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FINAL REPORT (Project BDV 25 977-81)

Synthesis of galvanized steel reinforcement corrosion performance

Final Report to Florida Department of Transportation

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Universal conversion table

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
AREA				
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				
MASS				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
TEMPERATURE (exact degrees)				
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
ILLUMINATION				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
FORCE and PRESSURE or STRESS				
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
AREA				
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
VOLUME				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
TEMPERATURE (exact degrees)				
°C	Celsius	1.8C+32	Fahrenheit	°F
ILLUMINATION				
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
FORCE and PRESSURE or STRESS				
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

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16. Abstract

Hot dipped galvanized steel rebar (HDGR) is an alternative to plain carbon steel often considered for concrete reinforcement. The zinc rich layer serves as a sacrificial coating protecting the underlying carbon steel. While there is information reporting benefits from increased chloride threshold and reduced corrosion rates, there is also uncertainty regarding the conditions required to ensure this benefit and also avoid possible performance deficiency. Therefore prior to possible approval and implementation of this material in aggressive marine environments of interest to FDOT, existing evidence on HDGR corrosion performance needs to be evaluated. This report is a synthesis review of the technical literature, conducted per request of the FDOT, towards a determination of whether to eventually allow the use of HDGR to increase durability. The performance evidence reviewed was indicative of the need for further field exposure and experimental testing prior to consider specification of the material. A phased approach to that end was proposed for consideration.

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Executive summary

Hot dipped galvanized steel rebar (HDGR) is an alternative to plain carbon steel often considered for concrete reinforcement. The zinc coating is expected to provide some resistance to corrosion initiation as well as, afterwards, sacrificial protection to the internal carbon steel since zinc is less noble than carbon steel. However, in alkaline environments typical of concrete zinc readily corrodes, so adequate balance between various processes need to exist for successful corrosion performance. That performance has been documented in the literature for various service conditions, but there is question of its suitability for the marine splash zone in Florida structures, where extremely aggressive chloride ion regimes exist. This report is a synthesis review of the technical literature, towards a determination of whether to allow the use of HDGR to increase durability. The review is focused on splash zone conditions and is restricted to traditional HDGR production material. The review was framed as answers to key questions on characteristics and behavior of the material in anticipated service; potentials benefits or negative consequences of its deployment; whether there is at present enough documentation to make a determination to allow specification by FDOT or not, and if documentation is not enough, what could be a path forward toward making that determination.

The technical evidence examined in this Synthesis was deemed to be sufficient only for further consideration of HDGR as an allowable steel bar material for inclusion in the FDOT Standard Specifications for Roads and Bridges. While some potential benefits of specifying HDGR are attractive, immediate deployment of HDGR especially in Florida's highly aggressive marine substructure service is not recommended. Instead, a phased approach is advised, including targeted testing and incorporation as an experimental feature of selected components in coming FDOT bridge projects.

Based on the findings of the review, it is proposed that evaluation consist first of assessing whether necessary materials properties are met, including manufacture's ability to produce bar with a minimum eta layer coating thickness, to maintain proper control of preplacement corrosion as in the construction yard, and to assure that concrete mixture materials and proportioning is possible to keep pore water pH and related properties within a restricted value envelope. Should outcome of all the above phases be adequate, field performance tests by means of side-by-side PSR and HDGR test slabs with preexisting cracks in actual bridge locations need to satisfy specified resolution criteria over a decade-long exposure. If that outcome is in turn, fully adequate, it is recommended for consideration that tentative HDGR deployment be conducted in a limited form, as an experimental feature in the form of one bridge pier incorporated as part of a selected new bridge construction.

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1   Introduction

Hot dipped galvanized steel rebar (HDGR) is an alternative to plain carbon steel often considered for concrete reinforcement. The zinc coating is expected to provide some resistance to corrosion initiation as well as, afterwards, sacrificial protection to the internal carbon steel since zinc is less noble than carbon steel. However, in alkaline environments typical of concrete zinc readily corrodes, so certain early corrosion products need to form first to act as an initial protective barrier. Thus, adequate balance between those processes need to exist for successful corrosion performance. That performance has been thoroughly documented in the literature for various service conditions, but there still exists some disagreement in its suitability for other conditions such as in the marine splash zone in Florida structures, where extremely aggressive chloride ion regimes exist. This report is a synthesis review of the technical literature, conducted per request of the FDOT, towards a determination of whether to allow the use of HDGR to increase durability. Also by request, the scope of the review is focused on splash zone conditions and is restricted to traditional HDGR production material, as opposed to other recently implemented zinc coating variations such as continuously galvanized rebar.

The literature was reviewed considering a two-stage corrosion service life model. The review includes existing data, reports, case studies etc. on the use of HDG in reinforced concrete. From the synthesis of results, the extent to which the Department should consider allowing HDGR in their highway structures to increase service life is examined. A roadmap is presented for addressing any issues that were identified as needing resolution before a final determination.

1.1   Corrosion-limited service life concepts

This section introduces elemental concepts and terminology that are used in subsequent portions of this report.

Corrosion service life forecasting of reinforced concrete elements can proceed via a two-stage initiation/propagation model as shown Figure 1-1. [1] The initiation stage duration (t_i) is governed by the transport of penetrating substances into the concrete cover (C) and by the threshold concentration of those substances required to start the corrosion process (C_T). The propagation duration (t_p) is a function of the critical steel loss to cause cracking of concrete cover (X_{crit}), and corrosion rate of the steel reinforcement (CR), which is influenced by different factors such as moisture content, temperature around corroding areas, chemical composition of the concrete pore solution, concrete cover thickness, among others. A serviceability limit state, often pre-defined as some quantifiable degree of concrete cracking, bounds the end of the propagation stage. The amount of corrosion required to cause the concrete to crack (X_{crit}) is expressed as the radial loss of the reinforcement due to corrosion. The service life of the reinforced concrete structure is given by the summation of the time to corrosion initiation and the duration of the corrosion propagation period.
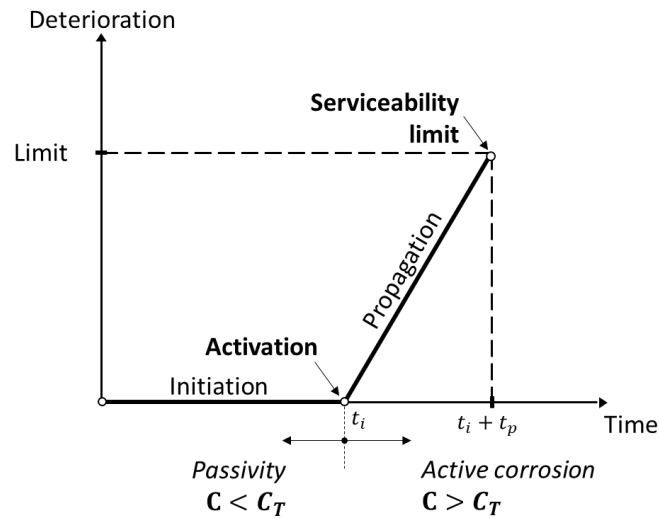
Figure 1-1: Simplified two-stage corrosion modeling for PSR reinforced concrete elements. Variations of this model have been proposed for HDGR [2].

In marine environments or regions where de-icing salts are used, chlorides transport into the concrete and cause corrosion of the reinforcing steel. The mechanism for chloride transport depends on the degree of saturation of the concrete pore solution. When concrete capillary pores are fully saturated, diffusion represents the primary transport mechanism of free chlorides into the concrete microstructure. On the contrary, when the concrete pore system is dry, absorption takes place as the main transport mechanism. Permeation can also occur when pores are filled with air or water due to an absolute pressure difference.

The time of chloride-induced corrosion initiation in sound concrete based on diffusion as the primary mechanism may be quantified based on exposure and materials conditions and system geometry. For example, for a simpler planar geometry one obtains equation 1-1, derived from the solution to Fick's second law of diffusion.

$$t_i = \frac{x^2}{4D_a \left(\mathrm{erf}^{-1} \left(1 - \frac{C_t}{C_s} \right) \right)^2}$$ 1-1

where:

t = time to initiation, s.
x = depth below exposed surface to the middle of the layer, m,
D_a = apparent chloride diffusion coefficient, m^2/s , and
C_t = chloride threshold, mass %,
C_s = predicted chloride concentration at the surface determined by regression analysis, mass %,
x = depth below exposed surface to the middle of the layer, m.

