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Optimal Approach for Addressing Reinforcement Corrosion for Concrete Bridge Decks in Illinois

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16. Abstract

This report presents the results of a comprehensive literature review focusing on corrosion performance of reinforced concrete bridge decks, with a particular emphasis on the relative performance of alternative corrosion-resistant reinforcement types. Examples of alternative corrosion-protection options examined herein include epoxy-coated, galvanized, stainless-steel, and A1035 bars, considering conventional black reinforcing bars as the standard. Based upon the results of the literature review, a framework for determining the optimal reinforcement option for a bridge deck is presented as a function of the properties of each reinforcement type and other factors, such as design service life, location of the bridge, estimated maintenance/repair cycles, and relative costs. Several examples are also provided to demonstrate the procedure for using the framework and its applicability for different bridge types with varying design considerations, such as a congested urban artery and a rural interstate. The literature review findings and the optimal approach framework were crafted for use by bridge design engineers as preliminary guidance when determining the type of reinforcement for a given bridge deck and its corresponding conditions. Furthermore, the approach can also be used by Illinois Department of Transportation officials when deciding whether to invest in higher performing corrosion-protection systems for a given application or for updating current bridge design policies to reflect the latest developments in alternative corrosion-resistant reinforcement options.

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EXECUTIVE SUMMARY

Corrosion of reinforcement in concrete bridge decks has remained a notable concern toward achieving longer design service lives for these structures in many parts of the United States where exposure to salts or other deterioration agents are common, including in Illinois. Currently, a popular solution to mitigate this corrosion and its potentially detrimental effects to the bridge deck is the use of epoxy-coated reinforcement in place of standard “black” reinforcing bars. Although epoxy-coated bars have been widely used in recent history for Illinois bridge construction, several alternative reinforcement options with enhanced corrosion-resistant performance have also gained popularity albeit with a common trade-off of higher initial material costs. A few examples of these alternatives include galvanized, stainless-steel, and A1035 bars—all of which have been demonstrated effective (to varying degrees) in mitigating corrosion and, as a result, facilitate longer durations between recommended maintenance or replacement of a given bridge.

The primary objective of this project was to develop a framework for selecting the optimal type / configuration of reinforcing bars with enhanced resistance to corrosion for a given bridge (and its inherent characteristics) in Illinois. The framework was built upon a comprehensive literature review that examined previous research focused on corrosion performance, cost-benefit analyses, as well as recommendations and design guidelines issued by other state departments of transportation or governmental agencies. The findings of the literature review were then used to assemble a quantitative methodology for calculating total life-cycle costs for a given bridge largely as a function of the reinforcing bar type used in its construction. Several other notable factors were also integrated in the holistic approach, including the location of the bridge (e.g., whether it is in a highly congested urban or rural area, spanning over a critical waterway, etc.), the size of the bridge, volume of traffic, and its initial cost of construction.

In its entirety, the framework was designed for use by practicing bridge engineers working on Illinois Department of Transportation (IDOT) projects. The framework will allow IDOT to recommend optimal design solutions for applications where enhanced corrosion resistance is desired and to assist IDOT officials with critical decision-making with respect to facilitating improved life-cycle performance and reduced life-cycle costs.

REPORT OUTLINE

This report contains four chapters. The narrative starts with an introduction to corrosion of reinforcement in concrete bridge decks and the major focus affecting the extent to which it can occur. This introduction is followed by an overview of corrosion-resistant reinforcement types and their respective characteristics and performance attributes. Lastly, the development of and guidelines for using a framework to select the optimal reinforcement type for a given bridge deck as a function of pertinent design parameters is presented. More specific details pertaining to each of the four chapters are highlighted below.

Chapter 1 covers the motivation for this research, a brief overview of the behavior of and physical mechanisms for corrosion of reinforcement embedded in concrete, and a thorough review of available corrosion-resistant reinforcement types and their corresponding properties.

Chapter 2 presents the outcomes of a comprehensive literature review focused on current understanding of reinforcement corrosion performance in concrete bridge decks. More specifically, this chapter will examine several factors affecting corrosion of reinforcement, including effects of coatings and bar defects, influence of chemical composition of the reinforcement, previous experimental testing on corrosion performance, life-cycle analyses of bridge decks constructed with various types of reinforcement, and currently adopted guidelines, recommendations, or standards by other governmental agencies, state departments of transportation, or industry organizations.

Chapter 3 details the development of an optimal corrosion-resistant reinforcement selection methodology based upon the results of the literature review presented in Chapter 2. The methodology allows the reader to estimate life-cycle costs for a given bridge deck considering several corrosion-resistant reinforcement types and other applicable factors, such as structural performance characterizations, location of the bridge, and cost of initial construction, among others.

Chapter 4 summarizes the findings and conclusions of this project.

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CHAPTER 1: INTRODUCTION

MOTIVATION OF THE RESEARCH

Corrosion of steel reinforcement in concrete bridges has been a longstanding challenge to satisfactorily meeting durability requirements over the intended service life of the structure. This observation is especially applicable for reinforced concrete bridge decks, which are often exposed to deicing salts or other deterioration agents, particularly in regions that experience moderate to severe winter weather conditions. Situated in the heart of the Great Lakes region of the United States, the state of Illinois is regularly exposed to winter snow, ice, and freezing rainstorms which, on average historically, has necessitated the state to purchase approximately 2,500 kg (5,511 lbm) of road salt per square kilometer (0.39 square mile) annually (Panno et al., 2005). Moreover, corrosion of reinforcement in concrete bridges was likely a leading source of an estimated $90 billion dollars in repair costs across the United States as recently as 30 years ago (Federal Highway Administration, 1991) and that cost estimate has likely increased significantly since. Reinforcement corrosion may result in several detrimental consequences (or combinations thereof) for a reinforced concrete structure. These consequences typically include but are not necessarily limited to loss of cross-section of the bars, which can result in a significant loss of the load-resisting capacity of a structural member; spalling of concrete, which may endanger people or objects situated or passing below the member; or significant cracking, which may be inherently troublesome for the structural performance of the member in addition to creating additional pathways for harmful deterioration agents to enter the internal cross-section(s) of the member—possibly accelerating the corrosion process moving forward.

Motivated by deterioration/damage concerns, several remediation strategies have been proposed and subsequently implemented to mitigate potential widespread damage to the structure caused by steel reinforcement corrosion. Because the initial driver of the aforementioned deterioration mechanism is commonly the permeability of the concrete, low-permeability concretes have been proposed as an effective solution (Mehta & Monteiro, 2006; Ozyildirim, 1998). A widely implemented alternative is the use of epoxy-coated reinforcing bars, especially within Illinois. The protective epoxy coating prevents migratory chlorides from undergoing the aforementioned chemical reaction with the steel bars, interrupting the corrosion mechanism and its subsequent consequences. Steels with enhanced corrosion resistance have also become increasingly popular, including stainless steel, galvanized bars, and A1035 steel—each of which will be discussed in more detail in the coming subsections. These materials inherently exhibit chemical properties that, with a similar end effect as the epoxy coating, interrupts the detrimental oxidation process. ASTM A1035 (2020) reinforcement contains an average chromium content of 9% (Aldabagh & Alam, 2020), which exhibits superior corrosion resistance to that of conventional steel bars (i.e., ASTM A615 [2020] or ASTM A706 [2016]), and is often a more economical alternative to stainless-steel bars. Furthermore, high-strength reinforcing steels, such as ASTM A1035 reinforcement, exhibit enhanced tensile strength properties, which may facilitate reduced congestion in structures such as concrete bridge decks—potentially reducing overall project costs, most notably labor costs.

The preceding discussion encourages the development of an optimal approach for selecting the type of reinforcement (or other corrosion mitigation strategies) best suited for facilitating superior corrosion resistance as a function of a concrete bridge's location, the intended life span of the structure, cost restrictions, or other applicable limitations or pertinent design considerations. Therefore, the main objective of the project will be to develop an optimal approach for addressing reinforcement corrosion for concrete bridges, especially concrete bridge decks, in Illinois. A comprehensive literature review on the subject will facilitate the compilation of existing research related to the holistic performance of various steel reinforcement options from the main perspective of corrosion performance. Preliminary and approximate estimations for the price per reinforcing bar option will be presented as a function of pertinent design criteria, including location of the structure, the intended life span of the bridge, and importance of the bridge (e.g., passing over a critical waterway, etc.). These recommendations and guidelines will assist bridge engineers and officials with determining the ideal reinforcement solution for a particular bridge and, ultimately, facilitate the implementation of an Illinois Department of Transportation (IDOT) policy on the subject by IDOT officials.

