use by government agencies is 5.1% for long-term projects. Therefore, a discount rate of 5.1% has been used in the following examples. However, it may be necessary to adjust the long-term discount rate to reflect the impacts of inflation.

Inflation Rates

It is common practice to use the CPI to compute the inflation of the general economy. However, the application of the inflation rate associated with the CPI to transportation related construction activities might not be valid because of the differences in the inflation of different products and services. To determine the appropriate rate of inflation for transportation construction activities three cost indexes were investigated: Engineering News Record (ENR) Construction Cost Index, FHWA Composite Construction Cost Index, and FHWA Structures Construction Cost Index. After the inflation rates are determined for the three construction-related indexes they will be compared to the CPI to determine if any significant differences are evident.

ENR Construction Cost Index (CCI)

The ENR CCI was developed as a means of tracking price trends in the construction industry. The index measures the cost to purchase a set package of goods and services compared to the cost of the same package in the base year. The base year for the CCI is 1913 and the index is composed of the 20-city average of the following components: (Grogan, 2006)

- 200 hours of common labor
- 25 cwt of standard structural steel shapes (fabricated)
- 1,128 tons of Portland cement
- 1,088 board-ft of 2x4 lumber

The index for the time period of 1918 – 2005 is presented in Figure 52.

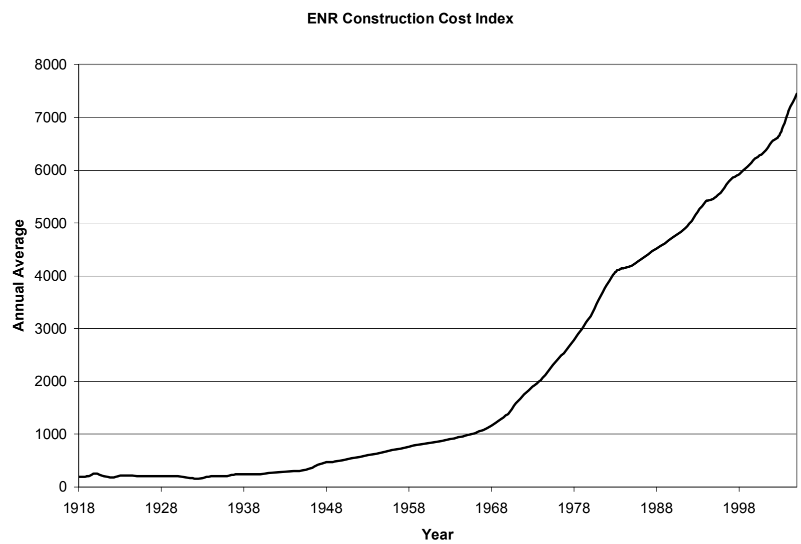


Figure 52 – ENR Construction Cost Index

Using the index values the average annual inflation rate was calculated and is presented in Figure 53.

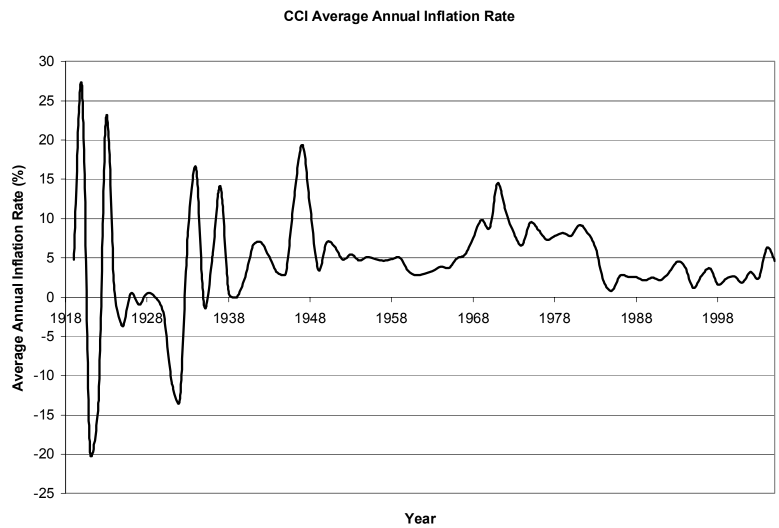


Figure 53 – CCI Average Annual Inflation Rate

As shown the CCI inflation rate was very volatile in the early 1900's ranging from –20%/year to 26%/year; however, it has become more stable in recent years. Over approximately the last 20 years the inflation rate has ranged between 2 and 6%.

FHWA Composite Construction Cost Index

The FHWA Composite CCI tracks the costs of transportation construction projects that are constructed using Federal funds. The Composite index consists of three subcategories: Excavation, Resurfacing, and Structures with weightings of 21%, 36%, and 43%, respectively. A base year of 1987 is used and the following components make up the index:

- 210,078,000 cubic yards of roadway excavation,
- 30,893,690 square yards of portland cement concrete surfacing with an average thickness of 9 inches,
- 37,760,443 tons of bituminous concrete surfacing,
- 577,753,544 pounds of reinforcing steel for structures,
- 444,924,141 pounds of structural steel, and
- 3,498,333 cubic yards of structural concrete.

Only construction projects that exceed $500,000 in total cost are considered when determining price trends for the index components.

In addition to a national cost index FHWA computes cost indexes for individual states. However, care should be taken when determining inflation rates from a state index due to high volatility. FHWA provides the following warning regarding state CCI's.

"In some instances, individual State indices may not be truly representative of long-term price trends because of comparatively low volumes of work for the period reported, or because of unusual projects awarded during the period. Also, differences in bid item specifications among the States might account for some of the differences in unit prices in the various States." (FHWA, 2006)

Due to the unreliability of state indexes, only the national index will be used in determining inflation rates. The FHWA National Composite CCI is presented in Figure 54 and the corresponding inflation rates are presented in Figure 55.

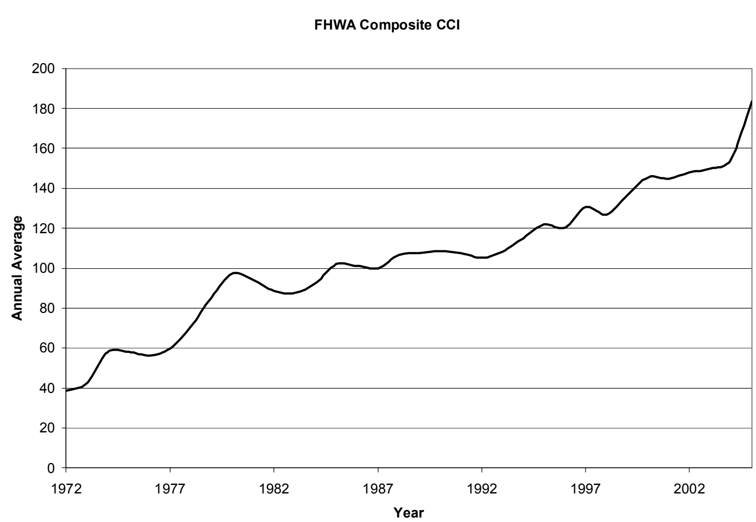


Figure 54 – FHWA Composite CCI

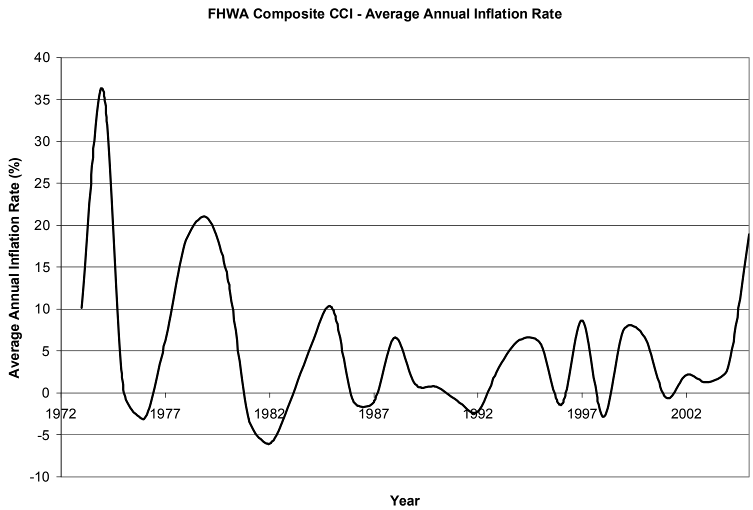


Figure 55 – FHWA Composite CCI – Average Annual Inflation Rate

As shown the FHWA Composite CCI is significantly more volatile than the ENR CCI with inflation rates over the last 20 years ranging from –3 to 19%. The reason for this increased volatility is not clear. Some fluctuation in cost data may be attributable to changes in funding levels for individual states.

FHWA Structures Construction Cost Index

The FHWA Stuctures CCI is calculated using the same methodology as the Composite CCI. The only differences between the two indexes are the components. The Structures CCI only includes items that pertain directly to the construction of highway structures and therefore may be a better predictor of inflation rates associated with bridge construction. The components included are:

- 577,753,544 pounds of reinforcing steel for structures,
- 444,924,141 pounds of structural steel, and
- 3,498,333 cubic yards of structural concrete.

The FHWA National Structures CCI is presented in Figure 56 and the corresponding inflation rates are presented in Figure 57.

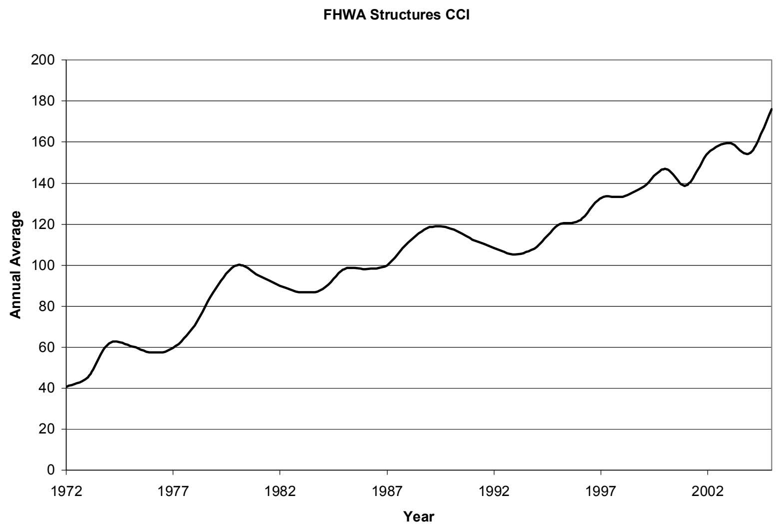


Figure 56 – FHWA Structures CCI

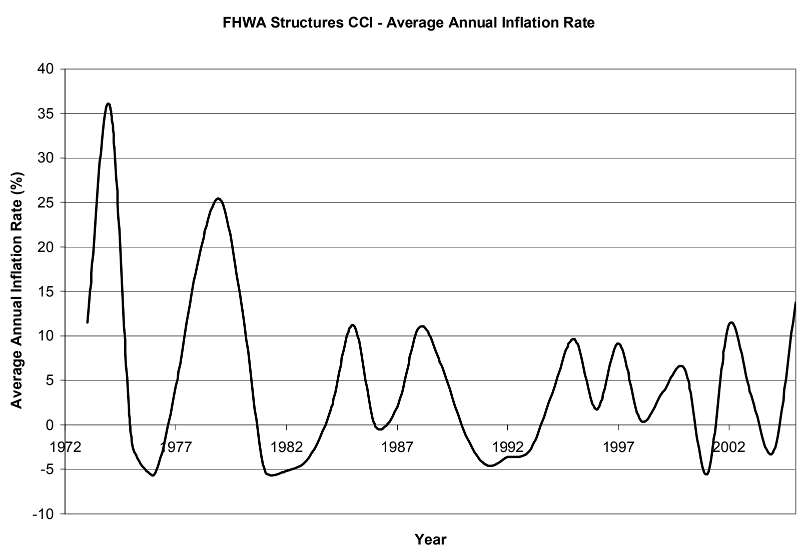


Figure 57 – FHWA Structures CCI – Average Annual Inflation Rate

A comparison of the Structures CCI to the Composite CCI indicates that the two indexes are very similar as might be expected.

