

## White Paper

# **A CRITICAL REVIEW OF CORROSION PERFORMANCE FOR EPOXY-COATED AND SELECT CORROSION RESISTANT REINFORCEMENTS IN CONCRETE EXPOSED TO CHLORIDES**

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With implementation of a clear roads policy in the United States some fifty years ago and the subsequent wintertime application of deicing salts to northern roadways and bridges, corrosion of reinforcing steel in concrete has evolved to become the single most costly problem of its kind. This has led to concerted efforts by the Federal Highway Administration (FHWA), American Association of State Highway and Transportation Officials (AASHTO), and state transportation agencies to better understand the issues involved and develop mitigation strategies in order to reduce and control damage from this cause. Similar damage also arises for coastal structures exposed to sea water splash and spray. Early research by the National Bureau of Standards (NBS)<sup>1</sup> (now the National Institute of Standards and Technology (NIST)) and FHWA<sup>2</sup> indicated that, for the time frame of the experiments and the conditions studied, powdered epoxy-coated reinforcing steel (ECR) performed well in salt contaminated concrete; and based upon these results, the North American highway community has made extensive use of ECR in bridges during the past 30-plus years.

Disclosure, however, of corrosion induced cracking and spalling of marine bridge substructures in the Florida Keys in the mid-1980's, just seven years after construction and at approximately the same time as projected for black bars (BB), resulted in a number of research studies, both in the United States and elsewhere,<sup>3,4,5,6,7,8,9,10,11,12,13,14,15,16,17,18,19</sup> for the purpose of, first, understanding the coating failure mechanism that had occurred and, second, projecting the long-term performance that should be anticipated with ECR. Of these, laboratory and field studies performed by the Florida Department of Transportation (FDOT) Materials and Corrosion Laboratory<sup>20,21</sup> and by Sagüés,<sup>5,6,12,22</sup> led to the FDOT discontinuing its use of ECR. A finding from a number of the above referenced studies was that ECR experiences corrosion at coating defects along with cathodic disbondment of the adjacent coating. Also noted was wet adhesion loss and underfilm corrosion. An example of this failure process is illustrated by the photographs in Figure 1,<sup>14</sup> which shows the general appearance of three ECR bars upon removal from identically exposed, chloride admixed test yard slabs that had been cyclically tap water ponded. The upper photograph of each bar pair shows black marker dots on the bars that identify presence of a coating defect at the indicated locations, as determined using a holiday detector. Clearly, there is a range of coating holiday densities that vary from low (uppermost bar) to high (lowest bar). The lower photograph of each pair documents the bar appearance subsequent to peeling away disbonded coating using a knife. A one-to-one correlation between defect presence/location, coating disbonding, and underfilm corrosion is apparent. These ECRs were acquired from different coating plants with the providers understanding that they were to be used for research studies resulting from the Florida Key's failures. Also, they were not subject to conventional transport and to jobsite storage, handling, and placement. However, despite failure of these coatings, macrocell current between a lower mat of BB, which was in

chloride-free concrete and remained passive and provided a driving force for corrosion, and the upper ECR mat was low even in the worst case. This latter finding (low macrocell current) has been reported by others as well and has led to the conclusion by some that corrosion at coating defects, adhesion loss, and underfilm corrosion are of little or no consequence. However, uncertainties regarding long-term ECR performance have lingered during the past 20-plus years; and so laboratory studies and bridge deck and substructure field surveys have continued to the present time. What is clear from these is that, where side-by-side comparisons have been possible, ECR has outperformed BB.<sup>7,8,23</sup> What remains uncertain is ECR performance in the long-term and the service life that can be expected in concrete exposed to chlorides. Some have projected an ECR service life of less than fifty years.<sup>11</sup> Examples of results from studies that contribute to this uncertainty are presented and discussed below.



Figure 1: Three ECR bars after exposure in Cl<sup>-</sup> contaminated concrete, first, with coating holidays identified (upper photograph of each bar pair) and , second, showing bar appearance upon removal of disbonded coating (lower photograph of each bar pair).

