RECENT REINFORCING BAR CORROSION TESTS: A SUMMARY OF TWO INDEPENDENT EVALUATIONS

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ABSTRACT

Over the past 40 years considerable research has been conducted on the corrosion protection performance of various types of reinforcing steel. Recently, two research programs considering various types of reinforcing steels have been conducted. The first was sponsored by Kansas Department of Transportation (KDOT) and the Federal Highway Administration (FHWA) where conclusions were obtained based upon extensive field and laboratory research on the performance and life-cycle costs of steels including: epoxy-coated reinforcing steel, uncoated steel in concrete containing corrosion inhibitors, epoxy-coated steel in concrete containing corrosion inhibitors and Type 2205 stainless steel. The second study was conducted at the Turner Fairbanks Laboratory of the FHWA where conclusions were developed based upon laboratory tests conducted on 12 different bar types from 11 sources. These included epoxy-coated, dual-coated, galvanized, low-carbon chromium, stainless clad and several types of stainless steel. This paper will present the two works and summarize findings.

KEY WORDS: Reinforcing Steel, Corrosion, Durability, Epoxy-Coated, Stainless

INTRODUCTION

During the past 40 years considerable research has been conducted into the protection of reinforcing steel in concrete against corrosion. This paper will present a historical background and provide details of two recent test programs that considered the performance of a variety of reinforcing steel types, including epoxy-coated and stainless steel reinforcing.

HISTORICAL STUDIES

Research on corrosion-resistant reinforcing steel largely began in the early 1970s. The details of several of the more important studies are discussed below.

In 1974 the National Bureau of Standards released a report on tests conducted on 47 coating materials for reinforcing bars including 15 epoxies[1]. This work was initiated in response to the rapid deterioration of concrete bridges following the implementation of an ice- and snow-free roads policy in many states to reduce the incidence of accidents. This research introduced a set of tests to ensure that coatings were impermeable to chloride ions and did not impair the bond of the coated reinforcing bar to the concrete. This research concluded that fusion-bonded epoxy coating should protect steel reinforcing bars from corrosion with acceptable bond and creep characteristics. Even prior to completion of this landmark study, fusion-bonded epoxy-coated bars were being used in concrete with the first bridge being constructed in 1973.

In 1983 Virmani and Clear produced a report for the FHWA on non-specification epoxy-coated bars[2]. Epoxy-coated bars used in the study were three years old, with over 25 holidays/ft (75 holidays/m) and up to 0.8 percent damage and failed to meet the then current ASTM requirements. These bars failed the bend test and the coating was to be readily peeled from the bars. They were placed into concrete with a water-cement ratio of 0.53 that contained 15 lb/yd^3 (8.9 kg/m^3) of admixed chloride ions and the bars only had 1 in. (25 mm) of cover. After two years the epoxy provided corrosion protection that was regarded as “very effective”. Significantly reduced corrosion was observed compared with uncoated bars, especially when used in both the upper and lower double mats. The corrosion rates determined from macro-cell current readings were 12 to 46 times less current than black bars.

In 1991 Clear published results from corrosion tests that were conducted over an 8.5-year period which exposed concrete to deicing salts and freeze-thaw[3]. These tests showed that the epoxy-coated bars reduced macro-cell corrosion by 57-165 times, whereas galvanized bar just slightly delayed onset of corrosion.

Another study was conducted by Clear and completed by Pfeifer et al. in 1993[4]. Tests used bent and straight epoxy-coated bars from seven suppliers and were conducted over three years. During the first 1.35 years of wet and dry testing all the epoxy-coated bars performed well; however, during the subsequent 10.5 months water ponding, bars from two sources remained passive while many bars from other sources started to corrode. It was found that those sources of bar had been patched prior testing, while others had significant holes and holidays. This testing led to a greater understanding of the impact of manufacturing processes on the performance of epoxy-coated reinforcing steel. Subsequently, in 1991 the Concrete Reinforcing Steel Institute (CRSI) developed and launched a voluntary epoxy plant certification program which is now used by almost all coaters in North America.