Consumer Price Index (CPI)

The Consumer Price Index (CPI) is a "measure of the average change over time in the prices paid by urban consumers for a market basket of consumer goods and services". (BLS, 2006) It is considered to be the most comprehensive measure of inflation for the U.S. economy.

The CPI represents all goods and services purchased for consumption by a reference population. The expenditure items included in the calculation of the CPI are arranged into eight major groups. The groups are as follows:

- FOOD AND BEVERAGES (breakfast cereal, milk, coffee, chicken, wine, service meals and snacks)
- HOUSING (rent of primary residence, owners' equivalent rent, fuel oil, bedroom furniture)
- APPAREL (men's shirts and sweaters, women's dresses, jewelry)
- TRANSPORTATION (new vehicles, airline fares, gasoline, motor vehicle insurance)
- MEDICAL CARE (prescription drugs and medical supplies, physicians' services, eyeglasses and eye care, hospital services)
- RECREATION (televisions, pets and pet products, sports equipment, admissions);
- EDUCATION AND COMMUNICATION (college tuition, postage, telephone services, computer software and accessories);
- OTHER GOODS AND SERVICES (tobacco and smoking products, haircuts and other personal services, funeral expenses). (BLS, 2006)

The CPI is presented in Figure 58 and the corresponding inflation rates are presented in Figure 59.

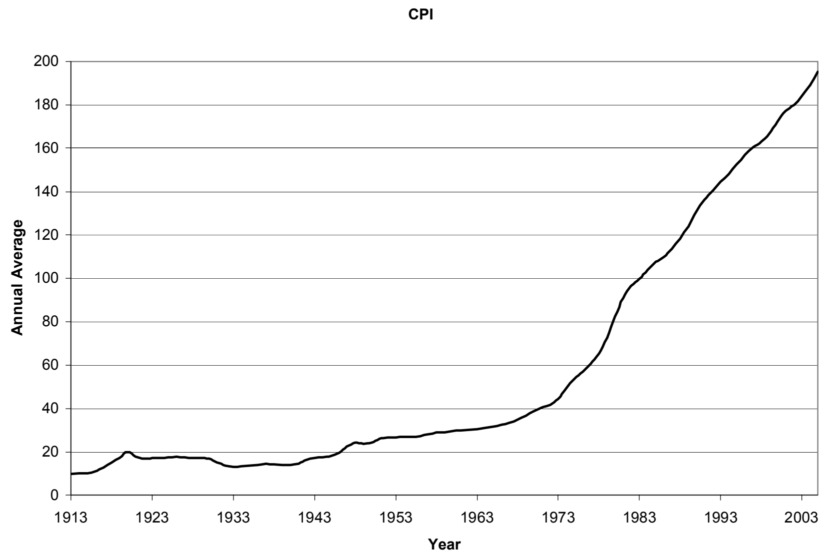


Figure 58 - CPI

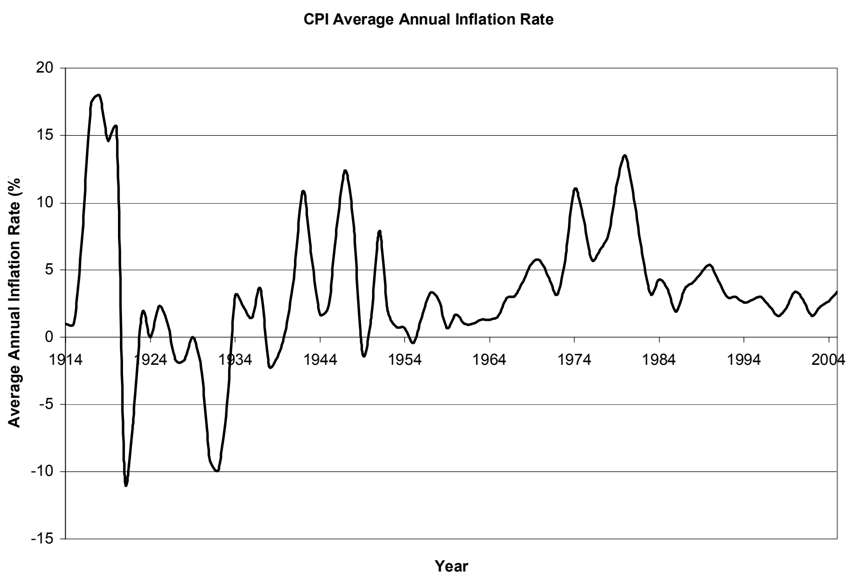


Figure 59 – CPI – Average Annual Inflation Rate

Index Comparison

After the inflation rates for each index were calculated comparisons were made to determine if there is any significant difference between indexes. To compare the indexes the total cumulative inflation was plotted for each index from a 1973 to 2005 as illustrated in Figure 60.

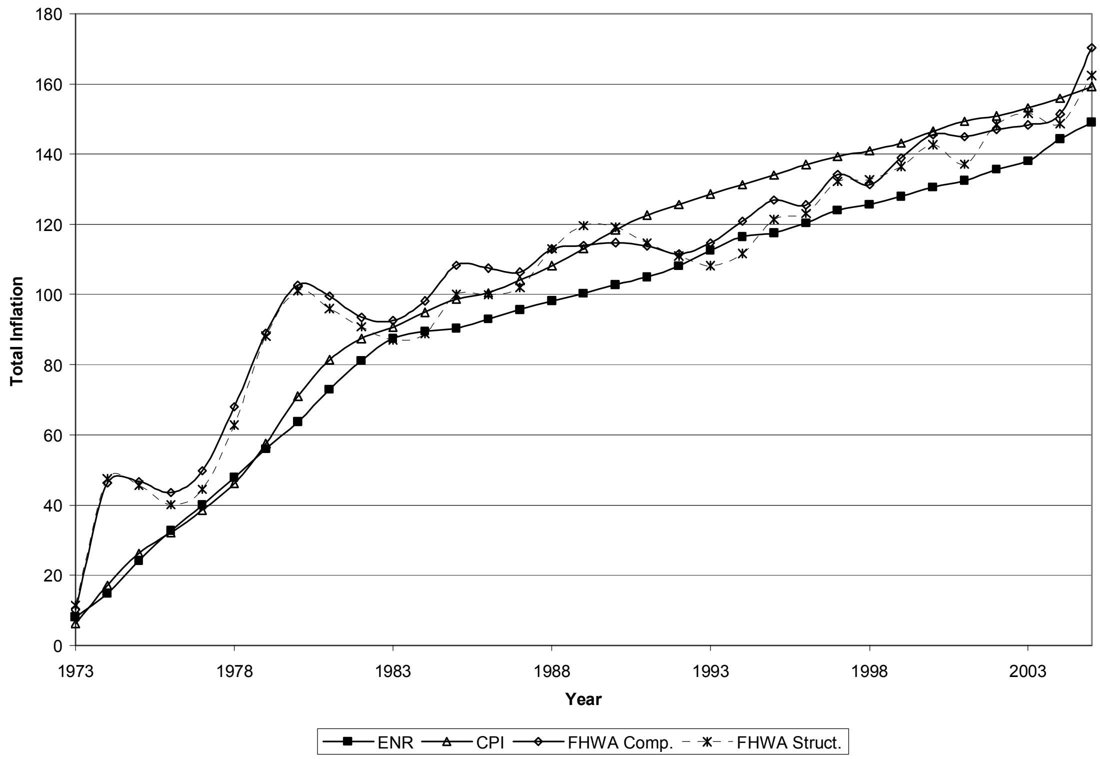


Figure 60 – Index Comparison

As shown all indexes follow the same general inflationary trend regardless of volatility within an individual index. Thus, it can be inferred that the indexes are not significantly different. Therefore, the real discount rates recommended for use by the Office of Management and Budget that are adjusted based upon the CPI may be used. Recall that the real discount rate is the discount rate adjusted for the effects of inflation.

As shown in Figure 60, the rate of inflation from 1983 to 2003 is relatively uniform and is slower than from 1973 to 1983, 8% versus 3%, respectively. Considering OMB's long-term interest rate of 5.1% and an inflation rate of 3%, the real discount rate would be approximately 2.1%. As of January 2007, the recommended real discount rate for projects exceeding 30-years in length is 3.0%, which is within reasonable agreement with the estimated real discount rate of 2.1%. (OMB, 2007)

Step 5: Analyze the Results

LCC comparison tables were developed for bridges where a maintenance strategy must be selected within the next 50 years and are presented in 10-year intervals (Tables 23 – 34). The tables are presented with and without traffic control costs and user costs are not considered. Where bridge maintenance decisions are to be made in years other than those presented in Tables 23 – 34, the LCC's may be interpolated using an exponential regression analysis.