As a part of research undertakings sponsored by the Concrete Reinforcing Steel Institute (CRSI) and the Canadian Strategic Highway Research Program (C-SHRP), K. C. Clear, Inc. performed 'Southern Exposures' upon a series of concrete slabs reinforced with ECR.<sup>24,25</sup> Such testing is quite harsh and results in high chloride concentrations [Cl<sup>-</sup>] at the bar depth. In spite of this, macrocell current remained low after one year; and the coating apparently continued to provide protection to the underlying steel. The testing was then terminated, and the slabs were continuously tap water ponded for an additional 10.5 months followed by outdoor exposure to natural weathering for 9.5 months. Upon reexamining these slabs, it was found that the coating had embrittled, disbonded, and cracked and active corrosion of the underlying steel was ongoing.

Based upon both laboratory and field studies, Weyers et al.<sup>26,27</sup> reported ECR coating disbondment and underfilm corrosion on bridges in Virginia; and as a consequence, the Virginia Department of Transportation (VDOT) discontinued use of ECR in 2010.

The Ontario Ministry of Transportation (MTO) has reported corrosion of ECR in concrete barrier walls and unprotected portions of bridge decks.<sup>28</sup> Because MTO specifies waterproofing membranes, the ECR attack on decks has been limited to the vicinity of expansion joints. That agency now specifies stainless steel for structure barrier walls and for decks of high traffic volume bridges for which repair/rehabilitation dictated lane closures would be particularly disruptive. Further, Clause 6.1.3 of the Canadian Standards Association<sup>29</sup> titled, "Special Reinforcement," states the following:

"Special reinforcement, such as epoxy-coated steel, galvanized steel, stainless steel, cadmium-plated steel, or other material not covered by Clause 6.1.1 or 6.1.4 shall be used only when specified or approved by the owner.

Note: Such reinforcement should be selected with caution, based on the severity of the concrete exposure and the desired service life of the concrete component or structure.

There is a growing body of knowledge suggesting that the benefits of epoxy coatings for long-term corrosion protection are not what was originally anticipated. Potential users should review recent literature on the subject for further information."

Findings from a comprehensive field survey and laboratory analysis of 240 extracted bar segments from 80 bridge decks with ECR in Pennsylvania and New York of age 4-18 years determined generally good performance but with identification of some locations where the above failure processes had commenced.<sup>30</sup> If the determined state of these ECR were linearly extrapolated to the future, one would conclude that a 75 year low maintenance service life could be realized. However, linear extrapolation is inappropriate for such failure processes, as is clearly evident in the case of BB, for which corrosion damage is nil as long as the steel remains passive but progressively accelerates once a critical  $\text{Cl}^-$  concentration is achieved at the bar depth. Inappropriateness of linear extrapolation is also apparent from the Canadian experience (described above) in that Hededahl and Manning<sup>4</sup> reported in 1989 that ECR was performing well in cracked, chloride contaminated barrier walls with shallow rebar cover and in bridge decks; however, by 1996 active ECR corrosion and concrete cracking and spalling were noted on barrier walls and at bridge deck expansion joints.<sup>17</sup>

The above findings have led independent experts to conclude that, to-date, ECR is performing well in high quality concrete with good cover but not in situations where either of these two conditions (good quality concrete and cover) is not met.

A fundamental difficulty in projecting future ECR performance is that, first, long-term deterioration processes are, for the most part, not necessarily reflected by results from short-term exposures and, second, it is not possible to perform first-principles based modeling of the coating disbondment, embrittlement, and underfilm corrosion processes. Such modeling can be performed for uncoated bars, however, provided, first, the critical  $\text{Cl}^-$  concentration,  $C_T$ , for passive film breakdown and active corrosion initiation of the alloy in question and, second, subsequent corrosion rate are known, as is now the case for BB and some corrosion resistant reinforcements. In this regard, numerous studies have concluded that  $C_T$  for stainless steels such as UNS-S31653 (316LN) and UNS-S31803 (2205) exceeds the  $\text{Cl}^-$  concentration that is reached at the reinforcement depth, even in the long-term; and so active corrosion does not initiate. For alloys of lesser noble element addition and, hence, reduced cost such as UNS-S41003 (3Cr12, a 12 wt% Cr ferritic stainless) and A1035 (MMFX2™),  $C_T$  data are available which indicate a mean value approximately four times greater than for BB;<sup>31,32,33,34</sup> and modeling studies have been performed that project long-term performance. Invariably, these utilize the first-principles based, one dimensional solution to Fick's second law of diffusion with relevant variables, in addition to  $C_T$ , being the 1) effective diffusion coefficient (D), 2)  $\text{Cl}^-$  concentration at