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According to this equation, it is assumed that the concentration of chlorides is constant and uniform across the surface. It is also assumed that the diffusion of chlorides occurs in a homogeneous material such that the diffusivity is not a function of position or time. Since this is not the case in actual concrete, the diffusivity is referred to as an apparent diffusivity. Nevertheless, this formulation has provided a reasonable means of estimating durability for sound concrete in regard to chloride induced corrosion. In the presence of preexisting cracks chloride transport to the rebar surface can be much faster, with consequent decrease in the value of t_i to an extent depending on many variables. Rough estimates of that reduced initiation time can nevertheless be conducted by suitable approximations, as detailed elsewhere. [3]

The corrosion propagation stage represents the period from corrosion initiation to the formation of corrosion-induced cracks in the reinforced concrete element. The stage duration is provided by the ratio of the critical corrosion penetration needed to crack the concrete (X_{crit}) and the corrosion rate (CR) if it is assumed to be constant over time. The corrosion rate of steel in concrete is controlled by exposure conditions and concrete characteristics. In this sense, it depends on several factors that include the cement chemical composition, concrete microstructure, temperature and moisture content, oxygen availability, and pH of the concrete pore solution. In the case of HDGR the propagation stage may be further conceived as a two stage process, first involving the loss of the galvanized layer, and then that of the base carbon steel each with its own effective CR value [2]. However, as in any modeling approach, the usefulness of more detailed treatments depends on the availability of sufficiently accurate input data, which is at present subject to considerable uncertainty.

The critical corrosion penetration to crack the concrete cover (X_{crit}) represents the tolerance to the accumulation of expansive steel corrosion products of a concrete element. As the reinforcement bar corrodes, corrosion products push into the concrete pore structure. Due to the expansive nature of corrosion products, once the transport of the corrosion products into the pores of the concrete becomes blocked, further corrosion imposes stress on the concrete matrix which eventually leads to cracking. X_{crit} values are highly influenced by variations in cover thickness, concrete mechanical properties such as compressive and tensile strength, and the concrete pore distribution. [4] The average cross-sectional steel loss to cause cracking of the concrete is represented in Figure 1-2.
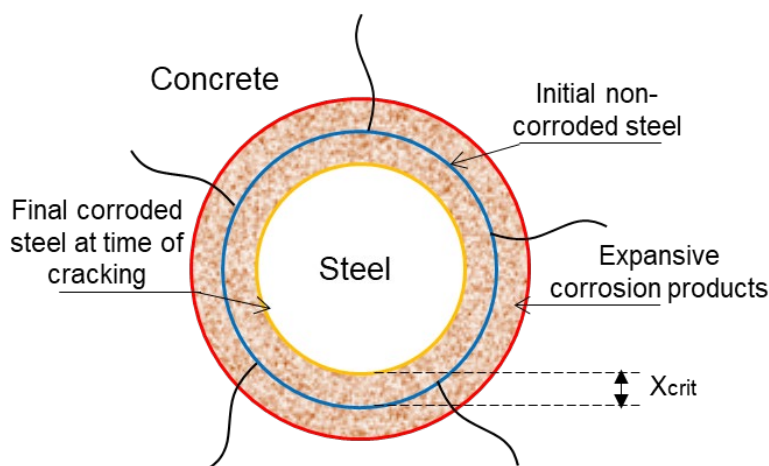

Figure 1-2: Average cross-sectional steel loss to cause cracking of the concrete.

Some empirical models have been proposed to determine X_crit for carbon steel reinforced OPC-based concrete, one of which is expressed in Equation 1-2.[4]

$$X_{crit} \sim 0.01 \; mm \frac{x}{\varphi}\left(1+\frac{x}{L}\right)^{n}$$

1-2

where:
X_{crit} = critical corrosion penetration needed to cause cracking of the concrete, mm,
x= concrete cover thickness, cm,
φ= rebar diameter, cm,
L= length of the corroding region of steel, cm.
n= constant ~1.5 to ~2

Alternative treatments to obtain the duration of both stages, with roughly comparable forecasting outcomes have been proposed elsewhere. [5, 6]

For the case of HDGR, it has been reported that the corrosion products coming from the dissolution of the galvanized layer tend to travel further away from the rebar surface than those form plain steel corrosion [7], resulting in less stress and hence increasing accordingly the value of X_crit. The latter in this case should be understood as an effective value, a function of the metal loss of the galvanized layer and of the base carbon steel which may be modeled. Reliable model treatments for that behavior are yet to be developed.

2   Methods

2.1   Search methods and survey scope

This is a synthesis overview of existing literature rather than a detailed enumeration of prior research. Several such exhaustive surveys are readily available elsewhere [2, 8-12] so they will not be replicated here. Instead, the work will focus on the overall findings of presently available evidence, with details noted as necessary to provide answers to the objective-related questions listed in the next section.

2.2   Questions addressing the objective of this synthesis

The synthesis review has been structured with a view to advise the FDOT on the following questions:

A. What is Hot-Dip Galvanized Rebar (HDGR) as available today?
   1- How is HDGR made, and what are its metallurgical properties?
   2- How does HDGR interact electrochemically and mechanically with the surrounding concrete?
   3- What is the history and present implementation of HDGR application to reinforced concrete, especially for conditions relevant to FDOT marine substructure service?
   4- What is the availability and cost of HDGR relative to plain steel "black" rebar (PSR)?

B. What would be, and to which extent, the potential benefits or negative consequences factors in specifying of using HDGR instead of PSR for FDOT marine substructure service?
   1- Would HDGR delay the onset of corrosion, and slow the propagation of active corrosion compared to PSR?
   2- Is there enough documentation of verified HDGR performance in actual field conditions to justify recommending its use instead of PSR?
   3- Could HDGR substitution for PSR result in risk of markedly reduced corrosion performance or adverse mechanical or structural consequences?

C. What is the recommended path forward toward deciding whether to specify HDGR?
   1- Is there sufficient data to make a determination, and if so which is it?
   2- If Item 1 is in the negative, what is the recommended test plan and criterion to make a determination?

3 A: What is Hot-Dip Galvanized Rebar (HDGR) as available today?

3.1 A1: How is HDGR made, and what are its metallurgical properties?

Common U.S. practice follows Standard Specification ASTM A-767 [13]. HGDR is produced by taking ordinary ASTM A615 rebar stock of the desired size and grade, acid-pickling it to remove mill scale, dipping in an aqueous solution (such as zinc ammonium chloride) to increase surface reactivity, immersing ("hot-dipping") in molten (~460 C) commercially pure zinc bath for a few minutes to form the solid galvanized layer on the steel surface, and slowly removing the bar from the bath while agitating/wiping surface to avoid excessive retention of molten zinc on the emerging bar[9]. ASTM A-767 requires a post- extraction chromate treatment intended to control surface reactivity during early curing. That treatment may be done by dipping the just-extracted hot bar into an aqueous sodium dichromate solution (quenching). Alternatively, air-cooled bars may be dipped into a sodium dichromate solution with sulfuric acid added as an activator.