Table 23 – LCC Comparison – 2008 – Without Traffic Control

2008 Without Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	Very High	High	Moderate	
Required Remaining Service Life	0	$30.19	$30.19	$30.19	$0.00
	5				$20.25
	10	$33.81			$17.46
	15		$29.17		$53.73
	20	$25.16			
	25			$21.70	
	30	$18.72	$18.72		$9.67
	35				$8.34
	40	$13.93			$25.66
	45		$12.02		
	50	$10.37		$10.37	
	55				$4.62
	60	$7.71	$7.71		$3.98
	65				$12.26
	70	$5.74			
	75		$4.95	$4.95	
	80	$4.27			$2.21
	85				$1.90
	90	$3.18	$3.18		$5.85
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
* Real Discount Rate 3.00%

*$1/SY = $1.19/m^2$

Table 24 – LCC Comparison – 2008 – With Traffic Control

2008 With Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	Very High	High	Moderate	
Required Remaining Service Life	0	$47.83	$47.83	$47.83	$0.00
	5				$20.25
	10	$46.94			$17.46
	15		$40.49		$87.70
	20	$34.93			
	25			$30.13	
	30	$25.99	$25.99		$9.67
	35				$8.34
	40	$19.34			$41.88
	45		$16.68		
	50	$14.39		$14.39	
	55				$4.62
	60	$10.71	$10.71		$3.98
	65				$20.00
	70	$7.97			
	75		$6.87	$6.87	
	80	$5.93			$2.21
	85				$1.90
	90	$4.41	$4.41		$9.55
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
Real Discount Rate 3.00%

Table 25 – LCC Comparison – 2018 – Without Traffic Control

2018 Without Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	Very High	High	Moderate	
Required Remaining Service Life	0	$22.46	$22.46	$22.46	$0.00
	5				$15.06
	10	$25.16			$12.99
	15		$21.70		$39.98
	20	$18.72			
	25			$16.15	
	30	$13.93	$13.93		$7.19
	35				$6.21
	40	$10.37			$19.09
	45		$8.94		
	50	$7.71		$7.71	
	55				$3.44
	60	$5.74	$5.74		$2.96
	65				$9.12
	70	$4.27			
	75		$3.68	$3.68	
	80	$3.18			$1.64
	85				$1.42
	90	$2.36	$2.36		$4.36
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
Real Discount Rate 3.00%

*$1/SY = $1.19/m^2

Table 26 – LCC Comparison – 2018 – With Traffic Control

2018 With Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	Very High	High	Moderate	
Required Remaining Service Life	0	$35.59	$35.59	$35.59	$0.00
	5				$15.06
	10	$34.93			$12.99
	15		$30.13		$65.26
	20	$25.99			
	25			$22.42	
	30	$19.34	$19.34		$7.19
	35				$6.21
	40	$14.39			$31.17
	45		$12.41		
	50	$10.71		$10.71	
	55				$3.44
	60	$7.97	$7.97		$2.96
	65				$14.89
	70	$5.93			
	75		$5.11	$5.11	
	80	$4.41			$1.64
	85				$1.42
	90	$3.28	$3.28		$7.11
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
Real Discount Rate 3.00%

Table 27 – LCC Comparison – 2028 – Without Traffic Control

2028 Without Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	Very High	High	Moderate	
Required Remaining Service Life	0	$16.72	$16.72	$16.72	$0.00
	5				$11.21
	10	$18.72			$9.67
	15		$16.15		$29.75
	20	$13.93			
	25			$12.02	
	30	$10.37	$10.37		$5.35
	35				$4.62
	40	$7.71			$14.21
	45		$6.65		
	50	$5.74		$5.74	
	55				$2.56
	60	$4.27	$4.27		$2.21
	65				$6.79
	70	$3.18			
	75		$2.74	$2.74	
	80	$2.36			$1.22
	85				$1.05
	90	$1.76	$1.76		$3.24
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
* Real Discount Rate 3.00%

*$1/SY = $1.19/m^2$

Table 28– LCC Comparison – 2028 – With Traffic Control

2028 With Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	Very High	High	Moderate	
Required Remaining Service Life	0	$26.48	$26.48	$26.48	$0.00
	5				$11.21
	10	$25.99			$9.67
	15		$22.42		$48.56
	20	$19.34			
	25			$16.68	
	30	$14.39	$14.39		$5.35
	35				$4.62
	40	$10.71			$23.19
	45		$9.24		
	50	$7.97		$7.97	
	55				$2.56
	60	$5.93	$5.93		$2.21
	65				$11.08
	70	$4.41			
	75		$3.81	$3.81	
	80	$3.28			$1.22
	85				$1.05
	90	$2.44	$2.44		$5.29
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
Real Discount Rate 3.00%

Table 29 – LCC Comparison – 2038 – Without Traffic Control

2038 Without Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	Very High	High	Moderate	
Required Remaining Service Life	0	$12.44	$12.44	$12.44	$0.00
	5				$8.34
	10	$13.93			$7.19
	15		$12.02		$22.14
	20	$10.37			
	25			$8.94	
	30	$7.71	$7.71		$3.98
	35				$3.44
	40	$5.74			$10.57
	45		$4.95		
	50	$4.27		$4.27	
	55				$1.90
	60	$3.18	$3.18		$1.64
	65				$5.05
	70	$2.36			
	75		$2.04	$2.04	
	80	$1.76			$0.91
	85				$0.78
	90	$1.31	$1.31		$2.41
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
Real Discount Rate 3.00%

*$1/SY = $1.19/m^2

Table 30 – LCC Comparison – 2038 – With Traffic Control

2038 With Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	Very High	High	Moderate	
Required Remaining Service Life	0	$19.71	$19.71	$19.71	$0.00
	5				$8.34
	10	$19.34			$7.19
	15		$16.68		$36.13
	20	$14.39			
	25			$12.41	
	30	$10.71	$10.71		$3.98
	35				$3.44
	40	$7.97			$17.26
	45		$6.87		
	50	$5.93		$5.93	
	55				$1.90
	60	$4.41	$4.41		$1.64
	65				$8.24
	70	$3.28			
	75		$2.83	$2.83	
	80	$2.44			$0.91
	85				$0.78
	90	$1.82	$1.82		$3.94
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
Real Discount Rate 3.00%

Table 31 – LCC Comparison – 2048 – Without Traffic Control

2048 Without Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	Very High	High	Moderate	
Required Remaining Service Life	0	$9.25	$9.25	$9.25	$0.00
	5				$6.21
	10	$10.37			$5.35
	15		$8.94		$16.47
	20	$7.71			
	25			$6.65	
	30	$5.74	$5.74		$2.96
	35				$2.56
	40	$4.27			$7.87
	45		$3.68		
	50	$3.18		$3.18	
	55				$1.42
	60	$2.36	$2.36		$1.22
	65				$3.76
	70	$1.76			
	75		$1.52	$1.52	
	80	$1.31			$0.68
	85				$0.58
	90	$0.97	$0.97		$1.79
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
* Real Discount Rate 3.00%

*$1/SY = $1.19/m²

Table 32 – LCC Comparison – 2048 – With Traffic Control

2048 With Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	Very High	High	Moderate	
Required Remaining Service Life	0	$14.66	$14.66	$14.66	$0.00
	5				$6.21
	10	$14.39			$5.35
	15		$12.41		$26.88
	20	$10.71			
	25			$9.24	
	30	$7.97	$7.97		$2.96
	35				$2.56
	40	$5.93			$12.84
	45		$5.11		
	50	$4.41		$4.41	
	55				$1.42
	60	$3.28	$3.28		$1.22
	65				$6.13
	70	$2.44			
	75		$2.11	$2.11	
	80	$1.82			$0.68
	85				$0.58
	90	$1.35	$1.35		$2.93
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
Real Discount Rate 3.00%

Table 33 – LCC Comparison – 2058 – Without Traffic Control

2058 Without Traffic Control		**Polymer Overlay ($/SY)**			**Concrete Overlay ($/SY)**
		Average Daily Traffic (ADT)			
	Year	**Very High**	**High**	**Moderate**	
Required Remaining Service Life	0	$6.89	$6.89	$6.89	$0.00
	5				$4.62
	10	$7.71			$3.98
	15		$6.65		$12.26
	20	$5.74			
	25			$4.95	
	30	$4.27	$4.27		$2.21
	35				$1.90
	40	$3.18			$5.85
	45		$2.74		
	50	$2.36		$2.36	
	55				$1.05
	60	$1.76	$1.76		$0.91
	65				$2.80
	70	$1.31			
	75		$1.13	$1.13	
	80	$0.97			$0.50
	85				$0.43
	90	$0.72	$0.72		$1.34
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
Real Discount Rate 3.00%

*$1/SY = $1.19/m^2

Table 34 – LCC Comparison – 2058 – With Traffic Control

2058 With Traffic Control		**Polymer Overlay ($/SY)**			**Concrete Overlay ($/SY)**
		Average Daily Traffic (ADT)			
	Year	**Very High**	**High**	**Moderate**	
Required Remaining Service Life	0	$10.91	$10.91	$10.91	$0.00
	5				$4.62
	10	$10.71			$3.98
	15		$9.24		$20.00
	20	$7.97			
	25			$6.87	
	30	$5.93	$5.93		$2.21
	35				$1.90
	40	$4.41			$9.55
	45		$3.81		
	50	$3.28		$3.28	
	55				$1.05
	60	$2.44	$2.44		$0.91
	65				$4.56
	70	$1.82			
	75		$1.57	$1.57	
	80	$1.35			$0.50
	85				$0.43
	90	$1.01	$1.01		$2.18
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
Real Discount Rate 3.00%

Life Cycle Cost Comparison Examples

Several examples are presented below to illustrate the use of the LCC comparison tables.

Example 1:

Given: A maintenance strategy for a 30-year old bridge deck is to be selected in the current year (2008). 0.5% of the bridge deck is presently deteriorated in the worst span lane and the total desired service life of the bridge deck is 100 years. The ADT for the bridge is 30,000 and traffic control is to be considered.

Solution: An ADT of 30,000 is considered high when determining the maintenance schedule for the polymer overlays. Given that the bridge deck is 30 years old and the desired service life is 100 years, the required remaining service life is 70 years. Using Table 24 the LCCs are summed from year 0 – 70 for the two maintenance alternatives being investigated (values to be summed are highlighted in Table 35).

Table 35 – LCCA – Example 1

2008 With Traffic Control		**Polymer Overlay ($/SY)**			**Concrete Overlay ($/SY)**
		Average Daily Traffic (ADT)			
	Year	**Very High**	**High**	**Moderate**	
Required Remaining Service Life	0	$47.83	$47.83	$47.83	$0.00
	5				$20.25
	10	$46.94			$17.46
	15		$40.49		$87.70
	20	$34.93			
	25			$30.13	
	30	$25.99	$25.99		$9.67
	35				$8.34
	40	$19.34			$41.88
	45		$16.68		
	50	$14.39		$14.39	
	55				$4.62
	60	$10.71	$10.71		$3.98
	65				$20.00
	70	$7.97			
	Sum		$141.69		$213.91
	75		$6.87	$6.87	
	80	$5.93			$2.21
	85				$1.90
	90	$4.41	$4.41		$9.55
	95				
	100				
	Sum	$218.42	$294.67	$99.22	$441.48

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)

Real Discount Rate 3.00%

*$1/SY = $1.19/m^2

Summing the LCCs for the two maintenance alternatives over the 70-year period yields total LCCs of \$141.70/SY (\$168.62/m^2) and \$213.90/SY (\$254.54/m^2) for polymer overlays and concrete overlays, respectively. Therefore, the decision in this case would be to use polymer overlays to extend the service life of the bridge deck.

Example 2:

Given: The same bridge deck presented in Example 1 is to be considered. However, now the ADT is 60,000 and the required remaining service life is only 25 years.

Solution: The LCCs to be summed are highlighted in Table 36.

Table 36 – LCCA – Example 2

2008 With Traffic Control		**Polymer Overlay ($/SY)** **Average Daily Traffic (ADT)**			**Concrete Overlay ($/SY)**
	Year	**Very High**	**High**	**Moderate**	
Required Remaining Service Life	0	$47.83	$47.83	$47.83	$0.00
	5				$20.25
	10	$46.94			$17.46
	15		$40.49		$87.70
	20	$34.93			
	25			$30.13	
	Sum	$129.69			$125.41
	30	$25.99	$25.99		$9.67
	35				$8.34
	40	$19.34			$41.88
	45		$16.68		
	50	$14.39		$14.39	
	55				$4.62
	60	$10.71	$10.71		$3.98
	65				$20.00
	70	$7.97			
	75		$6.87	$6.87	
	80	$5.93			$2.21
	85				$1.90
	90	$4.41	$4.41		$9.55
	95				
	100				
	Sum	$348.11	$152.98	$99.22	$352.98

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)

Real Discount Rate 3.00%

*\$1/SY = \$1.19/m^2

Summing the LCCs for the two maintenance alternatives over the 25-year period yields total LCCs of \$129.70/SY (\$154.34/m^2) and \$125.41/SY (\$149.24/m^2) for polymer overlays and concrete overlays, respectively. Therefore, the decision in this case would be to use concrete overlays to extend the service life of the bridge deck. However, since the difference between the two alternatives is small, the engineer may choose the higher cost alternative based on other extenuating circumstances.