OVERVIEW OF REINFORCEMENT CORROSION IN CONCRETE BRIDGE DECKS

The primary damage mechanism resulting from corrosion of reinforcing steel is characterized by three main drivers: production of an expansion product, concrete cracking, and, eventually, spalling of the concrete cover serving as protection for the bars (Mehta & Monteiro, 2006). The migration of chlorides commonly found in road salts through pores or microscopic cracks in the concrete eventually facilitates a chemical reaction on the surface of the reinforcing bars as metallic compounds are transformed into rust via oxidation. One of the main products of this reaction is significant volumetric expansion of the deteriorated material that in turn applies pressure to the surrounding concrete region, most notably the relatively thin region of concrete cover between the outer surface of the bars and the exterior surface of the concrete member. The expansion pressure can initially cause localized concrete cracking (due to significant internal tensile stresses), which can ultimately result in concrete spalling (i.e., the removal of concrete fragments from the larger concrete mass [American Concrete Institute, 2013]). Concrete spalling can result in the complete or partial loss of the concrete cover for the reinforcement. This mechanism can further complicate and worsen the deterioration process, as it may directly expose the residual bars to further deterioration agents and reduce the cross-sectional area of the structure, both of which may facilitate reduction in the load-carrying capacity of the member in some instances.

OVERVIEW OF CORROSION-RESISTANT REINFORCEMENT OPTIONS

Several corrosion-resistant reinforcement types are available that serve, to varying degrees, as alternatives to conventional black bars for satisfying pertinent structural design requirements. Enhanced corrosion resistance is typically provided by either placing a coating on black bars or using alternative steels that mitigate or even prevent corrosion products from forming when exposed to chloride ions when embedded in concrete. The enhancements are commonly associated with trade-offs in cost. Therefore, a major focus of this project is to provide a framework to estimate life-cycle costs for each corrosion-resistant bar option and subsequently aid in determining whether a higher initial material cost is justifiable for a given application and its inherent design objectives. Therefore, prior to presenting the framework (see Chapter 3 of this report), it is useful to first examine each reinforcement option and its corresponding properties, as highlighted in the following subsections. Chapter 2 discusses more details pertaining to specific outcomes of previous research studies that evaluated corrosion performance and corresponding life-cycle implications.

Epoxy-coated Bars

Epoxy-coated bars are currently the most widely used corrosion-resistant reinforcement type in concrete bridge decks (Darwin et al., 2020), including in Illinois. Generally, the coating is designed to prevent chlorides that have penetrated through the concrete cover from reacting with the otherwise unprotected metallic surface of the bar and consequently mitigating the development of the expansive corrosion product. ASTM A775 (2019) stipulates that bars receiving epoxy coating must meet A615 (2020), A706 (2016), A996 (2016), or A1035 (2020) and shall be free of any contaminants. Because of the importance of the coating to corrosion protection, ASTM A775 (2019) also implements thorough requirements for surface preparation of the bar prior to coating, the coating application process itself, and post-coating measurements of coating thickness, continuity, and flexibility to mitigate potential damage to the coating prior to concrete placement—which in turn may decrease its effectiveness in preventing the development of corrosion products. Furthermore, ASTM A775 (2019) defines a permissible amount of damaged coating and requirements for repair in the event damaged coating is identified. The maximum permissible repaired damaged coating must not exceed 1% of the surface area in each 0.305 m (1 ft) length of bar, and coating repairs must have a minimum thickness of 175 µm (0.007 in.).

Galvanized Bars

Like with epoxy-coated bars, galvanized reinforcement is also produced by placing a protective coating on plain (i.e., previously uncoated) steel reinforcing bars. Two main ASTM specifications govern galvanized bars: A767 (2019) for protective zinc coating applied by immersing the black bars in a molten zinc bath and A1094 (2020) for applying the zinc coating using a continuous hot-dip process. The coating on bars that meet A767 specifications inherently contains two distinct layers: an external layer consisting of pure zinc and an internal layer (i.e., closer to and adjacent to the surface of the underlying bar) comprised of mainly iron-zinc alloys—as a result of the interaction with the base steel. The alloys present in the external layer exhibit more brittle performance and thus may be prone to cracking, especially during the bending process (Darwin et al., 2020). In contrast, the coating on bars that are continuously galvanized in accordance with A1094 generally exhibit more flexible behavior (Darwin et al., 2020) and thus are likely less susceptible to cracking or other coating damage prior to embedment in concrete. Both ASTM A767 and A1094 allow the underlying bars to be galvanized to meet A615 (2020), A706 (2016), or A996 (2016). A1094 also allows for the use of A1035 (2020) underlying reinforcement.

Stainless-steel Bars

Unlike the epoxy-coated and galvanized bar options, the enhanced corrosion-resistant performance of stainless-steel reinforcement is attained through the chemical composition of the bar. ASTM A955 (2020) stipulates that the chemical composition of the steel shall contain somewhere between 16% to 24.5% chromium, depending upon the specific stainless-steel designation applicable for reinforcing bars. Similar to conventional A615 (2020) or A706 (2016) black bars, Grade 60 (i.e., a minimum nominal yield strength of 414 MPa [60 ksi]) stainless-steel bars are most common, while ASTM A955 (2020) also provides minimum tensile requirements for Grade 75 and Grade 80 variants as well. Stainless-steel bars are particularly effective in applications where exceptionally high corrosion potential exists, extended service life of the structure is desired, or for reinforced concrete structures that are more difficult or costly to repair or perform maintenance (Concrete Reinforcing Steel Institute, n.d.). Often the relatively higher up-front cost of stainless-steel reinforcement may prohibit its use in certain applications but may be more justifiable when evaluated as part of a comprehensive life-cycle assessment for a given reinforced concrete structure.

ASTM A1035 Bars

Similar to stainless-steel reinforcement, bars compliant with the ASTM A1035 (2020) specification achieve enhanced corrosion protection from the inherent chemical composition of the steel rather than a protective coating. Three alloy types (CL, CM, and CS) are defined by the A1035 specification as a function of chemical composition—particularly carbon and chromium composition. Maximum carbon contents of 0.3%, 0.2%, and 0.15% and chromium compositions of 2.0%–3.9%, 4.0%–7.9%, and 8.0%–10.9% are specified for Type 1035 CL, 1035 CM, and 1035 CS, respectively. Therefore, A1035 bars exhibit significantly less chromium content than stainless-steel bars; however, this attribute may be useful as an intermediate corrosion-resistant alternative with lower initial material costs relative to stainless-steel bars. Chapter 2 details additional information pertaining to previous research efforts focusing on A1035 bars.

In addition to exhibiting enhanced corrosion performance, A1035 reinforcement also offers higher tensile strength (relative to standard black bars as well as several other corrosion-resistant bar options), which may facilitate reduced congestion when embedded in concrete and more economical structural designs due to the higher nominal yield strength values. ASTM A1035 (2020) defines both Grade 100 and 120 (i.e., a minimum nominal yield strength of 689 MPa [100 ksi] and 827 MPa [120 ksi], respectively) for the three alloy classifications (i.e., 1035 CL, 1035 CM, and 1035 CS). As a trade-off for higher tensile strength, these bars typically exhibit lower ultimate tensile strains, which may limit the corresponding ductility of reinforced concrete structures fabricated using A1035 reinforcement. Furthermore, the stress-strain response of A1035 bars lacks a clearly defined yield point and plateau and thus the yield strength can be determined using one of several different methods—the 0.2% offset approach being the most common (Aldabagh & Alam, 2020). The 0.2% offset approach is where the yield strength is calculated as the stress at the intersection of a line parallel to but offset 0.002 strain along the horizontal axis from the initial slope of the stress strain curve—and the actual curve.

Note on Other Reinforcement Options

The aforementioned corrosion-protection options for reinforcing steel were the major focus of this study; however, other reinforcement options may also provide sufficient protection, depending upon the intended use and/or specific application. For example, fiber-reinforced polymer bars (in accordance with ASTM D7957 [2017]) have gained popularity for use in select applications requiring high corrosion resistance. However, the inherent lack of ductility often limits its use in larger scale infrastructure projects, where significant ductility is necessitated or deemed a prerequisite for achieving tension-driven failure modes with more economical strength reduction factors. The optimal framework detailed later in this report is designed to be universal and thus can incorporate pertinent performance aspects of bar options not specifically examined herein if sufficient information is available from research, industry experience, applicable codes, or design guidelines.

CHAPTER 2: CURRENT UNDERSTANDING OF REINFORCEMENT CORROSION PERFORMANCE IN BRIDGE DECKS

This chapter presents the results of a comprehensive literature review that focused on assessing the performance and life-cycle cost implications of several corrosion-resistant reinforcement types, specifically for concrete bridge decks.