the concrete surface ( $C_s$ ), and 3) concrete cover over the reinforcement ( $x$ ). A recent enhancement of this approach has been developed<sup>35,36</sup> that treats each of the above four variables as distributed rather than distinct, as is now known to be the case, because of, first, the heterogeneous nature of concrete and, second, construction variability. Table 1 lists mean ( $\mu$ ) and standard deviation ( $\sigma$ ) values for each of the controlling four variables which are representative of actual service and which were employed in the first-principles based analysis methodology to project the evolution of corrosion initiation for both BB and the two corrosion resistant reinforcements (3Cr12 and MMFX2™). In the analyses, the distribution for all four variables was assumed to be normal, as has been reported by others.<sup>37</sup> The indicated  $\mu(D)$  value is typical of good quality portland cement concrete, the  $\mu(C_s)$  is a conservative upper limit for northern US bridge decks that receive wintertime application of deicing salts,<sup>38</sup> the  $\mu(x)$  is a typical design value for the upper reinforcement mat on bridge decks, and the  $\mu(C_T)$  are as reported from a recently completed FHWA sponsored study.<sup>33,34</sup> Standard deviations reflect what has been reported in these same or related studies. The analysis considers that the concrete is comprised of identical, electrochemically independent elements, all of which receive the same exposure. In a case of varied exposure, then the model can be expanded to include these. The results are displayed as a cumulative distribution function (CDF) plot that relates the percentage of elements for which active corrosion has commenced as a function of time, as shown in Figure 2. The general trend is one where elements remain passive (no active corrosion) for some period of time (6.5 years for BB and 63 years for the corrosion resistant reinforcements (CRR) to achieve one percent of elements having initiated corrosion), beyond which corrosion initiation rate progressively increases with time until an inflection point is reached and then progressively decreases. Of particular significance is that, not only is the rate of first corrosion increased by an order of magnitude, but the progressive rate of corrosion initiation for the CRR subsequent to first occurrence is much more moderate than for the BB. This trend results because the rate at which chlorides accumulate at the reinforcement depth subsequent to the respective  $C_T$  being achieved is less for the CRR.<sup>36</sup> Table 2 lists the ratio of time-to-corrosion-initiation,  $T_i$ , for the CRR compared to BB at CDFs of 1, 10, 20, and 30 percent active. The results show that  $T_i$  at a CDF of one percent active is almost ten times greater for the CRR than BB, and this improvement factor becomes progressively greater with increasing CDF. This projection is in excellent agreement with what has been reported from test yard exposures.

Table 1: Listing of parameter values for the example analysis.

Parameter/units	Value
$\mu(D)$ , m <sup>2</sup> /s	$3 \cdot 10^{-12}$
$\sigma(D)$ , m <sup>2</sup> /s	$1 \cdot 10^{-12}$
$\mu(C_s)$ , wt% cement	4.00
$\sigma(C_s)$ , wt% cement	1.00
$\mu(x)$ , cm	7.50
$\sigma(x)$ , cm	0.75
$\mu(C_T)$ BB, wt% cement	0.896
$\sigma(C_T)$ BB, wt% cement	0.260
$\mu(C_T)$ CRR, wt% cement	3.580
$\sigma(C_T)$ CRR, wt% cement	0.358

Three factors are responsible for this good performance of the CRR in question, as listed below:

1. Higher concentration of noble metal alloying (Cr) and, in the case of MMFX2™, a refined, low carbide microstructure. This provides a factor of 4-5 enhancement in  $T_i(\text{CRR})/T_i(\text{BB})$ .
2. A slower rate of chloride uptake at a given depth into the concrete subsequent to corrosion initiation the higher  $\mu(C_T)$ .<sup>35</sup> This slows the rate of continued corrosion initiation proportionally.
3. Overlap of the  $C_s$  and  $C_T$  distributions in the case of the CRR but not BB, as recently demonstrated.<sup>36</sup> Thus, elements for which  $C_T$  exceeds  $C_s$  will not initiate corrosion irrespective of the exposure time involved. It is for this reason that the CDF trend for the CRR in Figure 2 asymptotically approaches an upper limit at long time that is well below 100 percent, whereas the BB trend approaches a limit of 100 percent active.

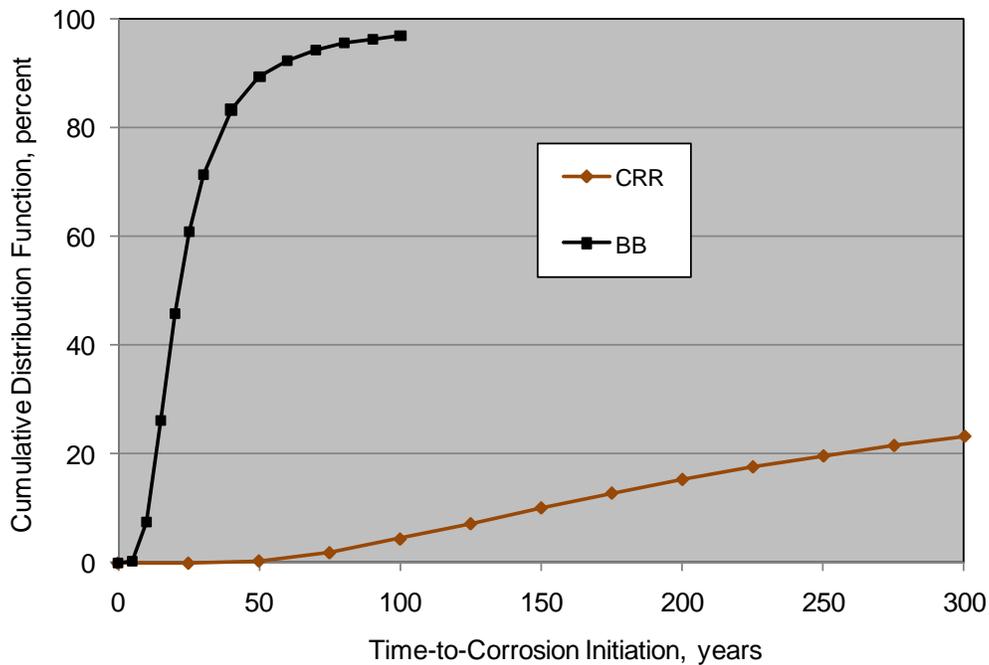


Figure 2: CDF plot of time for corrosion initiation of BB and CRR using a first-principles based analysis approach.

Table 2: Listing of the CRR-to-BB  $T_i$  ratio at various CDF.

CDF, percent	$T_i(\text{CRR})/T_i(\text{BB})$
1	9.7
10	13.7
20	18.9
30	-

Together, these three considerations provide a time enhancement factor of approximately ten for onset of initial active corrosion for the CRRs under consideration (3Cr12 and MMFX2™) compared to BB, and this time enhancement becomes even greater as corrosion initiation of

other, still passive elements progresses. The present analysis does not include the additional time required for corrosion to progress to the point where concrete surface cracks, delaminations, and spalls appear; but this adds further to the times reflected in Figure 2 and Table 2.

In summary, long-term performance of ECR in chloride contaminated concrete is uncertain; and no first-principles based analytical tools are available for projecting this. On the other hand, sufficient data are available, along with a first-principles based methodology, for projecting long-term performance of uncoated BB and CRRs such as 3Cr12 and MMFX2™. Results from the latter analysis indicate that in the case of these CRRs a low maintenance service life of 75 and even 100 years can be expected.

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