In 1993, the FHWA initiated a 5-year program considering a wide variety of bar types[5]. As part of this test program, 33 organic coatings, 14 ceramic, metallic or inorganic clad bars and 10 solid metallic bars were evaluated in screening tests. From these screening tests, 12 different bar types were selected for in-concrete testing. The work found that Type 316 stainless steel reinforcing steel should be considered for 100-year design lives with concerns that present costs would limit use of this material; however, the report supported continued use of epoxy-coated reinforcing steel.

More recently, two major research studies on the corrosion performance of various type of reinforcing steel have been conducted. Details of work conducted at the University of Kansas have been published and presentations of work conducted at the Turner-Fairbanks Laboratory in Virginia are summarized here.

UNIVERSITY OF KANSAS RESEARCH

INTRODUCTION

In 2011, a 487-page research report titled *"Evaluation of Multiple Corrosion Protection Systems for Reinforced Concrete Bridge Decks"* presenting an evaluation of the performance of several corrosion protection systems for concrete bridge decks was published[6]. Systems tested included:

- Uncoated reinforcing steel
- Epoxy-coated reinforcing steel (ASTM A775)
- Uncoated steel in concrete containing corrosion inhibitors
- Epoxy-coated steel in concrete containing corrosion inhibitors
- Type 2205 stainless steel

Extensive tests were conducted in plain concrete and concrete containing corrosion inhibitors. These tests included Southern Exposure, Cracked Beam and Corrosion Initiation specimens as well as Field Exposure slabs.

TEST METHODS

The concrete for these studies used a Type I/II cement with a crushed limestone coarse aggregate and a Kansas River sand. All concrete was air entrained. Reinforcing bars were obtained from commercial sources. Corrosion inhibitors used were added at the following concentrations:

- CN - calcium nitrite (3 gal/yd^3, 15 L/m^3)
- AE - combined amines and esters (1 gal/yd^3, 5 L/m^3)
- DTS - disodium tetrapropentyl succinate (1.54 gal/yd, 7.6 L/m^3)

The concrete used for tests used cement contents of 598 lb/yd^3 (355 kg/m^3) with a w/c of 0.45, a slump of 3 +/- 0.5 in. (75 +/- 12 mm) and an air content of 6 +/- 1%.

The Southern Exposure tests consisted of slabs measuring 12 x 12 x 7 in. (300 x 30 x 175 mm) containing two mats of No. 5 bars (16 mm) as shown in Fig. 1. The top mat consisted of two bars and the bottom consisted four bars with a clear cover of 1 in (25 mm). The bars are connected using a 10-ohm resistor to facilitate macrocell measurements. A 0.75-in. (18 mm) dam was integrally cast with the specimen to allow for ponding of these slabs with salt solutions. All epoxy-coated bars were intentionally damaged using a 0.125 inch (3.1 mm) diameter milling bit to simulate field damage. Bars were damaged with either 4 or 10 holes to provide different exposed areas, with half of the holes occurring on each side of the bar.

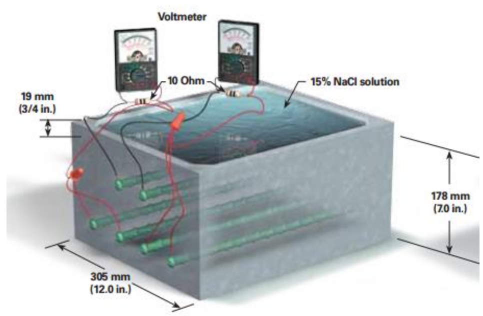

Fig. 1: Southern Exposure Specimen

Corrosion Initiation beams were similar to the Southern Exposure samples, except that they contained only three bars and were half the size of the Southern Exposure Samples.