The HDG rebar galvanized coating thus produced consists of 4 Fe-Zn alloy layers (in order starting from the base carbon steel: Gamma, Delta, Zeta and Eta) with a total thickness in the order of 100 μm. The layers relative thicknesses vary with processing and materials circumstances, but the example in Figure 3-1 is representative. Alloy Zn content increases from Gamma (~75%), to Delta (~90%), to Zeta (~94%) and Eta (nearly pure Zn). Detailed descriptions of these phases are available elsewhere [10, 14].
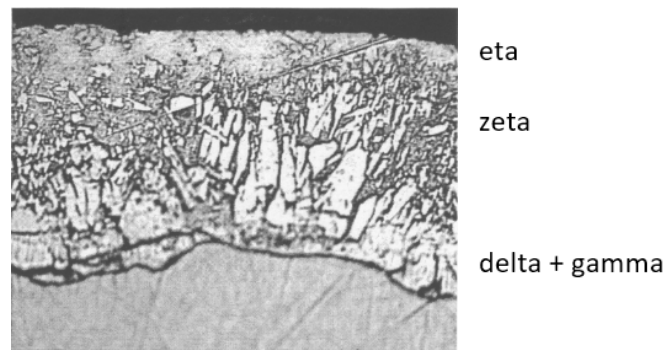
Figure 3-1. Representative metallographic cross section of galvanized coating. Picture field is 175 μm high.[15]

The effect of the various layers and chromate treatment on corrosion performance will be addressed when providing answers to the specific relevant questions. Bulk mechanical strength and ductility of normally produced HDGR is essentially the same as that of PSR [9]. However, sharp-radius bending of HDGR after galvanizing can lead to coating cracking and disbonding. Thus, if heavy fabrication is needed it is recommended that it take place with the bare steel bar prior to galvanizing. [2]

The fabricated rebar is made into a rebar assembly using tie wires that are recommended to be galvanized or plastic-coated steel. It is also recommended that cut ends and areas of damage of the bar coating be patched with zinc-rich paint or zinc solder. [16] Otherwise no provisions different from those used for PSR are deemed for rebar placement in forms, concrete casting, compacting (including no special type of vibrator) and finishing. [16]

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3.2 A2: How does HDGR interact electrochemically and mechanically with the surrounding concrete?

Embedded rebar interacts electrochemically with the liquid phase of concrete from the moment of casting and throughout the entire service life of the structure. The water in freshly mixed Portland cement concrete, with or without typically used admixtures in FDOT applications (notably Fly Ash and Microsilica), is a highly alkaline medium. Usually initial water pH tends to approach 12.5, reflecting the overwhelming presence of Portlandite ($Ca(OH)_2$). As the concrete sets and solid phases from cement hydration form a composite material with the aggregates, the liquid phase is present as pore water which is increasingly enriched by soluble alkali ions (mainly K+ and Na+). The pH of the pore water increases accordingly, to long-term values typically between 13 and 13.5 [17, 18]. It is noted that in some instances, for example with high slag content blended cement, the higher pH regime may evolve earlier [18, 19].

Zinc and Zn alloys oxidize readily in water, forming soluble ions at high (ZnO_2-, zincate) and low (Zn^{++}) pH values respectively. At intermediate pH regimes, nearly insoluble solid corrosion products form such as ZnO_2, $Zn(OH)_2$ and, in the presence of calcium ions as in concrete pore water, zinc calcium hydroxyzincate, $Ca(Zn(OH)_3)_2 \cdot 2H_2O$ (CHZ). The latter tends to accumulate as a compact deposit on the Zn surface, acting as a protective barrier and effectively suppressing further corrosion until aggressive agents arrive in sufficient quantity from the outside [14]. It is noted that while the blocking effect is comparable to that of an oxide passive film, the hydroxyzincate deposit is much thicker (μm vs nm range) and microstructurally textured than the former[20].

The conditions for formation and stability of the protective deposit prior to ingress of external aggressive agents are highly relevant to the applications of interest to FDOT, and will be examined first, followed by findings on performance upon external chloride ingress during service. These subjects have received attention, and the following salient points reflect the detailed 2004 review by Andrade[14], generally confirmed and supplemented by subsequent investigations in the last two decades unless otherwise noted. Other relevant work has been consulted as well. [21-30]

3.2.1 Deposit formation and stability

- In liquid phase solutions the protective CHZ deposit forms spontaneously in the pH range ~11.8 to ~13.2, with a galvanized layer consumption slow enough that usually much of the eta layer is not consumed.
- For water phase pH values increasingly closer to the upper end of that range the protective deposit becomes coarser grained and less organized, and the zinc corrosion rates during formation become larger. Work by others [19, 31] has confirmed that behavior. At the upper end of the noted pH range -and above it- the eta layer may be completely consumed, exposing the less Zn-rich zeta layer. That layer and the increasingly Fe-rich layers beneath it react with the water phase by selective dissolution of the Zn, leaving on the bar surface a porous deposit insufficiently protective of subsequent chloride ingress during service.

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- From the above, it is clearly important that the eta layer of the as-produced HDGR be thick enough to sustain the consumption needed for formation of the protective CHZ deposit (expected to be more demanding if the concrete pore water pH is near the high end of the desirable range), and still have a significant residual thickness afterwards. An as-produced minimum thickness of ~10 um has been recommended to that end [14]. Likewise, variations of HDGR such as post hot-dipping galvannealing, which converts the coating into a more uniform composition Zn-Alloy layer, were strongly discouraged[14].
- As long as the pH remains within the aforementioned range and the eta layer is still present, the presence of even appreciable amounts of Cl- (e.g., 0.5 M) in the liquid phase during early curing was not found to severely interfere with the protective deposit formation. However, mix contaminants such as alkaline chlorides (e.g. NaCl) can simultaneously increase the pH to beyond the upper stability deposit limit with detrimental results. On the other hand, admixed $CaCl_2$ which promotes $Ca(OH)_2$ formation with consequently less pH elevation, is expected to be less detrimental to deposit formation on that count. The overall effect of the Cl- ions in that admixture can nevertheless be detrimental as will be shown later.
- Once formed and in the absence of chlorides, the mature protective deposit has been found to resist pH values as high as 13.6 without dissolving. That is helpful because, from the circumstances noted above, for most concretes the less alkaline liquid phase pH values prevalent during early curing (e.g., ~12.5)[14, 17, 32] are within the range that readily promotes formation of the deposits. The higher pore water pH values (e.g. 13.5) that tend to develop later, [14, 17] and that might have interfered with deposit formation, encounter instead a mature deposit resistant to dissolution.
- It should be noted that agreement with the scenarios presented above about protective deposit formation and stability is not universal. For example, work by Pernicova [33] disputes the protective character of CHZ when the pH exceeds that of saturated calcium hydroxide solutions, and cites failures of small scale HDGR structures from the 1950s to early 1970s as being at least in part a result. While in the minority, these concerns as well as those reviewed here later merit attention when considering large scale deployment of HDGR.

3.2.2 Effects on surrounding concrete

- Formation of the protective deposit is not instantaneous, extending to a duration from hours to (in less propitious conditions) even days after concrete casting [14, 19, 32, 34]. The eta layer reaction that forms the CHZ, produces hydrogen gas during the intermediate step $Zn+2H_2O \rightarrow Zn(OH)_2+H_2$. The extent to which hydrogen evolution takes place could have important consequences, because H_2 gas bubbles at the rebar-concrete interface can potentially reduce mechanical bond strength as well as possibly facilitate onset of localized corrosion on exposure to chlorides. The chromate treatment specified in ASTM 767 [13] is intended to inhibit the H2 evolution reaction during early curing, thus alleviating bubble formation – although extending the duration of the protective deposit formation.[14] It is noted that some cements contain significant amounts of native Cr, with a resulting H_2 evolution decrease comparable to that of the chromate treatment [14, 28, 35]. It is also noted that the ASTM 767 treatment involves hexavalent Cr ions which are environmentally hazardous. That is a strong incentive for abandoning the practice if its benefits are marginal or uncertain, as well as finding replacements based on trivalent chromium [35].

- The potential hydrogen evolution-induced decrease in galvanized rebar bond strength (as well as from other sources such as thickening of the galvanized coating at low points of the ribs) has been reviewed in detail by Kayali [36, 37] who concluded that galvanizing of corrugated rebar has if anything a slight bond strength increase, and that chromate treatment was not necessary. Andrade [14] indicated that subsequent CHZ formation tended to fill the H2-induced porosity anyway so that adverse bond effects would be avoided and obviating the need for a chromate treatment. The 2009 review Federation International du Beton (fib) Task Group 9.7 [9] concluded that the chromate passivating treatment was "not at all necessary" and also that research "universally demonstrated that there was no reduction in bond strength for galvanized bars compared to equivalent non-galvanized bars". A more recent (2017) and highly detailed review by Pokorny [10] was however much less optimistic, citing more recent research that documented loss of bond between 4% and 15% for HDGR vs PSR, uncertainty derived from the mechanical test methods chosen, and urging continuing research on this issue.[38, 39] The aforementioned work by Pernicova showed also some evidence of severe gas formation at the rebar-concrete interface and cites the recent work by Pokorny and others highlighting instances of bond loss. Again these observations appear to be in the minority but suggest caution when considering HDGR deployment.