EPOXY-COATED BARS

Epoxy-coated reinforcement in concrete bridge decks may be used in either both mats of reinforcement or in the top mat only. Bridges that are subjected to lower magnitudes of applied loading and, consequently in many cases, less weathering (i.e., possible damage to the bridge deck—especially in the absence of a wearing surface—resulting from high repeated traffic levels) may be sufficient with epoxy-coated bars in the top mat only (i.e., closer to the surface and thus source of chlorides) for the sake of economizing the structure. Epoxy-coated reinforcement use in the top mat only restricts the anode formation and thus the ability for corrosion to occur. This mechanism is directly due to the reinforcement coating—corrosion still freely develops on the uncoated bottom reinforcement because there is no restriction to the cathode site. In the case of a mixed reinforcement system, the presence of the epoxy coating on the top reinforcement facilitates a reduction to the corrosion site by 60% to 93% (McDonald, 2010). Because the bottom mat still experiences typical corrosion, spalling and cracking may occur on the underside of the deck with minimal topside damage. In general, the use of epoxy coatings on steel reinforcement reduces the rate of cracking because the expansion from corroded reinforcement is slowed (McKenzie, 1993). Fundamentally, the nonconductive coating reduces the electrical conductivity to further reduce the potential for the corrosion cell. The corrosion that develops is heavily concentrated at the construction joints and whatever cracks in the coating may form.

Currently, most state departments of transportation opt for epoxy-coated reinforcement in both mats, if used (McDonald, 2010). As mentioned previously, the epoxy coating of both mats restricts the area of both the anode and cathode sites and thus corrosion overall, as its rate of development is dramatically reduced. In degradation testing of over 500 bridges with mixed or full epoxy-coated reinforcement, Treat and Dymond (2021) observed that bridges with epoxy coating in both reinforcement mats experienced less deterioration than bridges with epoxy-coated reinforcement in the top mat only. In every test executed, the control specimen with epoxy-coated reinforcement in both mats performed advantageously over the samples with mixed reinforcement. Another study observed that Minnesota-built bridges constructed in 1973 and 1978 with epoxy-coated reinforcement performed well in testing conducted by the Minnesota Department of Transportation in 2008 (McDonald, 2010). The New York Department of Transportation, in a study of 17,000 bridges, found that the epoxy-coated reinforcement in general performed better than uncoated reinforcement, especially in the later years of the reinforcement’s functional service life (McDonald, 2010).

A study conducted by Kessler and Lipscomb in 1994 suggested that epoxy coatings on reinforcement provide particularly effective corrosion resistance in the presence of excess chloride concentrations; 85% of their epoxy-coated reinforcement specimens did not exhibit any signs of corrosion at very high chloride levels. When exposed to heavily chlorinated environments, epoxy-coated reinforcement provides for bolstered resistance to corrosion and subsequent deterioration (Ahmed et al., 2017). Chloride concentration in concrete is depth-dependent; if cover depth of the reinforcement is increased, the onset of corrosion may be delayed. A comprehensive service life assessment of a reinforced concrete structure therefore needs to account for the rate of chloride penetration and how it consequently affects the rate of corrosion for the reinforcement.

The benefits of epoxy coatings on concrete reinforcement may be maximized if the reinforcing steel is sufficiently waterproofed in addition to the epoxy coating. Without sufficient waterproofing, one previous research study demonstrated that epoxy-coated bars experienced the onset of corrosion at 44.0 years instead of 46.9 years when waterproofed (Konečný & Lehner, 2016). Moreover, if a bridge deck has waterproofing in place, the structure can exhibit a service life of approximately six times longer than that of a bridge without reinforcement waterproofing (Konečný & Lehner, 2016). A similar observation is true for bridge decks exposed to more severe winter weather conditions, as epoxy-coated reinforcement often fares far better than black bars and will postpone the onset of corrosion significantly (Ahmed et al., 2017).

GALVANIZED BARS

Galvanized coatings, bonded metallurgically to the reinforcing steel, are generally better suited to resist deterioration from abrasion and handling relative to their plated or painted counterparts (Yeomans, 1994). The transportation and handling of galvanized steel is largely similar to that of black bars; the coating is fully integrated into the reinforcement and does not behave like a separate entity. However, galvanized steel reinforcement may not be as readily available and has a higher initial cost than uncoated or epoxy-coated reinforcement, which may limit the ease of access to the material (McDonald, 2010) or full implementation of its use in concrete bridge decks.

After adequately bonding to the underlying steel bar, the zinc-based coating first delays the onset of the corrosion process by providing a layer of protection between chlorides and the underlying bar. Once corrosion does initiate, the galvanized coating generally takes longer to degrade than other coatings (such as epoxy) and continues to partially insulate the underlying reinforcement from corrosion in the process as the zinc acts as an anode to protect the steel from corrosion (Ahmed et al., 2017). Although galvanizing the reinforcement is an effective means of resisting corrosion due to chlorides, the coating corrodes at an expedited rate in wet concrete. Furthermore, regarding the properties of the concrete mix design, an additional study highlighted that the use of Type V cement (i.e., designed for high sulfate resistance) slightly bolstered corrosion resistance over Type I cement for both galvanized and ASTM A108 steel specimens (Baltazar-Zamora et al., 2019).

The rate of corrosion on galvanized steel is generally very slow, even in the later years of a structure's functional service life. Sagüés et al. (2000) found the rate of corrosion in galvanized bars to be 15 μm/year (0.0006 in./year) over the first two years and 4 μm/year (0.0002 in./year) thereafter. For comparison, they found the corrosion rate for uncoated reinforcement to be closer to 12 μm/yr (0.0005 in./year) overall. Weathering galvanized steel has been found to more effectively lower corrosion rates upon embedment in concrete due to the additional protection from the zinc oxides and zinc carbonates that form during the weathering process (Tan & Hansson, 2008). In testing conducted by Andrade et al. (2001), galvanized steel did not show any signs of corrosion after immersion in natural seawater for 10 years, even where the reinforcement protruded from the concrete.

STAINLESS-STEEL BARS

The most common grades of stainless-steel reinforcement currently in use are 316LN and 2205, both of which provide excellent corrosion resistance (Ahmed et al., 2017). If either of these reinforcement types are utilized in a concrete structure, it is typical that the structure's service life can easily exceed 100 years; however initial costs of stainless-steel reinforcement can surpass five times that of conventional black bars, which may limit its use in certain applications. When subjected to a variety of simulated deterioration states, stainless-steel bars maintain corrosion levels well within the allowable threshold far beyond the 75-year service life of lower performance reinforcement (Guest et al., 2020). A review from Hebdon and Provines (2020) also suggests that stainless-steel reinforcement may have up to 10 times more corrosion resistance than uncoated weathering steel if the reinforcement is oriented in a near vertical direction. When subjected to corrosion-fatigue testing, stainless-steel reinforcement did not exhibit any corrosive wear, despite the development of fatigue cracks (DeJong et al., 2009).

ASTM A1035 BARS

Fundamentally, the corrosion resistance of ASTM A1035 (2020) steel bars is higher than that of conventional Grade 60 black bars (Thomas et al., 2013). As mentioned previously, A1035 steel also exhibits a relatively higher nominal yield strength (at least 689MPa [100 ksi]), which in turn facilitates reduced materials weights, as a smaller A1035 bar size can be used for comparable strength of a larger, heavier Grade 60 bar, which facilitates reduced labor and transportation costs (Harries et al., 2012). For example, a bridge under construction studied by Salomon and Moen (2017) highlighted a 23% weight and cost reduction when using No. 4 ASTM A1035 bars in place of No. 5 Grade 60 bars. In addition to the strength increase, ASTM A1035 steel has been shown to have anywhere from two to ten times as much corrosion resistance as conventional black bars, as well as increased resistance compared to ASTM A615 Grade 60 steel.

The publication of ASTM A1035 spurred the widespread adoption of ASTM A1035 steel as an acceptable means of reinforcement in concrete. Concrete samples made with ASTM A1035 steel were found to be compliant with the current AASHTO requirements and had a corresponding material savings of 36% (Salomon et al., 2014) in one study. To compensate for the reduced bar size, bridge deck cross-sections should be checked to ensure sufficient clear cover is maintained and overall flexural stiffness is similar to that of an equivalent section with Grade 60 reinforcement.