Example 3:

Given: A bridge deck is currently 20 years old and showing no signs of corrosion deterioration. Using measured chloride concentrations, and cover depths, engineers have estimated that the bridge deck will begin to reflect corrosion damage in approximately 20 years. The projected ADT of the bridge deck is 10,000 and the total desired service life is 80 years. Traffic control will not be considered.

Solution: Given that the maintenance strategy is not to be selected in the current year, but rather 20 years in the future, the Table for year 2028 will be used for the analysis. The ADT is considered moderate and at the time of analysis the bridge deck will be 40 years old. Therefore, the required remaining service life will be 40 years.

Table 37 – LCCA – Example 3

2028 Without Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	Very High	High	Moderate	
Required Remaining Service Life	0	$16.72	$16.72	$16.72	$0.00
	5				$11.21
	10	$18.72			$9.67
	15		$16.15		$29.75
	20	$13.93			
	25			$12.02	
	30	$10.37	$10.37		$5.35
	35				$4.62
	40	$7.71			$14.21
	Sum			$28.73	$74.81
	45		$6.65		
	50	$5.74		$5.74	
	55				$2.56
	60	$4.27	$4.27		$2.21
	65				$6.79
	70	$3.18			
	75		$2.74	$2.74	
	80	$2.36			$1.22
	85				$1.05
	90	$1.76	$1.76		$3.24
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
* Real Discount Rate 3.00%

*$1/SY = $1.19/m^2

As shown in Table 37, the total LCCs for the maintenance alternatives are \$28.74/SY (\$34.20/m^2) and \$74.81/SY (\$89.02/m^2) for polymer overlays and concrete overlays, respectively. Therefore, polymer overlays would be selected as the most cost effective alternative.

Example 4:

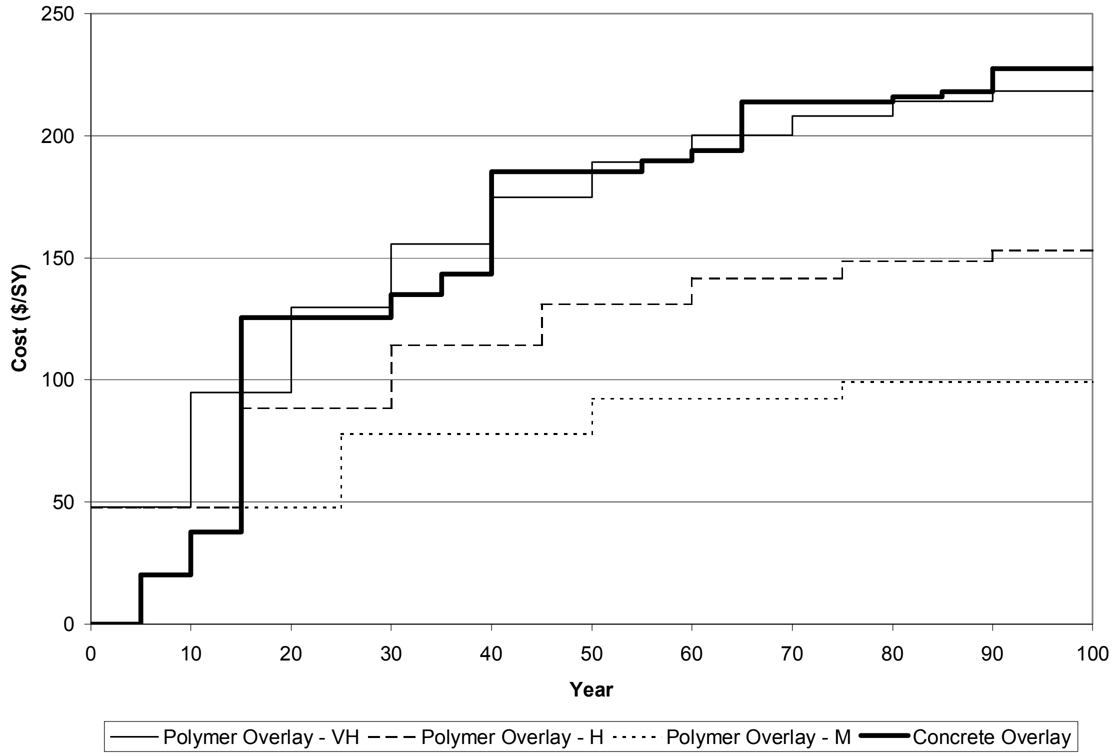
Given: A maintenance strategy for a 40-year old bridge deck is to be selected in the current year (2008). 4.0% of the bridge deck is presently deteriorated in the worst span lane and the total desired service life of the bridge deck is 100 years. The ADT for the bridge is 30,000 and traffic control is to be considered.

Solution: A LCCA is not required for this example because polymer overlays are not appropriate for bridge decks with total areas of corrsion deterioration greater than 1%. Therefore, concrete overlays should be selected as the maintenance alternative for this example.

Example 5:

Given: A bridge deck is reflecting between 0 and 1% corrosion deterioration in the worst span lane and a maintenance strategy is to be selected in the current year. Due to expected changes in traffic patterns the engineer is not able to accurately estimate future traffic loads. The engineer is also unsure of what the required service life of the bridge will be. Traffic control is to be considered.

Solution: Given the uncertainties in the problem the engineer can develop cash flow charts using the tabulated LCC data to investigate the sensitivity of the analysis to individual parameters. An example of a cash flow diagram for a bridge deck whose maintenance strategy is to be selected in the current year is presented in Figure 61.



*$1/SY = $1.19/m^2

Figure 61 – Cash Flow Diagram – Example 5

Using the cash flow diagram presented above, the engineer can investigate the effects of varying traffic volumes on the selection of the appropriate maintenance strategy. As shown, polymer overlays are the most economical option in nearly all cases. Concrete overlays are more cost-effective only if the ADT is expected to be very high and the required remaining service life falls within certain time frames. Therefore, due to the uncertainty of the project the most economical alternative will most likely be polymer overlays.

Maintenance Planning Costs

The maintenance alternative decisions presented in the previous section were based upon total LCCs calculated using a discount rate of 3.0%. LCCAs are useful for determining the most cost-effective maintenance alternative for a given bridge deck. However, it is often the case that maintenance officials are interested in the total cost of a maintenance alternative. Knowing the total cost of an alternative is useful in that there are inherent risks when conducting LCCAs using estimated discount rates because if actual discount rates differ from estimated rates the resulting LCCs may vary substantially. By estimating the total inflated costs of maintenance alternatives the engineer can compare alternatives based upon total expected costs. These comparisons will be void of any benefit that is anticipated from saving money in the short-term. Additionally, by computing actual costs the engineer can estimate the number of dollars that will be required to maintain a bridge in any given year.

To estimate the total cost of a maintenance alternative, the project costs in future years must be inflated from costs in the current year. Costs were increased using an inflation rate of 3.0%, which was estimated for construction projects previously. The resulting inflated costs for maintenance strategy decisions to be made within the next 50 years are presented in Tables 38 – 49.

Future costs were calculated using the following time-value of money relationship:

$$FV = PV \times (1+i)^n \qquad \text{Equation 40}$$

where:

FV = Future Value

PV = Present Value

i = Inflation rate

n = Number of years in the future

Table 38 – Inflated Cost Comparison – 2008 – Without TC

2008 Without Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	**Very High**	**High**	**Moderate**	
Required Remaining Service Life	0	$30.19	$30.19	$30.19	$0.00
	5				$27.21
	10	$61.07			$31.54
	15		$70.79		$130.42
	20	$82.07			
	25			$95.14	
	30	$110.29	$110.29		$56.97
	35				$66.04
	40	$148.23			$273.07
	45		$171.84		
	50	$199.20		$199.20	
	55				$119.28
	60	$267.71	$267.71		$138.28
	65				$571.74
	70	$359.79			
	75		$417.09	$417.09	
	80	$483.52			$249.74
	85				$289.52
	90	$649.81	$649.81		$1,197.09
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
* Inflation Rate 3.00%

*$1/SY = $1.19/m^2

Table 39 – Inflated Cost Comparison – 2008 – With TC

2008 With Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	**Very High**	**High**	**Moderate**	
Required Remaining Service Life	0	$47.83	$47.83	$47.83	$0.00
	5				$27.21
	10	$84.77			$31.54
	15		$98.28		$212.87
	20	$113.93			
	25			$132.08	
	30	$153.11	$153.11		$56.97
	35				$66.04
	40	$205.77			$445.69
	45		$238.54		
	50	$276.54		$276.54	
	55				$119.28
	60	$371.64	$371.64		$138.28
	65				$933.18
	70	$499.46			
	75		$579.01	$579.01	
	80	$671.23			$249.74
	85				$289.52
	90	$902.07	$902.07		$1,953.87
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
* Inflation Rate 3.00%

Table 40 – Inflated Cost Comparison – 2018 – Without TC

2018 Without Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	Very High	High	Moderate	
Required Remaining Service Life	0	$40.57	$40.57	$40.57	$0.00
	5				$36.57
	10	$82.07			$42.39
	15		$95.14		$175.27
	20	$110.29			
	25			$127.86	
	30	$148.23	$148.23		$76.56
	35				$88.75
	40	$199.20			$366.98
	45		$230.93		
	50	$267.71		$267.71	
	55				$160.30
	60	$359.79	$359.79		$185.83
	65				$768.37
	70	$483.52			
	75		$560.53	$560.53	
	80	$649.81			$335.63
	85				$389.09
	90	$873.29	$873.29		$1,608.79
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
* Inflation Rate 3.00%

*$1/SY = $1.19/m^2

Table 41 – Inflated Cost Comparison – 2018 – With TC

2018 With Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	Very High	High	Moderate	
Required Remaining Service Life	0	$64.28	$64.28	$64.28	$0.00
	5				$36.57
	10	$113.93			$42.39
	15		$132.08		$286.07
	20	$153.11			
	25			$177.50	
	30	$205.77	$205.77		$76.56
	35				$88.75
	40	$276.54			$598.97
	45		$320.58		
	50	$371.64		$371.64	
	55				$160.30
	60	$499.46	$499.46		$185.83
	65				$1,254.12
	70	$671.23			
	75		$778.14	$778.14	
	80	$902.07			$335.63
	85				$389.09
	90	$1,212.31	$1,212.31		$2,625.84
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
* Inflation Rate 3.00%

