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Alternate Reinforcements for Enhanced Corrosion Resistance in TxDOT Bridges

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16. Abstract

The corrosion of reinforcing steel in concrete is the leading cause of deterioration for reinforced concrete structures, especially bridges exposed to external chlorides. Practitioners and researchers have evaluated and implemented various technologies to combat this problem, including the use of high-performance concrete, chemical corrosion inhibitors, sealers and barriers, and alternative reinforcement. This synthesis project addressed the latter, specifically the use of alternative reinforcement (e.g., fiber-reinforced polymer (FRP) reinforcement, epoxy-coated steel, stainless steel, galvanized steel, etc.) to extend the service life of bridge structures subjected to external chlorides from marine environments or from de-icing salt applications. The primary goals of this project were to (a) review and synthesize published literature, (b) review and synthesize current DOT practice, (c) identify gaps in our current knowledge and state of practice, and (d) provide guidance, based on current knowledge, on how to evaluate and select alternative reinforcement for bridges subjected to external chlorides.

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Products

Chapter 6 represents the project's product, P1 *Conclusions and Recommendations for Alternative Reinforcements*.

Table of Contents

List of Tables

List of Figures

List of Acronyms

AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
AFRP	aramid fiber-reinforced polymer
BFRP	basalt fiber-reinforced polymer
CCTL	critical chloride threshold level
CFRP	carbon fiber-reinforced polymer
DOT	department of transportation
ECR	epoxy-coated reinforcement
FHWA	Federal Highway Administration
FRP	fiber-reinforced polymer
GFRP	glass fiber-reinforced polymer
LRFD	Load and Resistance Factor Design
LRP	linear polarization resistance
NCHRP	National Cooperative Highway Research Program
SCR	stainless steel-clad reinforcement
TEXMEX	Texas Marine Exposure Site

Chapter 1. Introduction

1.1. Introduction and Scope

The corrosion of reinforcing steel in concrete is the leading cause of deterioration for reinforced concrete structures, especially bridges. Practitioners and researchers have evaluated and implemented various technologies to combat this problem, including the use of high-performance concrete, chemical corrosion inhibitors, sealers and barriers, and alternative reinforcement. This synthesis project addressed the latter, specifically the use of alternative reinforcement (e.g., fiber-reinforced polymer (FRP) reinforcement, epoxy-coated steel, stainless steel, galvanized steel, etc.) to extend the service life of bridge structures subjected to external chlorides from marine environments or from de-icing salt applications. The primary goals of this project were to (a) review and synthesize published literature, (b) review and synthesize current department of transportation (DOT) practice, (c) identify gaps in our current knowledge and state of practice, and (d) provide recommendations, based on current knowledge, on how to evaluate and select alternative reinforcement for bridges subjected to external chlorides.

1.2. Organization of Report

This report is presented in the following sections:

- **Section 2** describes the various types of alternative reinforcement evaluated in this synthesis project, including coated reinforcement, high-chromium steel bars, and FRP bars.
- **Section 3** summarizes the corrosion resistance of alternative reinforcement, including a review of experimental techniques used to evaluate reinforcement and the corrosion resistance properties of the most commonly used alternative reinforcements.
- **Section 4** summarizes the structural aspects related to alternative reinforcement, including the mechanical properties of various alternative reinforcement types and the impact on the design and construction of bridges.
- **Section 5** presents the findings from a review of current practice regarding the use of alternative reinforcement in bridges, including the results of a survey distributed to 14 TxDOT districts and 17 other state DOTs.
- **Section 6** summarizes the main findings from this synthesis, including the identification of research needed to increase and improve the use of

alternative reinforcement in bridges. Guidance is presented, based on current knowledge, on how to evaluate the potential use of alternative reinforcement for bridges exposed to external chlorides.

Chapter 2. Overview of Materials

The properties and behavior of several types of alternative reinforcement materials will be discussed throughout this report. To minimize redundancy, these materials will often be grouped into the following categories: coated reinforcement, high-chromium steel bars, and FRP bars. The materials that comprise each category are listed in Table 2.1 and will be described in depth within the following sections.

Table 2.1: Alternative reinforcement material categories

Coated Reinforcement	High-Chromium Steel Bars	FRP Bars
Epoxy-Coated Steel Dual-Coated Steel	Stainless Steel	Glass FRP Bars
Galvanized Steel Stainless Steel-Clad Steel	Low-Carbon, Chromium Steel	Basalt FRP Bars

2.1. Coated Reinforcement

For the purpose of this report, types of alternative reinforcement that employ a physical barrier around the steel to prevent chlorides from reaching the bare metal surface will be referred to as coated reinforcement. Such materials include epoxy-coated steel, dual-coated steel, galvanized steel, and stainless steel-clad steel.

Epoxy-coated steel is specified by ASTM A775 *Standard Specification for Epoxy-Coated Steel Reinforcing Bars* or ASTM A934 *Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars*. Prior to coating, mill scale and oxidation are removed from the bars with grit blasting. The bars are then heated to approximately 450°F before undergoing an electrostatic powder coating process. The heat of the bars melts the powder on contact, which produces a polymer coating (Van Dyke, et al. 2017).

ASTM A775 specifies deformed and plain steel reinforcing bars with a protective, fusion-bonded epoxy coating applied by the electrostatic spray method. ASTM A934 covers deformed and plain steel reinforcing bars that are fabricated prior to surface preparation and then coated with a protective fusion-bonded epoxy coating by electrostatic spray or other suitable method. Pre-fabricated steel may also be coated using a coating meeting ASTM A775 (CRSI 2013). The epoxy used to manufacture A775 bars is pigmented green to distinguish it from A934 bars, which are pigmented purple. The cost of A934 (purple) bars is greater than that that of A775 (green) bars but use of A934 prevents coating damage encountered in bending operations (Van Dyke, et al. 2017).

Per both ASTM A775 and A934, the coating thickness measurements after curing shall be 7 to 12 mils for bar sizes Nos. 3 to 5 and 7 to 16 mils for Nos. 6 to 18. The acceptance process requires a minimum of ten recorded measurements of coating thickness per bar. The average of all recorded coating thickness measurements shall not be less than the specified minimum thickness or more than the maximum thickness. Coating continuity and flexibility requirements are also specified per ASTM A775. As such, there shall not be more than one holiday per foot on a coated steel reinforcing bar, and cracking or disbanding of the coating on the outside radius of a bar after the specified bending test is cause for rejection. The bend test requirements vary depending on diameter of the bar. These requirements are outlined within the standard.

Epoxy-coated reinforcing steel was first used for bridge construction in 1973 in Pennsylvania. The first American Society for Testing and Materials (ASTM) specification for epoxy-coated reinforcing steel was introduced in 1981 and has been modified and updated numerous times since then. Epoxy-coated steel is now one of the most commonly used materials/methods for increasing the corrosion resistance of reinforced concrete bridges.

Dual-coated steel reinforcing bars are deformed and plain reinforcing steel bars that are coated with a thermal-spray zinc layer followed by an exterior epoxy coating. These bars are specified by ASTM A1055 *Standard Specification for Zinc and Epoxy Dual-Coated Reinforcing Bars* (ASTM 2017). ASTM A1055 allows for bars meeting the specifications for ASTM A615, A706, and A996 to be coated. Several yield strengths are available according to each specification. Dual-coated steel reinforcing bars are available in all US conventional bar sizes and metric sizes used in Canada. A yellow coating is used to identify bars that meet ASTM A1055.

Production of dual-coated steel bars is achieved through the following process. The bars are cleaned and the surface heated to approximately 425°F (220°C) and then passed through a zinc arc spray, immediately followed with an electrostatic spray containing fine epoxy powder. The powder is attracted to the zinc layer based upon electrostatic forces. When the epoxy encounters the heated zinc-sprayed bars, it melts and fuses, forming a thermosetting polymer. The resultant dual coating is significantly more uniform in thickness than could be achieved using other methods (Concrete Reinforcing Steel Institute - CRSI 2015).

Per ASTM A1055, the minimum thickness required for the zinc-alloy coating ranges from 1.4 to 2.0 mils, depending method of application. The total coating thickness for the zinc-alloy and epoxy coating layers shall be 7 to 12 mils for bar sizes Nos. 3 to 5 and 7 to 16 mils for Nos. 6 to 18. This is the same thickness range required for epoxy-coated reinforcing (ASTM 2017).

Galvanized steel reinforcing bars are traditional steel bars conforming to ASTM A615, A706, or A996 that have been covered with a protective zinc coating applied by immersing the properly prepared reinforcing bars into a molten bath of zinc. The bars remain in the molten bath until the zinc reacts with the steel surface to form zinc-iron inter-metallic alloys. After solidification, the zinc coating must meet the minimum thicknesses of 5.1 to 5.9 mils for Class 1 bars and 3.4 mils for Class 2 bars as specified in ASTM A767 *Standard Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement* (ASTM 2016).

Stainless steel-clad reinforcement (SCR) is produced in a different manner than the other types of coated reinforcement. First, a strip of 316L stainless steel is formed into a pipe with an approximate diameter of 4-in. and the seam is welded together by plasma. Carbon steel filings are then compacted into the pipe with a hydraulic ram. Once filled, the pipe is heated and hot-rolled, creating a metallurgical bond between the two materials (Clemena, Kukreja and Napier 2003). Stainless steel-clad reinforcing was developed to obtain the benefits of stainless steel without its high cost. It can be less expensive than solid stainless steel, but it is a bit more expensive than epoxy-coated rebar. Issues with stainless-clad rebar result from imperfect bonding between the stainless steel and carbon steel, which can make the carbon steel vulnerable to corrosion (Van Dyke, et al. 2017).

Figure 2.1: Stainless steel-clad reinforcement (Clemena, Kukreja and Napier 2003)

2.2. High-Chromium Steel Bars

Stainless steels are iron-based alloys with a minimum chromium (Cr) content of 11%. This limit is such that the oxide layer formed on alloys with >11% Cr is

Cr_2O_3, whereas at lower levels of Cr content, an iron oxide is formed. Iron oxide occupies a considerably larger volume of space than Cr_2O_3, which leads to the spalling and debonding issues found in traditionally reinforced concrete structures experiencing corrosion. This is not an issue with stainless steel reinforced structures. Other alloying elements, such as nickel (Ni), molybdenum (Mo), copper (Cu), and nitrogen (N), are added to achieve the desired mechanical, fabrication, and corrosion resistance characteristics (Hansson 2016). The chemical composition requirements of the various stainless steel alloys available are specified in ASTM A955 *Standard Specification for Deformed and Plain Stainless-Steel Bars for Concrete Reinforcement.*

Low-carbon chromium steel is reinforcing steel with low carbon and high chromium content as specified in ASTM A1035 *Standard Specification for Deformed and Plain, Low-Carbon, Chromium, Steel Bars for Concrete Reinforcement*. The three alloy types of low-carbon, chromium steel are CS, CM, and CL. The alloy type is determined by the carbon and chromium content. The carbon content of each alloy decreases and the chromium content increases as you move up from alloy CL to CS. ASTM A1035 steel has two yield strengths: 100 and 120 ksi. The higher yield strength can effectively reduce the cross-section of members and reinforcement quantities; however, many specifications limit yield strength to 75 ksi for most applications (Shahrooz, et al. 2011).

2.3. FRP Bars

Fiber-reinforced polymer (FRP) reinforcing bars are made from filaments, or fibers, held in a polymeric resin matrix binder. The fibers used in FRP bars can be glass (GFRP), carbon (CFRP), or aramid (AFRP). Due to significantly lower cost compared to CFRP, GFRP bars are currently the most popular choice for FRP reinforcing bars and are fabricated in a pultrusion process that results in a bar having a cross section of glass fibers suspended in a polymer (usually vinyl ester) matrix. The glass fibers are made by extruding molten glass through an orifice. These fibers have a high tensile strength and a high modulus of elasticity and are the load-bearing component. The matrix is the bonding material used to hold the fibers together so as to transfer load between them, but also to protect the fibers and to maintain the dimensional stability of the GFRP bar (Benmokrane, Chaallal and Masmoudi 1995). The standard for GFRP bars is ASTM C7957/D7957M – 17, *Standard Specification for Solid Round Glass Fiber Reinforced Polymer Bars for Concrete Reinforcement*. Within the last decade, innovations in FRP technology have led to the increased use of basalt in FRP bars (BFRP). Compared to GFRP, BFRP has higher strength and modulus, similar cost, and greater chemical stability (Elgabbas, Ahmed and Benmokrane 2015). However, BFRP bars have been shown to exhibit poor durability when compared to GFRP bars as evidenced by moisture uptake and reductions in mechanical properties when tested after subjection to an alkaline environment at elevated temperature. Basalt fibers are mainly produced in Ukraine and, recently, in China (Benmokrane, et al. 2015). Basalt fiber properties are less controlled due to less control over the purity of the natural basalt stone (Ross 2006).

Chapter 3. Corrosion Resistance of Alternative Reinforcement

3.1. Introduction

Corrosion of reinforcing steel used in bridges and related infrastructure has plagued the nation for over 100 years; unfortunately, this issue becomes larger and more costly with each passing year. As of 2002, the estimated annual direct costs of corrosion in bridge structures was $8.3 billion (Koch, Brongers and Thompson 2002). In 2013, Davis and Goldberg (Davis and Goldberg 2013) released a study noting that there were 66,405 structurally deficient bridges within the U.S. The average age of these bridges was 65 years, well above their 50-year service life, which indicates that the annual corrosion costs have likely increased significantly beyond the 2002 estimate.

Practitioners and researchers have evaluated and implemented various technologies to combat corrosion of reinforcing steel, including the use of high-performance concrete, chemical corrosion inhibitors, sealers and barriers, and alternative reinforcement. Much is known on the corrosion resistance, or lack thereof, of traditional carbon steel. Recently, the use of alternative reinforcement materials has increased due to their purported superior corrosion resistance compared to traditional carbon steel; however, for some of these materials, long-term performance data to support these claims is lacking. The following sections provide a review of experimental techniques used to evaluate reinforcement and the corrosion resistance properties of the most commonly used alternative reinforcements.

3.1.1. Critical Chloride Threshold Values

It has long been assumed that corrosion of reinforcement in non-carbonated concrete can only occur once the chloride content at the steel surface has reached a certain threshold value. This is often referred to as the *critical chloride content* or *chloride threshold value*. A multitude of studies have been conducted with the aim of identifying one specific chloride threshold value; however, due to variances in measuring techniques, testing conditions, and definitions of the term, no consensus on a chloride threshold value has been achieved.

Two different ways of defining this threshold are common (Angst, et al. 2009): the scientific view and the engineering view. From a scientific point of view, the critical chloride content can be defined as the chloride content required for depassivation of the steel (Definition 1), whereas from a practical engineering point of view the threshold is usually the chloride content associated with visible or “acceptable” deterioration of the reinforced concrete structure (Definition 2). The first definition is concerned only with the initiation of corrosion while the second definition also considers the propagation of corrosion. This fundamental difference results in different threshold values. Definition 1 is considered more precise as it is directly related to depassivation. Issues with Definition 2 arise because there is no scientific scale for the ambiguous term, "acceptable deterioration."

The critical chloride content is most commonly expressed as total chloride content relative to the weight of the cement. Several standards (ASTM n.d.) (Andrade and Castellote 2002) document relatively simple methods for measurement of total chloride content. Despite there being simple methods for determining total chloride content in a concrete sample, varied testing conditions make it difficult to determine a universal chloride threshold value. Some of the main parameters that can influence the critical chloride content in concrete are pH of the pore solution, steel potential, steel-concrete interface, binder type, surface condition of steel, moisture content, and degree of hydration.

A standardized testing procedure for measuring critical chloride threshold does not exist; however, it is assumed that individual experimental setups must include the following (Angst and Vennesland 2008):

1. A steel electrode embedded in a cement based material (cement paste, mortar, or concrete) or immersed in a solution that simulates the concrete.

2. Chloride ions at the steel surface.

3. A method to detect depassivation of the steel (Definition 1) or for determining if the degree of corrosion has reached the acceptable limit (Definition 2).

4. A method to quantify the chloride content.

The most convenient setup that fulfills these requirements consists of a steel electrode immersed in an alkaline solution, where both the chloride concentration and pH can be easily and rapidly changed and accurately quantified (Angst, et al. 2009). However, this setup does not accurately model real concrete in the laboratory where the introduction of chlorides into hardened samples is much more time-consuming.