BAR AND COATING DEFECTS

Generally speaking, defects in the reinforcement network (i.e., in the coating where applicable or the underlying steel itself) have a significant impact on the service life and rate of corrosion development. Kessler and Lipscomb (1994) found epoxy-coated reinforcement to be effective against chloride-based corrosion, contingent on the efficacy and proper initial coating of the reinforcement. Should the bar have a defect such as low coating thickness or roughened patches, chloride ingress can reach the reinforcing steel beneath the coating and expedite the corrosion process. Epoxy coatings tend to be more fragile than other corrosion-resistant options; they are more easily damaged and, if chipped or deformed, could compromise the corrosion-resistance abilities of the coating (Ahmed et al., 2017). Should corrosion occur at sites of damage to the epoxy coating, there is a chance the corrosion will spread beneath the coating; however, the rate of corrosion on the steel beneath the coating is still slower than on the surface of uncoated reinforcement (McKenzie, 1993). When epoxy coatings dissolve, epoxy-coated reinforcement may experience corrosion at faster rates than galvanized steel reinforcement because the pH of the concrete around the reinforcement is higher (Tan & Hansson, 2008).

PHYSICAL PROPERTIES AND BENDING REQUIREMENTS

Corrosion has a more adverse effect on flexural strength deterioration in reinforced concrete members fabricated with smooth bars than deformed bars, and larger diameters are generally more desirable for safety and service life considerations, as larger reinforcement sizes take longer to corrode (Wang et al., 2015). On occasion, special requirements may stipulate that coated reinforcement types (i.e., epoxy or galvanized) should be coated after bending as a precautionary measure if mitigating damage to the coating is of particular concern. Lastly, special requirements may also exist for storing coated reinforcement bars prior to embedment in concrete. For example, the coating on epoxy bars will likely benefit from reduced exposure to UV rays or drastic/cyclic temperature cycles.

CHEMICAL COMPOSITION

Chromium Content

In galvanized steel reinforcement, the introduction of chromate films helps to increase corrosion resistance at high pH values (Tan & Hansson, 2008). In general, chromium-rich coatings on reinforcement help increase impedance, lower passivity current density, and slow the speed of corrosion (Murkute et al., 2021). In ASTM A1035 steel, the chromium content (which varies from 2% to 10.9% and is 9%, on average) allows for superior corrosion resistance when compared to conventional steel (Aldabagh & Alam, 2020). Lastly, the relatively high chromium content of stainless-steel bars facilitates its superior corrosion resistance and, consequently, reduced maintenance and longevity over a design life cycle of 100 years.

EXPERIMENTAL TESTING

Accelerated Corrosion Lab Tests

Testing conducted by Erdoğdu et al. (2001), which simulated seawater and harsh chloride environments, further supports the use of epoxy-coated reinforcement in both conditions. When compared to uncoated bars that exhibited extensive corrosive deterioration, after two years, the epoxy-coated reinforcement had minimal corrosion in the seawater environment and only some degree of corrosion in a 3% sodium chloride solution at sites of intentional damage to the coating. In both solutions, no corrosive deterioration occurred on the intact epoxy-coated reinforcement. In extreme chloride environments, galvanized reinforcement has only a minorly extended service life (Ahmed et al., 2017). However, the use of epoxy-coated reinforcement corresponded to minimal cracking and only one spall in a 30-year study conducted by Pincheira et al. (2015). Apart from the expected corrosion at joints and cracks, the epoxy-coated reinforcement showed very little, if any, significant corrosion in areas of concrete exposed to low chloride concentrations.

BOND BEHAVIOR

When subjected to pullout testing, concrete samples with post-installed (anchored using a chemical adhesive) epoxy-coated reinforcement can experience bond failure at the interface of the concrete and chemical adhesive, while concrete samples with uncoated reinforcing bars experienced bond failure at the interface of the chemical adhesive and the reinforcing steel (Mills & Dymond, 2021). The adhesives used for the purposes of this pullout study generally remained adhered to the epoxy-coated reinforcement after failure while it often de-bonded from the uncoated steel during and after failure. This is likely owing to the greater compatibility between organic resins with each other than with steel. However, note that other research studies also indicate that there is no direct relationship between adhesion loss and concrete distress (McDonald, 2009). Additionally, if no additional adhesive is provided between the concrete and the reinforcement, Paswan et al. (2020) found that uncoated bars had greater frictional resistance than coated bars, with more exaggerated differences in bond strength at smaller reinforcement sizes. The metallurgical bond between the steel reinforcement and the zinc-based coating in galvanized steel ensures sufficient adhesion between the two materials, but care should be taken to ensure the coating is properly applied to ensure adequate corrosion protection (Ahmed et al., 2017).

TEMPERATURE IMPACTS

Corrosion impacts are temperature dependent, with samples experiencing expedited rates of corrosion at increasing temperatures (Guest et al., 2020). The chloride threshold for reinforcing steel, or the reinforcing steel's capacity for chloride before the onset of corrosion, is an inverse function of temperature; increasing ambient temperature will reduce the chloride threshold of the reinforced concrete and bolster corrosive deterioration (Shirkhani et al., 2020).

LIFE-CYCLE ANALYSES

Cost Comparison

While epoxy-coated reinforcement may have a higher initial cost by approximately 3.7% on average, the repair cost is reduced and the comparable service life cost was decreased by at least 46% when compared to uncoated black bars in one study (Ahmed et al., 2017). Despite the increased initial cost associated with more corrosion-resistant reinforcement types (relative to black bars—see Chapter 3 for more details), the extension of service life and repair reduction may justify the increased up-front cost. An experimental cost comparison, using pricing provided by material suppliers, produced by O'Reilly et al. (2011) outlines installation (including material) costs as $1.92/kg ($0.87/lbm.) for uncoated black steel reinforcing bars, $2.14/kg ($0.97/lbm.) for epoxy-coated steel reinforcing bars, and $6.33/kg ($2.86/lbm.) for stainless-steel reinforcing bars. Chapter 3 provides more specific information regarding life-cycle costs.

STUDIES AND RECOMMENDATIONS BY OTHER DOTS AND AGENCIES

After signs of corrosion damage and epoxy-coating delamination, the Virginia Department of Transportation discontinued the use of epoxy-coated reinforcement in its highway bridges as of 2010. The Ontario Ministry of Transportation changed its specifications for highway bridge decks and structural walls following additional findings of corrosion damage in epoxy-coated reinforcement (Ahmed et al., 2017). There are currently 11 state departments of transportation in the United States that promote the use of ASTM A1035 steel reinforcement: Washington, Oregon, California, Montana, Idaho, Texas, Mississippi, Florida, Virginia, Maryland, and Maine (Aldabagh & Alam, 2020). Further research is likely needed to facilitate more widespread use of A1035 bars in other states, depending upon specific applications, exposure categories, and structural design considerations for specific cross-sections (especially due to the higher nominal yield strengths of A1035 reinforcement).

ASTM SPECIFICATIONS FOR EVALUATION OF CORROSION RESISTANCE

Of the alternative corrosion-resistant bar options examined in this report, only ASTM A955 (2020) (for stainless-steel reinforcement) provides specific details for evaluating corrosion resistance of the bars. The A995 specification permits the use of two methods to serve this purpose: a rapid microcell test and a cracked beam test. The former methodology involves subjecting a minimum of five stainless-steel bar specimens to a simulated concrete pour solution containing 15% sodium chloride for a duration of 15 weeks. The corresponding acceptance criteria is that the average corrosion rate (for the entire series of specimens) at any time during the test shall not exceed 0.25 µm/year (9.84×10^{-6} in./year) with no single specimen exceeding 0.50 µm/year (1.97×10^{-5} in./year). The latter methodology involves subjecting a small-scale deliberately cracked reinforced concrete beam specimen to repeated cycles of exposure to a chloride solution for a total duration of either 72 or 96 weeks (depending upon the application and the requester of the test). Corresponding acceptance criteria for the cracked beam methodology includes a maximum average corrosion rate (i.e., across the entire test series) of 0.20 µm/year (7.87×10^{-6} in./year) with no single specimen exceeding 0.50 µm/year (1.97×10^{-5} in./year).

CHAPTER 3: FRAMEWORK FOR DETERMINING OPTIMAL REINFORCEMENT TYPE

A comprehensive framework for determining the optimal corrosion-resistant reinforcement option for a given concrete bridge deck (specifically detailed for bridge decks in Illinois) is presented in this chapter. The following framework is largely based upon the results of the literature review presented in Chapter 2 of this report.