The Cracked Beam specimens were essentially half that of the Southern Exposure test specimens and measured 12 x 6 x 7 in. (300 x 150 x 175 mm) as shown in Fig. 2. Prior to casting, a 12-mil x 6 in. shim (0.30 x 150 mm) was cast into the concrete mold, creating a 6-in. (150 mm) long crack in the concrete exposing the top mat of steel. Similar specimens were previously used by McDonald et al.[5]

The Southern Exposure and Cracked Beam samples were tested over a 96 week period, using two test cycles each lasting 12 weeks. At the end of the 24 weeks, the program was repeated another three times until the total elapsed testing period was 96 weeks.

The first test cycle involved ponding the samples with a 15 percent sodium chloride salt solution on day 1. On day 4, measurements were conducted and the solution was removed.

The samples were then placed under a heat tent at 100 +/-3 °F (38 +/-2 °C) for three days. This cycle was repeated for 12 weeks. After the 12 weeks of testing, the samples were continuously ponded using a 15 percent NaCl solution. Readings were taken on a weekly basis. The corrosion-initiation samples followed a similar sequence, except that the testing was terminated following initiation of corrosion.

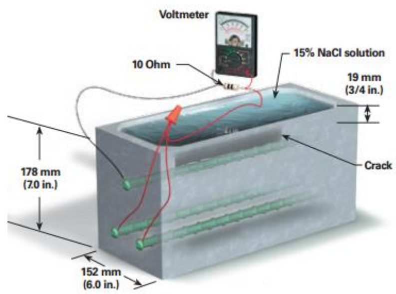

Fig. 2: Cracked beam specimen

Field Test specimens, measuring 48 x 48 x 6.5, (1200 x 1200 x 165 mm) were cast containing two mats of No. 5 (5/8-in. diameter/16 mm) reinforcing steel as shown in Fig. 3. Each mat consisted of two layers of seven bars, spaced 6 in. (150 mm) on center. The top mat was run perpendicular to the bottom layer. Specimens were tested in simulated cracked and non-cracked conditions. All epoxy-coated bars were intentionally damaged using a 0.125-in. (3.2 mm) drill bit. Each bar was damaged with 16 holes, half on each side of the bar. The Field Test specimens were stored outside and were ponded with 10 percent rock salt solution, applied every 4 weeks. Test slabs were placed in the field for approximately 4.8 years.

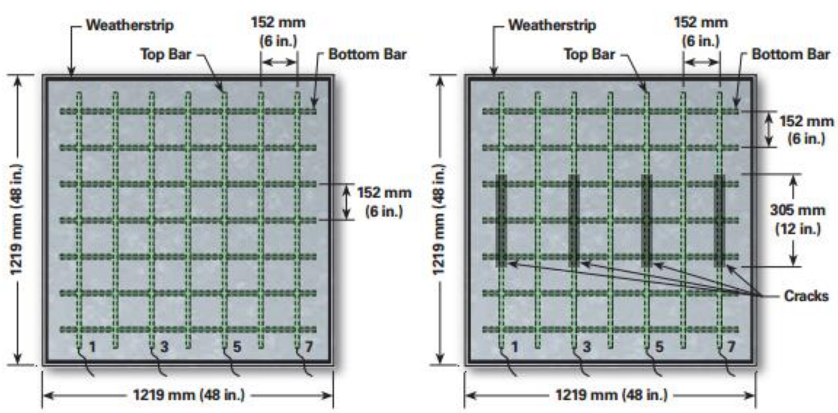

Fig. 3: Field test specimen with and without cracks

MEASUREMENTS

Measurement for the Southern Exposure and Cracked Beam specimens included macrocell voltage, mat-to-mat resistance, corrosion potential and linear polarization resistance. The amount of chloride in the concrete during the 96 week period was also determined using AASHTO T260-94 at the initiation of corrosion, and after 48 and 96 weeks of testing.

Measurements for the field specimens included macrocell voltage drop, mat-to-mat resistance, and corrosion potential, taken every four weeks for the first 96 weeks and then every 8 weeks. Chloride samples were obtained at the end of the test period.