3.2.3 Performance upon external chloride penetration in service

- Much of the available literature, summarized in a detailed 2018 state of the art report by NACE International [8] reports CT values for HDGR that are in the order of 1.5 to 4 times larger than those of PSR in similar circumstances. An increased value of CT for Zinc over that of plain steel would be expected from laboratory work comparing behavior of both materials in calcium hydroxide solutions, which showed pitting initiation chloride concentrations of 0.08 M for steel compared to 0.45 M for Zinc [11, 14]. The relative effect however may be different in solutions with added alkali species that increase the pH above that obtained with calcium hydroxide only. Furthermore, Andrade [14] warns that increases in threshold may be expected when there is a remaining external, nearly pure Zn eta layer after the earlier protective CHZ deposit formation, but not necessarily if the eta layer has been stripped exposing the Fe-Zn intermetallic layers beneath. The extent of the benefit observed in sound concrete exposed to external chloride sources varies significantly, including cases of no significant benefit. For example, one of the most thorough laboratory investigations available [40] with ponding tests indicated a HDGR/PSR CT ratio in the order of 1.6, and later work by the same group indicated a CT ratio of average values of about 1.25 [41]. Work elsewhere for marine shore exposure [42] indicated a CT ratio of about 3. Other ponding tests by McDonald [43, 44] indicated no significant difference in the duration of the initiation stage of HDGR and PSR. Limited tests by Moreno in concrete exposed to partial immersion in saltwater [15] showed likewise no significant difference in the duration of the initiation stage of HDGR and PSR in concrete. Work by Clear with ponded concrete [45] showed no distinct improvement of HDGR over PSR for the duration t_i.

- Once the threshold is reached at the bar surface, corrosion propagation affects first the galvanized layer with accumulation of Zn-rich corrosion products in the immediate surroundings. It has been often repeated in the literature that those products tend to diffuse further away from the bar surface than the products produced by corrosion of steel [2, 7, 12, 14] but the evidence for that assertion is to this day still quite limited [7, 46]. As indicated in the Introduction, such extended transport of corrosion products should result in a favorable contribution to increase the duration of the propagation stage, as discussed further below.
- As the galvanized layer becomes increasingly consumed, the corrosion potential can be quite negative [47] and it may provide substantial cathodic corrosion prevention to any small base carbon steel spots that may become exposed by locally greater galvanized layer consumption. However, as full galvanized layer consumption affects increasingly larger fractions of the bar surface, the preventive effect becomes less important due to limited throwing power of the anodic regions. Corrosion modality then effectively reverts to that of PSR. The overall effect of those process on any extension of the duration of the propagation stage relative to that of PSR can be modeled but reliable input data remain elusive. Experimental evidence is limited. Work by O'Reilly [48] found that galvanized reinforcement required over twice the corrosion loss to reach a concrete cracking limit state, information indicative of X_{crit} but not necessarily of the value of t_p (see Introduction). This uncertainty has led some authors [5] to assign no additional credit on the duration t_p for HDGR over PSR, although in fairness it should be noted that similar sweeping assumptions have been made even for stainless steel reinforcement [49].

3.3 A3: What is the history and present implementation of HDGR application to reinforced concrete, especially for conditions relevant to FDOT marine substructure service?

HDGR has been used to reinforce concrete in appreciable quantities since mid-20th century [2]. Use of HDGR for highway construction applications in the U.S. is a small fraction of the total rebar tonnage, which is dominated by PSR and Epoxy-coated Rebar (ECR). Numerous laboratory studies have been conducted to assess corrosion performance, but documented experience under field conditions is comparatively limited. In the field, HDGR has been used in multiple occasions to construct bridge decks and related components exposed to deicing salt service, providing a decades-long experience base. In that service, surface chloride levels and concrete moisture levels are usually not as severe as in lower marine substructure, but variability of conditions resulted in some cases of chloride contamination high enough to be comparable to that in marine service. Of greater interest, HDGR has also been used for some decades-long service in marine field applications and outdoor exposure tests in conditions directly relevant to those of interest to FDOT. The outcome of evaluations is discussed later.

3.4 A3-What is the availability and cost of HDGR relative to plain steel "black" rebar (PSR)?

Production of HDGR has been in sufficient capacity, quality control and sustained commercial availability to timely supply construction needs of multiple full-size projects. Those range in volume from medium [50] (e.g., bridge decks, typically >10 metric tons) to a recent, very large structure (Governor Mario M. Cuomo bridge, 59,000 tons of HGDR [51, 52]). The unit weight cost of HDGR depends on market variables and volume of a project, but it is estimated to be up to 50% greater than that of PSR [53]. Added cost from any special handling or design requirement in placing the material is considered minimal, if any. A possible exception is for cases where extensive prefabrication should be conducted prior to galvanizing is needed [54], with consequent custom handling requirements.

4 B: What would be, and to which extent, the potential benefits or negative consequences factors in specifying of using HDGR instead of PSR for FDOT marine substructure service?

4.1 B1: Would HDGR delay the onset of corrosion, and slow the propagation of active corrosion compared to PSR?

The review in the previous sections shows sufficient evidence to expect that CT for properly prepared HDGR in sound concrete will exceed the CT value for PSR, by a factor in the order of 1.5 to perhaps as much as 4. As an example of the impact of that increase on the initiation time duration ti, assume a medium quality concrete without Pozzolanic admixtures, and a cement factor of 444 kg/m^3 exposed to severe substructure conditions with a 23.6 kg/m^3 surface chloride content and a rebar cover of 10 cm on a flat slab configuration. Based on previous FDOT research [55] the estimated chloride ion diffusion coefficient for such concrete is ~1.27 E-8 cm2/s. If CT for PSR were ~2 kg/m3 (a typical value equal to 0.45% of the cement content), the projected t_i duration per Eq.1 would be 42 years. For a CT multiplier for HDGR somewhere in the above range, for example 2, the resulting t_i value would increase to 66 years, or about 1.5 times greater than for PSR. This result is typical of the roughly square root power law benefit increase in t_i for a linear increase in CT expected for these type of conditions [56, 57]. Thus, although HDGR would still be subject to corrosion initiation, an initiation stage increase roughly in the order of 25% to as much as 100% (for threshold multipliers of ~1.5 to ~4 respectively) might take place under conditions of interest to FDOT for the sound concrete part of the substructure. A caveat exists in that projections for long periods (e.g., 50-100 years) before chloride ions arrive to the rebar surface are contingent on the galvanized coating (especially the outer eta layer) have not been consumed by them by normal quasi-passive metal dissolution. Work by Moreno [15, 58] examined the rate of eta-layer consumption for chloride free blended cement concretes use for FDOT construction and found mostly very low corrosion rates (<0.3 um year). However, at that rate much of the eta layer (some 15 um) would be consumed after about 50 years, suggesting that if the high value of CT for HDGR depends on the presence of the eta layer, credit for the benefit estimated above might not accrue.

The preceding comments applied to sound concrete without preexisting cracks or similar deficiencies. As noted in the introduction, preexisting cracks could seriously reduce the duration of the initiation stage compared to that in sound concrete. There are fewer investigations of HDGR performance on pre-cracked concrete [46, 59, 60] compared to those in sound concrete, but the indications generally confirm the expectation of greatly decreased values of t_i at the cracks vs sound locations. Further aggravation of conditions may exist in those cases because arrival of chloride ions could under certain conditions take place while the protective deposit has not yet fully formed. This concern is especially important for FDOT substructure applications because of the recognized presence of a certain incidence of preexisting cracking in footers and other components even under careful construction conditions[3, 55].

Regarding the effective extent of corrosion during the propagation stage in sound concrete, the material presented in Section 3.2.3 illustrates the present uncertainty on that issue. The aforementioned work by O'Reilly [48] showed that while HDGR required about twice the corrosion loss of PSR to develop cracking, it took about 4 times longer exposure; however the data presented was not sufficient to clearly separate t_i from t_p. In the absence of precise new information clearly demonstrating improvement over that of PSR, the t_p duration for HDGR in similar exposure service in sound concrete appears to be best conservatively estimated to be like that of PSR. Considerations of possible added detrimental issues is given in the next section.