Since long-term data is needed to acquire critical chloride threshold values, many new alternative steels have been using simulated pore solutions to acquire threshold values. However, without being in a composite system, the values obtained from simulated pore solutions may not provide the most accurate chloride threshold values. One potential test procedure for quantitatively determining the critical chloride threshold is the Accelerated Chloride Threshold (ACT) method (Trejo and Pillai 2004). The procedure creates an accelerated chloride transport system, which gradually increases the chloride concentration near the embedded steel. A corrosion initiation detection system is in place to measure the polarization resistance values as a function of potential time. A program is used to identify when the steel reinforcement transfers from a passive to an active corrosion state. Once corrosion is determined, the specimen is cut to determine the chloride threshold at the steel mortar interface (Trejo and Pillai 2004). That value is then reported as the critical chloride threshold value. This procedure seems to be a suitable method to determine the critical chloride threshold value; however, more alternative reinforcements need to be verified with this method.

A number of factors may change the critical threshold values for alternative reinforcement steels. Critical threshold values can be influenced by surface treatments on the reinforcement bar to remove the mill scale. Pallai and Trejo (2005) showed that milling the surface of SS316LN reinforcement greatly decreased the critical threshold value; however, milling an A706 reinforcement bar only slightly decreased the critical threshold value. Each alternative steel may behave differently due to surface treatments.

There have also been attempts to set critical chloride threshold limits for newly constructed concrete. The American Concrete Institute (ACI) 201.2R *Guide to Durable Concrete* (ACI Committee 201 2016) provides chloride limits for several environmental conditions; however, as stated earlier, the large range of reported critical chloride thresholds makes it difficult to determine an accurate value for specifications.

3.1.2. Corrosion Resistance Evaluation Methods

3.1.2.1. ASTM G109

The test method described in ASTM G109 (ASTM 2013) covers a procedure for determining the effect of chemical admixtures on the corrosion of metals in concrete; however, this method is also often used to evaluate the performance of alternative reinforcement in concrete without chemical admixtures. Test specimens are prepared by casting three reinforcement bars into concrete measuring 11 x 6 x 4.5 in. One end of each bar is drilled and tapped with a stainless steel screw and two nuts. After the specimens have cured for 28 days, they are allowed to dry in a 50% humidity room for two weeks before the four vertical sides are sealed with an epoxy sealer. A plastic dam is also placed on top of the specimen (Figure 3.1) and epoxy sealer is applied around the dam on the top surface of the concrete. Two-week cycles of ponding with a 3% sodium chloride solution and drying are repeated throughout the duration of the test. The voltage across the resistor and corrosion potential measurements are recorded every four weeks until the average integrated macrocell current of the control specimens is 150 C or greater. At the end of the test, the specimens are broken and the extent of corrosion of the reinforcement is examined.

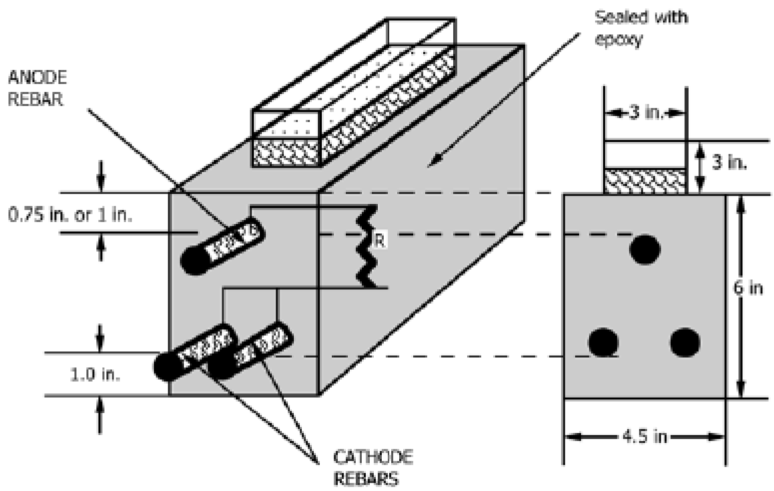

Figure 3.1: Specimen configuration for ASTM G109

3.1.3. Active Corrosion Detection Methods

3.1.3.1. Steel Potential

Actively corroding steel has a significantly more negative potential than passive steel in concrete; therefore, the onset of corrosion can be detected by potential measurements. Potential measurements are collected by connecting the positive terminal of a voltmeter to steel and the negative terminal to a reference electrode. Typical reference electrodes are composed of silver in a silver chloride solution or copper in a saturated copper sulfate solution. Each half-call reference electrode has a known and steady potential that is used to measure the relative potential of the steel being evaluated. Very negative potentials can be found in saturated conditions where there is no oxygen to form a passive layer, but with no oxygen there can be no corrosion. This shows the weakness of potential measurements. The problem is that the potential is not purely a function of the corrosion condition but also other factors, and that the corrosion condition does not equate with corrosion rate (Broomfield 2007).

3.1.3.2. Linear Polarization Resistance (LPR) Measurements

Another nondestructive technique used to detect depassivation of reinforcing steel is the measurement of the LPR. This resistance is inversely proportional to the corrosion current, according to the Stern-Geary equation (Angst, et al. 2009). The use of this method allows for an instantaneous determination of the corrosion rate in a given specimen. LPR can be measured through the use of a three-electrode system comprising 1) a working electrode (reinforcing bar), 2) counter electrode, and 3) a reference electrode, as illustrated in Figure 3.2. The principal of LPR is based on disturbing the corrosion equilibrium on the surface of steel reinforcing bars by introducing a small perturbative electrical signal using a surface counter electrode. Monitoring the relationship between electrochemical potential and current generated between electrically charged electrodes under the test allows the calculation of the corrosion rate (FHWA Research and Technology 2015).

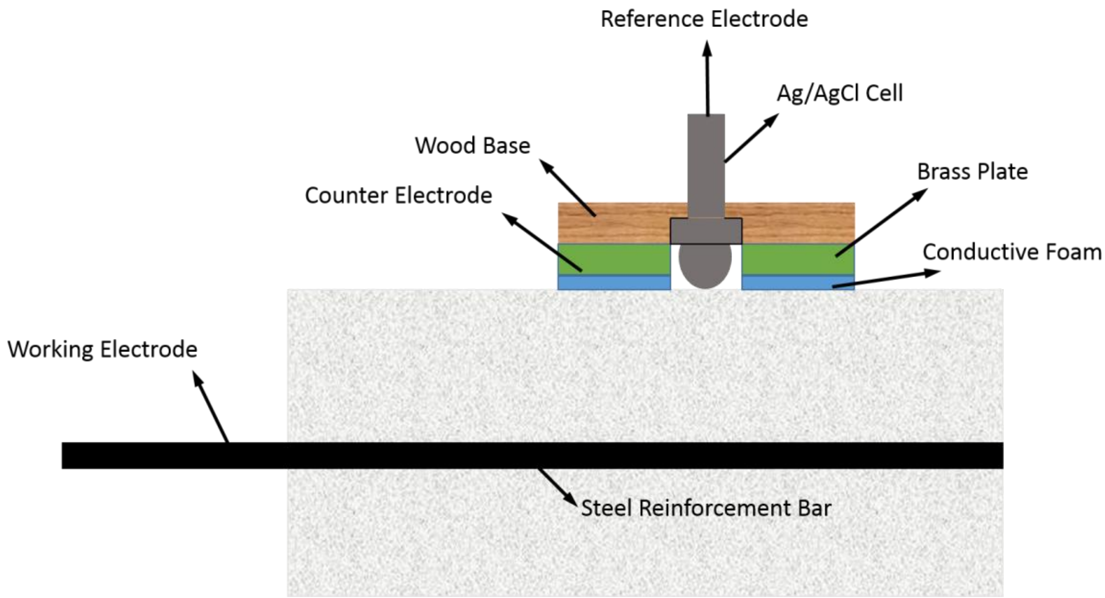

Figure 3.2: Three-electrode system for LPR measurement

3.2. Corrosion Resistance Properties of Alternative Reinforcement

3.2.1. Coated Reinforcement

3.2.1.1. Epoxy-Coated Steel

It wasn't until the late 1960s/early 1970s that engineers and transportation officials identified the use of deicing salts as the cause of early deterioration of steel reinforcement. The Federal Highway Administration (FHWA) investigated the issue and epoxy-coated reinforcement (ECR) was identified as the most viable option for combatting early corrosion in reinforced concrete structures exposed to deicing salts and marine environments. Since its first use in bridge construction in 1973, epoxy-coated reinforcing steel has become one of the most commonly used materials to attempt to inhibit steel corrosion.

ECR is produced through an electrostatic powder coating process that creates a physical barrier between traditional carbon steel and potentially corrosive environments. Damage to the epoxy coating barrier is recognized as one of the greatest weaknesses when using epoxy-coated reinforcing steel.

Research was conducted by the FHWA in the 1980s using 31 reinforced concrete slabs constructed with poor-quality concrete and non-specification reinforcing steel. The bars were 3 years old, had greater than 25 holidays/ft, failed the bend test, and epoxy readily peeled from the bars. Despite these noted deficiencies, the corrosion rates for these bars was still 12 to 46 times less than uncoated bars cast in similar concrete (McDonald 2019). Macrocell corrosion can develop between top mat steel in chloride contaminated concrete and bottom mat steel in chloride-free concrete; therefore, the higher range of corrosion resistance is achieved when ECR is used for both top and bottom mats of reinforcement.

Concern regarding the use of ECR began in 1986 after significant corrosion-induced spalling of concrete piers was observed in five major bridges in the Florida Keys built between 1978 and 1983 (Sagues, Powers and Kessler 2001). Analysis of the practices used during construction of these bridges indicated that the epoxy coating on the reinforcement had suffered damage prior to placement. Additionally, poor quality concrete was used, which allowed for very fast chloride penetration.

In 2006, the Virginia Transportation Research Council (Weyers, Sprinkel and Brown 2006) reported on a long-term study on the performance of ECR. This study included a review of past ECR performance throughout the nation as well as field evaluations and the development of new service life models. Their study concluded that the epoxy coating on ECR naturally degrades in the highly alkaline moist environment within concrete. The subsequent loss of bond, coupled with the inevitable flaws in the coating induced by construction, leads to an estimated service life benefit of ECR of as little as 3 to 5 years beyond that of traditional carbon steel. Consequently, the report recommended that the Virginia DOT amend its specifications regarding the use of ECR to require the use of corrosion-resistant metallic reinforcing bars such as low-carbon, chromium, stainless steel-clad, and solid stainless steel instead.

Due to the observed unsatisfactory performance of ECR in both states, Florida and Virginia no longer use ECR. Instead, both DOTs specify the use of stainless steel or low-carbon chromium reinforcement. Florida also utilizes FRP reinforcement in some applications.

ECR has the lowest average material cost per pound compared to other alternative reinforcement options (Van Dyke, et al. 2017); however, due to its shorter life span, its overall life-cycle cost is considerably higher than the highest priced alternatives such as stainless steel and SCR, which can offer corrosion resistance for 75+ years. For this reason, in-depth cost-benefit analysis should be conducted for each individual project prior to choosing ECR as the primary type of reinforcement.

3.2.1.2. Dual-coated Reinforcement

The multi-layer system provided by dual-coated reinforcement consists of a protective zinc inner layer applied to traditional carbon steel followed by an epoxy coating as the outer layer. Ideally, the zinc inner layer provides cathodic protection for the steel in the event of damage to the outer epoxy-coating layer. This type of alternative reinforcement is relatively new; therefore, little performance data exist for this material. Clemena (G. G. Clemena 2003) evaluated the corrosion resistance of dual-coated reinforcement compared to low-carbon chromium steel rebar; 304 and 361LN stainless steel; stainless steel-clad; LDX 2101 duplex stainless steel; and carbon steel. Three condition scenarios were established for the dual-coated reinforcement: (1) no damage to bar coating, (2) cut in outer epoxy coating, (3) cut in both outer epoxy coating and inner zinc coating to expose carbon steel. Each type of reinforcement was cast into concrete blocks similar to the configuration specified in ASTM G109. The macrocell current flowing between the top and bottom bars in each sample was measured weekly. The half-cell potential of the top bars of each sample was also measured weekly. The samples were exposed to weekly cycles of salt solution ponding and drying for at least 105 weeks. Within 13 weeks the carbon steel had depassivated and showed signs of corrosion. Depassivation and corrosion was noted in the LDX and low-carbon chromium bars after 21 and 35 weeks, respectively. The dual-coated reinforcement with cuts through both the zinc and epoxy layer remained passive for 76 weeks, while the other two scenarios for dual-coated reinforcement and stainless steel bars remained passive throughout the 105 weeks. In summary, Clemena concluded that dual-coated reinforcement offers similar corrosion resistance to that of stainless steel.

3.2.1.3. Galvanized Reinforcement

Galvanized, or zinc-coated, steel can be produced through several different methods. For steel products greater than 5–6 mm thick, such as reinforcement, hot dipping is the preferred and most common method. This method provides a durable and long lasting zinc coating. Hot-dipped galvanizing, often called “batch galvanizing,” involves immersing clean and prefluxed steel in molten zinc at 840°F. During the immersion time while the steel is being heated to the temperature of the zinc, a metallurgical reaction occurs between the steel and zinc (S. R. Yeomans 2016). Time of immersion varies from just a few minutes to 20 to 30 minutes depending on the size of the steel being galvanized.

Zinc, applied to steel in the hot-dip galvanizing process, reacts with iron in the steel to form a series of metallurgically bonded zinc-iron alloy layers as shown in Figure 3.3. The hot-dip galvanized zinc coating provides an impenetrable barrier, protecting the steel from corrosive elements in the environment. The zinc coating also provides cathodic protection for the underlying steel. Due to the electric potential difference between the two materials, zinc corrodes preferentially to steel.

Steel exposed at cut edges or from severe mechanical damage will not corrode, as the adjacent zinc will sacrifice itself and isolate corrosion until all of the surrounding zinc is consumed (American Galvanizers Association 2011). Additionally, corrosion products produced from the dissolution of the galvanized coating are significantly less voluminous than those produced by the corrosion of carbon steel, thus reducing the production of internal stresses that can lead to cracking and spalling of concrete.

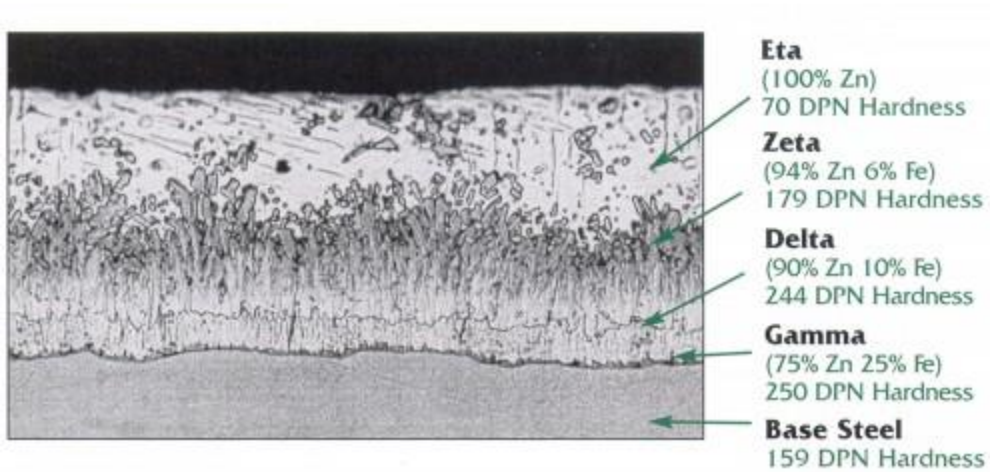

Figure 3.3: Typical zinc-iron alloy layers (American Galvanizers Association 2011)

A key feature of hot-dip coatings is that the outer eta layer (~100% zinc) is generally 40–50 µm thick. It is the presence of this eta layer that controls much of the behavior of zinc when in contact with saturated concrete. Depending on the steel chemistry and processing conditions, the coating may only contain one or two of the alloy layers. The galvanized coating on steels with higher silicon content tend to consist almost entirely of the enlarged zeta crystals that grow in an uncontrolled manner and consume the outer layer of pure zinc. Similarly, this zeta layer is promoted at the expense of the pure zinc layer when galvanized steels are heated above about 842°F (450°C).