Table 42 – Inflated Cost Comparison – 2028 – Without TC

2028 Without Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	Very High	High	Moderate	
Required Remaining Service Life	0	$54.53	$54.53	$54.53	$0.00
	5				$49.14
	10	$110.29			$56.97
	15		$127.86		$235.55
	20	$148.23			
	25			$171.84	
	30	$199.20	$199.20		$102.89
	35				$119.28
	40	$267.71			$493.19
	45		$310.35		
	50	$359.79		$359.79	
	55				$215.43
	60	$483.52	$483.52		$249.74
	65				$1,032.62
	70	$649.81			
	75		$753.31	$753.31	
	80	$873.29			$451.06
	85				$522.90
	90	$1,173.63	$1,173.63		$2,162.08
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
* Inflation Rate 3.00%

*$1/SY = $1.19/m^2$

Table 43 – Inflated Cost Comparison – 2028 – With TC

2028 With Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	Very High	High	Moderate	
Required Remaining Service Life	0	$86.39	$86.39	$86.39	$0.00
	5				$49.14
	10	$153.11			$56.97
	15		$177.50		$384.46
	20	$205.77			
	25			$238.54	
	30	$276.54	$276.54		$102.89
	35				$119.28
	40	$371.64			$804.97
	45		$430.84		
	50	$499.46		$499.46	
	55				$215.43
	60	$671.23	$671.23		$249.74
	65				$1,685.43
	70	$902.07			
	75		$1,045.75	$1,045.75	
	80	$1,212.31			$451.06
	85				$522.90
	90	$1,629.25	$1,629.25		$3,528.91
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
* Inflation Rate 3.00%

Table 44 – Inflated Cost Comparison – 2038 – Without TC

2038 Without Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	**Very High**	**High**	**Moderate**	
Required Remaining Service Life	0	$73.28	$73.28	$73.28	$0.00
	5				$66.04
	10	$148.23			$76.56
	15		$171.84		$316.56
	20	$199.20			
	25			$230.93	
	30	$267.71	$267.71		$138.28
	35				$160.30
	40	$359.79			$662.80
	45		$417.09		
	50	$483.52		$483.52	
	55				$289.52
	60	$649.81	$649.81		$335.63
	65				$1,387.76
	70	$873.29			
	75		$1,012.39	$1,012.39	
	80	$1,173.63			$606.19
	85				$702.74
	90	$1,577.27	$1,577.27		$2,905.66
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
* Inflation Rate 3.00%

*$1/SY = $1.19/m^2

Table 45 – Inflated Cost Comparison – 2038 – With TC

2038 With Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	**Very High**	**High**	**Moderate**	
Required Remaining Service Life	0	$116.10	$116.10	$116.10	$0.00
	5				$66.04
	10	$205.77			$76.56
	15		$238.54		$516.68
	20	$276.54			
	25			$320.58	
	30	$371.64	$371.64		$138.28
	35				$160.30
	40	$499.46			$1,081.81
	45		$579.01		
	50	$671.23		$671.23	
	55				$289.52
	60	$902.07	$902.07		$335.63
	65				$2,265.07
	70	$1,212.31			
	75		$1,405.40	$1,405.40	
	80	$1,629.25			$606.19
	85				$702.74
	90	$2,189.57	$2,189.57		$4,742.56
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
* Inflation Rate 3.00%

Table 46 – Inflated Cost Comparison – 2048 – Without TC

2048 Without Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	Very High	High	Moderate	
Required Remaining Service Life	0	$98.48	$98.48	$98.48	$0.00
	5				$88.75
	10	$199.20			$102.89
	15		$230.93		$425.43
	20	$267.71			
	25			$310.35	
	30	$359.79	$359.79		$185.83
	35				$215.43
	40	$483.52			$890.75
	45		$560.53		
	50	$649.81		$649.81	
	55				$389.09
	60	$873.29	$873.29		$451.06
	65				$1,865.03
	70	$1,173.63			
	75		$1,360.56	$1,360.56	
	80	$1,577.27			$814.67
	85				$944.42
	90	$2,119.72	$2,119.72		$3,904.96
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
* Inflation Rate 3.00%

*$1/SY = $1.19/m^2

Table 47 – Inflated Cost Comparison – 2048 – With TC

2048 With Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	Very High	High	Moderate	
Required Remaining Service Life	0	$156.02	$156.02	$156.02	$0.00
	5				$88.75
	10	$276.54			$102.89
	15		$320.58		$694.37
	20	$371.64			
	25			$430.84	
	30	$499.46	$499.46		$185.83
	35				$215.43
	40	$671.23			$1,453.86
	45		$778.14		
	50	$902.07		$902.07	
	55				$389.09
	60	$1,212.31	$1,212.31		$451.06
	65				$3,044.07
	70	$1,629.25			
	75		$1,888.74	$1,888.74	
	80	$2,189.57			$814.67
	85				$944.42
	90	$2,942.60	$2,942.60		$6,373.61
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
* Inflation Rate 3.00%

Table 48 – Inflated Cost Comparison – 2058 – Without TC

2058 Without Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	Very High	High	Moderate	
Required Remaining Service Life	0	$132.35	$132.35	$132.35	$0.00
	5				$119.28
	10	$267.71			$138.28
	15		$310.35		$571.74
	20	$359.79			
	25			$417.09	
	30	$483.52	$483.52		$249.74
	35				$289.52
	40	$649.81			$1,197.09
	45		$753.31		
	50	$873.29		$873.29	
	55				$522.90
	60	$1,173.63	$1,173.63		$606.19
	65				$2,506.45
	70	$1,577.27			
	75		$1,828.49	$1,828.49	
	80	$2,119.72			$1,094.84
	85				$1,269.22
	90	$2,848.72	$2,848.72		$5,247.94
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
* Inflation Rate 3.00%

*$1/SY = $1.19/m^2

Table 49 – Inflated Cost Comparison – 2058 – With TC

2058 With Traffic Control		Polymer Overlay ($/SY) Average Daily Traffic (ADT)			Concrete Overlay ($/SY)
	Year	Very High	High	Moderate	
Required Remaining Service Life	0	$209.68	$209.68	$209.68	$0.00
	5				$119.28
	10	$371.64			$138.28
	15		$430.84		$933.18
	20	$499.46			
	25			$579.01	
	30	$671.23	$671.23		$249.74
	35				$289.52
	40	$902.07			$1,953.87
	45		$1,045.75		
	50	$1,212.31		$1,212.31	
	55				$522.90
	60	$1,629.25	$1,629.25		$606.19
	65				$4,090.98
	70	$2,189.57			
	75		$2,538.31	$2,538.31	
	80	$2,942.60			$1,094.84
	85				$1,269.22
	90	$3,954.61	$3,954.61		$8,565.59
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
* Inflation Rate 3.00%

The maintenance planning costs presented in Tables 38 – 49 were derived from the maintenance activity timings presented in Table 50. These maintenance activities are the same as those used in the previous LCCA examples.

Table 50 – Maintenance Activity Timings – Total Costs

Year	Polymer Overlays			Concrete Overlays
	ADT			
	VH	H	M	
0	PO & 1% P	PO & 1% P	PO & 1% P	
5				6% P
10	OR, PO, & 1% P			6% P
15		OR, PO, & 1% P		M, CO, & G
20	OR, PO, and 1%P			
25			OR, PO, & 1% P	
30	OR, PO, and 1%P	OR, PO, & 1% P		6% P
35				6% P
40	OR, PO, and 1%P			M, CO, & G
45		OR, PO, & 1% P		
50	OR, PO, and 1%P		OR, PO, & 1% P	
55				6% P
60	OR, PO, and 1%P	OR, PO, & 1% P		6% P
65				M, CO, & G
70	OR, PO, and 1%P			
75		OR, PO, & 1% P	OR, PO, & 1% P	
80	OR, PO, and 1%P			6% P
85				6% P
90	OR, PO, and 1%P	OR, PO, & 1% P		M, CO, & G
95				
100				

* PO = Polymer Overlay; P = Patching; OR = Overlay Removal; M = Milling; CO = Concrete Overlay; G = Bridge Deck Grooving

Inflated Cost Comparison Examples

The examples presented in the LCCA section are revisited below to compare decisions based upon LCCA methodology and total cost methodology.

Example 1:

Given: A maintenance strategy for a 30-year old bridge deck is to be selected in the current year (2008). 0.5% of the bridge deck is presently deteriorated in the worst span lane and the total desired service life of the bridge deck is 100 years. The ADT for the bridge is 30,000 and traffic control is to be considered.

Solution: An ADT of 30,000 is considered high when determining the maintenance schedule for the polymer overlays. Given that the bridge deck is 30 years old and the desired service life is 100 years, the required remaining service life is 70 years. Using Table 39 the inflated costs are summed from year 0 – 70 for the two maintenance alternatives being investigated (values to be summed are highlighted in Table 51).

Table 51 – Inflated Costs – Example 1

2008 With Traffic Control		**Polymer Overlay ($/SY)**			**Concrete Overlay ($/SY)**
		Average Daily Traffic (ADT)			
	Year	**Very High**	**High**	**Moderate**	
Required Remaining Service Life	0	$47.83	$47.83	$47.83	$0.00
	5				$27.21
	10	$84.77			$31.54
	15		$98.28		$212.87
	20	$113.93			
	25			$132.08	
	30	$153.11	$153.11		$56.97
	35				$66.04
	40	$205.77			$445.69
	45		$238.54		
	50	$276.54		$276.54	
	55				$119.28
	60	$371.64	$371.64		$138.28
	65				$933.18
	70	$499.46			
	Sum		$909.40		$2,031.05
	75		$579.01	$579.01	
	80	$671.23			$249.74
	85				$289.52
	90	$902.07	$902.07		$1,953.87
	95				
	100				
	Sum	$3,326.35	$3,299.89	$1,035.45	$6,555.24

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
* Inflation Rate 3.00%
*$1/SY = $1.19/m^2

Summing the inflated costs for the two maintenance alternatives over the 70-year period yields total inflated costs of $909.40/SY ($1082.19/m^2) and $2031.05/SY (2416.95/m^2) for polymer overlays and concrete overlays, respectively. Therefore, the decision in this case would be to use polymer overlays to extend the service life of the bridge deck. As shown, the maintenance alternative recommendation does not change when the total inflated costs are considered in place of the total LCCs for this example.

Example 2:

Given: The same bridge deck presented in Example 1 is to be considered. However, now the ADT is 60,000 and the required remaining service life is only 25 years.

Solution: The inflated costs to be summed are highlighted in Table 52.