Corrosion progression of HDGR under pre-cracked conditions and aggressive salt exposure has been evaluated in some detail in recent investigations. Darwin [41] observed HDGR corrosion losses in cracked beams that were some 3 times less than for PSR. Ogunsanya [61] reported HDGR bars at crack locations having a 12-14 times lower corrosion current density than PSR. Berke [54] reported lower corrosion (upon autopsy, limited observations) for HDGR than for PSR in ponded cracked concrete specimens. Presuel-Moreno[46, 62] reported (limited analysis) improved performance of HDGR over PSR in cracked ponded concrete over a period of 9 years. The results are generally favorable for HDGR, although confirmation under realistic marine substructure conditions is still to be obtained.

4.2   Is there enough documentation of verified HDGR performance in actual field conditions to justify recommending its use instead of PSR?

Yeomans [12, 16] and others [7] have presented evidence of various investigations of field performance in marine environments relevant to FDOT applications. However, much of that evidence concerns either very short age evaluations where chloride presence at the rebar surface is still minor, or examinations of corrosion condition in a relatively small number of samples where conditions were severe[63]. Bridge deck examinations in deicing salt service have shown instances of a degree of improved performance of HDGR over PSR [64, 65]. Additional evidence with some mixed performance is noted in the next section. FDOT has been conducting a long term (since 1993) test exposure of side-by-side PSR and HDGR with low cover (2 inch) in partially submerged piles constructed with Class V concrete at the Matanzas inlet site. That exposure, although disturbed by long-term burying of much of the piles in shifting sand, can generate corrosion performance data directly relevant to the issue of interest and with materials following modern FDOT specifications. However, while some activation of the HDGR and PSR bars may have taken place already, as of last examination (May 2022) there are no visual indications in any of the piles of corrosion induced cracking or discoloration. The information at present is valuable in that it indicates that no severe HDGR corrosion damage seems to have taken place after nearly 30 years of exposure with relatively low cover, but longer time exposure will be needed to make a determining assessment.

4.3 Could HDGR substitution for PSR result in risk of markedly reduced corrosion performance or adverse mechanical or structural consequences?

This question is probably the most important in the content of the project statement because any substantial risk of reduction of structural service life from use of an alternative to PSR would be unacceptable. Apart from severe structural damage from uncontrolled events (e.g., collision, fire, tidal surges, scouring of foundations), damage from reinforcement corrosion is the chief service life limiting process anticipated in present FDOT marine substructure design. HDGR is nominally expected, if anything, to improve reinforcement corrosion performance relative to PSR, [16]. That improvement is often cited, when promoting the use of HDGR to justify the added cost of galvanizing [53]. Thus, it is essential to carefully evaluate the risk that special circumstances may arise where the expected advantage is diminished or –more critically- even reversed.

Concern on this issue relates mainly to two potential cases discussed in the literature: faster or more damaging corrosion of the galvanized coating upon chloride exposure, and reduction of mechanical bond strength between the HDGR and the immediately surrounding concrete due to interfacial gas production during early curing. The next subsections focus respectively on each case, recognizing that they may act concurrently either in a mitigating or aggravating manner.

This question merits careful consideration because instances have been documented where corrosion performance of HDGR was deemed to be only marginally better, if at all, than that of PSR [45, 66]. A salient example, highly relevant to this review, addresses long term field experience in severe deicing salt service [67]. That work evaluated the condition of three galvanized rebar bridge decks in Ontario, Canada, after about 30 years of service. It was concluded that HDGR bars initiated corrosion soon after the CT value estimated for PSR was reached, that corrosion of HDGR caused significant damage to the concrete, and that HDGR bars were not recommended as the primary or sole means of corrosion protection for structures in that application.

In the context of an initiation-propagation stages corrosion durability model, this question can be stated as to whether t_i and/or t_p could be shortened enough for their sum ts to be significantly lower for HDGR than for PSR. Significant reduction in t_i would result if somehow CT for HDGR were to be markedly lower than that for PSR. Strong reduction in t_p could take place if the rate of corrosion of HDGR were sufficiently higher, and/or the corrosion products more expansive (or less rapidly accommodated by the surrounding concrete), than those for PSR.

As noted earlier the CT value of properly conditioned HDGR is generally accepted to be higher (e.g., 1.5 to 4 times) than that of PSR for the same exposure regime. It is thought that the CT benefit can be reduced if pre-placement conditioning in the form of chromate treatment is absent. However, that decrease in CT does not appear to have been significant enough to reach beneath the level of PSR, and several recent investigations have questioned the need for chromate treatment, as indicated in subsequent sections[19]. It is noted that even when keeping material and exposure conditions nominally the same, CT values for both PSR and HDGR show large variability (e.g., coefficient of variation values in the order of 0.5 or more] [6, 68, 69]. Thus in a side by side comparison of similar populations of HDGR and PSR samples, corrosion initiation of HDGR could take place earlier than for PSR in an appreciable number of instances [40]. Also as noted above, slow wastage of the eta layer may take place over long periods of time so the CT multiplier for HDRW might not be creditable over the long time. Duly noting those qualifications, examination of the

literature did not reveal clear evidence of cases where CT of HDGR has been found to be significantly lower than that of PSR. Consequently, the review looked in detail for possible issues of lower comparative performance in the corrosion propagation stage.

Possible decreases in the value of t_p stemming from HDGR corrosion products being more expansive than those of PS have received scant but, still concerning attention in the literature. Hime [70] reported the presence of highly expansive zinc hydroxychloride ($Zn_5[OH]_8Cl_2.H_2O$) on galvanized rebar that was embedded in mortar having a chloride-containing admixture. The mortar had been used as filling in cores of masonry bricks of a commercial building, that exhibited corrosion-induced cracking of mortar and bricks. Some of the affected bars showed corrosion products of the underlying steel, but others showed thick, white corrosion products with the above composition. Based on crystallographic considerations, the authors noted that the corrosion product/zinc volume ratio would be >3.6%, much greater than the ~1.5 ratio for the "usual Zn corrosion product, ZnO".

The possibility of decreased mechanical bond of HDGR vs PSR has been indicated above, including the diversity of opinions on that issue. This matter awaits full resolution but in the meanwhile an increasing volume of HDGR is placed without chromating treatment [51]. New data are becoming available that may further determine the extent, if any of this effect [71].

5 C: What is the recommended path forward toward deciding whether to specify HDGR?

5.1 C1: Is there sufficient data to make a determination, and if so which is it?

The technical evidence examined in this Synthesis is deemed to be sufficient for only for further consideration of HDGR as an allowable steel bar material among those listed in the FDOT Standard Specifications for Roads and Bridges, Section 931 and further defined under the Structure Design Guidelines. While some potential benefits of specifying HDGR are attractive, immediate deployment of HDGR especially in Florida's highly aggressive marine substructure service is not recommended. Instead, a phased approach is advised, including targeted testing and incorporation as an experimental feature of selected components in coming FDOT bridge projects. It is noted that the following proposes how to obtain needed evidence but does not address which should be the source of support for such effort.

5.2 C2: If Item 1 is in the negative, what is the recommended test plan and criterion to make a determination?

The following key issues are proposed for a phased approach evaluation that will lead to an eventual determination of acceptability of HDRG, if resolution in each and all cases were favorable.

5.2.1 Necessary Materials Properties

5.2.1.1 Eta layer thickness

As indicated by the review results, presence of a suitably thick external eta layer is deemed to be essential to not negatively affect corrosion performance. Thus in addition to satisfying the minimum material property requirements under ASTM 767 [13], suppliers need to be able to demonstrate with high confidence their ability to supply a product with a designated minimum eta layer thickness. The minimum is recommended to exceed the 10 µm value suggested by Ref. [14] by an extra pre-installation wastage allowance amount determined as indicated next.

Resolution: It may be established by identifying FDOT HDGR suppliers willing to satisfy the requirement, and establishing quality control methods such as statistical metallographic analysis to verify that the desired objective can be obtained in actual practice.