When hot-dipped galvanized coatings come in contact with portland cement paste (or simulated solution), it is generally observed that about 10 µm of zinc from the outer layer of the coating is consumed by the passivation reaction. This process continues throughout the first 1–2 hours while the initial reaction between zinc and the pore solution is quite vigorous. Studies have shown that galvanized coatings with outer layers of iron-zinc intermetallic phases required longer to passivate than those with a pure zinc surface layer, confirming the importance of the presence of pure zinc in the passivation reaction (S. R. Yeomans 2016). Typically, traditional carbon steel embedded in concrete depassivates when the pH of the concrete drops below 11.5; however, galvanized steel is able to remain passivated until the pH drops below 9.5 (S. R. Yeomans 2004). This quality provides additional resistance to carbonation-induced corrosion.

Studies of galvanized reinforcing bars recovered from field structures indicate that after the initial consumption of zinc from the outer layer of the coating during passivation, the galvanized coating remains intact for extended periods. As long as the conditions of the concrete do not change significantly, little metal loss will occur until the zinc is depassivated and active corrosion begins (S. R. Yeomans 2004).

Galvanized steel has been used since the 1930s for corrosion protection in many types of reinforced concrete structures and elements. Bermuda was an early adopter of the use of galvanized reinforcement with the construction of the Longbird Bridge in 1953. The Bermuda marine environment is highly corrosive; however, a 1978 inspection of bridges conducted by Construction Technology Labs concluded that 98% of the initial galvanized coating remained intact despite high levels of chloride present within the surrounding concrete (American Galvanizers Association 2011). Due to the positive performance of galvanized steel in Bermuda, it is currently used exclusively on all reinforced concrete structures, including docks, jetties, bridge decks, substructures, and industrial/commercial construction throughout the country.

Extensive research has been conducted on the corrosion resistance of galvanized reinforcement for concrete bridge and highway construction exposed to high levels of chlorides due to use of deicing salts or exposure to marine environments. Early studies yielded significantly varied results, which led to confusion and doubts regarding the corrosion resistance of galvanized reinforcement. In 1982 the FHWA convened an expert panel to address the concerns over conflicting data by critically examining the available literature on the performance of galvanized reinforcement in concrete. The main finding of the panel pointed to a lack of standardization in experimental methodology as the main source of the conflicting results and subsequent confusion. As a result, numerous comparative studies and investigations of galvanized steel reinforced bridges were conducted in the 1980s and 1990s. The main conclusions drawn from these studies were the following: (1) galvanized reinforcement can tolerate chloride levels at least 2.5 times higher than those causing corrosion of black steel in equivalent concrete; (2) galvanized reinforcing steel can prevent the onset of corrosion for a period of 4 to 5 times longer than black steel (S. R. Yeomans 2004).

The general trend of more recent results indicates that the level of corrosion for galvanized bars is extensively less than that of black steel. One specific study indicated that, for a 0.5 w/c concrete, galvanized bars performed better than black bars, although in a 0.4 w/c concrete there was similar performance for both galvanized and black bars after 8 years of exposure and meaningful comparisons could not be made. It was also noted that the worst corrosion occurred when top mat galvanized bars in high chloride concrete were coupled to black steel bars in relatively chloride-free concrete at the bottom of the slab; the best case was when galvanized bars were used in both the top and bottom mats (S. R. Yeomans 2004) (Pfeifer, Landgren and Zoob 1987).

While damage to galvanized coating is possible through small chips or scratches, it is far less common than damage to ECR. Also, due to the protective, sacrificial properties of galvanizing, the consequences of such damage are far less severe than with traditional carbon steel.

Additionally, due to the robust nature of galvanized coatings, there are no special transportation and handling requirements except that appropriate bend radii need to be used to minimize cracking of the coating (S. R. Yeomans 2016). Minimum finished bend diameters are specified in ASTM A767 (ASTM 2016). The standard also specifies that all coating damaged due to fabrication and handling, up to the point of shipment to the job site, shall be repaired with a zinc-rich formulation in accordance with ASTM A780 (ASTM 2016).

3.2.1.4. Stainless Steel-clad Reinforcement

In an attempt to utilize the corrosion resistance of solid stainless steel without its high cost, SCR was developed. SCR can provide corrosion resistance comparable to solid stainless steel; however, the production of high quality SCR can be challenging as corrosion resistance can be diminished if the fabrication of the clad bar is not completed properly. Occasionally, gaps may form between the cladding and core material. This can lead to cracking and voids which can decrease corrosion resistance and ultimate service life. Additionally, the manufacturing process results in a stainless steel cladding along the length of the bars and an exposed carbon steel core at the ends of the bars. These ends must be sealed to prevent corrosion of the carbon steel core.

The results of early research conducted by Rasheeduzzafar indicated stainless-clad reinforcement had highly superior corrosion resistance compared to traditional galvanized and epoxy-coated bars. In the study, the four types of reinforcement were cast into concrete specimens with additions of sodium chloride to accelerate corrosion. The specimens were exposed to the coastal flats of Eastern Saudi Arabia and monitored until concrete failure. Cracking was noted in the traditional, galvanized, and epoxy-coated bars within 6 months, whereas no cracking was noted in the stainless steel-clad bar specimens after seven years (Rasheeduzzafar, Badar and Khan 1992).

A later study, conducted by the Virginia Transportation Research Council, aimed to verify the reported corrosion resistance of stainless-clad steel reported by

Rasheeduzzafar and others (Rasheeduzzafar, Badar and Khan 1992) (McDonald, Pfeifer and Sherman 1998), and to determine the chloride corrosion threshold of a specific brand of clad bar. The performance of clad bars embedded in concrete blocks subjected to weekly cycles of ponding/drying in saturated sodium chloride solution was compared to parallel samples of traditional black steel and stainless steel. After close to three years of testing, the clad and solid stainless steel bars remained passive while the traditional black bars started showing signs of corrosion within the first 90 days. The mean chloride content for each reinforcement type was estimated for the black bars at corrosion initiation at and for the other bars at the end of the testing period. The estimated chloride content for the concrete containing the clad and solid stainless steel bars was 0.5990 (by weight of concrete), with no signs of corrosion; in contrast, the estimated chloride content of the traditional black bars was only 0.0430 with visible signs of corrosion. With these results the researchers concluded that the clad bars exhibited the same corrosion resistance as solid 316LN stainless steel and provided a chloride corrosion threshold at least 16 times that of traditional black steel (Clemena, Kukreja and Napier 2003).

The performance of SCR has shown to be superior to ECR. This is due to the toughness of the cladding, which prevents most of the chipping and cracking that is often seen during the handling of epoxy-coated bars. Damage to the cladding is still possible, however, and the material should be handled with care during fabrication to minimize damage (Darwin, et al. 2013). Also, the use of plastic caps instead of two-part epoxy for sealing the ends of clad bars can result in handling related corrosion issues. Plastic end caps can often fall off during handling and placement exposing the carbon core to potential chloride ingress.

Limited availability of stainless steel-clad bars seems to be one of its largest drawbacks. In the early 2000s, the Michigan DOT (MDOT) attempted to use stainless-clad reinforcement in several bridge construction projects. At the time, the only manufacturer of SCR was located in England, which increased lead time on acquiring materials. As the construction date of the first bridge project neared, MDOT was notified by the manufacturer that they would not receive all necessary SCR in time. This delay forced MDOT to switch to using solid stainless steel instead of SCR for the project. During a second attempt to use SCR in Michigan, materials were stockpiled well in advance of construction to avoid issues experienced by the previous project; however, #5 bars were the minimum size of SCR available, resulting in the use of solid stainless steel bars for all #3 bars required by the design (Kahl 2012). DOTs in South Dakota, Virginia, and Florida have also reported issues with availability of the product.

3.2.2. High-Chromium Steel

3.2.2.1. Stainless Steel Reinforcement

The main qualification for stainless steel alloys is a minimum chromium content of 11%. Four types of stainless steel are commercially available and classified based on their microstructure: martensitic, ferritic, austenitic, and duplex (ferritic-austenitic). The microstructure and properties of stainless steel are dependent on the type and concentrations of the different alloying elements. The corrosion resistance of martensitic and ferritic stainless steels is not sufficient for their use as reinforcing steel, and therefore these stainless types are not cost-effective alternatives to carbon steel rebar. Austenitic stainless steel contains 16–26% chromium and 6–22% nickel, while duplex steel contains 18–29% chromium and 4–9% nickel depending on specifications. Several grades also contain molybdenum, which increases pitting resistance. Austenitic grades are highly ductile and work-hardenable, whereas duplex steels are less ductile but stronger. The costs of the raw materials needed to produce stainless steel, specifically nickel and molybdenum, are high and variable. Manufacturers have experimented with new formulations in an attempt to reduce the price. The most common methods replace some portion of the nickel with manganese or reduce or eliminate molybdenum entirely (Hansson 2016).

Alloys with >11% chromium form an oxide layer of Cr_2O_3 instead of Fe_2O_3. The Cr_2O_3 oxide layer remains very thin and is not detectible by the naked eye. However, an iron oxide film will continue to grow and expand over time, resulting in visible rust. This oxide layer creates a passive layer preventing corrosion of stainless steel.

Stainless steel reinforcement is considerably more costly than traditional carbon steel; therefore, it is mainly used in harsh marine environments and climates that require the routine use of deicing salts. The earliest application of stainless steel found in literature was for a marine pier built from 1938 to 1941 in Progresso, Mexico. The steel used for this pier had a chemistry similar to AISI 304 steel, and the environment is considered one of the most corrosive exposures possible. A condition survey conducted in 2002 showed no signs of corrosion after nearly 60 years of marine exposure. Conversely, a parallel pier built 30 years later with black steel had completely disintegrated. The condition of both piers in 2002 is illustrated in Figure 3.4 (Hansson 2016).

Figure 3.4: Two piers built in Progresso, Mexico. Far right, pier built with stainless steel, completed in 1941. Left, remains of pier built with black steel 30 years later (Hansson 2016)

The stainless steel alloys specified by ASTM A955 provide varying levels of corrosion resistance, as the standard provides minimum requirements for each type. It is generally agreed that types 316L and 316LN can provide the greatest amount of corrosion resistance in harsh environments due to their high chromium and nickel contents and their incorporation of molybdenum. However, other types of stainless steels, such as 304, XM-28, and XM-29, have also shown good corrosion resistance in less severe environments despite containing lower amounts of chromium and nickel. These alloys are also available at a cost that is more comparable to carbon steel than the higher chromium/nickel content alloys. Therefore, it is important to consider the severity of the service environment, expected chloride exposure, and service life when specifying a certain alloy (CRSI 2012) (Serdar, Zulj and Bjegovic 2013) (Hansson 2016).

Serdar et al. (Serdar, Zulj and Bjegovic 2013) evaluated the long-term corrosion behavior of six stainless reinforcing steels that had been embedded in mortar, exposed to chloride media and monitored for two years. The steels evaluated were AISI types 410, 204Cu, 2101, 304, 2304, and 316. A summary of key chemical components of these steel types is listed in Table 3.1. The samples were partially submerged in a 3.5% NaCl solution for two years. At the end of this period AISI 2304 (duplex) showed similar corrosion resistance to AISI 410 (austenitic). Another duplex steel, AISI 2101, showed good long-term corrosion resistance compared to AISI 410, but lower than the austenitic steels AISI 316 and 304.

Grimault et al. evaluated the stress corrosion cracks by hydrogen embrittlement of two types of stainless steels (austenitic and duplex steels). The two steels were placed into a corrosive environment and placed under stress to 80% of their strength. The austenitic steel failed at a much earlier time than the duplex steel;

however, both steels evaluated after one week showed damage. It was determined that strain induced martensite has an influence on hydrogen embrittlement and stress corrosion cracking of the two stainless steels in this study (Grimault et a. 2011).

Table 3.1: Chemical composition of evaluated steels [adapted from (Serdar, Zulj and Bjegovic 2013)]

Steel Grade (AISI)	Chemical Composition, wt%				
	C	Cr	Ni	Mo	Mn
410	0.018	12.37	0.46	0.02	0.56
316	0.027	15.82	12.33	2.60	1.98
204Cu	0.038	16.23	2.11	0.32	7.94
304	0.058	18.24	7.93	0.04	1.46
2101	0.045	19.88	1.22	0.10	5.28
2304	0.020	22.22	3.57	0.28	1.09

Hansson (Hansson 2016) discusses the challenges of evaluating the long-term performance of stainless steel. Considering the anticipated 75- to 100-year lifetime of a stainless steel reinforced structure, the use of accelerated test methods are necessary to estimate corrosion resistance. Unfortunately, any type of acceleration method introduces unrealistic variables that can underestimate chloride threshold limit and time to corrosion. Some commonly used accelerated test methods for evaluating stainless steel reinforcement are described below.

3.2.2.1.1. Rapid Macrocell Test per ASTM A955

All stainless steel reinforcing bars sold in North American must comply with the corrosion criteria specified within ASTM A955. The corrosion resistance shall be evaluated based on the criteria described in either A1.2 or A1.3 as follows:

> A1.2 A rapid macrocell test series shall consist of a minimum of five specimens. Using the rapid macrocell test of bars in a simulated concrete pore 15% sodium chloride solution over a 15-week period...[as illustrated in Figure 3.5 of this report], the average corrosion rate for the test series shall at no time during the test exceed 0.25 μm/year, with no single specimen exceeding a corrosion rate of 0.50 μm/year. (ASTM 2018)
>
> A1.3 A cracked beam test series shall consist of a minimum of five specimens. Using the cracked beam test as specified in Annex A3 [of ASTM A955]...the average corrosion rate for the test series shall at no time during the test exceed 0.20 μm/year, with no single specimen exceeding a corrosion rate of 0.50 μm/year. (ASTM 2018)

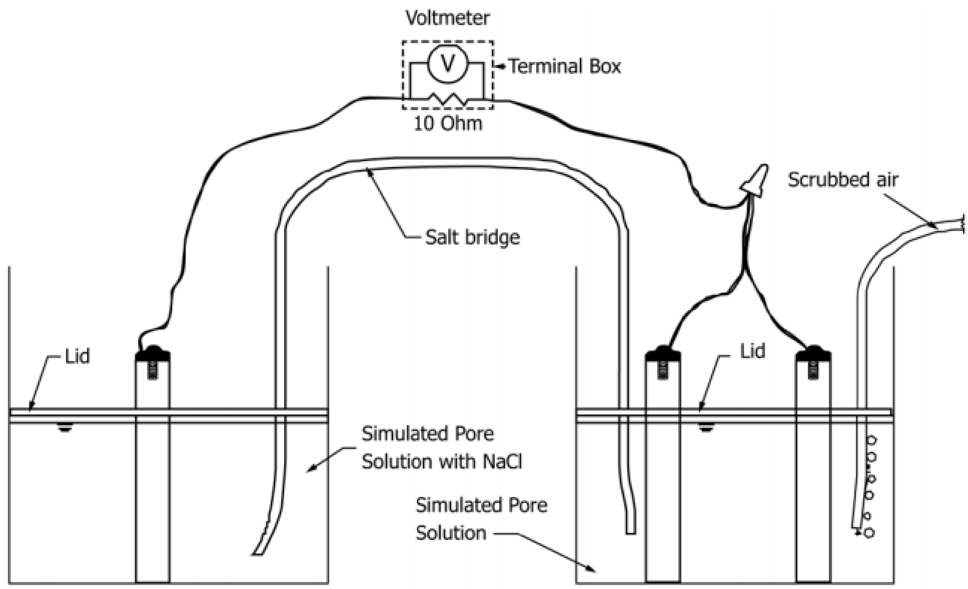

Figure 3.5: Experimental set-up for the ASTM A955 Macrocell Test, version A1.2 (ASTM 2018)

Method A1.3 is illustrated in Figure 3.6. In this test, a prism specimen is cast with a 0.03 mm stainless steel shim parallel to and above the top rebar to create a standard "crack" after the shim is removed from the hardened concrete. Each week, the ponding well is filled with 15% NaCl for four days, and the potential drop across the 10 Ω standard resistor is recorded and converted to corrosion rate in μm/year. A three-day drying period is then initiated by siphoning the NaCl solution from the ponding well and the prism is held at 100.4°F (38°C). A series of different exposure cycles are followed for a 75-week period. Hansson (Hansson 2016) states that few researchers tend to use this method. Instead, most investigations have been conducted on uncracked concrete or mortar. However, structural concrete in the field is always cracked so it relevant to attempt to recreate the same type of exposure. The issue with the method described in ASTM A955 is that the "shim" produces a parallel-sided slot that allows direct access of the exposure solution to the rebar rather than the more tortuous path provided by naturally occurring cracks in the field.