Table 52 – Inflated Costs – Example 2

2008 With Traffic Control		**Polymer Overlay ($/SY)** Average Daily Traffic (ADT)			**Concrete Overlay ($/SY)**
	Year	**Very High**	**High**	**Moderate**	
Required Remaining Service Life	0	$47.83	$47.83	$47.83	$0.00
	5				$27.21
	10	$84.77			$31.54
	15		$98.28		$212.87
	20	$113.93			
	25			$132.08	
	Sum	$246.53			$271.61
	30	$153.11	$153.11		$56.97
	35				$66.04
	40	$205.77			$445.69
	45		$238.54		
	50	$276.54		$276.54	
	55				$119.28
	60	$371.64	$371.64		$138.28
	65				$933.18
	70	$499.46			
	75		$579.01	$579.01	
	80	$671.23			$249.74
	85				$289.52
	90	$902.07	$902.07		$1,953.87
	95				
	100				
	Sum	$3,572.88	$2,390.48	$1,035.45	$4,795.80

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)

* Inflation Rate 3.00%

*$1/SY = $1.19/m^2

Summing the inflated costs for the two maintenance alternatives over the 25-year period yields total inflated costs of $246.53/SY ($293.37/m^2) and $271.61/SY ($323.22/m^2) for polymer overlays and concrete overlays, respectively. Therefore, the decision in this case would be to use polymer overlays to extend the service life of the bridge deck. Thus, $25.08/SY ($29.85/m^2) of bridge deck would be saved. This money could then be invested into transportation-related research projects or other required maintenance activities. In this example, using the total inflated costs resulted in a different recommendation as compared to the same example using LCCA. This is due to the specific timings and magnitudes of the maintenance activities presented.

Example 3:

Given: A bridge deck is currently 20 years old and showing no signs of corrosion deterioration. Using measured chloride concentrations, and cover depths, engineers have estimated that the bridge deck will begin to reflect corrosion damage in approximately 20 years. The projected ADT of the bridge deck is 10,000 and the total desired service life is 80 years. Traffic control will not be considered.

Solution: Given that the maintenance strategy is not to be selected in the current year, but rather 20 years in the future, the Table for year 2028 will be used for the analysis. The ADT is considered moderate and at the time of analysis the bridge deck will be 40 years old. Therefore, the required remaining service life will be 40 years.

Table 53 – Inflated Costs – Example 3

2028 Without Traffic Control		Polymer Overlay ($/SY)			Concrete Overlay ($/SY)
		Average Daily Traffic (ADT)			
	Year	Very High	High	Moderate	
Required Remaining Service Life	0	$54.53	$54.53	$54.53	$0.00
	5				$49.14
	10	$110.29			$56.97
	15		$127.86		$235.55
	20	$148.23			
	25			$171.84	
	30	$199.20	$199.20		$102.89
	35				$119.28
	40	$267.71			$493.19
	Sum			$226.36	$1,057.01
	45		$310.35		
	50	$359.79		$359.79	
	55				$215.43
	60	$483.52	$483.52		$249.74
	65				$1,032.62
	70	$649.81			
	75		$753.31	$753.31	
	80	$873.29			$451.06
	85				$522.90
	90	$1,173.63	$1,173.63		$2,162.08
	95				
	100				

* ADT: Moderate = 5,000 to 25,000; High = > 25,000; Very High = > 50,000 (Weyers et al., 1993)
* Inflation Rate 3.00%

*$1/SY = $1.19/m^2

As shown in Table 53, the total inflated costs for the maintenance alternatives are \$226.37/SY (\$269.38/m^2) and \$1057.02/SY (\$1257.85/m^2) for polymer overlays and concrete overlays, respectively. Therefore, polymer overlays would be selected as the most cost effective alternative.

Example 4:

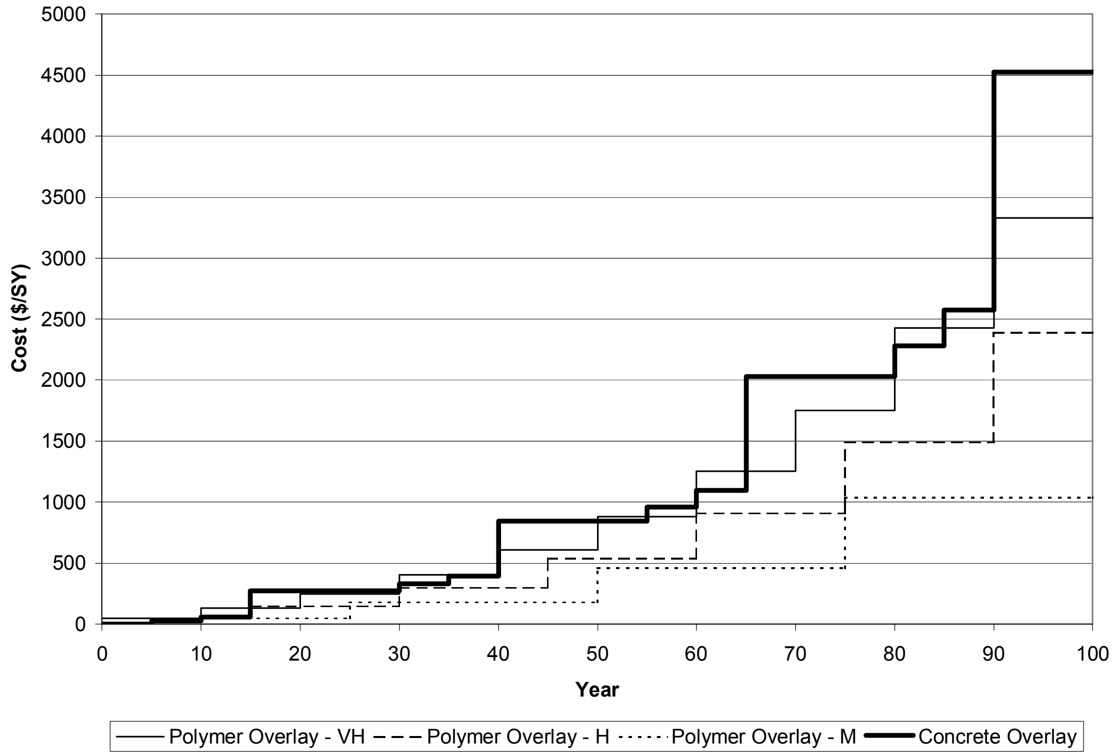
Given: A maintenance strategy for a 40-year old bridge deck is to be selected in the current year (2008). 4.0% of the bridge deck is presently deteriorated in the worst span lane and the total desired service life of the bridge deck is 100 years. The ADT for the bridge is 30,000 and traffic control is to be considered.

Solution: A total maintenance cost analysis is not required for this example because polymer overlays are not appropriate for bridge decks with total areas of corrsion deterioration greater than 1%. Therefore, concrete overlays should be selected as the maintenance alternative for this example.

Example 5:

Given: A bridge deck is reflecting between 0 and 1% corrosion deterioration in the worst span lane and a maintenance strategy is to be selected in the current year. Due to expected changes in traffic patterns the engineer is not able to accurately estimate future traffic loads. The engineer is also unsure of what the required service life of the bridge will be. Traffic control is to be considered.

Solution: Given the uncertainties in the problem the engineer can develop cash flow charts using the tabulated inflated cost data to investigate the sensitivity of the analysis to individual parameters. An example of a cash flow diagram for a bridge deck whose maintenance strategy is to be selected in the current year is presented in Figure 62.



*$1/SY = $1.19/m^2

Figure 62 – Cash Flow Diagram – Inflated Costs – Example 5

Using the cash flow diagram presented above, the engineer can investigate the effects that varying traffic volumes will have on the selection of the appropriate maintenance strategy. As shown, polymer overlays are the most economical option in nearly all cases. Concrete overlays are more cost-effective only if the ADT is expected to be very high and the required remaining service life falls within certain time frames. Therefore, due to the uncertainty of the project the most economical alternative will most likely be polymer overlays. It is important to note that this total cost analysis results in the same recommendations as the LCCA.

The total cost approach is not expected to affect the maintenance alternative selected in most cases. However, it will provide useful data concerning the anticipated costs to be incurred in the future. By knowing the timings and amounts of future maintenance expenditures the engineer can better plan and budget maintenance activities.

Table 54 presents the LCCA and Total Inflated Costs that were calculated for Examples 1 – 3. As shown, the maintenance alternative selected remained the same for Examples 1 and 3, regardless of the analysis method. The recommended maintenance alternative changed for Example 2, however, the argument can be made that the the cost difference computed for LCCA method is so minute that either maintenace alternative could be selected.

Table 54 – LCCA/Total Inflated Cost Comparison

		Example 1		Example 2		Example 3	
	Methodology	Polymer	Concrete	Polymer	Concrete	Polymer	Concrete
Cost/SY	LCCA	$141.70	$213.90	$129.70	$125.41	$28.74	$74.81
	Total Inflated Cost	$909.40	$2,031.06	$246.53	$271.62	$226.37	$1,057.02

*$1/SY = $1.19/m^2

BLCCA Cost Analysis Computer Model

BLCCA is a computer model developed by NCHRP Project number 483 to analyze the LLC's of bridges. BLCCA allows for a rigorous analysis of bridge maintenance alternatives. The LCCAs presented previously only considered one-time agency expenditures at discrete times. BLCCA improves upon that basic model in many ways, some of which are presented below:

- Uncertainty of event timings can be incorporated using statistical distributions for each event.
- User costs can be estimated based upon ADT and other construction activity parameters.
- Differences in the vulnerability of maintenance alternatives may be considered, such as vulnerability to earthquake loadings
- User costs and Agency costs can be adjusted to reflect changes in costs due to increasing or decreasing levels of traffic.
- The sensitivity of the analysis to variations in input parameters such as discount rate and activity timing can be investigated.

- Before, during, and after maintenance activity effects can be considered. For example user costs associated with accidents may be reduced due to the widening of a bridge.

BLCCA is capable of providing a much more detailed analysis than would typically be required. Input to the computer model can be adjusted to provide the level of analysis desired for any individual project. Many times the engineer will only be concerned about the Agency costs associated with maintenance alternatives, in which case the use of a robust computer model such as BLCCA may not be necessary. Additionally, care should be taken when assigning values to input parameters as the effects on the analysis can be substantial. To provide a complete analysis with BLCCA detailed input data concerning anticipated ADT growth, vunerability costs, activity-timing distributions, and discount rate variability is required.

An example of a LCCA using BLCCA is presented in the following section. The example uses the same input data that was used in an example presented previously and also incorporates estimated values for user costs, uniform maintenance costs, and ADT.

1.0 BLCCA Example

This is an example of a LCC comparison between two bridge maintenance alternatives. The two maintenance alternatives are the same as those presented previously (Polymer Overlays and Concrete Overlays). Values presented are intended to provide an approximate representation of a typical bridge deck. This example is a modified version of Example 1 presented in the BLCCA Users Manual. (BLCCA, 2003) The BLCCA Users Manual is to be consulted for an in-depth description of the computer model.

1.1 Structure Description

The bridge to be analyzed has a total area of 2000 SF (222 SY) with a design life of 100 years.

1.2 BLCCA Data Input

1.2.1 Bridge

The first step in conducting a LCCA in BLCCA is to select a bridge and comparison type from the provided bridge inventory. The BLCCA inventory includes a variety of predefined life cycle comparison templates for alternatives such as overlay type and structure widening. BLCCA also contains a database of NBI structure numbers that allows for the user to select the specific bridge to be analyzed. If the bridge is included in the NBI database, the parameters of the bridge such as the length and width will be automatically defined. The bridge in this example is not in the NBI database, so the "Not Defined" bridge must be selected, and appropriate data corrected/added throughout the four Inventory tabs of BLCCA data input.

1.2.2 New Scenario Definition

New scenario definition involves the specification of the analysis period, discount rate and the selection of primary model types to be used for all life-cycle analyses.