CORROSION INITIATION

The initiation period is defined as the time at which chloride penetrates in sufficient quantity to initiate corrosion. In order to determine this time, the amount of chloride required to initiate corrosion was required. This value is termed the chloride corrosion threshold. The reported corrosion threshold has varied considerably. Typically in North America a value of 0.2 percent by weight of cement is used as the chloride corrosion threshold, while in Europe a value of 0.4 percent by weight of cement is more commonly applied.

The onset of corrosion was defined in these tests as occurring when the measured macrocell corrosion rate exceeded 0.3µm/yr (11.8 x 10^{-6} in/yr) or when the corrosion potential became more negative than -0.275V CSE. The average critical chloride threshold values determined using the Southern Exposure tests and Initiation beam tests are shown in Table 1.

Table 1: Critical Chloride Corrosion Thresholds for Corrosion Protection Systems.

System	Corrosion Threshold (lb/yd^3)	Corrosion Threshold (kg/m^3)
Uncoated reinforcing steel	1.58	0.94
Epoxy-coated reinforcing steel	8.42	5.01
Stainless 2205 reinforcing	26.4	15.71
Corrosion inhibitors		
• AE	2.27	1.35
• CN	3.05	1.81
• DTS	0.83	0.49
Corrosion inhibitors and epoxy-coated reinforcing		
• AE	8.43	5.02
• CN	9.82	5.84
• DTS	1.82	1.08

DETERMINATION OF LIFE

The results obtained from the laboratory and field tests were combined with the information on chloride ingress, corrosion rates and the amount of corrosion to cause cracking to determine the period before concrete repair was required.

CHLORIDE INGRESS

Chloride ingress for analysis of the life of bridge decks was based upon work presented by Lindquist et al[7]. Data on the effect of cracks on chloride ingress for bridge decks with average annual daily traffic greater than 7,500 is shown in Fig. 4. An equation was developed to enable prediction of chloride at 3 in. (75 mm) depth in the concrete for bridges with ADDT >7500 (Eqn 1).

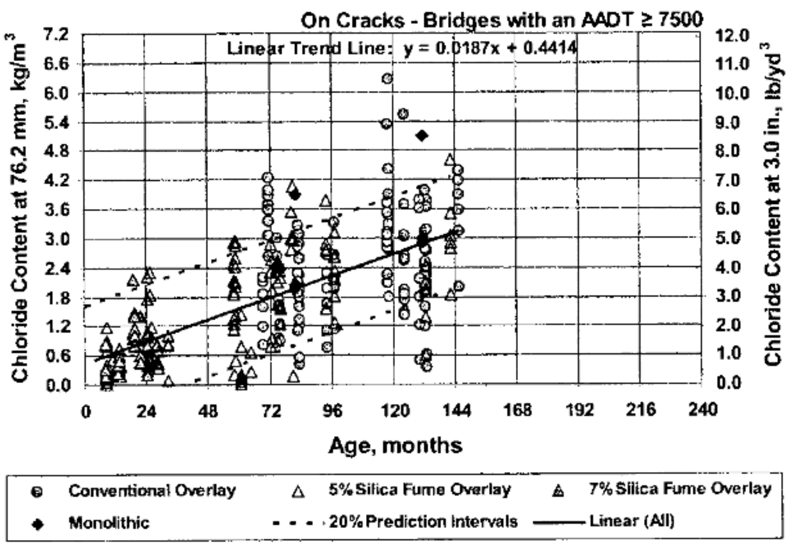

Fig.4: Chloride Ingress in Cracked Concrete for Bridges with AADT > 7500

$$C(t) = 0.0187.t + 0.4414 \quad \ldots (1)$$

where

$t =$ time (months)

$C(t) =$ chloride content (kg/m^3)

CALCULATED INITIATION PERIOD

Using the threshold values shown in Table 1 and the rate of chloride ingress shown by Equation 1, the time to initiation was predicted (Table 2).