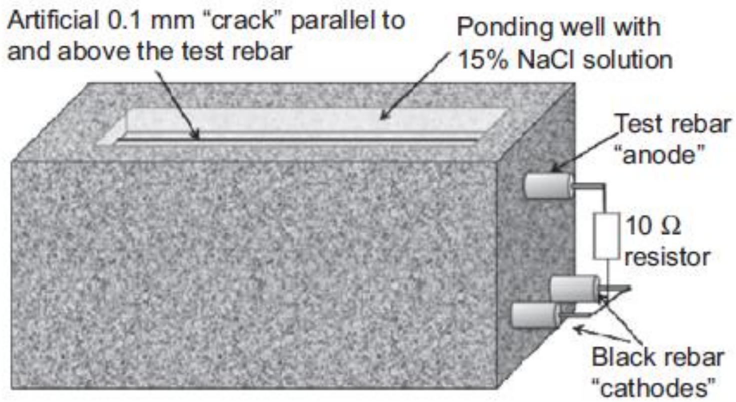

Figure 3.6: Schematic representation of prism for macrocell corrosion testing according to ASTM A955 A1.2 (ASTM 2018) (Islam, Bergsma and Hansson 2013)

3.2.2.1.2. Critical Chloride Threshold Level (CCTL) Testing

The simplest method for testing the CCTL of stainless steel is by measuring the corrosion current density of the steel in synthetic pore solution with increasing chloride additions, otherwise known as the potentiostatic method. Levels calculated with this method will always be significantly higher than those measured in mortar or concrete, because the chlorides are largely concentrated in the pore solution. CCTL values can vary greatly depending on evaluation method used and surface condition prior to testing. Table 4.3 of (Hansson 2016) summarized the CCTL results of several investigations. The results ranged from 3 to 15% by weight cement or solution. Generally, the CCTL is higher for 304 and 316 stainless steels compared to the 302 and 315 grades. The CCTL of duplex steels, 2304 and 2101, tends to fall between the average values of the low and high alloyed austenitic steels.

3.2.2.2. Low-carbon Chromium Reinforcement

In the early 1990s, the FHWA, American Iron and Steel Institute, and the US Navy collaborated on a project to develop low-carbon, high-performance steels for use in bridge construction. From this research emerged a new type of steel generically referred to as low-carbon chromium steel, or ASTM A1035 Types CS, CM, and CL reinforcement. The chemical requirements and controlled rolling manufacturing process specified by ASTM A1035 result in a steel with a fine-grained lath martensite/austenite microstructure (Van Dyke, et al. 2017) (CRSI 2017), with compositions as shown in Table 3.2. Higher chromium contents provide greater corrosion resistance compared to traditional carbon steel.

Table 3.2: Chemical composition of alloy types for ASTM A1035 steel: maximum composition by percentage (ASTM 2016)

Alloy Type	Carbon	Chromium	Manganese	Nitrogen	Phosphorus	Sulfur	Silicon
CL	0.3	2.0 – 3.9	1.5	0.05	0.035	0.045	0.5
CM	0.2	4.0 – 7.9					
CS	0.15	8.0 – 10.9					

Since low-carbon chromium steel is a relatively new alloy, less performance and corrosion data exists for the material compared to traditional black steel. The consensus among those that have investigated the corrosion resistance of ASTM A1035 is that it its corrosion performance is on the range of two to ten times greater than black steel (Shahrooz, et al. 2011). For comparison, ASTM A955 316 stainless steel is considered to have a corrosion performance of >20 times that of black steel. Clemena and Virmani (Clemena and Virmani 2004) investigated the corrosion resistance and chloride threshold of several types of reinforcing steels embedded in concrete test blocks subjected to weekly cycles of ponding with a saturated solution of NaCl for 3 days and drying for 4 days. Based on the observed time to corrosion it was estimated that ASTM A1035 reinforcement types were capable of resisting 4.7 to 6.0 times more chloride ions than carbon steel.

3.2.3. FRP Bars

The use of composites expanded after World War II in the 1940s. Various industries took advantage of the high strength and light weight of composite materials, but it was not until the expansion of the national highway system in the 1950s and 1960s that these materials were seriously considered for use as reinforcement in concrete. Agencies concerned with corrosion of reinforcing steel due to the use of deicing salts investigated several materials as an alternative to black steel; however, epoxy-coated steel was determined to be the most viable option and FRP bars were not widely used commercially until the 1970s. In the 1980s, a resurgence in interest arose when new developments were launched to apply FRP reinforcing bars in concrete that had special performance requirements, such as nonmagnetic properties or in areas that were subjected to severe chemical attack. FRP reinforcing bars have been most notably used in applications where electrically nonconductive reinforcement is necessary, such as in facilities for MRI equipment; however, their use in bridge construction has steadily increased since the 1990s (ACI 440.1R-15 2015).

FRP bars are generally considered immune to electrochemical corrosion, as well as resistant to chloride ion and chemical attack, and provide significant cost savings over the use of stainless steel reinforcement. Due to the linear elastic stress-strain behavior of FRP (i.e., brittle failure, no yielding), ACI Committee 440 (ACI 440.1R-15 2015) suggests the use of FRP reinforcement should be limited to structures that will significantly benefit from its other properties, such as the noncorrosive or nonconductive behavior. FRP bars have shown to have excellent resistance to chloride-induced corrosion; however, they are prone to alkali-induced degradation. This is discussed more fully in Section 4.2.3 of this report.

3.3. Field Performance of Alternative Reinforcement Exposed to Texas Gulf Coast Marine Environment

3.3.1. Introduction

Monitoring concrete in large-scale field exposure is the most effective means of obtaining realistic data regarding material performance, as accelerated laboratory test methods often provide inaccurate assessments of long-term performance. Sadly, it is often difficult to find sufficient outdoor space and/or funding for the establishment of outdoor exposure sites. In fact, until recently there were only three large-scale marine exposure sites in North America. One of the most notable marine exposure sites was developed by the US Army Corps in the 1950s on Treat Island, Maine. This site has provided a wealth of information regarding the performance of concrete subjected to very cold waters as well as information regarding the performance of concrete in aggressive freeze/thaw cycle zones. However, less information is available for transportation structures exposed to marine or other similarly aggressive environments in warm water climates. Fortunately, The University of Texas was able to establish a new site in Port Aransas, TX in 2014, and with time the site will be able to produce valuable long-term performance data for various types of binders and repair materials subjected to marine environments.

In 2013 a comprehensive feasibility study was conducted to identify the specific areas related to marine exposure in need of further research to advance the current practice and use of materials in such environments. As a result of this study, the Texas Marine Exposure Site (TEXMEX) was developed and established in 2014. TEXMEX is located along the ship channel that connects Port Aransas to the Gulf of Mexico on property owned by The University of Texas Marine Science Institute. TEXMEX is a real-world proving ground aimed at extending the service life of transportation structures exposed to marine environments or similar aggressive environments, such as those exposed to chloride deicing salts. This site is one of only four marine exposure sites located in North America. Currently, the site focuses on evaluating corrosion of concrete-embedded steel reinforcing, chloride resistance of cementitious materials, and the potential for alkali-silica reactivity and carbonation in concrete exposed to marine environments; however, the site has the potential to also evaluate structural steel, signage, lighting, and other infrastructure-related components and materials.

The site was established out of a need for more data on marine exposure in warm weather sites. Most data from marine exposure sites has come from very cold sites, such as Treat Island, Maine. The data obtained from this newly established exposure site will help better define the specific exposure conditions along the Texas coastline and provide more information on how other durability mechanisms, such as alkali-silica reaction, delayed ettringite formation, and carbonation, are affected by marine exposure.

As listed in Table 3.3, currently four types of alternative reinforcement are under evaluation at the TEXMEX site: dual-coated, galvanized, stainless steel, and low-carbon chromium. A traditional carbon steel is also under evaluation for comparison.

Table 3.3: Alternative reinforcement under evaluation at TEXMEX

Type/Brand	Governing Standard	Grade	Size
Carbon Steel	ASTM A615	60	#4
Dual-coated (Z-bar)	ASTM A1055/A615	60	#4
Galvanized	ASTM A767/A615	60	#4
Stainless Steel	ASTM A955	60	#4
Low-carbon Chromium (MMFX)	ASTM A1035 CS	100	#4

3.3.2. Marine Exposure Blocks

Two large-scale concrete blocks were cast for each type of alternative reinforcement. The dimensions of the marine exposure blocks are 305 x 150 x 1145 mm (12 x 6 x 45 in.). One block was designed to evaluate reinforcement performance while the other was intended to evaluate resistance to chloride penetration. The reinforced block was instrumented with four reinforcement bars, each placed at different depths to provide 12.5, 25, 37.5, and 50 mm (½, 1, 1-1/2, and 2 in.) of cover. No. 4 black steel reinforcement was used in all reinforced marine exposure blocks. The end of each reinforcement bar was tapped and threaded to allow a threaded stainless steel rod to be inserted. This serves two purposes: 1) it provides a means for accurately securing the reinforcement bars at the exact cover, and 2) it provides a connection between the stainless steel (which is accessible on the outside of the concrete) with the steel being evaluated. This connection helps with non-destructive techniques such as half-cell potential. A stainless steel bar was cast in the center of each block. This bar was hooked after casting and allows for hanging the blocks off the side of the ferry channel wall. Figures 3.7 and 3.8 show the layout of the steel within the reinforced exposure blocks.

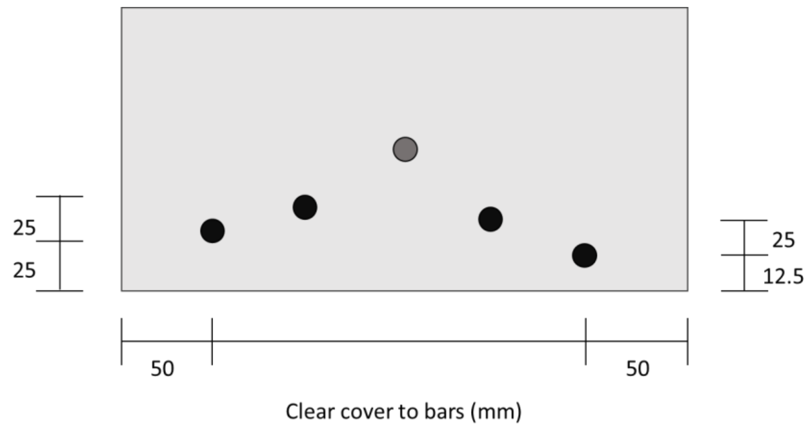

Figure 3.7: Diagram of steel reinforcement locations within reinforced marine blocks

Figure 3.8: Photo of marine exposure block showing locations of reinforcing steel

After casting, the exposure blocks were cured in the laboratory at ambient temperature for approximately 28 days before being placed at the exposure site in Port Aransas. The hooked bar cast into each block was used to attach each exposure block to a chain that is anchored into the top side of the ferry channel wall. This allows for submersion of approximately one-half of each exposure block as illustrated in Figure 3.9. The blocks are placed such that the mid-tide level is situated at or slightly below the mid-height of the specimen; the zone at or immediately above the high-tide level has been found to be the most aggressive in terms of inducing corrosion of the embedded reinforcement.

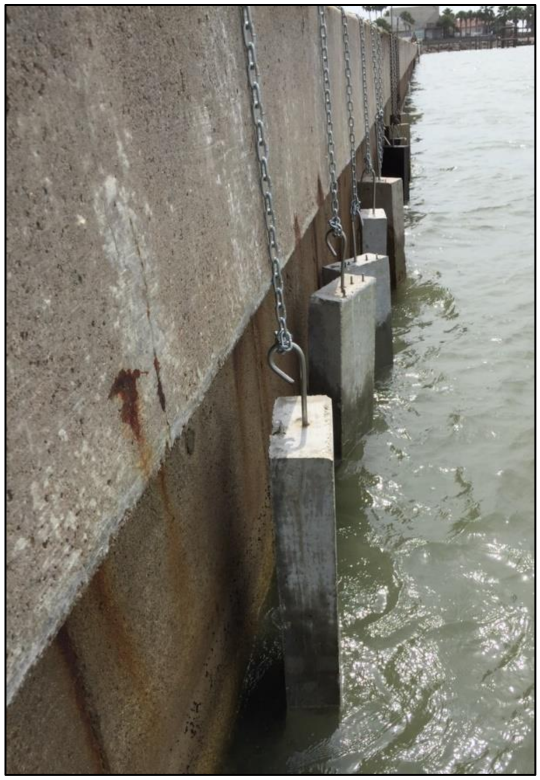

Figure 3.9: Photo showing partially submerged marine exposure blocks

3.3.3. Visual Observations

A visual assessment of the blocks was conducted each time they were removed from the water. The assessment included the identification of crack locations and widths as well as the locations of rust staining.

3.3.4. Evaluation of Corrosion Resistance

The corrosion potential of the embedded steel was evaluated through half-cell potential measurements. Initial measurements were taken when the exposure blocks were delivered to the site. Visual assessments and subsequent half-cell potential measurements were taken periodically every year. Corrosion potential measurements were obtained per ASTM C876 *Standard Test Method for Corrosion Potentials of Uncoated Reinforcing Steel in Concrete* (ASTM 2009).

3.3.5. Results and Conclusions

A copper-copper sulfate reference electrode was used to determine the corrosion potential values for the marine exposure blocks. The measured corrosion potential of each alternative reinforcement at various cover depths can be interpreted to determine a relative rate of corrosion as described in ASTM C876 and listed in Table 3.4. The corrosion potentials and corresponding rates of corrosion listed in this table were color-coded; these colors were then applied to Tables 3.5 through 3.7 to illustrate the relative progression of corrosion over time. The blocks were deployed to the exposure site over a seven-month period; therefore, measurements of the blocks were recorded at various exposure dates.

In August of 2017 Hurricane Harvey struck the Gulf Coast of Texas. The storm made landfall in Port Aransas as a Category 4 hurricane, resulting in extensive damage to the city and our exposure site. Unfortunately, the galvanized and low-carbon chromium exposure blocks were washed out to sea during the storm. The remaining blocks were measured in April 2018 and are discussed below.

The last reported measurements were taken in April 2018 when the period of exposure for each block ranged from 17 to 24 months. Tables 3.5 through 3.7 show the progression of corrosion potential of each block over the 17- or 24-month period. The readings indicate high levels of corrosion activity early on for the two blocks containing galvanized steel and low-carbon chromium steel; however, the initial high readings may be the result of preliminary corrosion taking place to form a passivation layer on the steel (Broomfield 2007). This phenomenon could also explain the shift to a more positive corrosion potential between the initial measurements and those taken at seven months for the dual-coated steel blocks.

Table 3.7 shows the latest reported corrosion potential of the remaining exposure blocks. All of the stainless steel and dual-coated blocks continue to show a low rate of corrosion after 24 months of exposure. The visual inspections of these blocks showed no outward signs of corrosion such as rust staining or cracking. Both of the black bar exposure blocks showed a severe rate of corrosion at a cover depth of 0.5-in. Some rust staining was evident at 0.5-in cover on the black bar with the fly ash exposure block. It is of interest to note that the corrosion potentials of the dual-coated and stainless steel reinforced blocks have generally become more positive over the 24-month exposure period. This shows an increase in corrosion resistance over time. Meanwhile, a steady increase in rate of corrosion is observed in the specimens reinforced with traditional black bar.