RELATIVE COSTS

The greatest determining factor for most departments of transportation (DOTs), including IDOT, is the cost of implementing each reinforcement bar type. For most DOTs, until the early 2000s, the only cost evaluated was the initial cost of construction and perhaps some simple forecasting of potential rates of corrosion through several sample bridge decks that would serve as a basis of research on the performance of epoxy-coated bars relative to standard black bars. Using such a simplified approach to aid in reinforcement type selection, epoxy-coated bars were determined to be more cost beneficial than the standard black bar for most concrete bridge decks applications, especially in Illinois where epoxy-coated bars became, and still remain, the overwhelming choice. At the time, other corrosion-resistant reinforcement alternatives (e.g., stainless steel, galvanized, etc.) were largely deemed cost-prohibitive when considering largely only initial construction costs. However, since the early 2000s, more DOTs have become much more mindful of life-cycle cost implications for concrete bridge decks, specifically how each corrosion-resistant bar option contributes to extending the design life cycle of a given bridge deck. Initially, life-cycle analyses were implemented due to updated research results (beyond older studies performed in the 1980s and 1990s) now revealing that epoxy-coated bars may not be as economical as previously believed when now considering the full expected service life of the bridge deck. A prime example is a Florida bridge study conducted from 1986 to 2000 that observed several bridges within the Florida Keys with epoxy-coated bars utilized in the construction of concrete bridge decks (McDonald, 2009). It was believed that the corrosion-resistant (epoxy-coated) reinforcement would ultimately save the State of Florida future expenses in repair costs; however, within 20 years of construction, 5 of the examined bridges exhibited high levels of corrosion with multiple locations of delamination and spalling of concrete. In the aftermath of that study, life-cycle cost analysis has become more common in many cases as an important tool to determine the type of reinforcing bar used to enhance corrosion performance in concrete bridge decks.

GENERAL FRAMEWORK PROCEDURE

For the purposes of this report, the life-cycle cost analyses (LCCA) that follow were utilized to facilitate normalized comparisons of the relative costs associated with each reinforcing bar type (i.e., facilitate streamlined assessments of alternative reinforcement options relative to each other and conventional black bars). Within this following procedure, the term “normalized” relates to the base standard in relative pricing of each reinforcement in comparison to black bars. For the initial and material repair costs within the tables below, black bar is standardized as a base of \$0.84/ sq. m. (\$1.00/sq. yd.). The prominent factors included in the LCCA are initial material/construction cost, the duration (and consequently the total number) of repair periods over a 100-year design service life, and unit repair cost (i.e., cost per each repair completed). The breakdown of each of these costs for each bar type is shown in Table 1. It is important to note that while 100 years was used as the theoretical design service life, concrete bridge decks constructed with several of the examined bar types, including standard black and epoxy-coated bars, may not reach this 100-year life span as a result of the high levels of corrosion on the bridge deck that will often warrant a complete bridge reconstruction rather than continuous efforts in repair costs and/or heavy maintenance.

For each respective bar type, Table 1 highlights how often a bridge deck comprising that particular corrosion-protection system would warrant a repair of the deck (adopted from Darwin et al. [2020]). Please note that these repair costs include concrete replacement as well. This particular basis of repair time does not take into consideration the extent of the repair, nor does it account for other external factors that may warrant a sudden repair or accelerate the timeline of repairs, such as severe vehicular accidents resulting in damage to the bridge. Rather, it establishes a cost comparison for the reinforcing material required to repair a given bridge reinforcing corrosion-resistance system. Table 1 also breaks down the cost of materials associated with annual repair for five years. Uncoated black bars require repairs every year, whereas stainless steel will not require repairs during the service life of a bridge deck. Thus, the unit material repair cost for stainless steel is $0.00 and the larger initial cost may be justified for stainless steel reinforcement, given the constraints of the bridge design. The initial material construction costs, and ultimately the unit material repair costs as well, were normalized relative to the cost of standard black bars (set at $1.00 per 0.84 square meters [1 square yard] of bridge deck, **pursuant to pricing adopted from Darwin et al. [2020])**, to establish a baseline comparison of the uncoated and untreated black bars. The costs of the remaining reinforcement types were then set by scaling the base cost of the black bar.

Table 1. Breakdown of Calculation for Material Repair Costs over a 100-year Service Life

Bar Type	**Initial Cost ($/0.84m^2) [$/yd^2]**	**Repair Event Number** 1	2	3	4	5	**Unit Material Repair Cost ($/0.84m^2) [$/yd^2]**
		Time to Repair (Years)					
Black Bar	$1.00	24.33	48.63	72.98	85.20	96.10	$1.00
Epoxy Coated	$1.10	49.45	85.20	x	x	x	$1.06
Galvanized A767	$1.15	56.00	99.70	x	x	x	$1.06
Galvanized A1094	$1.11	56.00	99.70	x	x	x	$1.04
Stainless Steel	$3.97	x	x	x	x	x	$0.00
A1035	$1.27	49.75	89.40	x	x	x	$1.10

The estimated theoretical material life-cycle cost ($/0.84 m^2 [1 yd^2])—see Table 2—was determined using a present value (PV) cost methodology adopted from Darwin et al. (2020) and calculated using the equation shown in Figure 1, where *f* is the unit material repair cost, *n* is the time to repair for each repair activity, and *i* is taken as 0.02. This methodology includes higher fidelity mathematical strategies and inflation in an attempt to more accurately present holistic life-cycle cost estimates—as opposed to a more simplified approach, which essentially sums all expected total costs over the design life span (as will be highlighted later in this report).

$$PV = Initial\ Cost + \sum_{0}^{5} [f(1+i)^{-n}]$$

Figure 1. Equation. Present value cost-estimate equation.

$$SLCC = Initial\ Cost + fN_R$$

Figure 2. Equation. Simplified equation for life-cycle cost estimate.

A simplified alternative to estimating life-cycle costs is also depicted in Table 2. Rather than use the more sophisticated present value methodology, the simplified approach follows the equation shown in Figure 2, where *SLCC* stands for *Simplified Life-Cycle Cost*, f is the unit material repair cost, and N_R is the number of repairs needed within the 100-year design service life (see Table 1). The simplified method assumes a uniform repair cost across each repair milestone and is designed to be a more straightforward method of estimating life-cycle costs perhaps suitable for preliminary design. The equation shown in Figure 2 can also be further manipulated to include an inflation adjustment term, if desired.

Table 2. Calculation of Estimated Theoretical and Simplified Life-Cycle Costs

Bar Type	Initial Cost ($/0.84m^2) [$/yd^2]	Material Repair Cost ($/0.84m^2) [$/yd^2]	Estimated Theoretical Material Life-Cycle Cost ($/0.84m^2) [$/yd^2]	Simplified Life-Cycle Cost		
				50-Year Life Cycle	75-Year Life Cycle	100-Year Life Cycle
Black Bar	$1.00	$1.00	$2.57	$3.00	$4.00	$6.00
Epoxy Coated	$1.10	$1.06	$1.69	$2.15	$2.15	$3.21
Galvanized A767	$1.15	$1.06	$1.65	$1.15	$2.21	$3.26
Galvanized A1094	$1.11	$1.04	$1.59	$1.11	$2.14	$3.18
Stainless Steel	$3.97	$0.00	$3.97	$3.97	$3.97	$3.97
A1035	$1.27	$1.10	$1.86	$2.36	$2.36	$3.46

Note: Columns two and three in Table 2 are adopted from the results in Table 1. The results in column four are calculated using the equation in Figure 1. The results in columns five through eight are calculated using the equation in Figure 2.

In Table 2, the initial cost ($/yd^2) and material repair cost ($/yd^2) are obtained from the values in Table 2. The estimated material life-cycle cost ($/yd^2) is calculated in Figure 1. The data within the simplified life-cycle cost section of Table 2 (50-year life cycle, 75-year life cycle, and 100-year life cycle) are calculated from Figure 2. These values calculated are represented in Figure 3.

A simplified alternative to estimating life-cycle costs is also depicted in Table 2. Rather than use the more sophisticated present value methodology, the simplified approach follows the equation shown in Figure 2, where *SLCC* stands for *Simplified Life-Cycle Cost*, *f* is the unit material repair cost, and N_R is the number of repairs needed within the 100-year design service life (see Table 1). The simplified method assumes a uniform repair cost across each repair milestone and is designed to be a more straightforward method of estimating life-cycle costs perhaps suitable for preliminary design. The equation shown in Figure 2 can also be further manipulated to include an inflation adjustment term, if desired.