The length of analysis period is set to 100 years for this example. This selection was made to reflect the requirement for both alternatives to last at least 100 years. It is also assumed that at year 100 neither of the two alternatives will have any residual (salvage) value, and that at this year the construction-repair cycle will restart.

Federal Infrastructure Projects use discount rates published in OMB Circular No. A-94. Since the length of analysis will be 100 years, a Discount Rate of 3.00% will be used just as in the examples presented previously.

The primary model types that may be considered in BLCCA are Traffic Projection, Condition Index, and Load Capacity. By defining an ADT that increases with time the associated increase in user costs will be included in the LCCA. Changes in the Condition Index and Load Capacity primary models will not affect the LCC's computed by BLCCA, but can be used by the engineer to monitor the overall condition of the structure over its service life. The primary models are by default set to "generic"; this selection will accommodate the data input for this example.

1.2.3 Initial Condition Definition

The first primary model to be defined is the Average Daily Traffic (ADT). The example conditions are such that at year 1 the ADT is equal to 20,000 vehicles, and that at year 100 the ADT is equal to 40,000 vehicles. A two-point curve definition is sufficient to define the ADT projection line.

Table 55 – ADT Primary Model

Traffic Summary Table	
Year	ADT
0	20000
100	40000

The second primary model is the Condition Index. A Condition index of 100 reflects a perfect condition (immediately after initial construction), while 50 would require significant repair. In this example the bridge deck is expected to require significant repair at 35 years of age. The intermediate condition index at year 20 was chosen to reflect the case where the bridge condition deteriorates faster as time progresses.

Table 56 – Condition Index Primary Model

Condition Index	
Year	Index
0	100
20	85
35	50

The third primary model is the Load Capacity. The Load Capacity is taken to be 36 tons and remains constant. This indicates that corrosion of the bridge deck will not affect the load capacity of the structure. The load capacity of the structure is also irrelevant for a LCCA concerning bridge deck overlays.

Table 57 – Load Capacity Primary Model

Load Capacity	
Year	Capacity (tons)
0	36
100	36

1.2.4 Initial Cost Definition

This example does not consider any vulnerability costs. The only initial cost that the bridge will experience is due to NBI Inspections (Distributed Agency Cost): $150 every two years.

BLCCA requires input in equivalent annual cost. Therefore, the inspection costs are set at a constant rate of $75 per year. Furthermore, it is expected that the inspections will not begin until 2 years after the construction. To reflect this delay, a timeshift of 3 years will be used.

Table 58 – Agency Maintenance Costs

Distributed Agency Costs	
Year	**Cost**
0	75
100	75

* Timeshift of 3 years

1.3 Alternative 1 – Polymer Overlays

BLCCA does not require detailed item-by-item cost data input. It is only necessary to provide the total costs associated with a particular event. For example, for an overlay it is not necessary to provide indvidual costs for patching, traffic control, and overlay materials if all of the costs occur in the same year. The user can simply provide an estimated cost for the project as a whole.

The definition of alternatives can be performed using either the Wizard or the Event/Effects tab. To illustrate both options, Alternative 1 will be defined using the Wizard, and Alternative 2 using Event/Effects tab.

1.3.1 Wizard – Polymer Overlays

The Wizard guides the user through a series of steps to define the events and their effects. Alternative 1, Polymer Overlays, involves 5 events and each event lasts 1 year (or less):

Table 59 – Alternative 1 Event Timeline

Event Description		Start Year	Duration
Event 1	Deck Overlay	26	1
Event 2	Deck Overlay	41	1
Event 3	Deck Overlay	56	1
Event 4	Deck Overlay	71	1
Event 5	Deck Overlay	86	1

For example, if an event lasts only 3 months (summer), the duration period will be still 1 year. This number is used to position the agency cost in time. The actual time impact of the repair on the user, vulnerability, and agency costs is defined separately in the Effects Tab.

Each of the events consists of one or more activities/repairs. The user can choose from nearly 50 cost types provided in a drop-down list or create their own. In this case, each event can be fully defined by one type of work and its corresponding cost.

1.3.2 Primary Conditions – Polymer Overlays Wizard

Each of the events can affect the primary conditions. In this alternative after the overlay events some changes are required:

- ADT will remain unchanged. In other words, ADT will increase according to the model and values defined in the Scenario (Initial Conditions).
- Condition Index needs to be re-defined. As the bridge deteriorates with time, the value of the Condition Index decreases. After each event (after each overlay), the condition is improved and the Condition Index has to reflect this change.
- Load Capacity will remain unchanged.

Note the definition of Primary models must be performed in sequential order: 1) ADT, 2) Condition Index, and 3) Load Capacity. To redefine the Primary models the follwing steps are used:

- Click on the arrow in the status box in the Category Index, and a pick-list of the Primary models appears. Double-click the Condition Index to select it.

- Click on the arrow in the status box in the Events Index, and a pick-list of the defined event appears. Double-click the desired Event to select it.
- Click on the Define button in the center of the form, and a Data input form appears.
- Input the values shown in the table below.

Table 60 – Condition Index – Polymer Overlay Redefined

Condition Index	
Year	Index
0	90
10	85
15	80

1.3.3 During Event Costs – Polymer Overlay Wizard

- Vulnerability costs – the repair procedure can leave the structure weaker and thus more vulnerable to damage due to earthquake, scour, etc. In this example, no such effects will be considered.
- User costs – the repair procedure will cause traffic to slow down and increase accident rates.

BLCCA allows the user to select one of two models for the calculation of user costs. The North Carolina model provides comprehensive modeling of user costs with fairly simple input while the generic model allows the user to input the costs directly.

The following table summarizes the data input required to define the user costs for the five events using the generic model type. The values provided are estimated values based upon data provided in Example 1 of the BLCCA Users Manual.

Table 61 – User Costs – Polymer Overlays

	Cost ($)	
Year	Accident	Delay
26	500	5000
41	500	5000
56	500	5000
71	500	5000
86	500	5000

1.3.4 After Event Costs – Polymer Overlays Wizard

- User – bridge overlaying may decrease the user costs due to decreased accident rates.
- Vulnerability – bridge strengthening may decrease the vulnerability costs due to earthquake, scour, etc.
- Distributed Agency – use of new or different technology during bridge repairs, may dictate a different maintenance and inspection schedule.

None of these effects are considered, and the conditions are left unchanged.

1.3.5 Events/Effects – Alternative 1 – Polymer Overlays

This tab shows the events and their timing and costs as defined using the Wizard. Since no uncertainties and no relationships between events were considered, no changes to the event definitions are required.

1.4 Alternative 2 – Concrete Overlays

Alternative 2 has the same initial conditions and deterioration rate as Alternative 1. The only difference will be in the maintenance method used to extend the service life of the bridge deck.

1.4.1 Events – Alternative 2 – Concrete Overlays

The definition of alternatives can be performed using either the Wizard or the Event and Effect tabs. To illustrate both options, Alternative 1 was defined using the Wizard, and Alternative 2 will be defined using the Event/Effects tab.

First add a New alternative in the Alternatives tab.

Then by clicking on the Events/Effects tab you can add the 16 events that comprise Alternative 2. Before you add any events, you must check that you are in the correct bridge, scenario, and alternative.

Table 62 – Alternative 2 Event Timeline

Maintenance Alternative 2	
Year	**Required Maintenance**
37	2% Patching
41	2% Patching
44	2% Patching
47	2% Patching
50	2% Patching
53	2% Patching + Overlay
62	2% Patching
66	2% Patching
69	2% Patching
72	2% Patching
75	2% Patching
78	2% Patching + Overlay
87	2% Patching
91	2% Patching
94	2% Patching
97	2% Patching

Note: by default BLCCA creates the first event at year one.

To define the activities and costs that are included in each event,

1 Select the event for which you would like to define the cost by clicking on its title in the table.
2 Select the Agency Cost sub-tab (in the Events tab); the Cost Component (Agency) table will be gray with no data.
3 Click on the add button, and select the type of cost-component from the pick-list by double-clicking on the item.
4 Click on the Save button.
5 Type in the cost, and click OK; the new Cost component will appear in the table.
6 Repeat the steps for all Cost components; in this case only one cost per event is needed.

7 Select the event Definition sub-tab, click on Event 2, and repeat the above-described steps.

Please refer to the previous examples for the type of Cost components and associated costs for Alternative 2.

1.4.2 Effects – Alternative 2 – Concrete Overlays

The effects for Alternative 2 on the primary conditions are the same as those for Alternative 1 with the exception of the effect on the Condition Index. The Condition Index after the event is adjusted as follows:

Table 63 – Condition Index – Concrete Overlay Redefined

Condition Index	
Year	**Index**
0	90
15	80
25	40

The after event and during event costs are also the same as for Alternative 1 with the exception of user costs. The user costs associated with Alternative 2 are higher due to increased construction times. The appropriate values for Alternative 2 are presented below.

Table 64 – User Costs – Concrete Overlays

	Cost ($)	
Year	**Accident**	**Delay**
53	1500	15000
78	1500	15000

1.5 Analysis Results

There are many reports and charts that can be generated by BLCCA to analyze the results of the analysis, some of which are presented below.