Table 3.4: Corrosion condition/rate based on Corrosion Potential Measurements

Cu/CuSO3 Reference Electrode	
Corrosion Potential (mV)	**Rate of Corrosion**
> -200	Low (<10%) rate of corrosion
-200 to -350	Intermediate (~50%) rate of corrosion
<-350 to -500	High (>90%) rate of corrosion
<-500	Severe rate of corrosion

Table 3.5: Initial half-cell potential readings for marine exposure blocks

Cover Depth (in)	Half-Cell Potential (mV)									
	Black (Control)	Black w/Ash	Dual-coated	Dual-coated w/Ash	Galvanized	Glavanized w/Ash	Stainless	Stainless w/Ash	LCC	LCC w/Ash
0.5	-90	-350	-230	-160	-740	-800	-160	-160	-310	-390
1.0	150	-310	-230	-190	-560	-50	-190	-130	-430	-340
1.5	-100	-370	-300	-210	-500	-720	-200	-160	-510	-380
2.0	-120	-310	-310	-220	-260	-200	-200	-160	-650	-430
Months of exposure	0 months expsoure - initial half cell potential readings									

Table 3.6: Half-cell potential readings at 7 months

Cover Depth (in)	Half-Cell Potential (mV)					
	Black (Control)	Black w/Ash	Dual-coated	Dual-coated w/Ash	Stainless	Stainless w/Ash
0.5	-320	---	-120	-60	-60	-60
1.0	-290	---	-30	-110	-10	-70
1.5	-260	---	-80	-70	-30	-80
2.0	-300	---	-180	-280	-20	-70
Months of exposure	7	---	7	7	7	7

Table 3.7: Half-cell potential readings at 17–24 months

Cover Depth (in)	Half-Cell Potential (mV)					
	Black (Control)	Black w/Ash	Dual-coated	Dual-coated w/Ash	Stainless	Stainless w/Ash
0.5	-648	-577	-80	118	-86	15
1.0	-458	-514	-129	-111	-50	20
1.5	-502	-380	142	159	-33	-9
2.0	-400	-127	96	85	34	-47
Months of exposure	24	17	24	24	24	24

Chapter 4. Structural Performance of Alternative Reinforcement

4.1. Introduction

The mechanical properties of reinforcing bars have a critical influence on the performance of reinforced concrete structures. The ultimate strength and ductility of reinforced concrete members are typically controlled by the yield strength and elongation capacity of reinforcing bars, while deflections depend to a large extent on the elastic modulus of tension reinforcement. In addition, the bond behavior between the concrete and the reinforcing bar affects both the serviceability (cracking) and strength design (required development and lap-splice lengths) of reinforced concrete members.

The alternative reinforcement materials discussed in previous sections provide enhanced corrosion resistance compared to traditional carbon steel. This is achieved through either altering the chemistry of more traditional steels or by embracing new composite materials. These changes in chemistry or material selection may result in different mechanical and structural properties that need to be taken into account in structural design. The following section provides a review of the mechanical properties and other relevant structural aspects of the alternative reinforcement materials considered in this study.

4.2. Mechanical Properties of Alternative Reinforcement

The strength, ductility, and weldability of steel are some of the most important mechanical properties necessary for design and construction of reinforced concrete structures. Acceptable limits for yield and tensile strengths of all steel used for reinforcement are specified by ASTM standards. Bending test requirements are also specified to ensure proper ductility is provided.

4.2.1. Coated Reinforcement

4.2.1.1. Epoxy-coated and Dual-coated Reinforcement

The strength of coated reinforcements is the same (for design purposes) as the equivalent grade of black bar, regardless of coating type. ASTM standards for epoxy-coated bars (ASTM A775 or A934) and dual-coated bars (ASTM A1055) require the steel reinforcing bars to be coated to meet the material specifications for black bars (ASTM A615, A706, or A996). Design guidelines described in ACI 318 *Building Code Requirements for Structural Concrete* and the American Association of State Highway and Transportation Officials (AASHTO) publication *LRFD Bridge Design Specifications* treat epoxy-coated, zinc and epoxy dual-coated, and galvanized steel bars the same as black bars in terms of structural design, except that additional development length is required for bars with epoxy coating and dual coating, as discussed in Section 4.3.

4.2.1.2. Galvanized Reinforcement

Structural design guidelines treat galvanized bars like black bars. ASTM standards for galvanized bars (ASTM A767 or A1094) require the steel reinforcing bars to be coated to meet the material specifications for black bars (ASTM A615, A706, or A996). However, bars bent prior to galvanizing may be affected by strain-aging embrittlement, i.e., reduction of ductility. Strain-aging embrittlement is caused by the combination of high stresses and elevated temperatures. Stresses are induced into the steel by cold working or bending bars during fabrication. Exposure to a high heat source, such as the galvanizing kettle, provides the second component necessary to instigate strain-aging embrittlement. Rebar is more susceptible to strain aging than other galvanized steels because it is commonly made from low quality steel. These types of steel are more susceptible to strain aging because the steel has many impurities that congregate at the highly stressed points in the steel, making strain-aging more likely. To this end, excessive cold-working should be avoided prior to galvanizing (American Galvanizers Association 2011). For this reason, ASTM 767 limits the bend diameter of bars that are bent prior to galvanizing to between 6 and 10 times the nominal diameter of the bar (d_b) depending on the steel grade and bar size. The reinforcement may be fabricated with smaller bend diameters, but then the bars shall be stress relieved at a temperature of 900 to 1050°F for one hour per inch of bar diameter.

4.2.1.3. Stainless Steel-clad Reinforcement

Stainless steel-clad bars undergo a distinctly different manufacturing process than the other types of coated reinforcement. Rather than applying a coating to an ASTM-compliant reinforcing bar, the stainless steel cladding is manufactured first, and steel filings are used to fill the cladding. The filings are heated and compacted, and then the composite material is hot-rolled to create a steel reinforcement bar that complies with ASTM A955 and ASTM A615 (Clemena, Kukreja and Napier 2003) (NX Infrastructure Ltd. 2008). Theoretically, the mechanical properties of stainless steel-clad bars should be comparable to equivalent grade 60 black bars; however, the production of high-quality SCR can be challenging as corrosion resistance and the concrete/steel bond strength can be diminished if the fabrication of the clad bar is not properly done (Tanks and Sharp 2015).

Mechanical testing conducted by the South Dakota DOT showed that the core and cladding behaved separately, with the core failing prior to the cladding in tensile testing. Figure 4.1 shows the bar failure after the test. They also noted that the yield strength of the bars had only a 67% probability of being greater than that required by the AASHTO MP13 specification for stainless-steel clad bars (Cross, et al. 2001). Later research conducted by the Virginia DOT showed an improvement in the production process that provided stainless steel-clad bars with comparable mechanical properties to equivalent carbon steel bars (Clemena, Kukreja and Napier 2003).

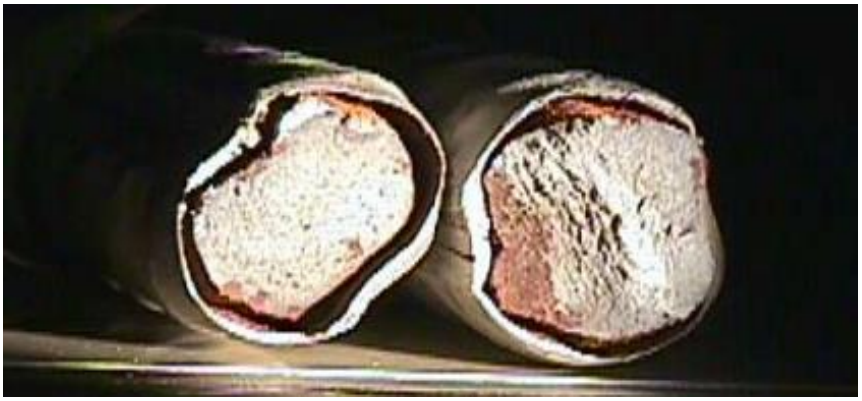

Figure 4.1: Stainless steel-clad bar failure following tensile test (Cross, et al. 2001)

4.2.2. High-Chromium Steel Bar

4.2.2.1. Stainless Steel Reinforcement

The characteristics of stainless steel reinforcement are specified by ASTM A955. Stainless steel bars conforming to this specification are of two minimum yield strength levels: 60,000 psi and 75,000 psi, designated as Grade 60 and Grade 75, respectively. Some advantages to stainless steel reinforcement are good ductility, homogeneity of microstructure, cut ends that do not have to be protected, and good weldability. Stainless steel bars have greater ductility and a greater capacity for work hardening than conventional steel bars. The minimum elongation required by ASTM A955 is 20%, which is significantly higher than that required for black bars in ASTM A615 or A706.

Presently, ACI 318 (ACI Committee 318 2014) and AASHTO LRFD (American Association of State Highway and Transportation Officials (AASHTO) 2017) specifications generally treat stainless steel reinforcing bars the same as carbon steel reinforcing bars in terms of structural design. The appropriate yield strength will have to be used in the design computations. Stainless steel does not present a well-defined yield plateau, as shown in Figure 4.2. The yield strength is taken as either the stress value corresponding to a tensile strain of 0.0035 or as the intersection point of the stress-strain curve with a line parallel to the elastic branch with a 0.2% offset strain, as illustrated in Figure 4.2.

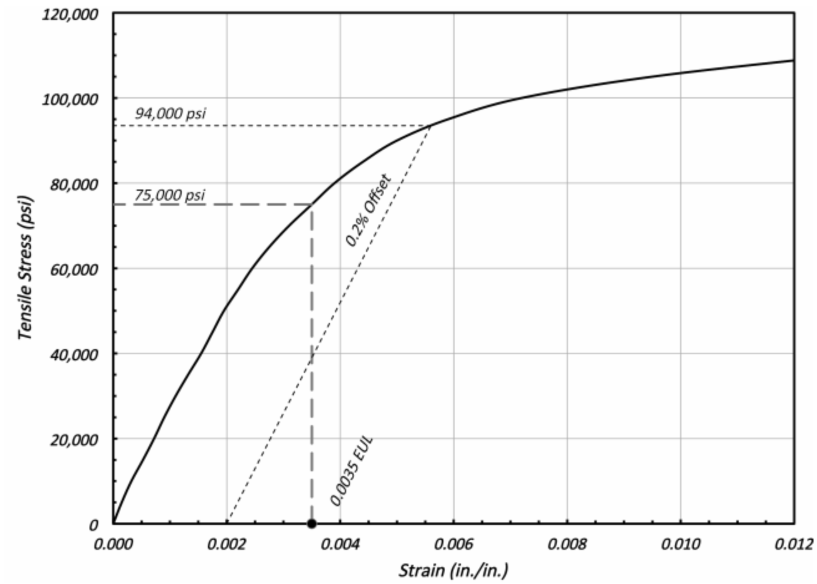

Figure 4.2: Representative tensile stress-strain behavior and equivalent yield strength of stainless steel (CRSI 2012)

4.2.2.2. Low-carbon Chromium Reinforcement

Low-carbon chromium steel reinforcement is specified under ASTM A1035. The chemical and manufacturing requirements specified by this standard result in a high-strength reinforcing steel with two available yield strengths: 100 ksi and 120 ksi. High-strength reinforcing steels, such as low-carbon chromium steel, often do not exhibit a distinct yield plateau. Since the yield point is less well defined than for conventional steels, the yield strength needs to be calculated with the 0.2% method shown in Figure 4.2.

The International Code Councilevaluated ASTM A1035 Grade 100 steel for its compliance with the International Building Code from 2009, 2012, and 2015 (International Code Council 2017). The following limitations for the use of ASTM A1035 reinforcement were identified:

- The reinforcing bars must not be used as longitudinal reinforcement in special moment frame members, special structural wall boundary elements, or coupling beams.
- ASTM A1035-16b Types CS, CM, and CL Grade 100 reinforcing bars must not be welded.
- ASTM A1035-16b Types CS, CM, and CL Grade 100 reinforcing bars must not be used as headed deformed bars in tension.
- The specified compressive strength for concrete must range from 4,000 psi to 12,000 psi.

Additionally, the National Cooperative Highway Research Program (NCHRP) conducted an extensive review of low-carbon chromium steel and the use of high-strength reinforcing steel in AASHTO *LRFD Bridge Design Specifications*. After extensive review of current literature and research, NCHRP provided recommendations for inclusion in the AASHTO *LRFD Bridge Design Specifications* that will permit the use of high-strength reinforcing steel with specified yield strength not greater than 100 ksi (Shahrooz, et al. 2011). Based on this research, AASHTO *LRFD Bridge Design Specifications* (American Association of State Highway and Transportation Officials (AASHTO) 2017) allow the use of reinforcing steel with specified yield strengths of up to 100 ksi.

ACI 318-14 *Building Code Requirements for Structural Concrete* (ACI Committee 318 2014) allow the use of ASTM A1035 reinforcement as transverse reinforcement for confinement in special earthquake-resistant structural systems and spirals in columns. The maximum value of the specified yield strength permitted for the design calculation is 100 ksi.

4.2.3. FRP Bars

GFRP bars exhibit a linear elastic behavior in tension, i.e., they behave elastically until rupture. The typical tensile strength of GFRP varies between 70 and 230 ksi (ACI 440.1R-15 2015), which is in general higher than that of steel. However, the elastic modulus of GFRP bars is 4 to 6 times smaller. Figure 4.3 shows the moment-curvature relations of a concrete section reinforced with steel and GFRP bars. For a similar area of reinforcement, GFRP bars provide a higher moment strength than steel bars, but lower ductility and flexural stiffness. As a result, permissible deflections under service loads may control the design of FRP-reinforced sections (ACI 440.1R-15 2015).

Table 4.2.1—Typical tensile properties of reinforcing bars[*]

	Steel	GFRP	CFRP	AFRP
Nominal yield stress, ksi (MPa)	40 to 75 (276 to 517)	NA	NA	NA
Tensile strength, ksi (MPa)	70 to 100 (483 to 1600)	70 to 230 (483 to 690)	87 to 535 (600 to 3690)	250 to 368 (1720 to 2540)
Elastic modulus, $\times 10^3$ ksi (GPa)	29.0 (200.0)	5.1 to 7.4 (35.0 to 51.0)	15.9 to 84.0 (120.0 to 580.0)	6.0 to 18.2 (41.0 to 125.0)
Yield strain, percent	0.14 to 0.25	NA	NA	NA
Rupture strain, percent	6.0 to 12.0	1.2 to 3.1	0.5 to 1.7	1.9 to 4.4

[*]Typical values for fiber volume fraction ranging from 0.5 to 0.7.

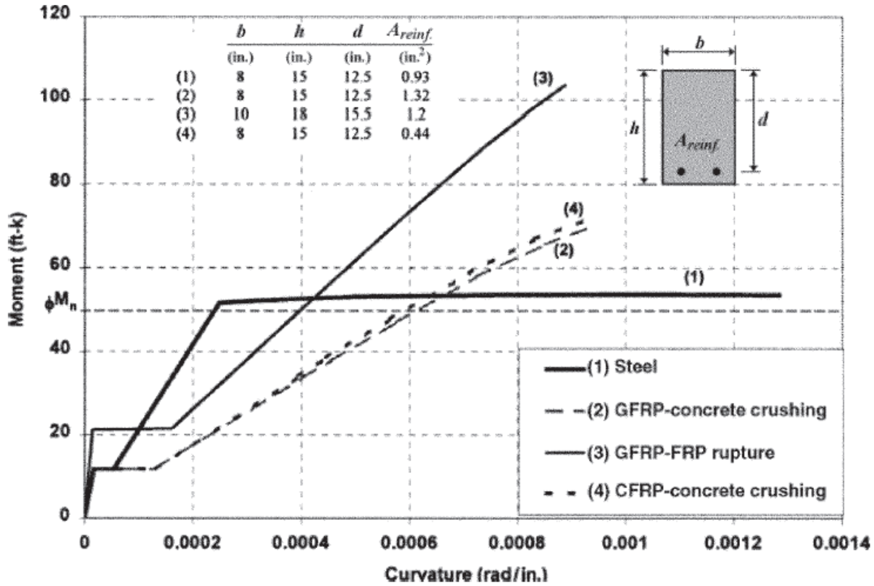

Figure 4.3: Moment-curvature relationship for RC sections using steel and GFRP bars (ACI 440.1R-15 2015)

The mechanical properties of FRP bars can vary greatly depending on the type of fiber used. Table 4.1 shows the typical tensile properties of several FRP bars compared to steel. FRP bars cannot be bent once they have been manufactured, but they can be fabricated with bends. In FRP bars produced with bends, a strength reduction of 40 to 50%, compared with the tensile strength of a straight bar, can occur in the bend portion due to fiber bending and stress concentrations (ACI 440.1R-15 2015) (Nanni, et al. 1998). BFRP reinforcement has been developed recently as a potential alternative to GFRP. Basalt fibers provide comparable modulus and strength, improved strain to failure, and lower cost compared to GFRP. However, the mechanical properties of basalt fibers are highly dependent on the source where the basalt rock is mined. Most BFRP that has been marketed in the US is from rock mined in the Ukraine. Long-term availability of this high-quality basalt is uncertain.