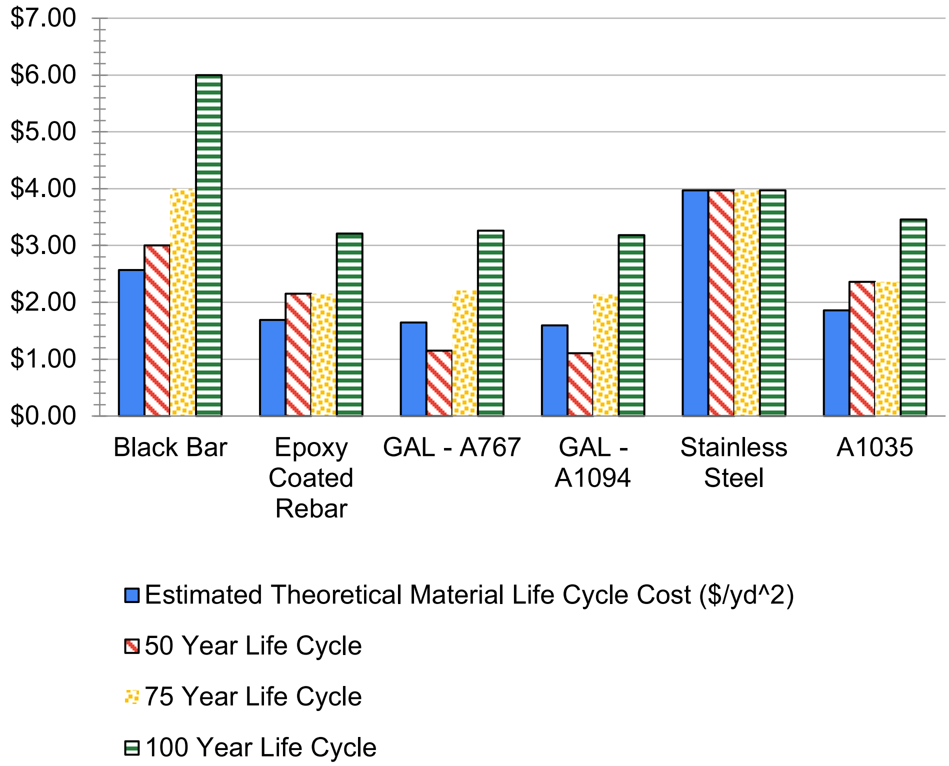

Figure 3. Graph. Graphical representation of summarized life-cycle costs.

The results in Table 2 (and graphically summarized in Figure 3) demonstrate that while standard black bars have the lowest initial cost, over the course of the 100-year design service life, they exhibit the highest estimated theoretical material life-cycle cost at $2.57/0.84 m^2 (1 yd^2) when compared to all other bar types, excluding stainless steel at $3.97/0.84 m^2 (1 yd^2). Using this approach, the bar option with the lowest estimated theoretical material life-cycle cost is A1094 galvanized at $1.59/0.84 m^2 (1 yd^2) followed only slightly by A767 galvanized at $1.65/0.84 m^2 (1 yd^2). Using the simplified methodology, the A1094 galvanized bars again exhibit the lowest cost over both 50- and 100-year service lives. It is important to note, however, that as the length of the design service life increases, the difference between the life-cycle cost of using stainless-steel bars versus the other alternatives significantly decreases, demonstrating the value of stainless-steel bars over a longer service life. In this scenario, the standard black bars exhibit a significantly higher life-cycle cost relative to all of the alternatives despite having the lowest initial material/construction cost. These results demonstrate the value of utilizing higher-priced (initially) corrosion-resistant reinforcement options in order to lower total life-cycle costs. As will be examined in more detail in the following subsections, this claim generally is more applicable to larger projects, as smaller bridges are generally less likely to merit the higher up-front costs. This observation is accounted for in an expanded version of the life-cycle cost methodology by considering other external factors and design parameters, such as annual average daily traffic (AADT) and, consequently, the corresponding amount of road salt usage at that location.

ADDITIONAL BRIDGE DESIGN AND PERFORMANCE CONSIDERATIONS

During the preliminary and design stages for a concrete bridge deck, several design-oriented factors must be considered before determining the selection of the reinforcement bars (both type and placement configuration/properties). With regards to corrosion performance, major factors for consideration are the location of the bridge and the severity of the winter weather typically experienced at that location. Because of the relatively high usage of road salts and snow removal trucks within most of the northern part of the United States, most concrete bridge decks in this region will likely be susceptible, to some extent, to high corrosion rates in concrete reinforcing bars, depending on the extent of exposure to sodium chloride on the deck. In order to correct damage resulting from high levels of bar corrosion, repairs will be made to either localized regions exhibiting high corrosion or to entire sections of the deck where general corrosion at varying degrees is observed.

Annual average daily traffic is another significant factor to consider when selecting the optimal reinforcement type, because, while two bridges may share a similar location and weather conditions, they may be subjected to differing levels of AADT. The higher traffic volume may expedite the time required for repairs or facilitate increased repair costs as construction services may become more difficult as traffic volume increases (such as lane closure restrictions, limited storage of equipment on-site, etc.).

Additionally, the structural design of the bridge deck can also play an important role in initial material and repair costs. Fundamentally, the quantity of higher strength reinforcing steel required for a given design will be less than that when using standard Grade 60 bars because of the inherently higher nominal yield strength value. Further examination would be required to determine the most economical reinforcement configuration for structural design alone; however, a very simplified way to account for the relative yield strengths between corrosion-resistant bar alternatives is still useful for the purposes of this study. A simple but useful example would be to compare the nominal yield strengths of Grade 60 (i.e., minimum of 414 MPa [60 ksi]) epoxy-coated and high-strength A1035 bars (minimum of 689 MPa [100 ksi]). In addition to potential savings on initial and material costs, the use of fewer quantities of high-strength reinforcement is expected to also reduce construction and labor costs over the service life of the structure.

EXPANDED FRAMEWORK

An expanded version of the originally presented framework is detailed in this section to reflect the other factors that are expected to play significant roles in estimating life-cycle costs for a particular concrete bridge deck. These additional factors for the purposes of this study include (but may not be limited to only these parameters, depending upon the application): weather, AADT, bridge size, and nominal yield strength. Collectively, these factors can be used to extrapolate the base life-cycle costs calculated previously to help facilitate an enhanced estimate of life-cycle costs for a given bridge deck and ultimately help determine the most viable design solution for a given bridge deck.

The goal for using the weather factor (WF) is to determine the correlation between the bridge location and the weather conditions it will experience. On average, IDOT personnel will apply 136–363 kg (300–800 lb) of salt per two lane-miles (1 mile = 1.61 km) (IDOT, 2006). This is an important factor to consider when determining the severity of salt exposure for a given bridge deck. Additionally, with increasingly lower temperatures, the quantity of salt or the frequency of its application will be increased. Therefore, using Table 3, the user can determine the amount of salt (normalized for every two lanes of bridge width) as a function of the temperature at the bridge location and subsequently determine the weather factor (WF).

Table 3. Parameters for Calculation of the Weather Factor

Degrees (°C) [°F=(9/5)*°C+32)]	kg (lbm.) salt / two lanes	WF
T >= 0	136 (300)	0.375
T<0 & T>=-3.56	181 (400)	0.500
T<-3.56 & T>=-7.11	227 (500)	0.625
T<-7.11 & T>=-10.7	272 (600)	0.750
T<-10.7 & T>=-14.2	318 (700)	0.875
T<-14.2	363 (800)	1.000

Additionally, a nominal yield strength factor (NYSF) can be used to account for the aforementioned advantages to life-cycle costs when using high-strength reinforcing bars. The equation shown in Figure 4 simply normalizes the nominal yield strength of a given bar (in ksi) relative to the standard 414 MPa (60 ksi) yield strength of conventional Grade 60 black bars. Therefore, the higher the nominal yield strength, the lower the NYSF, consequently resulting in reduced life-cycle costs. Please note that all bar types examined in this study exhibit a minimum nominal yield strength of 60 ksi (higher strength variants are also available) with the exception of A1035 bars, which shall have a minimum nominal yield strength of 689 MPa (100 ksi) (827 MPa [120 ksi] steel is also available). Note that 1 ksi is equal to 6.895 MPa when using the equation in Figure 4.

$$Nominal\ Yield\ Strength\ Factor\ (NYSF) = \frac{60\ ksi}{f_y\ of\ bar\ (in\ ksi)}$$

Figure 4. Equation. Calculation of nominal yield strength factor.

The remaining two factors, AADT and bridge size (SF), are, to some degree, inherently related and thus will be presented together for the purposes of this study. The former factor is used to help determine how life-cycle costs are related to traffic demands (e.g., costlier construction costs for a more heavily traveled urban highway, etc.), while the latter considers the overall physical footprint of the bridge, which influences, among other things, the total quantity of salt needed and the extent of repair needed for a given bridge deck size. According to the Illinois Traffic Monitoring Program, IDOT (2004) classifies road groups into the following categories: interstate-rural, interstate-urban, non-interstate-rural, and non-interstate-urban. These categories are used in Table 4 to correspond to distinct AADT and size factors for the purposes of this study. AADT factors were adopted from IDOT's (n.d.) live map of AADT.