5.2.1.2 Controllable preplacement corrosion

Excessive wastage of the outer portions of the galvanized layer may occur due to post-manufacturing atmospheric corrosion of the rebar in aggressive Florida exposure conditions [72-74]. The wastage may occur during factory storage and later transportation, but principally while awaiting placement in the construction yard. That is particularly important for marine structures, because for cast in place elements the rebar may be previously sitting for extended periods (weeks or longer) on a shore-side construction yard or even on a bridge-side barge on seawater. Based on previous Florida considerations [56], either

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regime can be a source of extensive chloride contaminated moisture deposition on the rebar surface, with consequent corrosion and some loss of eta layer thickness before it is placed in concrete. Pre-placement corrosion needs to be limited.

Resolution can be made by, first, measurement of corrosion loss of galvanized thickness in anticipated FDOT coastal construction sites, construction schedule times and workable protection procedures such as tarping. Outcome is corrosion rate values leading to determination of whether there is a useful envelope of tolerable storage parameters (e.g., duration of exposure, distance from coastline if inland, protection by tarping), and a practical field method of coating condition assessment, that can be implemented in future construction to keep the wastage below an acceptable level

5.2.1.3 Concrete pore water pH

The review showed [14] that successful corrosion performance of HDGR depends on forming a compact CHZ protective deposit formed by reaction of the eta layer with the concrete alkaline pore water during the early curing stage. However, if the initial pore water pH is too high (e.g., >~ 12.8) the deposit is less protective plus much of the eta layer may be consumed in the process, to the detriment of chloride-induced corrosion resistance later in the service life of the structure. However, once the protective deposit has formed, some later pH increase up to ~13.3 may still be tolerated without adverse consequences. Also as noted earlier, It turns out that concrete formulations comparable to those specified for FDOT substructure requirements tend to follow a pore water pH evolution with time that approximately matches the desired behavior for protective deposit stability [17]. The specific behavior for a given concrete depends on several factors that include cement alkali and sulfate content, content and type of Pozzolanic admixtures as well as possibly other admixtures, w/c ratio and in some cases aggregates used. Due to variability in the dosage and origin of the various components as well as composition tolerances, there isn't at present sufficient information to provide assurance that the nominal mixture proportions presently specified by FDOT will reliably produce a pH time evolution adequate to HDGR performance.

Resolution: Testing by established methods [17] needs to be conducted with representative samples of concretes produced per current FDOT marine substructure and material specifications, to determine evolution of pore water pH (and of other possibly relevant species such as chromate content) with time, as well as the expected variability ranges. The results need to be compared with the tolerable conditions indicated above to determine suitability. If some or all mixture proportions are revealed to be inadequate, the feasibility of specifying customized proportioning for acceptable HDGR performance without undue added burden needs to be established.

5.2.1.4 Mechanical bond

Given the previous discussions, this issue is left open at this time pending the result of ongoing investigations (e.g., [71]) and possible availability of further evaluation of large HDGR structures now in place such as the Governor Mario M. Cuomo bridge [51, 52] (AKA Tapan-Zee bridge). Special consideration of whether work in this direction should take place is recommended to be postponed to after or during the closing stages of the previous phases, assuming that their resolution has been favorable.

5.2.2   Field Performance Requirements

5.2.2.1   Meet or exceed PSR corrosion performance on accelerated test:

The review showed that extensive laboratory testing has already been conducted to answer the most pressing questions on HDGR corrosion mechanisms and anticipated performance. Field evaluations have been relatively more limited, as many have not been of enough duration to have clearly exceeded the corrosion threshold and establish behavior under more aggressive conditions that may approach Florida substructure exposure. The main challenge for this application is surface chloride levels that approach saturation due to evaporative concentration, combined with the presence of high moisture and temperature. Conditions are particularly aggravated by the inevitable incidence of preexisting cracks in the otherwise highly impermeable concrete normally used in FDOT marine structure [3, 55]. Indeed, because concrete quality and cover thickness are designed to greatly delay the onset of corrosion on PSR in sound concrete, corrosion at preexisting cracks is likely to be the most important challenge to corrosion performance of HDGR rebar, assuming it has otherwise satisfied the early age issues addressed in the previous section [59].

Thus, there is need to determine whether HDGR can sufficiently withstand the most aggressive anticipated marine substructure conditions, attaining performance that meets or exceeds that of the currently used PSR. Ideally, that could be elucidated from side-by side long duration test panel exposures with concrete cover, quality and cracking incidence replicating those of actual construction conditions. However, such determination could take a very long time (e.g. many decades) with risk of discarding benefits from a specification transition from PSR to HDGR, should the latter demonstrate clearly superior relative performance. Thus, some judicious acceleration of test conditions needs to be exercised to reveal shortcomings or benefits in a shorter period.

Resolution: A suitable compromise between test duration and usefulness of outcome may be obtained by reduction in concrete cover and increased incidence of cracking, while keeping the concrete composition like that used in actual practice. An example among many possible test configurations could be a 4ft x 4ft reinforced concrete slab, with rebar placed at selected portions with e.g., ½", 1" and 2" cover, and with a pattern of longitudinal and transversal cracks created initially by a temporary external bending excessive load application. Slabs should be replicates in configuration but prepared in pairs, one with only PSR and the other with only HDGR. After a sufficient duration of curing (e.g., 1 year) to allow for a sizable amount of Pozzolanic admixtures reaction, the slab pairs would be placed vertically and side by side as removable attachments to the submerged, tidal and splash zones of selected FDOT marine bridge piers. Alternatively, or in addition, freshly cast slabs could be installed to simulate cast in place construction practice whereby molds may be removed while curing has been still short, a condition that may be particularly averse to HDGR when the protective deposit is still maturing. Multiple slab replications are recommended.

Nondestructive measurements of corrosion rate and cracking or spalling evolution, as well as records of visual or other evidence of corrosion, would be collected periodically over a period in the order of one decade. Autopsies would then be performed for a subset of the slabs to assess the corrosion performance to date, leaving the rest for follow up monitoring. While it is anticipated that corrosion initiation in sound concrete locations might take place over that period for the lowest values of cover sampled, some

corrosion development is highly likely to be observed in the PSR slab at all the crack locations and for all cover values [3, 59]. The extent of corrosion in the paired HDGR slabs should be carefully compared with the PSR results.

Favorable determination on the use of HDGR for this phase is contingent on establishing whether the statistically assessed corrosion damage for HGDR is distinctly less severe than in the companion PSR slabs in each of the exposure elevations tested. If test acceleration is such that favorable determination can be reasonably anticipated from exposure shorter than one decade, that determination will be provisionally accepted for the purposes of proceeding to experimental feature deployment addressed next. Conversely, uncertain outcome after one decade indicates postponement of a determination decision until future examination of the remaining subset(s) of field slabs.

5.2.2.2   Demonstration as experimental feature in a full bridge pier.

Should all the evaluation phases described above result in clearly positive actual or provisional determinations, it is recommended for consideration that HDGR be deployed first as an experimental feature in a single pier or similar component of a new bridge, where at least one twin portion with comparable environment exposure has traditional PSR reinforcement. The experimental HDGR feature and the peer PSR control should both be fitted with similar sensors sufficient for periodically nondestructively assessing corrosion performance as well as any other relevant structural indicators as the structure ages, and to flag by comparative evaluation any differences, adverse or positive, between behaviors. The experimental feature will also serve to determine whether HGDR construction can be performed properly with FDOT methods and materials provided from approved suppliers with appropriate quality control. Concurrent with this phase as well as with the accelerated tests, it is recommended that a collaborative liaison be secured with the New York State Thruway Authority (NYSTA) to jointly benefit from HGDR performance information that may be obtained in the substructure of the recently constructed (2017) Governor Mario M. Cuomo Bridge, crossing the chloride-rich estuarial region of the Hudson River [52, 75, 76].

Resolution: Favorable determination from this phase is contingent on evidence that the corrosion, mechanical, and other relevant behavior the HDGR component is not inferior, and preferably improved, than that of the PSR control. It is recognized that the experimental feature component will be likely to age very slowly, but positive results from some period of confirmation of e.g. one or two decades duration will serve as an important source of added confidence on an eventual decision to fully allowing specification of HDGR for FDOT marine substructure applications.