Table 4.1: Typical tensile properties of reinforcing bars [from (ACI 440.1R-15 2015)]

	Steel	GFRP	CFRP	AFRP
Nominal yield stress, ksi	40 to 75	n/a	n/a	n/a
Tensile strength, ksi	70 to 100	70 to 230	87 to 535	250 to 368
Elastic modulus, x10^3 ksi	29.0	5.1 to 7.4	15.9 to 84.0	6.0 to 18.2
Yield strain, percent	0.14 to 0.25	n/a	n/a	n/a
Rupture strain, percent	6.0 to 12.0	1.2 to 3.1	0.5 to 1.7	1.9 to 4.4

FRP reinforcing bars subjected to a constant tension over time can suddenly fail after a time period called the endurance time. This phenomenon is known as creep rupture or static fatigue. In general, carbon fibers are the least susceptible to creep rupture, whereas aramid fibers are moderately susceptible, and glass fibers are the most susceptible (ACI 440.1R-15 2015). BFRP reinforcement has been developed recently as a potential alternative to GFRP.

Long-term exposure to the high pH inside concrete and high levels of moisture are known to degrade the tensile strength and stiffness of FRP bars. For this reason, the material properties used in design equations should be calculated with the environmental reduction factors provided by ACI 440. The reduction factors listed in Table 4.2 are based on the type of FRP and exposure conditions.

Table 4.2: ACI 440 environmental reduction factors for tensile property design calculations (ACI 440.1R-15 2015)

Exposure Condition	Fiber Type	Environmental reduction factor C_E
Concrete not exposed to earth and weather	Carbon	1.0
	Glass	0.8
	Aramid	0.9
Concrete exposed to earth and weather	Carbon	0.9
	Glass	0.7
	Aramid	0.8

The glass fibers in GFRP are damaged due to a combination of two processes: (1) chemical attack on the glass fibers by the alkaline cement environment; and (2) concentration and growth of hydration products between individual filaments. The embrittlement of fibers is due to the nucleation of calcium hydroxide on the fiber surface (Benmokrane, et al. 2002).

Most test methods that aim to accelerate the degradation of FRP rely upon exposure to higher temperatures to spur the corrosion process, with the tensile behavior and capacity evaluated as a function of temperature and time of testing. Because it is assumed that degradation of FRP is caused by one primary chemical reaction, researchers have used an Arrhenius approach to analyze the test results and extrapolate to real-world conditions. This approach assumes that the Arrhenius time temperature relationship is valid for the whole temperature range considered. Recently, Huang and Aboutaha (Huang and Aboutaha 2010) utilized a new scientific approach in an attempt to obtain accurate environmental reduction factors by taking the application temperature and humidity into account. The study concluded that current environmental reduction factors specified in ACI 440 achieve sufficient safety margin for the majority of concrete applications.

Exposure of FRP bars to ultraviolet rays and moisture before their placement in concrete could also adversely affect their tensile strength due to the degradation of the polymer constituents, including aramid fibers and all resins.

4.3. Bond and Development Length of Alternative Reinforcement

Bond of reinforcement affects both the in-service (cracking) and ultimate response (development length) of reinforced concrete members. Bond is provided by (i) chemical adhesion between the bar and the concrete, (ii) frictional forces at the bar surface, and (iii) mechanical interlock provided by the bar ribs. The different material and/or surface characteristics of alternative reinforcement may affect the bond performance of the bars. This section reviews the bond properties of alternative reinforcement and its design implications in development and lap-splice length requirements.

4.3.1. Coated Reinforcement

4.3.1.1. Epoxy-coated and Dual-coated Reinforcement

Bond performance of epoxy-coated bars has been extensively studied experimentally [e.g., (Johnston and Zia 1982), (Treece and Jirsa 1989), (Zuo and Darwin 2000)]. These studies have consistently shown that epoxy coatings result in lower bond strengths as compared to uncoated bars (ACI-ASCE 408 2003). For example, Johnston and Zia (Johnston and Zia 1982) studied the bonds of uncoated and epoxy-coated bars using slab and beam-end specimens. The bars were confined by transverse reinforcement, and the coating thickness of epoxy-coated bars varied between 6.7 and 11.1 mils (0.0067 and 0.00111 inches). The study found that the average bond strength of coated bars was about 4% lower than that of uncoated bars for the slab specimens and 15% lower for the beam-end specimens. In addition, the slab specimens with coated bars were shown to produce slightly larger deflections and wider cracks than those with uncoated bars. Treece and Jirsa (Treece and Jirsa 1989) studied the bonds of uncoated and epoxy-coated bars using beam-splice specimens without transverse reinforcement in the splice region. The epoxy-coated bars had coating thicknesses between 4.5 and 14 mils. On average, the bond strength of coated bars was 34% smaller than that of uncoated bars. The reduction bond strength of epoxy-coated bars can be explained by the lower coefficient friction between coated bars and the concrete. Idun and Darwin (Idun and Darwin 1999) obtained results indicating that the coefficient of friction of uncoated and coated reinforcing steel were 0.56 and 0.49, respectively.

Based on the reduced bond strength of epoxy-coated bars observed in tests, ACI 318 (ACI Committee 318 2014) and AASHTO LRFD (American Association of State Highway and Transportation Officials (AASHTO) 2017) require that the tension development length and the lap-splice length of bars of epoxy-coated bars be increased by 50% (for bars with cover less than 3 bar diameters or with clear spacing between bars less than 6db) or 20 % (for the rest of cases). Likewise, the development length of hooked bars and headed bars needs to be increased by 20% according. Additionally, ACI 318 requires the same modification factors for dual-coated (zinc and epoxy) bars.

4.3.1.2. Galvanized Reinforcement

Several studies conducted by Kayali and Yeomans (Kayali and Yeomans 1995) (Kayali and Yeomans 2000) investigated the bond of galvanized reinforcing steel in concrete. The results confirmed there is no adverse effect on bond with the use of galvanized steel. They also identified the very strong adhesion between concrete and the galvanized coating that is almost lacking between black steel and concrete. As a result, ACI 318 (ACI Committee 318 2014) does not require the use of any modification factors for the calculation of development length when designing with galvanized reinforcement. While galvanized bars are considered to provide comparable bond strengths as conventional bars, some studies [(Hamad and Mike 2003), (Hamad and Fakhran 2006)] have shown a bond reduction for galvanized bars in high-strength concrete. The authors of this study argued that there are mechanisms both positively and negatively affecting the adhesion between galvanized bars and normal-strength concrete, which result in an insignificant difference in bond strength with black bars, and that the role of mechanisms affecting positively adhesion in high-strength concrete is reduced.

4.3.1.3. Stainless Steel-clad Reinforcement

While there is much discussion regarding the bond between carbon steel and the outer cladding in SCR, little information exists on the steel-to-concrete bond for this material. With adequate bond between the core and cladding is achieved, it can be assumed that the concrete-steel bond for SCR would be similar to that of solid stainless steel reinforcement. However, debonding between the core and the cladding will limit the transfer of stresses between the reinforcement and the concrete.

4.3.2. High-chromium Steel Bars

4.3.2.1. Stainless Steel Reinforcement

A number of studies have shown that the bond strength of stainless steel bars is comparable to that of uncoated carbon steel bars (Ahlborn and Dengartigh 2002), (Moen and Sharp 2016). Ahlborn and Dengartigh (Ahlborn and Dengartigh 2002) conducted 191 bond tests with beam-end specimens containing ASTM A615 carbon steel, 316LN stainless steel, and 2205 Duplex stainless steel reinforcement. The results of their study showed that there was no reason to believe that the stainless steel bond strength was different than the carbon steel bond strength.

Design codes treat the development lengths and lap-splice lengths for stainless steel the same as for carbon steel reinforcing bars (CRSI 2012).

4.3.2.2. Low-carbon Chromium Reinforcement

The bond characteristics of ASTM A1035 steel bars are similar to traditional reinforcing steel grades since the modulus and bar deformations of both types are identical, but the high-strength of low-carbon chromium bars results in the development of higher bar stress and the need for longer development lengths (Shahrooz, et al. 2011). A number of studies have indicated that increasing splice lengths to satisfy splice strength requirements may not be sufficient if high stress levels are to be developed without the use of transverse reinforcement (Seliem, et al. 2009), (Raafat El-Hacha 2006). With long splice lengths, the bond stresses at the lead end of a splice with poor confinement begin to drop before the bond along the rest of the splice can be fully developed. As a result, it is not possible to mobilize high bond stresses along the entire length of a long splice. Adding transverse reinforcement will increase the local bond strength and overall splice strength. Tests have indicated the current lap-splice length requirements are not only sufficient but even conservative for high-strength bars in confined lap-splices (Seliem, et al. 2009), (Michael Briggs 2007).

In 2010, ACI International Task Group 6 developed a design guide for the use of ASTM A1035 steel in concrete. These guidelines state that development and splice length for ASTM A1035 Grade 100 reinforcement may be determined by the ACI-318 equations provided the splice is confined. The guide also provides modified equations for the calculation of splice length for unconfined spliced bars (ACI Innovation Task Group 6 2010).

4.3.3. FRP Bars

While the bond mechanisms of GFRP bars are similar to those of steel bars, these bars show in general a smaller bond strength due to smaller size of the bar ribs. Due to the large number of combinations of fibers, epoxy resins and surface shapes, it is more difficult to develop general bond provisions for FRP bars (Federation Internationalte du Beton (fib) 2000). Nevertheless, ACI 440.1R-15 (ACI 440.1R-15 2015) presents development and lap-splice lengths equations for FRP reinforcing bars, which have been derived from an experimental database composed mainly of GFRP.

Similar to the issues discussed regarding long-term mechanical properties of FRP bars, certain environmental conditions can adversely affect the bond mechanisms of FRP reinforcement. The FRP-to-concrete bond can degrade when exposed to continual freeze/thaw or wet/dry cycles, alkaline solutions, and high temperature. Reduction in bond strength is highly dependent on type of fiber used to manufacture the bar. Mashina et al. (Mashima and Iwamoto 1993) investigated the bond behavior of CFRP, GFRP, and AFRP bars in concrete subjected to freeze/thaw cycles. No reduction in bond capacity was observed in the CFRP and GFRP bars whereas a 40% reduction in bond strength was noted in AFRP bars after 600 freeze/thaw cycles. Long-term exposure to the alkaline environment present within concrete can degrade GFRP, which results in the reduction of tensile strength and bond strength of the material.

4.4. Implications of Using Alternative Reinforcement on Structural Behavior and Design

The use of alternative steel reinforcement does not introduce changes in strength design other than a different nominal yield strength of steel for those bars with a higher grade. In terms of ductility, alternative steels have generally similar or higher elongation capacities than black bars. However, cold working and bending of galvanized reinforcement has to be limited to avoid embrittlement of steel. Slightly different strength design procedures need to be followed for FRP bars given their linear elastic nature. Design guidelines for FRP bars are provided in ACI 440.1R-15.

Regarding the serviceability of the structures, employing FRP bars will decrease the flexural stiffness of reinforced concrete elements as compared to steel reinforcement given the same level of reinforcement or flexural strength. This is due to the lower elastic modulus of FRP composites as compared to steel. As a result, structural design may be governed by permissible deflections. The lower bond resistance of ECR can also have implications in the cracking behavior of reinforced concrete members. Epoxy-coated bars have been reported to cause wider cracks than conventional bars (Krauss and Rogalla 1996). The lower bond stiffness of these bars reduced the restraint of reinforcement on the crack opening.

The required development and lap-splice lengths for alternative steel reinforcement are the same as those of conventional reinforcement, except for epoxy-coated and dual-coated bars, as prescribed in current design specifications. The minimum development and lap-splice lengths of FRP bars may be calculated using the equations provided in ACI 440.1R-15.

The use of alternative reinforcement with significantly higher corrosion resistance, such as stainless steel, could also open to possibility of relaxing some design requirements related to durability. This could optimize structural designs and compensate for the increase of material costs associated to corrosion-resistant reinforcement. For example, a reduction of the concrete cover would be acceptable and would lower the size and weight of the structure, and thus its cost. A smaller cover would also reduce the width of flexural cracks, contributing to enhanced corrosion resistance.

Chapter 5. Review of Current Practice Related to Alternative Reinforcement

5.1. Survey of State DOTs and TxDOT Districts

A written survey was developed with the aim of gaining a better understanding of the use of alternative reinforcements throughout the US. The survey was distributed to 17 state DOTs and 14 TxDOT district offices. The surveyed states and districts were chosen based on the assumption that deicing or anti-icing chemicals were used regularly within the DOT and/or a significant amount of their infrastructure was exposed to marine environments.

5.1.1. State DOT Survey Results

Responses were received from 12 state DOTs—a response rate of 71%. Figures 5.1 and 5.2 illustrate the reported use of alternative reinforcement by state. Of the state DOT responses, 75% of states reported use of alternative reinforcement with 78% of those states reporting the use of multiple types of alternative reinforcement. The most cited alternative was epoxy-coated rebar, with eight states reporting its use, followed by five states reporting the use of stainless steel and four states reporting use of low-carbon chromium steel. All states using alternative reinforcement reported the use of epoxy-coated rebar, except Florida. Alabama and Louisiana reported that they do not use alternative reinforcement. Massachusetts stated that they have used epoxy-coated rebar and galvanized reinforcement in the past, but they do not currently use any alternative reinforcements.

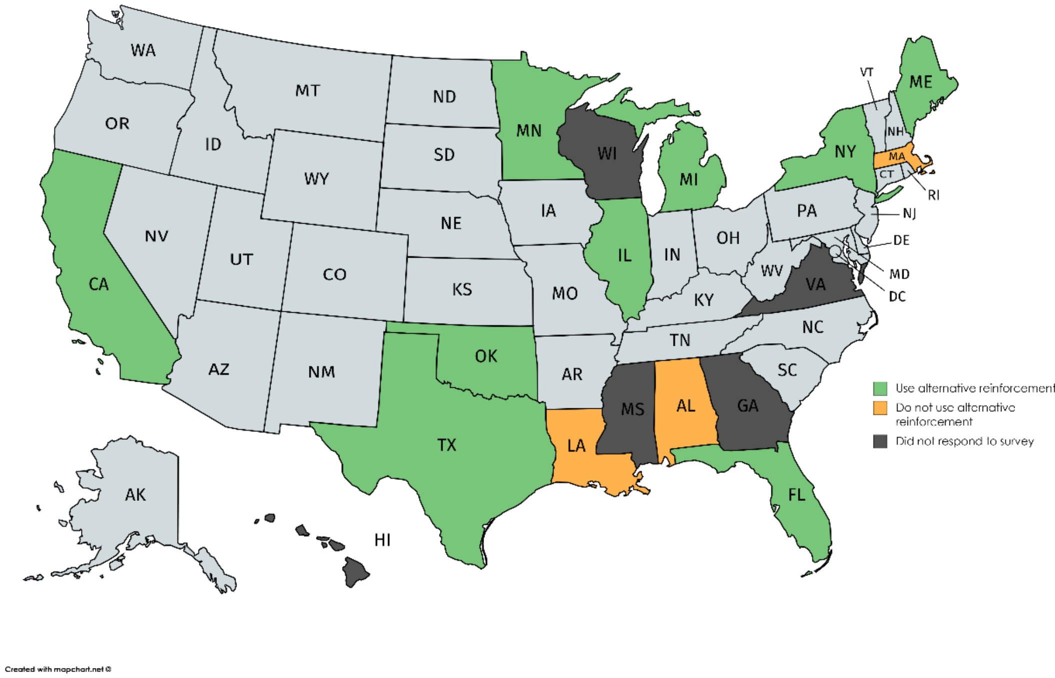

Figure 5.1: Alternative reinforcement use by state

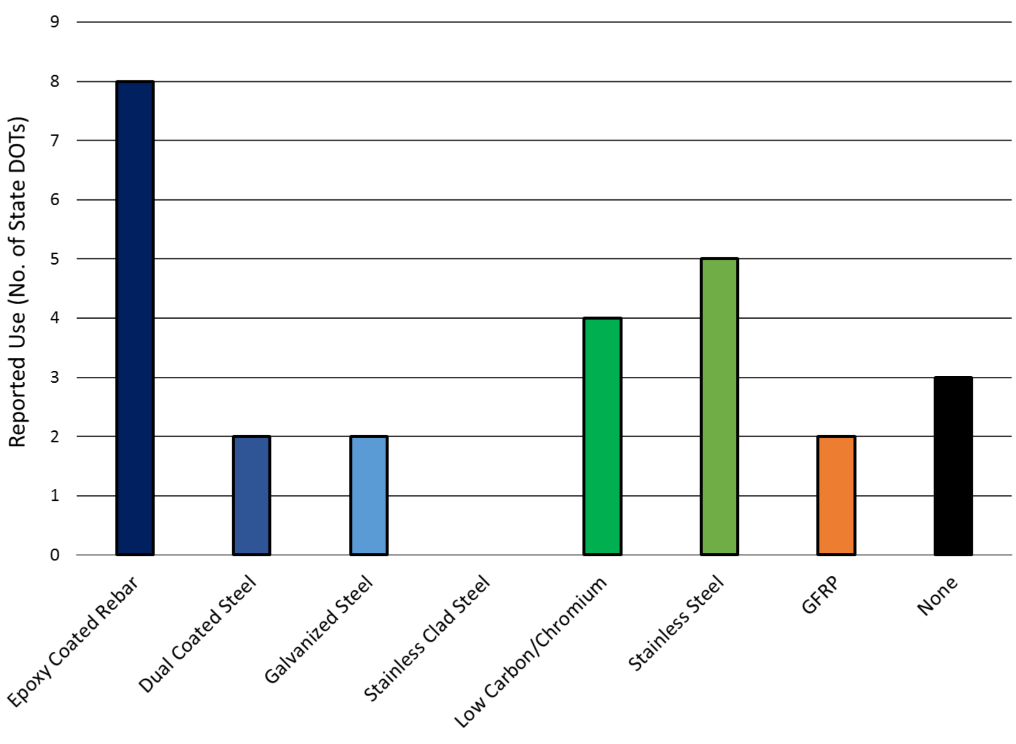

Figure 5.2: Types of alternative reinforcement used by state DOTs

One main goal of this survey was to determine the amount of alternative reinforcement used annually within each state surveyed. Unfortunately, few responders were able to provide this information. Five states did provide quantities and in all instances epoxy-coated rebar was the most abundantly used material. A breakdown of the quantities reported in the survey are provided in Table 5.1.