Table 4. Parameters for Calculation of the AADT and Size Factors

Road Groups	AADT Limit	AADT Factor	SF
Interstate-Rural	25,000	0.25	0.65
Interstate-Urban	50,000	0.50	0.85
Non-Interstate-Rural	75,000	0.75	0.5
Non-Interstate-Urban	100,000	1.00	0.75

Finally, once the four factors have been calculated, the expanded life-cycle cost (*ELCC*) for a given bridge deck can be estimated using the equation in Figure 5, where *LCC* is the unfactored life-cycle cost and can utilize either the theoretical or simplified life-cycle costs (as depicted in Table 5) .

$$ELCC = LCC(WF)(NYSF)(AADT)(SF)$$

Figure 5. Equation. Calculation of expanded life-cycle cost.

SELECT EXAMPLES

To illustrate the effectiveness of the expanded version of the framework, two brief examples are included. The first case is a bridge located in a congested region of Cook County (near the western Chicago suburbs) with a relatively high AADT. The second case is a bridge on a typical rural interstate in southern Illinois. In both examples, the framework and its respective equations are applied to obtain life-cycle cost comparisons between each bar type considering the general (or unfactored) framework procedure (i.e., strictly based on initial and repair costs) and the expanded framework considering specific factors related to the performance of a given bridge deck. For the first example, the weather factor selected was 0.625 due to the average temperature during snowfall in that area taken as −6.67°C (20°F), which is in the range of −3.61°C (25.5°F) > −7.11°C (19.2°F). Size and AADT factors of 0.85 and 0.50, respectively, were selected based on an AADT of 41,900, which ranges between 25,000 and 50,000. For the second case, a rural interstate bridge in southern Illinois was assumed with a corresponding average temperature above 0°C (32°F) and thus was assigned a weather factor equal to 0.375. Additionally, AADT and size factors of 0.25 and 0.65, respectively, were then selected in accordance with Table 4. Please note that the nominal yield strength factor is dependent upon the bar type selected (and not the other general properties of the bridge deck). Therefore, it will be listed directly in the example for each corrosion-resistant reinforcement option. The life-cycle cost estimate results for both examples are summarized in Table 5 and graphically represented in Figure 6 and Figure 7 for examples 1 and 2, respectively. Select example calculations can also be found in the appendix of this document.

Table 5. Summary of Life-Cycle Costs for Two Examples

Example Selection		Simplified Life-Cycle Cost Analysis				Bar Selection Factors				Expanded Life-Cycle Cost Comparison			
Example Case	Bar Type	Theoretical Material Life-Cycle Cost* ($/0.84m²) [$/yd²]	Simplified Life-Cycle Cost*			WF	SF	AADT Factor	NYSF	Theoretical Material Life-Cycle Cost ($/0.84m²)** [$/yd²]	Simplified Life-Cycle Cost		
			50 Years	75 Years	100 Years						50 Years	75 Years	100 Years
Mannheim Bridge (Lake and St. Charles), Cook County	Black Bar	$2.57	$3.00	$4.00	$6.00	0.625	0.85	0.50	1	$0.68	$0.80	$1.06	$1.59
	Epoxy Coated	$1.69	$2.15	$2.15	$3.21	0.625	0.85	0.50	1	$0.45	$0.57	$0.57	$0.85
	A767 Galv.	$1.65	$1.15	$2.21	$3.26	0.625	0.85	0.50	1	$0.44	$0.31	$0.59	$0.87
	A1094 Galv.	$1.59	$1.11	$2.14	$3.18	0.625	0.85	0.50	1	$0.42	$0.29	$0.57	$0.85
	Stainless Steel	$3.97	$3.97	$3.97	$3.97	0.625	0.85	0.50	1	$1.05	$1.05	$1.05	$1.05
	A1035	$1.86	$2.36	$2.36	$3.46	0.625	0.85	0.50	0.6	$0.30	$0.38	$0.38	$0.55
Rural Interstate, Southern Illinois	Black Bar	$2.57	$3.00	$4.00	$6.00	0.375	0.65	0.25	1	$0.16	$0.18	$0.24	$0.37
	Epoxy Coated	$1.69	$2.15	$2.15	$3.21	0.375	0.65	0.25	1	$0.10	$0.13	$0.13	$0.20
	A767 Galv.	$1.65	$1.15	$1.15	$3.26	0.375	0.65	0.25	1	$0.10	$0.07	$0.13	$0.20
	A1094 Galv.	$1.59	$1.11	$1.11	$3.18	0.375	0.65	0.25	1	$0.10	$0.07	$0.13	$0.19
	Stainless Steel	$3.97	$3.97	$3.97	$3.97	0.375	0.65	0.25	1	$0.24	$0.24	$0.24	$0.24
	A1035	$1.86	$2.36	$2.36	$3.46	0.375	0.65	0.25	0.6	$0.07	$0.09	$0.09	$0.13

*From Table 2; **Calculated using equation in Figure 5.

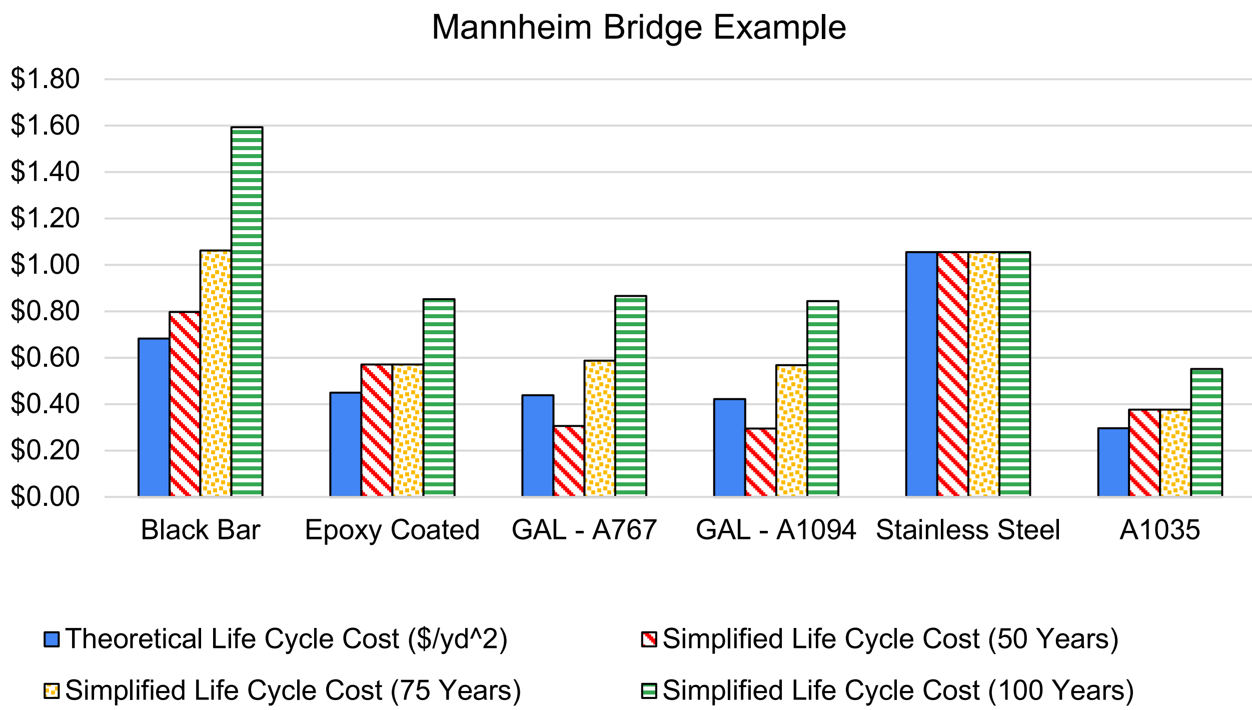

Figure 6. Graph. Graphical representation of life-cycle cost estimates for example 1.