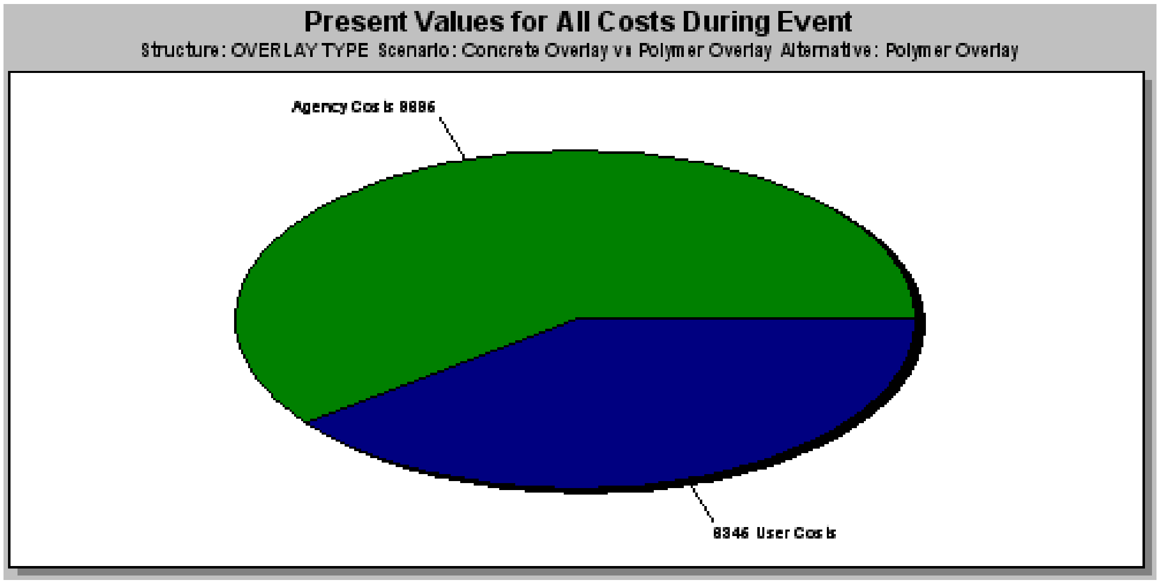


Figure 63 – Present Value During Event Costs – Alternative 1 – Polymer Overlays

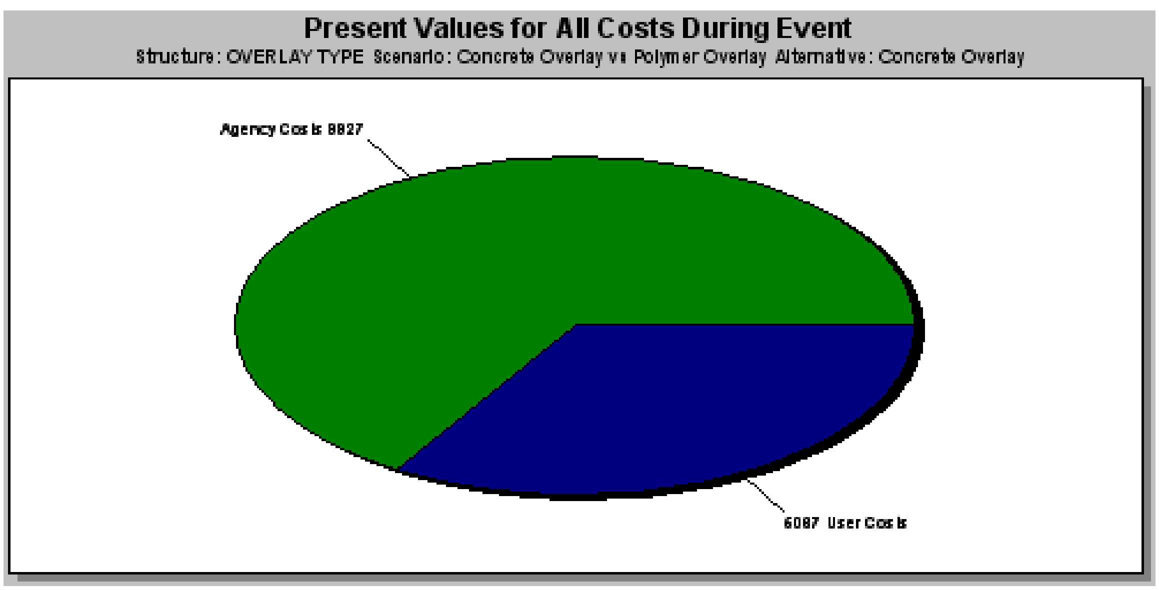


Figure 64 – Present Value During Event Costs – Alternative 2 – Concrete Overlays

Figure 65 – Cumulative During Event Costs – Alternative 1 – Polymer Overlays

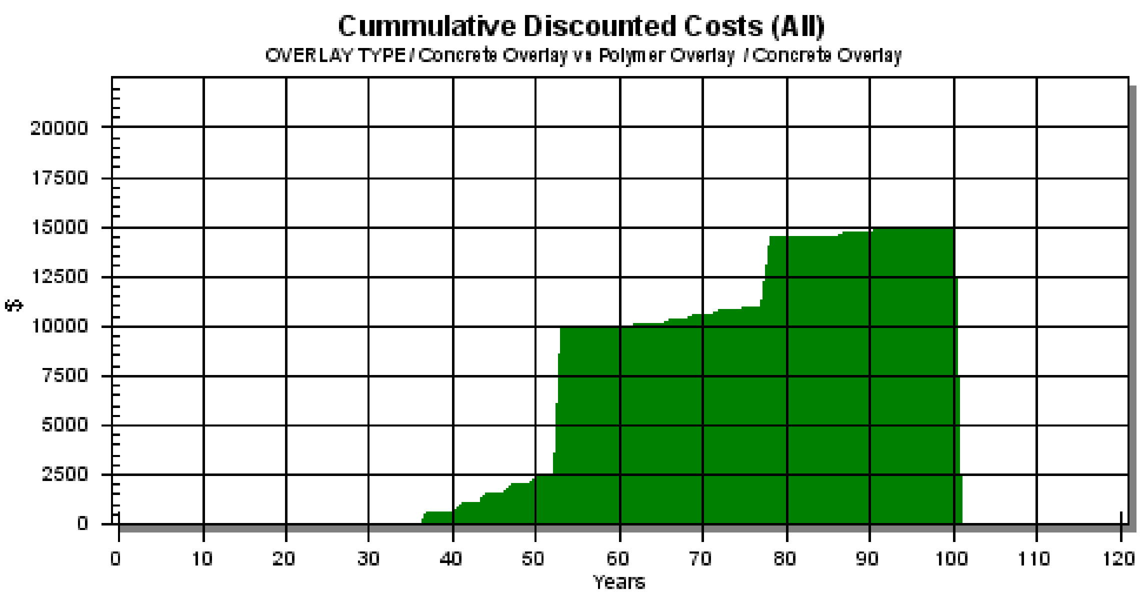


Figure 66 – Cumulative During Event Costs – Alternative 2 – Concrete Overlays

As shown, using concrete overlays is more cost effective than using polymer overlays. However, if user costs are not considered polymer overlays may be more economical. BLCCA can be used to investigate the affect of the different cost components individually. This allows engineers to focus their efforts on optimizing the activities that have the greatest impact on the LCC of a structure. Additionally, a sensitivity analyses may be conducted on the discount rate, event timings, or event costs. A sensitivity analysis for a discount rate that ranges from 2 – 5% is presented in Figure 67.

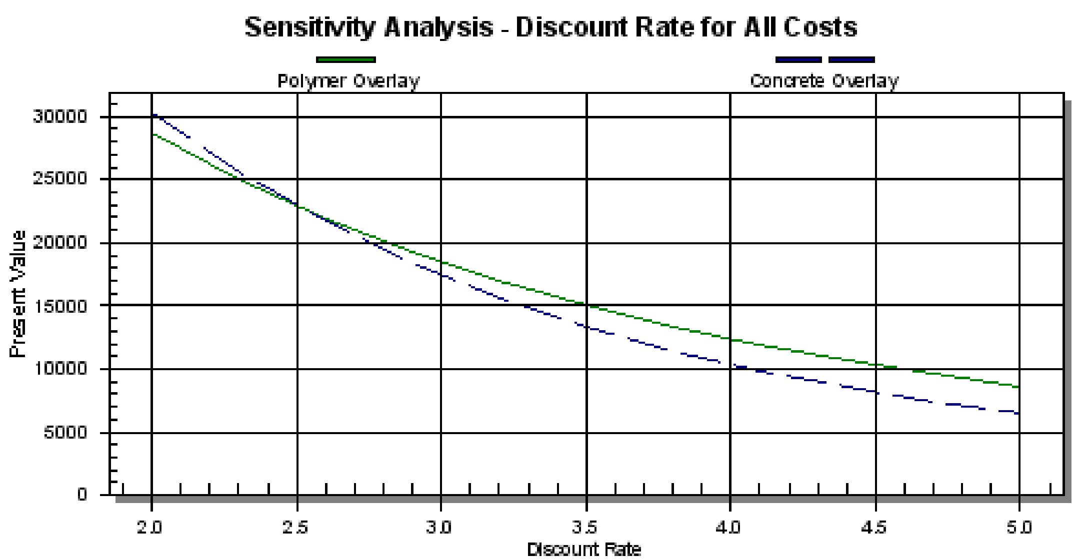


Figure 67 – Discount Rate Sensitivity Analysis

CONCLUSIONS

The following conclusions can be made from the results of this study:

1. The time to first repair and rehabilitation of bridge decks can be modeled using a probabilistic approach, which allows for the incorporation of variability related to chloride exposure conditions, bridge deck construction, and corrosion initiation.

2. Fick's second law of diffusion can be used to model the apparent diffusion rate of chlorides into a concrete bridge deck. Additionally, the effective diffusion rate at any point in time can be projected using available diffusion decay models.

3. The time required for corrosion to induce cracking in the cover concrete can be estimated using existing corrosion-cracking models. An estimated time to corrosion cracking of 6 years for bare steel reinforcement was determined for this study using the Liu/Weyers model in conjunction with the Vu, Stewart, and Mullard model.

4. The time to first repair for the set of 10 – 0.47 $^{w}/_{c}$ bridge decks in Virginia can be accurately predicted using the probabilistic service life model. However, accurate service life predictions for individual bridge decks do not appear to be possible given the limited amount of data available.

5. A reasonable estimate of the time required for corrosion deterioration to progress from a level of 2% to a level of 12% was determined from damage surveys conducted on actively corroding bridge decks. The corrosion propagation time for bare steel bridge decks is estimated to be 16 years.

6. The reduction of the $^{w}/_{c}$ ratio from 0.47 to 0.45 appears to have a negligible effect on the diffusion properties of concrete.

7. The addition of pozzolan to the concrete mixture appears to dramatically reduce the diffusion rate of chlorides into concrete.

8. The service lives of bridge decks constructed under current specifications (0.45 $^{w}/_{c+p}$ and 50 mm (in.) cover depth) are expected to exceed a design life of 100 years regardless of reinforcement type.

9. Surface chloride concentrations, C_o, have been determined for the Commonwealth of Virginia and are only a function of the environmental exposure conditions not the type of concrete.

10. Apparent diffusions, D_{ca}, have been determined for the Commonwealth of Virginia for $^{w}/_{c}$ = 0.47 and $^{w}/_{c}$ = 0.45 with and without pozzolan.

11. Clear concrete cover depths, x, have been determined for two construction specification periods: 0.47 $^{w}/_{c}$ and 0.45 $^{w}/_{c}$.

12. Life Cycle Costs Analyses can be conducted to determine optimum maintenance strategies for individual bridge decks.

13. It is not appropriate to specify a single maintenance strategy for all bridge decks within a system, as the most economical alternative will be dependent upon individual structure parameters.

14. Long-term inflation rates associated with transportation-related construction appear to correspond with the inflation rate of the general economy.

15. A total maintenance cost approach to maintenance strategy selection generally results in the same conclusions as a LCCA approach.

16. Available LCCA computer software can be used to provide a detailed analysis of bride deck maintenance alternatives.

RECOMMENDATIONS

The following recommendations can be made based upon the results of this study:

1. It is recommended that newly constructed bridge decks be built under current specifications with bare steel reinforcement. The decision to use bare steel reinforcement over available alternative reinforcements was made based upon the determination that the service lives of bridge decks constructed under current specifications are expected to exceed 100 years regardless of reinforcement type. Therefore, reinforcement type should be selected on a first-cost basis. Bare steel being the least costly alternative would typically be the reinforcement of choice. However, alternative reinforcements such as galvanized steel, stainless steel, and MMFX-2 may be used in place of bare steel as a secondary corrosion protection measure in cases where extreme chloride exposure is expected.

2. Life Cycle Cost Analyses should be conducted to determine the most economical maintenance alternative for individual bridge decks. Real discount rates provided by the Office of Management and Budget should be used to discount future expenditures. (OMB, 2007) It is also recommended that the engineer account for costs incurred by the traveling public (User costs) as well as Agency costs in order to provide the most economical alternative to the public as a whole.

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VITA

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