Table 5.1: Reported annual use of alternative reinforcement (tons)

	Epoxy-Coated	**Stainless Steel**	**Galvanized**	**Low-Carbon Chromium**	**Dual-Coated**	**GFRP**
California	50,000	---	5,000	---	---	---
New York	830	100	2	10	2	---
Maine	68	80	---	10	---	---
Illinois	10,000	---	---	---	---	---
Michigan	4,300	1	---	---	---	75

All state DOTs reported that all alternatives used by their DOT provided superior corrosion resistance to traditional steel, except Maine. Maine has observed inferior corrosion resistance with ECR compared to traditional steel. This is likely due to damage to the epoxy coating during handling and placement. When asked "*Have you identified construction or placement issues with the use of alternative reinforcing?"*, all responders reported issues related to damage to epoxy-coated rebar and the need to touch-up such areas during placement.

The survey included several questions related to design methods and inspection techniques used for bridge construction. All responders reported that all bridges were designed according to AASHTO LRFD bridge design requirements, regardless of type of reinforcement used. However, when ECR (epoxy-coated and dual-coated) was used the lap splices and embedment lengths were increased according the AASHTO LRFD specifications to compensate for reduced bond strength. All responders also stated that the same inspection techniques are used for alternative and traditional steel construction. These techniques typically include visual inspections during placement to identify epoxy coating damage, and visual and non-destructive inspection techniques for existing structures. During replacements, widenings, and rehabilitation projects TxDOT reports routine observation of the condition of existing epoxy coating reinforcement.

In terms of detailing practices, one state mentioned that GFRP cannot be bent in the field so stainless steel was used when bent bars were specified for GRFP-reinforced structures. Aside from this, no other differences in detailing practices were reported. Additionally, none of the responders reported instances of excessive bridge deflections due to the use of alternative reinforcement.

5.1.2. TxDOT District Survey Results

Responses were received from 71% of the 14 TxDOT districts surveyed. Figures 5.3 and 5.4 illustrate the reported use of alternative reinforcement within these districts. Eighty-percent (80%) of the responding districts reported the use of ECR. The Houston, Amarillo, and Dallas districts reported using GFRP. The use of dual-coated reinforcement was reported in two districts, and the use of galvanized and stainless steel reinforcement was reported by one district for each type. No specific correlation could be made between the type of alternative used and location of use; however, the coastal districts do seem to be more willing to use more novel types of reinforcement whereas the northern districts tend to use primarily epoxy-coated bars.

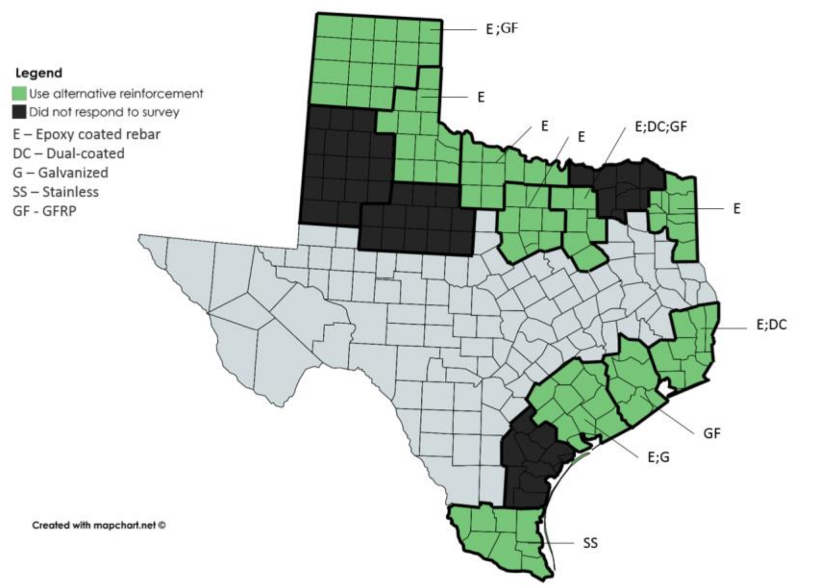

Figure 5.3: Alternative reinforcement use by TxDOT district

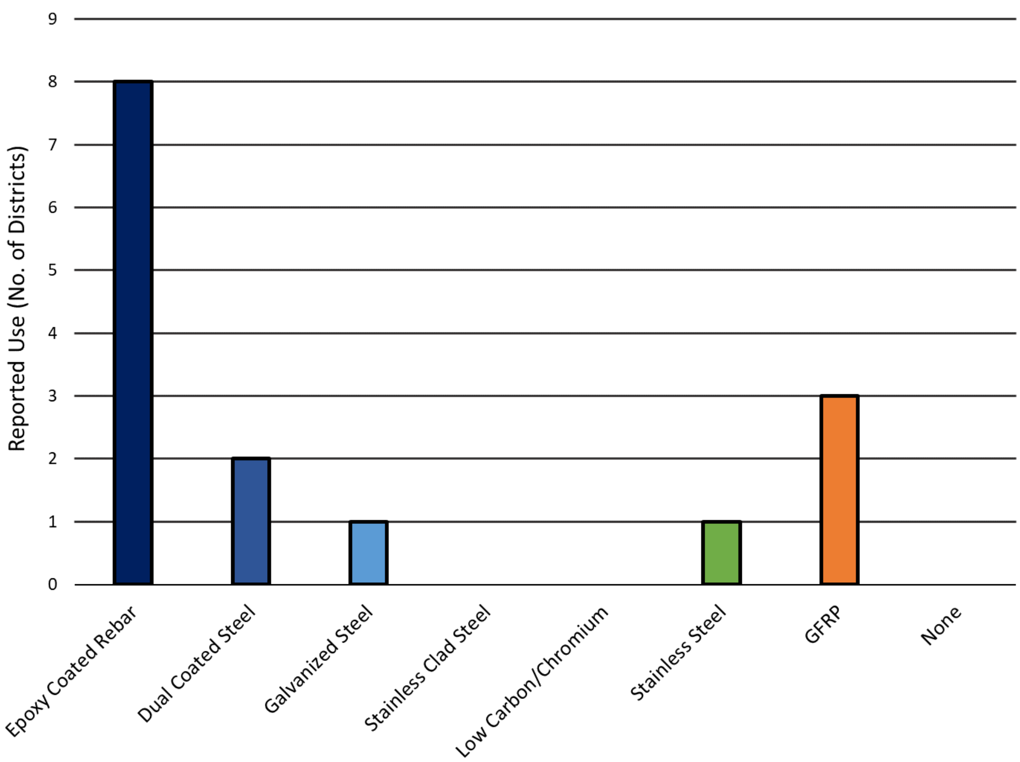

Figure 5.4: Types of alternative reinforcement used by TxDOT Districts

5.2. Other Reported Use of Alternative Reinforcement

None of the states surveyed reported use of SCR. This is likely due to availability issues and lack of long-term performance data on the material. Several states have reported its use in experimental applications with varying degrees of success. Virginia, Michigan, and Oklahoma have all conducted research to evaluate the use of SCR in bridge construction; however, Michigan and Oklahoma do not currently use the material in bridge construction.

Chapter 6. Conclusions and Recommendations

6.1. Material Costs and Consideration

This chapter presents a brief summary of the main findings from this synthesis study on the use of alternative reinforcement aimed at increasing the service life of bridges and other reinforced concrete subjected to external chlorides. Based on the literature review and survey of current practice, recommendations are presented on how to evaluate alternative reinforcement options for a given exposure condition. Lastly, based on the gap analysis performed, a list of research/implementation needs is presented aimed at addressing the critical technical/practical issues identified in this report.

Some of the main conclusions that can be drawn from this synthesis study include the following:

- There is an increasing need for improving the corrosion resistance of reinforced concrete bridges (and other structures) that are subjected to external chlorides, such as those from anti-icing/de-icing salts or seawater. This need is compounded by the recent shortage of fly ash, a material that has been used for many years to combat the potential for corrosion by reducing the rate of chloride diffusion. With this recent market situation, the need for alternative reinforcement (or other products) to fill this void in corrosion protection has increased.

- Several commercially available alternative reinforcement products have been used in bridges nationwide, with epoxy-coated steel by far the most commonly used to date, followed distantly by galvanized steel, stainless steel, low-carbon chromium steel, and FRP (primarily GFRP). Table 6.1 summarizes the types and estimated amounts of alternative steel used by various state DOTs, based on a survey developed and distributed under this synthesis project. Table 6.1 also summarizes other relevant information on the various alternative reinforcement types currently being used, including information on their relative cost, corrosion resistance, and specific technical or construction-related issues. Other potential alternative reinforcements, such as stainless steel cladded rebar or basalt FRP, are not included in this table due to lack of availability and/or lack of data received from the DOT survey.

- The alternative reinforcement types summarized in Table 6.1 can be evaluated for use in a given bridge by using service-life prediction software, such as ConcreteWorks V3.0. Some alternative reinforcement, such as stainless steel and epoxy-coated steel, are already included as preset options within ConcreteWorks, with appropriate modifications to the chloride threshold value included as defaults. Other alternative reinforcement could also be considered in ConcreteWorks, but one would need to manually enter the appropriate chloride threshold value. By evaluating a specific bridge to be constructed in a given chloride-laden environment, it is possible to quantify the potential increase in service life imparted by a specific alternative reinforcement type. ConcreteWorks allows for evaluating the service life of reinforced concrete exposed to either anti-icing/de-icing salts or a marine environment, with estimated values of surface chloride and chloride build-up based on historical, national databases. Although ConcreteWorks does not intrinsically allow for performing an economic analysis, one can use the cost data in Table 6.1, coupled with the predicted service life (or time to corrosion initiation) from ConcreteWorks, to compare the various alternative reinforcements for a given exposure condition.

- As shown in Table 6.1, some alternative reinforcement types require modifications to standard construction and design methodologies. Increased development lengths are required for some alternative reinforcement types, such as epoxy-coated steel or FRP. Appropriate specifications and codes (ACI, AASHTO, etc.) should be followed to ensure that any necessary modifications are applied to account for reduced bonding with the concrete and for the potential, long-term degradation of FRP, particularly in a wet environment.

Table 6.1: Summary of alternative reinforcement types

Reinforcement type	Estimated total annual usage from state DOT survey[1] (tons)	Typical cost[2] ($/lb)	Relative corrosion resistance	Chloride threshold value[3] (% of concrete)	Required increase in lap-splice lengths	Other relevant information
Black steel (low carbon)	---	0.32 - 0.52	Low	0.07	---	
Epoxy-coated steel	65,198	0.46 - 0.80	Low/Med	0.07	50%/20%[4]	Must minimize epoxy coating damage during field operations and installation; Not able to use electrochemical NDT methods to assess epoxy-coated steel
Galvanized steel	5,002	0.50 - 0.73	Med/High	---	None required	
Stainless steel	181	1.82 - 3.44	High	0.70	None required	May be susceptible to stress corrosion cracking.
Dual-coated steel	---	---	Low/Med	---	50%/20%[4]	Has been used in two TxDOT districts (quantities unavailable)
Low-carbon chromium steel	20	0.65 - 0.94	Low/Med	---	None required[5]	ASTM A1035 includes two yield strengths, 100 and 120 ksi
GFRP	75	1.00 - 1.44[6]	High	---	See ACI 440.1R-15	Some FRP may be subject to long-term degradation in high-pH concrete pore solution (see ACI 440 environmental reduction factors, C_E, in Table 4.2).

[1] Data received from California, New York, Maine, Illinois, and Michigan (see Chapter 5 for more specific details)
[2] Data from Van Dyke et al. 2017
[3] Assumed inputs for ConcreteWorks V3.0, expressed in % chlorides per mass concrete
[4] 50% increase for bars with cover less than 3db or with bar spacing less than 6 db; 20% for other case. Likewise, the development length of hooked bars and headed bars needs to be increased by 20%.
[5] ACI 318 bond length equations can be used, provided that splice is confined
[6] GFRP is usually sold by the foot and not by the pound

By synthesizing the information and data gathered during the course of this 12-month project, it became quite apparent that there were significant gaps in the state of the art and the current practice regarding alternative steel. Some of the most important gaps in our technical and practical understanding of alternative reinforcement types are listed below, along with ideas on how to close these gaps through future research or implementation work:

- Currently, there is not a standardized test method to accurately measure the chloride threshold value for a given alternative reinforcement type. Such a standardized test is need in order to better capture the impact of different reinforcement bars on the quantity of chlorides needed at the bar surface to initiate corrosion. Data from such a standardized test needs to be correlated directly with long-term field performance in order to generate confidence in an accelerated, short-term test.

- Very little information is available on the long-term corrosion resistance of alternative reinforcement types. Monitoring of existing structures containing alternative reinforcement is essential, and new construction projects should integrate long-term monitoring into the project scope. Additional samples should be cast using alternative reinforcing bars and monitored at the TEXMEX site in Port Aransas. This site has been found to be extremely aggressive, yielding corrosion within only a couple years for some reinforcement types.

- The use of alternative reinforcement may affect the structural performance and design of reinforced concrete members if the mechanical properties or bond characteristics of the bars differ from those of conventional reinforcement. Implications for design include the cracking and deformability of structural members (to be considered for serviceability design), the ultimate load-carrying capacity and ductility of the members (to be considered in strength design), and the required development and lap-splice lengths of the reinforcing bars. Such differences are in general considered in current design practice and specifications. However, a number of issues concerning the structural design with alternative reinforcement deserve more detailed study:

  - Limited research conducted on stainless-clad reinforcement has shown that the core and the cladding of the bars may separate prior to the developing its full tensile capacity. The bond performance and composite action of the core and cladding need to be further investigated to ensure an efficient use of this type of reinforcement.

  - Current design specifications present a number of limitations regarding the use of alternative reinforcement with nominal yield strengths higher than 60 ksi. Current codes limit the application of high-strength bars to certain applications. Research is needed to support a more general use of steel bars with excess of Grade 60 to exploit the benefits of their higher tensile strength. In addition, current development and lap-splice equations may not be adequate for higher-strength bars.

- The use of alternative reinforcement with significantly higher corrosion resistance, such as stainless steel, could allow for more relaxed durability design requirements (e.g., concrete cover) to optimize structural design and compensate the higher cost of the reinforcement. The design possibilities offered by the use of corrosion-resistant bars should be further explored to minimize initial construction costs and life-cycle costs.