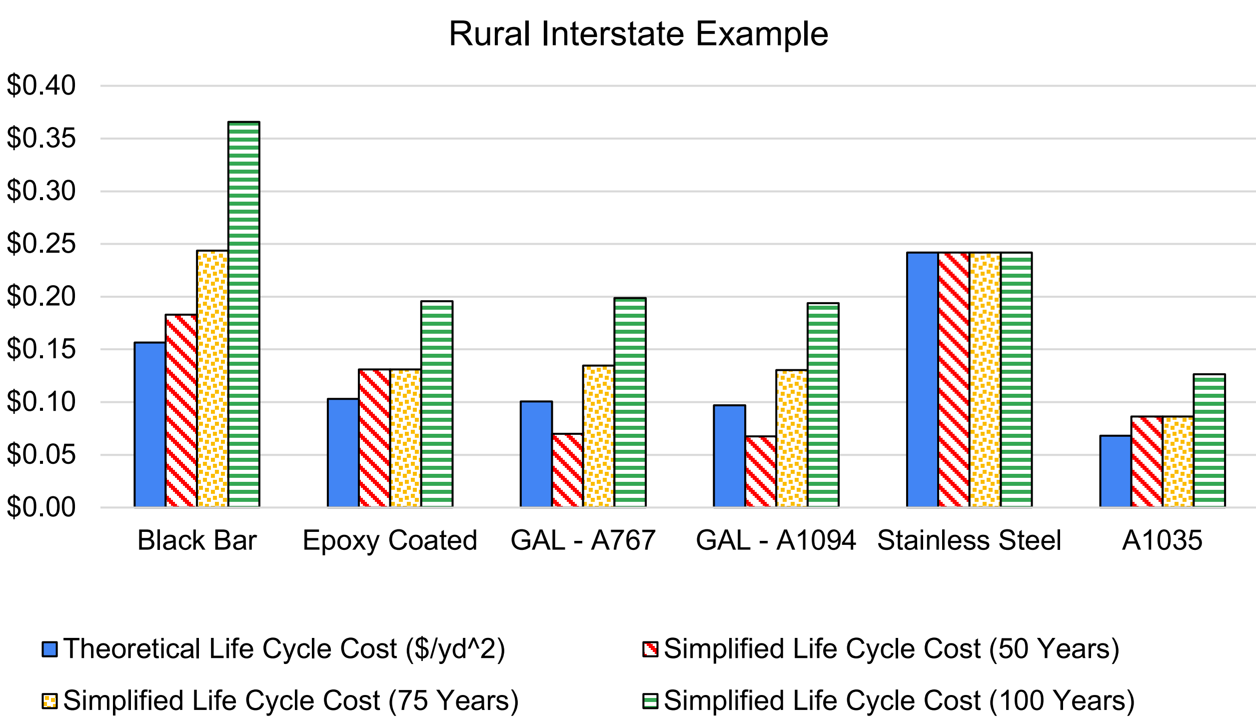

Figure 7. Graph. Graphical representation of life-cycle cost estimates for example 2.

For the first example, the expanded life-cycle cost estimates for the theoretical estimate and 100-year simplified approach were lowest for A1035 bars (see Table 5 and Figure 6), largely due to the effects of the higher nominal yield strength and its implications toward structural performance and expected reduced construction/labor costs. Both galvanized options exhibited the approximately similar lowest costs when using the 50-year simplified approach. Contrarily, the stainless-steel bars exhibit the highest costs using the theoretical approach as well as the simplified 50-year service life method. However, using the simplified 100-year service life estimate, stainless steel is no longer the most expensive (black bars in that case) and is seemingly much more cost competitive. Therefore, these results show strong correlation between the length of the service life and the value of using stainless-steel reinforcement in bridge deck construction. Similar results are shown for the second example (see Table 5 and Figure 7). However, the magnitudes of life-cycle cost differences between reinforcement types are greatly reduced. This observation highlights the increased significance of using such an optimal approach framework for bridge decks in congested areas and consequently heavier traffic demands.

CHAPTER 4: SUMMARY AND CONCLUSIONS

In this study, the results of a comprehensive literature review focusing on the corrosion resistance of several alternatives to standard black reinforcing bars were presented. The outcomes of the review were then used as the basis for developing a framework to assist bridge design engineers and IDOT officials in selecting the optimal type of reinforcement for a given bridge deck considering applicable factors influencing corrosion performance over the design service life of the structure. A short case study was used to highlight the effectiveness of the framework for two example bridges in drastically different parts of Illinois with different design parameters as well as overall implications for the most effective and economical corrosion resistant solutions. More specifically, the framework considered six reinforcing bar types, including standard black bars (as the control), as well as epoxy-coated, stainless-steel, A767 galvanized, A1094 galvanized, and A1035 bars. Note that the data and supplemental information used to construct this framework was based on existing research for each bar type, as detailed in the literature review in Chapter 2. While plenty of existing literature demonstrates that epoxy-coated bars have been an effective solution in combating corrosion over the past several decades, recent studies have also increased motivation and encouragement for using other bar types. Furthermore, after examining the total life-cycle costs and corresponding performance of each bar type, stainless steel is likely to be a very justifiable solution for larger and more expensive bridge structures that may be exposed to significantly more severe corrosion environments as a result of lower average temperatures and higher salt usage in winter months. Enhanced corrosion performance often comes with a significant trade-off of higher initial material costs, which for some applications may restrict the use of higher performance corrosion-resistant bar options. Because of its relatively lower chromium content (when compared to stainless-steel bars) and its lower material cost, the use of A1035 reinforcement is likely to serve as a commendable intermediate option between standard black bars and stainless steel. With regards to A1035 bars, further research is likely needed to fully assess its coupled corrosion and structural performance—especially due to its relatively limited ductility (when compared to conventional Grade 60 underlying black bars). However, it is expected—as shown in this report—that the higher strength steel can facilitate reduced labor, repair, and construction costs when accounted for in a life-cycle analysis.

Based on the results of the literature review and subsequent deployment of the optimal corrosion-resistant reinforcement selection framework, the following conclusions can be drawn:

1. As expected, the duration of the chosen design service life of the concrete bridge deck plays a major role in determining the most cost-effective corrosion-resistant reinforcing bar option. This is especially evidenced for stainless-steel bars where the difference between its life-cycle costs and those of the other bar alternatives decreases significantly as the design service life approaches 100 years.

2. The life-cycle costs between different bar options seem to scale as a function of external factors, such as the bridge's location, AADT, and weather; however, their relative cost differences become especially magnified for larger projects in more congested areas. Thus, performing comprehensive life-cycle analysis on these structures may be much more warranted compared to smaller, less traveled bridges in rural areas.

3. Without consideration of external factors specific to a given bridge deck, the galvanized bar types (i.e., A767 and A1094) generally exhibit the lowest life-cycle costs independent of the method used (i.e., estimated theoretical or simplified) and chosen design service life duration (up to 100 years examined in this study). For the purposes of this study, it can be concluded that this observation is largely due to the performance enhancement provided by the metallic coating (when not prematurely damaged) and cost savings resulting from using standard black bars as the underlying product.

4. When considering bridge-specific design parameters, A1035 bars exhibited the lowest overall life-cycle costs in all but one case: galvanized bars using the 50-year simplified method. This is largely due to their relatively higher nominal yield strength of the A1035 bars and the corresponding effect on reducing labor and repair costs due to the likely fewer quantity of bars required.

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APPENDIX: SAMPLE CALCULATIONS

ESTIMATED THEORETICAL MATERIAL LIFE-CYCLE COST

$$PV = Initial\ Cost + \sum_{0}^{5}[Repair\ Cost + (1+i)^{-n}]$$

For Black Bar:

$$PV\ =\ \$1.00\ + [\$1.00 + (1+0.02)^{-24.33} + (1+0.02)^{-48.63} + (1+0.02)^{-72.98} + (1+0.02)^{-85.20}$$
$$+(1+0.02)^{96.10}]\ =\ \$2.57$$

SIMPLIFIED LIFE-CYCLE COST

$$SLCC = Initial\ Cost + fN_R$$

For Black Bar (Year 50):

$$SLCC\ = Initial\ Cost\ (\$/yd\hat{}2)\ +\ Material\ Repair\ Cost\ (\$/yd\hat{}2)\ *\ N_R$$
$$SLCC\ =\ \$1.00/yd^2\ +\ \$1.00/yd^2\ *\ 2\ Repairs$$
$$SLCC\ =\ \$3.00/yd^2$$

NOMINAL YIELD STRENGTH FACTOR

$$Nominal\ Yield\ Strength\ Factor\ (NYSF) = \frac{60\ ksi}{f_y\ of\ bar\ (in\ ksi)}$$

For Black Bar:

$$NYSF\ =\ \frac{60\ ksi}{60\ ksi} = 1.0$$

EXPANDED LIFE-CYCLE COST

$$ELCC = \boldsymbol{Estimated\ Theoretical\ Material\ Life\ Cycle\ Cost}\ (\$/\boldsymbol{yd\hat{}2}) * (WF)(NYSF)(AADT)(SF)$$

For Rural Bridge Example for Black Bar:

$$ELCC\ = \$2.57/yd^2\ * (0.375)(0.65)(0.25)(1.0)\ =\ \$0.16/yd^2$$