Table 2: Time to Initiation, Time to Cracking and Time to First Repair

System	Initiation Period (years)	Propagation Period (years)	Time to First Repair (years)
Uncoated reinforcing	2.2	6.8	14
Epoxy-coated reinforcing	20.3	24.8	50
Stainless 2205 reinforcing	67.6	224	297
Corrosion inhibitor			
• AE	4.1	6.8	16
• CN	6.1	6.8	18
• DTS	1.0	26.6	33
Corrosion inhibitor and epoxy-coated reinforcing			
• AE	20.3	24.8	50
• CN	24.0	34.0	63
• DTS	2.8	45.6	53
*Note that the authors assumed a time to first repair 5 years after cracking.			

AMOUNT OF CORROSION TO CAUSE CRACKING

The amount of corrosion to cause cracking was extensively studied by O'Reilly et al.[6] using experimental and finite element analyses. Based upon this work, an equation was developed for the amount of corrosion to cause cracking, based upon the concrete cover, bar diameter, fraction of bar corrosion and fractional area of the corroding, that is shown in Equation 2.

$$x_{crit} = 0.53\left(\frac{C^{2-A_f}}{D^{0.38}{L_f}^{0.1}{A_f}^{0.6}} + 0.6\right) \times 3^{A_f-1} \quad \text{... (2)}$$

Where
X_{crit} = corrosion loss at crack initiation (mil)
C = cover (in.)
D = bar diameter (in.)
L_f = fractional length of bar corroding, $L_{corroding}/L_{bar}$
A_f = fractional area of bar corroding $A_{corroding}/A_{bar}$

For uncoated and stainless steel reinforcing bars, this critical corrosion value was calculated to be 56 µm (0.002 in.). For epoxy-coated bars, L_f was calculated to be 0.024 and A_f was calculated to be 0.0023 and the critical corrosion value was calculated to be 2,434 µm (0.096 in.). The value for epoxy-coated bars was substantially greater than that for the uncoated bars as corrosion was assumed to only occur at the damage site locations.

CORROSION RATES

For uncoated reinforcing with inhibitors, no field tests were cast, so an estimate was made using relationships developed between bench-scale and field tests. Further, in the bench-scale test program for corrosion inhibitors, the uncoated bars in the control concrete exhibited significantly greater corrosion rates than in the test program conducted using the coated bars and thus, additional scaling of the measured corrosion rates was required.

An estimate of the corrosion rate for Type 2205 stainless steel bars was also determined, based upon bench studies, as the field specimens had not exhibited any corrosion during the 4.8 year test program.

The measured corrosion rates assumed that the entire area of the reinforcing steel was corroding; however, the autopsy results showed that for uncoated bars, corrosion occurred in localized areas. Thus, the corrosion rates determined from the field results was multiplied by a factor to obtain a localized corrosion rate.

Finally, as both macrocell and microcell corrosion contribute to corrosion losses, the macrocell values were also factored to account for the microcell corrosion.

PROPAGATION PERIOD

The propagation period was calculated from the amount of corrosion required to crack concrete; i.e., 56 µm for uncoated bars and 2,434 µm for coated bars. The corrosion rates from cracked concrete only were used in the analysis as "*…bridge decks inevitably develop cracks over the reinforcement, the comparisons using the corrosion rates in cracked concrete likely provide the more accurate representation of corrosion in bridge decks."* Calculated values are shown in Table 2.

TIME TO REPAIR

The time to repair is determined by adding the initiation period to the propagation period. An additional five year period was provided to account for time from the first crack to the repair of the deck. The report explains that "*The latter period is based on the observation that a bridge deck is not fully repaired at the development of the first crack. Rather, the bridge typically undergoes a series of short-term temporary repairs. To account for the period of temporary repairs, a five year delay between first cracking and repair is assumed for all corrosion protection systems.* The calculated time until repair is shown in Table 2.