References

ACI 440.1R-15. 2015. *Guide for the Design and Construction of Structural Concrete Reinforced with Fiber-Reinforced Polymer (FRP) Bars.* Farmington Hills, MI: American Concrete Institute.

ACI Committee 201. 2016. *ACI 201.2R-16 Guide to Durable Concrete.* Farmington Hills, MI: American Concrete Institute.

ACI Committee 318. 2014. *Building Code Requirements for Structural Concrete (ACI 318-14).* Farmington Hills, MI: American Concrete Institute.

ACI Committee 440. 2008. *Specification for carbon and glass fiber-reinforced polymer bar materials for concrete reinforcement (ACI 440.6M-08).* Farmington Hills, MI: American Concrete Institute.

ACI Innovation Task Group 6. 2010. *Design Guild for the Use of ASTM A1035/A1035M Grade 100 (690) Steel Bars for Structural Concrete.* Farmington Hills, MI: American Concrete Institute.

ACI-ASCE 408. 2003. *Bond and Development of Straight Reinforcing Bars in Tension (ACI 408R-03).* Farmington Hills, MI: American Concrete Institute.

Ahlborn, T. M., and T. C. Dengartigh. 2002. *A Comparative Bond Study of Stainless Steel Reinforcement in Concrete, Report No. RC1406.* Michigan Tech, Houghton, MI: Center for Structural Durability.

American Association of State Highway and Transportation Officials (AASHTO). 2017. *AASHTO LRFD Bridge Design Specifications (8th Ed.).* Washington, DC: AASHTO.

American Association of State highway and Transportation Officials (AASHTO). 2014. *AASHTO LRFD Bridge Design Specifications, 7th ed.* Washington, DC: AASHTO.

American Galvanizers Association. 2011. *Hot-Dip Galvanized Reinforcing Steel: A Specifier's Guide.* Centennial, CO: AGA.

—. 2011. "Mechanical Properties of Galvanzied Steel." Accessed February 5th, 2018. https://www.galvanizeit.org/images/uploads/drGalv/Strain_Age_Embrittlement_During_Galvanizing.pdf.

Andrade, C., and M. Castellote. 2002. "Recommendation of RILEM TC 178-TMC: 'Testing and modelling chloride penetration in concrete'." *Material and Structures* 35: pp. 583-585.

Angst, U., and O. Vennesland. 2008. "Critical chloride content in concrete - state of the art, in: Concrete Repair, Rehabilitation and Retrofitting II." *Proceedings of the 2nd Int. Conf. on Concrete Repair, Rehabilitation and Retrofitting II.* Cape Town, South Africa.

Angst, Ueli, Bernhard Elsener, Claus K. Larsen, and Oystein Vennesland. 2009. "Critical chloride content in reinforced concrete - A review." *Cement and Concrete Research* 39: pp. 1122-1138.

ASTM. 2016. *ASTM A1035 Standard Specification Deformed and Plain, Low-Carbon, Chromium, Steel Bars for Concrete Reinforcement.* West Conshokocken, PA: ASTM International.

—. 2017. *ASTM A1055: Standard Specification for Zinc and Epoxy Dual-Coated Steel Reinforcing Bars.* West Conshokocken, PA: ASTM International.

ASTM. 2016. *ASTM A767 Standard Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement.* West Conshohocken, PA: ASTM International.

ASTM. 2018. *ASTM A955 Standard Specification for Deformed and Plain Stainless Steel Bars for Concrete Reinforcement.* West Conshokocken, PA: ASTM International.

—. n.d. *ASTM C1152: Standard test method for acid-soluble chloride in mortar and concrete.* West Conshokocken, PA: ASTM International.

—. 2009. *ASTM C876 Standard Test Method for Corrosion Potentials of Uncoated Reinforcing Steel in Concrete.* West Conshohocken, PA: ASTM International.

ASTM. 2013. *Standard Test Method for Determining Effects of Chemical Admixtures on Corrosion of Embedded Steel Reinforcement in Concrete Exposed to Chloride Environments.* West Conshohocken, PA: ASTM International.

Benmokrane, B, F Elgabbas, E A Ahmed, and P Cousin. 2015. "Chracterization and Comparative Durability Study of Glass/Vinylester, Basalt/Vinylester and Basalt/Epoxy FRP Bars." *Journal of Composites for Construction* Vol. 19 (No. 6).

Benmokrane, B., O. Chaallal, and R. Masmoudi. 1995. "Glass fibre reinforced plastic (GFRP) rebars for concrete structures." *Construction and Building Materials* 9 (6): pp. 353-364.

Benmokrane, Brahim, Peng Wang, Tan minh Ton-That, Habib Rahman, and Jean-Francois Robert. 2002. "Durability of Glass Fiber-Reinforced Polymer Reinforcing Bars in Concrete Environment." *Journal of Composites for Construction* 6 (3): pp.143-153.

Broomfield, J. 2007. *Corrosion of Steel in Concrete: Understanding, Investigation and Repair.* New York: Taylor & Francis.

Clemena, Gerardo G. 2003. *Investigation of the Resistance of Several New Metallic Reinforcing Bars to Chloride-Induced Corrosion in Concrete.* Charlottesville, VA: Virginia Transportation Research Council.

Clemena, Gerardo G., Dina N. Kukreja, and Calude S. Napier. 2003. *Trial Use of a Stainless Steel-Clad Steel Bar in a New Concrete Bridge Deck in Virginia.* Charlottesville, VA: Virgina Transportation Research Council.

Clemena, Gerardo, and Yash Virmani. 2004. "Comparing the Chloride Resistances of Reinforcing Bars." *Concrete International* 26 (11): pp. 39-49.

Concrete Reinforcing Steel Institute - CRSI. 2015. ""Frequently Asked Questions (FAQ) about Dual-Coated Steel Reinforcing Bars", CRSI Tech Note ETN-M-7-15." Schaumburg, IL, 5 pp.

Council, International Code. 2017. *ICC ES Report ESR-2107.* ICC Evaluation Service.

Cross, William M., Ed F. Duke, Jon J. Kellar, Ken N. Han, and Dan Johnston. 2001. *Stainelss Steel Clad Rebar in Bridge Decks.* Pierre, SD: South Dakota Department of Transportaion.

CRSI. 2017. *FAQ about Low-Carbon, Chromium ASTM A1035 Types CS,CM and CL Steel Reinforcing Bar, CRSI Technical Note ETN-M-11-17.* Schaumburg, IL: Concrete Reinforcing Steel Institute.

CRSI. 2012. *Frequently Asked Questions About Stainless Steel Reinforcing Bars: CRSI Technical Note ETN-M-2-12.* Schaumburg, Illinois: Concrete Reinforcing Steel Institute.

CRSI. 2013. *Specialty and Corrosion-Resistant Steel Product Guide.* schaumburg, IL: Concrete Reinforcing Steel Institute.

Darwin, David, Mathew O'Reilly, Isaac Somogie, Jayne Sperry, James Lafikes, Scott Storm, and JoAnn Browning. 2013. *Stainless Steel Reinforcement as a Replacement for Epoxy Coated Steel in Bridge Decks.* Lawrence, Kansas: University of Kansas Center for Research, Inc.

Davis, S., and D. Goldberg. 2013. *The Fix We're in for: The State of Our Nation's Bridges* . Washington, DC: Transportation for America.

Elgabbas, F., E. A. Ahmed, and B. Benmokrane. 2015. "Physical and mechanical characteristics of new basalt-FRP bars for reinforcing concrete structures." *Construction and Building Materials* 95: pp. 623-635.

Federation Internationalte du Beton (fib). 2000. *Bond of Reinforcement in Concrete (fib bulletin 10).* Lausanne, Switzerland: fib.

FHWA Research and Technology. 2015. *Nondestructive Evaluation Web Manual: Linear Polarization (LPR).* Washington, DC: FHWA.

Grimault, B., L. Gaillet, M.Driss-Habti, T. Chaussadent, E. Chauveau, and M. Manel. "Stress corrosion cracking and hydrogen embrittlement behaviour of high strength duplex and austenitic stainless steels in simulated concrete pore solution". *EuroCorr*. September. 2011.

Hamad, B. S., and M. F. Fakhran. 2006. "Effect of Confinement on Bond Strength of Hot-Dip Galvanized Lap Splices in High-Strength Concrete." *ACI Structural Journal* 103 (1).

Hamad, B., and J. Mike. 2003. "Experimental Invetigation of Bond Strength of Hot-Dip Galvanized Reinforcement in Normal and High-Strength Concrete." *ACI Structural Journal* 100 (4).

Hansson, C. M. 2016. "Corrosion of Stainless Steel in Concrete." In *Corrosion of Steel in Concrete Structures*, 59-85. Cambridge, MA: Elsevier.

Huang, Jianwei, and Riyad Aboutaha. 2010. "Environmental Reduction Factors for GFRP Bars used as Concrete Reinforcement: New Scientific Approach." *Journal of Composites for Construction* 14 (5): pp. 479-486.

Idun, E. K., and D. Darwin. 1999. "Bond of Epoxy-Coated Reinforcement: Coeffiecient of Friction and Rib Face Angle." *ACI Structural Journal* 96 (4): pp. 609-615.

Islam, M. A., B. P. Bergsma, and C. M. Hansson. 2013. "The chloride-induced corrosion behaviour of stainless steel and carbon steel reinforcing bars in sound and cracked concrete." *Corrosion* 69: pp. 303-313.

Johnston, D. W., and P. Zia. 1982. *Bond Characteristics of Epocy Coated Reinforcing Bars, Report No. FHWANC-82-002.* Washington, DC: Federal Highway Administration, pp. 163.

Kahl, Steven. 2012. *Stainless and Stainless-Clad Reinforcement for Highway Bridge Use.* Lansing, Michigan: Michigan Department of Transportation.

Kayali, O., and S. Yeomans. 1995. "Bond and slip of coated reinforcement in concrete." *Construction and Building Materials* 9 (4): pp. 219-226.

Kayali, O., and S. Yeomans. 2000. "Bond of ribbed galvanized reinforcing steel in concrete." *Cement and Concrete Composites* 22: pp. 459-467.

Koch, Gerhardus H., Michiel Brongers, and Neil G. Thompson. 2002. *Corrosion Costs and Preventive Strategies in the United States.* McLean, VA: Federal Highway Administration.

Krauss, P. D., and E. A. Rogalla. 1996. *Transverse Cracking in Newly Constructed Bridge Decks.* 492 NCHRP Report 380, Washington, DC: Transportation Research Board, Project 12-37.

Mashima, M., and K. Iwamoto. 1993. "Bond Characteristics of FRP Rod and Concrete after Freezing and Thawing Deterioration." *ACI Special Publication* 138: pp. 51-70.

McDonald, D. B. 2019. "Corrosion of epoxy-coated steel in concrete." In *Corrosion of Steel in Concrete Structures*, pp 87-110. Cambridge, Ma: Elsevier.

McDonald, D. B., D. W. Pfeifer, and M. R. Sherman. 1998. *Corrosion Evaluation of Epoxy-Coated, Metallic-Clad and Solid Metallic Reinforcing Bars in Concrete.* McLean, VA: FHWA.

Michael Briggs, Shelby Miller, David Darwin, JoAnn Browning. 2007. *Bond Behavior of Grade 100 ASTM A 1035.* Lawrence, Kansas: The University of Kansas Center for Research, 2007.

Moen, C. D., and S. R. Sharp. 2016. "Bond Properties between Concrete and Corrosion-Resistant Reinforcing Steels." *ACI Structural Journal* 113 (2).

Moffatt, E. 2016. *Durability of Rapid-set (ettringite based) Concrete (Dissertation).* Fredericton, New Brunswick: University of New Brunswick.

Nanni, A., S. Rizkalla, C. E. Bakis, J. O. Conrad, and A. A. Abdelrahman. 1998. "Characterization of GFRP Ribbed Rod Used for Reinforced Concrete Construction." *Proceedings of the International Composites Exhibition (ICE-98).* Nashville, TN. pp. 16A/1-6.

NX Infrastructure Ltd. 2008. "NX Infrastructure." November. Accessed January 2018. http://www.nxinfrastructure.com/sites/default/files/usa.rebars.pdf.

Pfeifer, D. W., J. R. Landgren, and A. Zoob. 1987. *Protective systems for new prestressed and substructure concrete, Report No FHWA-RD-86-193.* Washington, DC USA: Federal Highway Administration.

Raafat El-Hacha, Hossam El-Agroudy, and Sami H. Rizkalla. 2006. "Bond Characteristics of High-Strength Steel Reinforcement." *ACI Structural Journal* 103 (6): 771-782.

Raafat El-Hacha, Hossam El-Agroudy, and Sami H. Rizkalla. 2006. "Bond Characteristics of High-Strength Steel Reinforcement." *ACI Structural Journal* 103 (6).

Rasheeduzzafar, Fahd H., Mahar A. Badar, and Mohammed M. Khan. 1992. "Performance of Corrosion Resisting Steels in Chloride-Bearing Concrete." *ACI Materials Journal* 89 (5): pp. 439-448.

Rogalls, P.D Krauss and E.A. 1996. *Transverse Cracking in Newly Constructed Bridge Decks.* 492, NCHRP Report 380, Project 12-37, Washington, D.C.: Transportation Research Board.

Ross, Anne. 2006. *Basalt Fibers: Alternative to Glass.* August 1. Accessed August 31, 2018. https://www.compositesworld.com/articles/basalt-fibers-alternative-to-glass.

Sagues, Alberto A., Rodney G. Powers, and Richard Kessler. 2001. *Corrosion Performance of Epoxy-Coated Rebar in Florida Keys Bridges, Final Paper 01642.* Houston, TX: NACE International.

Seliem, H., A. Hosny, S. Rizkalla, and P. Zia. 2009. "Bond Characteristics of ASTM A1035 Steel Reinforcing Bars." *ACI Structural Journal* 106 (4): pp.530-539.

Serdar, Marijana, Lidija V. Zulj, and Dubravka Bjegovic. 2013. "Long-term corrosion behaviour of stainless reinforcing steel in mortar exposed to chloride environment." *Corrosion Science* 69: pp. 149-157.

Shahrooz, Bahram M., Richard A. Miller, Kent A. Harries, and Henry G. Russell. 2011. *NCHRP Report No. 679: Design of Concrete Structures Using High-Strength Steel Reinforcement.* Washington, DC: Transportation Research Board.

Tanks, Jonathon D., and Stephen R. Sharp. 2015. "Characterization of a Stainless-Clad Reinforcing Bar." *TRB 94th Annual Meeting Compendium of Papers.* Washington, DC: Transportation Research Board.

Treece, R. A., and J. O. Jirsa. 1989. "Bond Strength of Epoxy-Coated Reinforcing Bars." *ACI Materials Journal* 86 (2): pp. 167-174.

Trejo, David, and Radhakrishna Pillai. 2004. "Accelerated Chloride Threshold Testing-Part II Corrosion-Resistant Reinforcement." *ACI Materials Journal* 101 (1): pp. 57-64.

Van Dyke, Chris, Bobby Meade, Danny Wells, Sudhir Palle, and Theodore Hopwood II. 2017. *Long-Term Corrosion Protection of Bridge Elements Reinforcing Materials in Concrete.* Frankfort, KY: Kentucky Transportation Cabinet.

Weyers, R. E., M. M. Sprinkel, and M. C. Brown. 2006. *Summary Report on the Performance of Epoxy-Coated Reinforcing Steel in Virginia.* Charlottesville, VA: Virginia Transportation Research Council.

Yeomans, S. R. 2016. "Galvanized Steel Reinforcement." In *Corrosion of Steel in Concrete*, pp. 111-129. Cambridge, MA: Elsevier.

Yeomans, Stephen R. 2004. "Galvanzied Steel in Concrete: An Overview." In *Galvanized Steel Reinforcement in Concrete*, pp. 1-30. Kidlington, Oxford, UK: Elsevier Ltd.

Yeomans, Stephen R. 2004. "Laboratory and Field Performance of Galvanized Steel in Concrete." In *Galvanized Steel Reinforcement in Concrete*, pp. 145-197. Kidlington, Oxford, UK: Elsevier.

Zuo, J., and D. Darwin. 2000. "Splice Stength of Conventional and High Relative Rib Area Bars in Normal and High-Strength Concrete." *ACI Structural Journal* 97 (4): pp. 630-641.