6 Conclusions

- Updated examination of the documented performance of HDGR over about one century of utilization and testing largely confirms earlier indications of some promising performance during the corrosion initiation stage. There is still also indication of mixed performance, or insufficient documentation, to consistently demonstrate and improvement over PSR in the corrosion propagation stage.

- Performance uncertainty is a concern especially for t the severe marine exposure conditions prevalent in FDOT marine substructure. Thus, immediate deployment of HDGR for that service is not recommended.

- Because of the possibility of improved performance and consequent benefit to FDOT, further consideration of using HDGR is recommended, but only contingent upon clearly satisfactory outcome of a phased evaluation program.

- Proposed evaluation consists first of assessing whether necessary materials properties are met, including manufacture's ability to produce bar with a minimum eta layer coating thickness, to maintain proper control of preplacement corrosion as in the construction yard, and to assure that concrete mixture materials and proportioning is possible to keep pore water pH and related properties within a restricted value envelope.

- Should outcome of all the above phases be fully adequate, field performance tests by means of side-by-side PSR and HDGR test slabs with preexisting cracks in actual bridge locations need to satisfy specified resolution criteria over a decade-long exposure. If that outcome is in turn adequate, it is recommended for consideration that tentative HDGR deployment be conducted in a limited form, as an experimental feature in the form of one bridge pier incorporated as part of a selected new bridge construction.

References

1.   Tuutti, K., *Corrosion of steel in concrete*. 1982: Cement-och betonginst.

2.   Yeomans, S., *Galvanized Steel Reinforcement in Concrete; An Overview.* 2004.

3.   Lau, K., A. Sagüés, and R. Powers, *Corrosion of epoxy-coated rebar in marine bridges—Part 2: Corrosion in cracked concrete.* Corrosion, 2010. **66**(6): p. 065002-065002-16.

4.   Torres-Acosta, A.A. and A.A. Sagues, *Concrete cracking by localized steel corrosion--Geometric effects.* Materials Journal, 2004. **101**(6): p. 501-507.

5.   Williamson, G.S., et al., *Concrete and steel type influence on probabilistic corrosion service life.* ACI Materials Journal, 2009. **106**(1): p. 82-88.

6.   O'Reilly, M., et al., *Corrosion-Induced Concrete Cracking for Uncoated and Galvanized Reinforcing Bars.* ACI Materials Journal, 2018. **115**(6).

7.   Kinstler, T.J. *RESEARCH AND UPDATE ON GALVANISED REINFORCING STEEL*. in *PROCEEDINGS OF THE INTERNATIONAL CONFERENCE ON CORROSION AND REHABILITATION OF REINFORCED CONCRETE STRUCTURES HELD 7-11 DECEMB 1998, ORLANDO, FLORIDA, USA*. 1999.

8.   NACE International Task Group (TG) 057, *State of the Art Report on Corrosion-Resistant Reinforcement, NACE Publication 21429-2018*. 2018, NACE International: Houston.

9.   fib Task Group 9.7, *Corrosion protecion of reinforcing steels - fib Bulletin 49*. 2009: Lausanne, Switzerland.

10.   Pokorný, P., P. Tej, and M. Kouřil, *Evaluation of the impact of corrosion of hot-dip galvanized reinforcement on bond strength with concrete – A review.* Construction and Building Materials, 2017. **132**: p. 271-289.

11.   Cornet, I. and B. Bresler, *Galvanized Steel in Concrete: Literature Review and Assessment of Performance*, in *Galvanized Reinforcement for Concrete - II*. 1981, International Lead Zinc Research Organization, Inc.: New York. p. 1-56.

12.   Yeomans, S.R., *Laboratory and field performance of galvanized steel in concrete*, in *Galvanized Steel Reinforcement in Concrete*. 2004, Elsevier. p. 145-197.

13.   ASTM Intrnational, *ASTM A 767/A 767M, Standard Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement*.

14.   Andrade, C. and C. Alonso, *Electrochemical aspects of galvanized reinforcement corrosion*, in *Galvanized steel reinforcement in concrete*, S. Yeomans, Editor. 2004, Elsevier. p. 111-144.

15.   Moreno, E.I. and A.A. Sagüés, *PERFORMANCE OF ALTERNATIVE MATERIAL REBAR*. 1993, University of South Florida: Tampa. p. 43.

16.   Yeomans, S., *Galvanized steel reinforcement: Recent developments and future opportunities*, in *Corrosion of Steel in Concrete Structures*, A. Poursee, Editor. 2021, Woodhead Publishing.

17.   Vollpracht, A., et al., *The pore solution of blended cements: a review.* Materials and Structures, 2015. **49**(8): p. 3341-3367.

18.   Galan, I., et al., *Continuous optical in-situ pH monitoring during early hydration of cementitious materials.* Cement and Concrete Research, 2021. **150**.

19.   Tan, Z.Q. and C.M. Hansson, *Effect of surface condition on the initial corrosion of galvanized reinforcing steel embedded in concrete.* Corrosion Science, 2008. **50**(9): p. 2512-2522.

20.   Maeda, M., et al., *Passivation Mechanism of Galvanized Steel Rebar in Fresh Concrete.* ISIJ International, 2020. **60**(2): p. 337-345.

21.   Tittarelli, F. and T. Bellezze, *Investigation of the major reduction reaction occurring during the passivation of galvanized steel rebars.* Corrosion Science, 2010. **52**(3): p. 978-983.

22.   Tittarelli, F., M. Carsana, and T. Bellezze, *Corrosion behavior of reinforced no-fines concrete.* Corrosion Science, 2013. **70**: p. 119-126.

23.   Tittarelli, F., et al., *Corrosion behaviour of bare and galvanized steel in geopolymer and Ordinary Portland Cement based mortars with the same strength class exposed to chlorides.* Corrosion science, 2018. **134**: p. 64-77.

24.   Tittarelli, F. and G. Moriconi, *The effect of silane-based hydrophobic admixture on corrosion of galvanized reinforcing steel in concrete.* Corrosion Science, 2010. **52**(9): p. 2958-2963.

25.   Wu, M. and J. Shi, *Beneficial and detrimental impacts of molybdate on corrosion resistance of steels in alkaline concrete pore solution with high chloride contamination.* Corrosion Science, 2021. **183**: p. 109326.

26.   Yadav, A., A. Nishikata, and T. Tsuru, *Evaluation of impedance spectra of zinc and galvanised steel corroding under atmospheric environments.* Corrosion engineering, science and technology, 2008. **43**(1): p. 23-29.

27.   Alonso, C.M., I; Fullea, J; Ibarra,V, *In-Situ Monitoring of Galvanized Reinforced Concrete Structured-The Spanish Experience.* 2009.

28.   Ebell, G., et al., *Electrochemical investigations on the corrosion behaviour of galvanized reinforcing steels in concrete with chromate-reduced cements.* Materials and corrosion, 2012. **63**(9): p. 791-802.

29.   Feng, X., Y. Tang, and Y. Zuo, *Influence of stress on passive behaviour of steel bars in concrete pore solution.* Corrosion Science, 2011. **53**(4): p. 1304-1311.

30.   Figueira, R.M., et al., *Corrosion protection of hot dip galvanized steel in mortar.* 2013.

31.   Sergi, G., N. Short, and C. Page, *Corrosion of Galvanized and Galvannealed Steel in Solutions of pH 9.0 to 14.0.* Corrosion, 1985. **41**(11): p. 618-624.

32.   Macías, A. and C. Andrade, *Galvanized steel behaviour in Ca (OH) 2 saturated solutions containing $SO_{4=}$ ions.* Cement and Concrete Research, 1987. **17**(2): p. 307-316.

33.   Pernicova, R., D. Dobias, and P. Pokorny, *Problems connected with use of hot-dip galvanized reinforcement in concrete elements.* Procedia Engineering, 2017. **172**: p. 859-866.

34.   González, J., et al., *Effect of four coating structures on corrosion kinetic of galvanized reinforcement in concrete.* Materials and Structures, 1984. **17**(6): p. 409-414.

35.   Tittarelli, F., et al., *Effect of the Type of Surface Treatment and Cement on the Chloride Induced Corrosion of Galvanized Reinforcements.* IOP Conference Series: Materials Science and Engineering, 2017. **245**.