For cracked concrete, the authors indicated that uncoated bars would require repair after 14 years. Epoxy-coated bars in the cracked concrete would be repaired after 50 years. The bars in concrete containing corrosion inhibitors would be repaired after 16 to 33 years and 50 to 63 years for uncoated and epoxy-coated bars, respectively. No repairs would be needed for the stainless steel bars during the 75-year analysis period.

COST EFFECTIVENESS

In economic analysis it is common to use net present value (NPV) to determine the effectiveness of any strategy. The NPV is calculated as shown in equation 3. Calculation of the net present value of the building and maintenance of bridge decks depends strongly on the discount rate and the timing of maintenance operations.

$$NPV = \sum \frac{R_t}{(1+i)^t} \qquad \ldots (3)$$

Where
R_t = Net cash flow at time t
i = discount rate
t = time of cash flow

O'Reilly et al.[6] reported costs of uncoated, epoxy and 2205 stainless steel reinforcing as \$0.35, \$0.45 and \$2.35 per lb (\$0.77, \$0.99, \$5.17 per kg), respectively. Placement costs were estimated at \$0.52 per lb (\$1.14 per kg) . Further, they reported that the average amount of steel in a deck was approximately 275 lb/yd^3 (165 kg/m3) based upon an average determined from review of 12 bridges. They also reported that the in-place cost of normal concrete was \$562/yd^3 (\$735/m^3)and repair costs were \$283/yd^2 (\$338/m^2) It was assumed that these repairs would last 25 years before an additional, similar repair would be required.

For corrosion inhibitors, the costs for AE, CN and DTS were \$23.00/gal, \$5.00/gal and \$18.75/gal (\$6.07/L, \$1.30/L, \$4.85/L), respectively. To counteract the reduction in strength and low freeze-thaw resistance observed in concrete containing DTS an additional 60 lb/yd^3 (36 kg/m^3) of portland cement at \$0.0625/lb (\$0.1375/kg) was also required.

Using values from Table 2, and the initial and discounted repair costs, life-cycle costs were determined as shown in Table 3. Fig. 5 shows the initial and life-cycle costs for the various systems based upon a reasonable long-term discount rate of 4 percent.

Table 3: Initial and Life-cycle Costs for Various Systems Using a Discount Rate of 4 Percent.

System	Initial Cost \$/yd^2	LCC \$/yd^2
Uncoated reinforcing	\$189	\$444
Epoxy-coated reinforcing	\$196	\$237
Stainless 2205 reinforcing	\$319	\$319
Corrosion inhibitor	\$192 - 197	\$308 - 432
Corrosion inhibitor and Epoxy-coated reinforcing	\$199 – 203	\$224 - 242

System	Initial Cost $\$/m^2$	LCC $\$/m^2$
Uncoated reinforcing	226	531
Epoxy-coated reinforcing	234	283
Stainless 2205 reinforcing	382	382
Corrosion inhibitor	230 - 236	368 – 517
Corrosion inhibitor and Epoxy-coated reinforcing	238 – 243	268 - 289

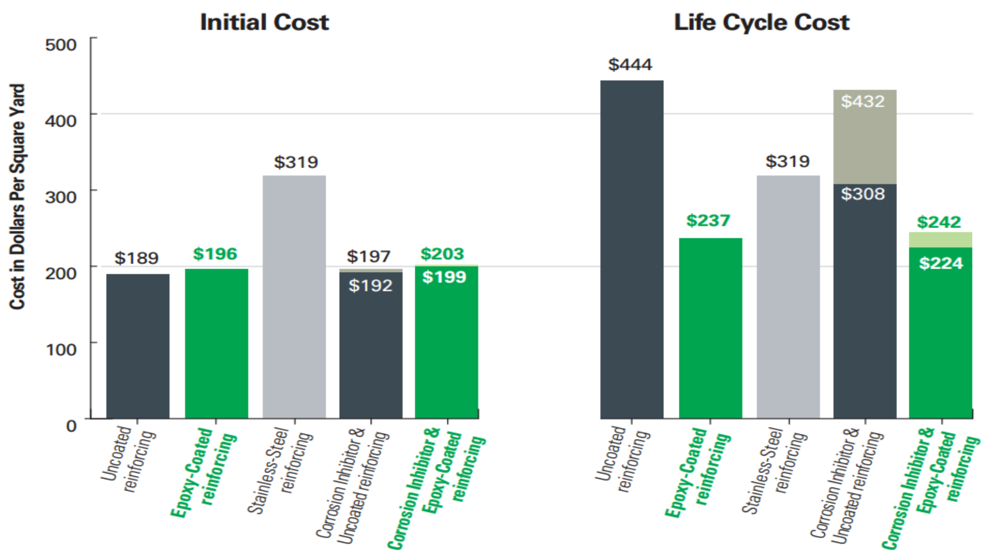

Fig. 5: Initial and Life-cycle costs using a discount rate of 4 percent

CONCLUSIONS FROM O'REILLY ET AL.

Conclusions presented in the report by O'Reilly et al. include:

1. Conventional reinforcement exhibits the highest corrosion rates among all systems studied.
2. While corrosion inhibitors reduce the corrosion rates observed for conventional reinforcement, the combination of conventional reinforcement and corrosion inhibitors is not as cost-effective as epoxy-coated reinforcement.
3. Epoxy coatings significantly reduce corrosion rates compared to conventional reinforcement.
4. Corrosion inhibitors, in conjunction with both epoxy-coated reinforcing steel and conventional reinforcement, reduce corrosion rates in uncracked concrete; however, corrosion inhibitors are significantly less effective in cracked concrete. Corrosion inhibitors also show relatively less effect when used with epoxy-coated reinforcing steel than when used with conventional reinforcement.

5. For bare conventional steel reinforcing bars, the corrosion losses required to crack concrete are directly proportional to the clear concrete cover. For isolated regions of corrosion, such as occurs at damage sites on epoxy-coated reinforcing steel, the relationship changes to one that is directly proportional to square of the concrete cover as the exposed region on the bar decreases. An equation is developed to predict the corrosion losses required to crack concrete for both bare reinforcement and damaged epoxy-coated reinforcement.
6. For the exposure conditions seen on a typical bridge deck in Kansas, stainless steel reinforcement has a present cost over a 75-year design life that is 10 to 20 percent more expensive than epoxy-coated reinforcement.
7. A bridge deck containing conventional reinforcement has the shortest design life of all corrosion protection systems tested. The use of corrosion inhibitors in conjunction with conventional reinforcement increases the design life of the bridge deck; however, the design life remains less than that of conventional epoxy-coated reinforcing steel.

FHWA RESEARCH IN PROGRESS

In 2010, work was initiated by SK Lee at the Turner Fairbanks laboratories of the FHWA that considered 12 different types of reinforcing steel[8]. The objective of this work was to determine the chloride threshold for the various materials and the time to corrosion. This data would be used to determine design guidelines for bridge decks that considered corrosion and life-cycle costs.

Type of reinforcing steel studied included:

- Uncoated reinforcing steel (ASTM A615)
- Epoxy-coated reinforcing (ASTM A775)
- Dual-Coated Bar (ASTM A1055)
    - Reinforcing bars covered with dual coating of zinc alloy and an epoxy coating.
- Galvanized (ASTM A1055)
- Low-Carbon Chromium (ASTM A1035)
- Stainless clad reinforcing (316 stainless)
- Stainless steels:
    - Duracorr (ASTM A1010)
    - 3CR12
    - 2201
    - 2304 (S32304)
    - Enduramet 32 (S24100)

These twelve types of #5 or #6 (16 or 19 mm) reinforcing materials were acquired from 11 sources and embedded in eight concrete slabs as shown in Figs. 6 and 7. The coated products (epoxy, dual-coated and galvanized) had defects prior to placement into the concrete specimens to simulate field damage. These defects represented 0.15, 0.5 and 1.0 percent of the bar surface area. The reinforcing steels were placed in the top and bottom mats.