36.   Kayali, O., *Bond of Steel in Concrete and the Effect of Galvanizing*, in *Galvanized steel reinforcement in concrete*, S. Yeomans, Editor. 2004, Elsevier. p. 229-270.

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

38.   Hamad, B.S. and J. Ghaida'K, *Bond strength of hot-dip galvanized hooked bars in high strength concrete structures.* Construction and Building Materials, 2008. **22**(10): p. 2042-2052.

39.   Hamad, B.S. and J.A. Mike, *Experimental investigation of bond strength of hot-dip galvanized reinforcement in normal-and high-strength concrete.* Structural Journal, 2003. **100**(4): p. 465-470.

40.   Darwin, D., et al., *Critical Chloride Corrosion Threshold of Galvanized Reinforcing Bars.* ACI Materials Journal, 2009. **106**(2): p. 176-183.

41.   Darwin, D., et al., *EVALUATING THE PERFORMANCE OF EXISTING REINFORCEMENT FOR OKLAHOMA BRIDGES*. 2020, Oklahoma Department of Transportation: Oklahoma City. p. 194.

42.   Maldonado, L., *Chloride threshold for corrosion of galvanized reinforcement in concrete exposed in the Mexican Caribbean.* Materials and corrosion, 2009. **60**(7): p. 536-539.

43.   D.B. McDonald, D.W. Pfeifer, and M.R. Sherman, *CORROSION EVALUATION OF EPOXY-COATED, METALLIC-CLAD AND SOLID METALLIC REINFORCING BARS IN CONCRETE*. 1998: McLean, VA.

44.   McDonald, D.B., D.W. Pfeifer, and G.T. Blake, *THE CORROSION PERFORMANCE OF INORGANIC-, CERAMIC-, AND METALLIC-CLAD REINFORCING BARS AND SOLID METALLIC REINFORCING BARS IN ACCELERATED SCREENING TESTS*. 1996: McLean, VA,.

45.   Clear, K.C., *Time-to-Corrosion of Reinforcing Steel in Concrete Slabs, Vol. 4: Galvanized Reinforcing Steel*. 1981, United States. Federal Highway Administration.

46. Presuel-Moreno, F.J. and D. Rourke. *Review of galvanized rebar performance on G-109 specimens after 9 years*. in *CORROSION 2009*. 2009. OnePetro.

47. Ogunsanya, I.G. and C.M. Hansson, *The Influence of Coating Thickness and Composition on the Corrosion Propagation Rates of Galvanized Rebar in Cracked Concrete.* Corrosion, 2017. **74**(1): p. 134-143.

48. O'Reilly, M., et al., *Corrosion-Induced Concrete Cracking for Uncoated and Galvanized Reinforcing Bars.* Aci Materials Journal, 2018. **115**(6): p. 825-832.

49. Saire-Yanez, J., C.L. Alexander, and A.A. Sagüés, *The Corrosion Propagation Stage of Stainless Steel Reinforced Concrete: A Review.* Corrosion, 2021. **77**(8): p. 812-828.

50. Galvanized Rebar Alliance. *Two large and unique projects in different climates rely on hot-dipped galvanized reinforcement*. 2021 June 29, 2022]; Available from: https://www.roadsbridges.com/galvanized-rebar/article/10707866/two-large-and-unique-projects-in-different-climates-rely-on-hot-dipped-galvanized-reinforcement.

51. American Galvanizers Association. *Governor Mario M. Cuomo Bridge*. 2018 June 27, 2022]; Case History]. Available from: https://galvanizeit.org/uploads/project-gallery-content/Governor_Mario_Cuomo_Bridge_2018.pdf.

52. Barbas, J. and M. Paradis, *Scalable, Modularized Solutions in the Design and Construction of the Governor Mario M. Cuomo Bridge.* Structural Engineering International, 2019. **29**(1): p. 123-127.

53. Yeomans, S., *Galvanized Reinforcement In Concrete Structures Questions and Answers in Collaboration with the Galvanizers Association of Australia Edition 2.0.* 2018.

54. Berke, N.S., *Enhanced performance zinc coating for steel in concrete*. 2016.

55. Sagüés, A.A., et al., *Corrosion forecasting for 75-year durability design of reinforced concrete*. 2001, University of South Florida: Tampa. p. 174.

56. Sagüés, A., et al., *Corrosion of epoxy-coated rebar in marine bridges—Part 1: A 30-year perspective.* Corrosion, 2010. **66**(6): p. 065001-065001-13.

57. Sagüés, A.A., *Corrosion of Metals in Concrete*, in *Galvanized Steel Reinforcement in Concrete*, S. Yeomans, Editor. 2004, Elsevier. p. 71-86.

58. Moreno, E.I., A.A. Sagüés, and R. Powers. *Performance of plain and galvanized reinforcing steel during the initiation stage of corrosion in concrete with Pozzolanic additions*. in *CORROSION 96*. 1996. OnePetro.

59. Ogunsanya, I. and C. Hansson, *The influence of coating thickness and composition on the corrosion propagation rates of galvanized rebar in cracked concrete.* Corrosion, 2017. **74**(1): p. 134-143.

60. Moreno, P.R., D, *Towards Achieving the 100 Year Old Bridge Using Galvanized Rebar.* 2009.

61.   Ogunsanya, I. and C. Hansson, *The Influence of Coating Thickness and Composition on the Corrosion Propagation Rates of Galvanized Rebar in Cracked Concrete.* Corrosion, 2018. **74**(1): p. 134-143.

62.   Presuel Moreno, F.J., *Achieving the 100 Year Bridge Using Galvanized Rebar,Performance of Galvanized Rebar up to 14 Years: G-109 Reinforced Concrete Specimens*. 2013, Florida Atlantic University: Florida.

63.   Allan, N.D., *The Bermuda experience: leading the way on galvanized reinforcement*, in *Galvanized Steel Reinforcement in Concrete*, S. Yeomans, Editor. 2004, Elsevier. p. 199-291.

64.   Nagi, M. and S. Alhassan. *Long Term Performance of Galvanized Reinforcing Steel in Concrete Bridges-Case Studies*. in *CORROSION 2005*. 2005. OnePetro.

65.   Wiss, J., *Iowa state highway 92 over drainage ditch# 25: performance evaluation-galvanized reinforcing bars, Louisa County, Iowa*. 2015, Iowa. Dept. of Transportation. Office of Bridges and Structures.

66.   Arup, H., *Galvanized Steel in Concrete.* Materials Performance, 1979. **19**(4): p. 41-44.

67.   Pianca, F. and H. Schell, *The long term performance of three ontario bridges constructed with galvanized reinforcement.* Ontario Ministry of Transportation, Downsview, Ontario, 2005.

68.   Angst, U.M., et al., *Beyond the chloride threshold concept for predicting corrosion of steel in concrete.* Applied Physics Reviews, 2022. **9**(1): p. 011321.

69.   Darwin, D., et al., *Critical chloride corrosion threshold for galvanized reinforcing bars*. 2007, University of Kansas Center for Research, Inc.

70.   Hime, W. and M. Machin, *Performance variances of galvanized steel in mortar and concrete.* Corrosion, 1993. **49**(10): p. 858-860.

71.   Patnaik, A., *Reduction of Bridge Deck Cracking through Alternative Material Usage*. 2017, The University of Akron: Akron. p. 344.

72.   Ormellese, M., et al. *Atmospheric Corrosion Behavior of Zinc and Zinc Alloys: Comparison Between Natural and Accelerated Exposure*. in *CORROSION 2021*. 2021. OnePetro.

73.   de Rincon, O., et al. *Behavior of Galvanized Steel in Tropical Marine Environments*. in *CORROSION 2005*. 2005. NACE International

74.   Romero, N., et al., *Behavior of galvanized steel in tropical marine environments.* Materials performance, 2006. **45**(2): p. 24.

75.   American Galvanizers Association. *Governor Mario M. Cuomo Bridge*. 2018 June 21, 2022]; Available from: https://galvanizeit.org/uploads/project-gallery-content/Governor_Mario_Cuomo_Bridge_2018.pdf.

76.   *TAPPAN ZEE HUDSON RIVER CROSSING PROJECT Final Environmental Impact Statement and Section 4(f) Evaluation*, Federal Highway Administration, Editor. July 2012.