Fig. 6: Concrete slabs prior to casting of concrete (Lee[8])

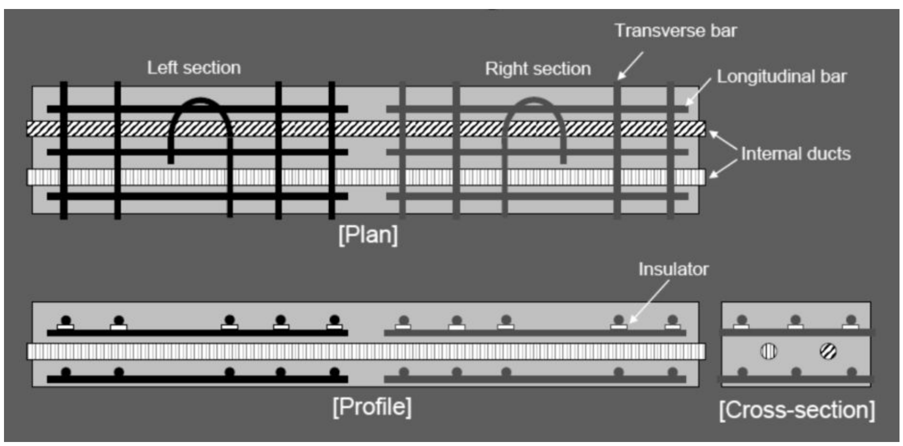

Fig. 7: Schematic diagram of reinforcing bar location in concrete slabs (Lee[8])

The slabs were designed to enable measurement of individual bar performance. These bars were electrically isolated from each other within the concrete slab to facilitate corrosion measurements.

The testing involved ponding the slabs using a weekly cycle with a 15 percent sodium chloride solution. This involved 3 days of wetting, followed by 0.5 days of data collection and 3.5 days of drying. Testing was conducted over a period of approximately 450 days.

During the testing the following data was obtained.

- AC resistance between top and bottom mats
- Corrosion potential of individual top mat bars
- Mixed potential of entire reinforcing in both mats

- Corrosion rate
- Marco-cell current between top and bottom mats
- Linear polarization resistance of individual top mat bars

RESULTS

The final report for this testing has not been presented at the time of writing; however, preliminary results presented by Lee have separated the data into four levels of corrosion protection as shown in Table 4.

Table 4: Corrosion Ranking of Various Bars

Poor	**Fair**	**Good**	**Excellent**
Duracorr Black	Stainless 2201 Stainless 3CR12 Low-Carbon Chromium	Galvanized	Epoxy-Coated Dual-Coated Stainless 2304
Data from corrosion tests for stainless-clad and Enduramet 32 bars are currently inconclusive and further details will be presented in future reports.			

PRELIMINARY FINDINGS

Preliminary findings from the work of Lee include:

1. Use of epoxy-coated, dual-coated and Type 2304 bars offered the best corrosion resistance from the 12 different bars studied.
2. According to AC resistance data, their performance is attributed in part to the large electrochemical resistance between top and bottom mats.
3. Further investigation is required for Enduramet 32 and stainless-clad bars due to mixed corrosion readings.
4. Galvanized bars may be used in moderately corrosive environments.
5. The alloyed bars did not provide adequate corrosion resistance, evidenced by high macro-cell current and/or low polarization resistance.

SUMMARY AND CONCLUSIONS

This paper presents information on various research reports conducted over the past 40 years on corrosion–resisting reinforcing steel. During this period, significant research has been conducted on the performance of epoxy-coated reinforcing steel and generally excellent corrosion performance may be expected. Corrosion tests have been conducted on stainless steel reinforcing steel indicating that the particular chemistry of the steel may play a significant role. Further, life-cycle cost analyses show that the costs of stainless steel may be substantially greater than that of epoxy-coated reinforcing steel.

The paper highlights two recent studies that have been conducted by independent researchers using a wide variety of different corrosion-resistant reinforcing steel types.

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