VISIT REPORT

UNIVERSITY OF WATERLOO

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REPORT BY FRANK GOODWIN

Represented by Professor C. Hansson and I. Ogunsanya, Graduate Student

Accompanied by M. Gagne

Subject: MD-71 Rebar Market Development

The purpose of this meeting was to review the autopsied concrete slabs that contained several varieties of black or galvanized rebar after 450 days of exposure. The graduate student expects to complete his thesis by the end of the year, which will be based on these data. Overall we are very pleased with the data that showed its great advantage compared with black steel and performance not too dissimilar from hot dip galvanized rebar and MMFX semi-stainless alloy. The Sample C1 is from Dubai, the Sample C2 is from Xiamen and the Sample C3 is a laboratory-dipped "Galfan" coating of rebar produced in the laboratory by Daniel Liu at Teck. The average thicknesses for the samples were: air wiped, Southeast Galvanizing sample, 150 microns. Sample C1 (Dubai) 41 microns; Sample C2 (Xiamen Newsteel), 33 microns; Sample C3, (Galfan from Teck) 26 microns. The standard deviations for the Southeast Galvanizing, Dubai and Nusteel samples were about 30% of the average coating thickness, whereas in the case of the Galfan coating the standard deviation was half of the coating thickness. Thus, there is great variability, which is not unexpected given the ribbed surface of the rebar. Measurements were made by cross-section microscopy. X-ray fluorescence was used to determine the coating composition. There is a great deal of pickup of surrounding phases, or the steel substrate in the X-ray results that were shown. For example, the Galfan coating showed 16.1% iron and it is known that Galfan is almost insoluble in iron and so this must have come from the steel. Accuracy improved with coating thickness. For the corrosion solution, 21% Cl brine, corresponding to Ontario MOT brine used in practice, was used rather than 3.5pct used by other researchers.

Electrochemical test results were shown. For the corrosion potentials, ASTM C876 was referenced that characterizes black steel corrosion in concrete. These were taken bi-weekly over the 450-day period for three types of concrete samples: sound (non-cracked), transverse cracked and longitudinal cracked samples. In the second, the crack is perpendicular to the rebar, whereas in the other the crack is parallel to the rebar. This simulates concrete deck flexing that can cause cracks in the concrete deck. In this case, a normal concrete deck would have cracks transverse to the rebar running transverse in the bridge mat, whereas the cracks would be perpendicular to the rebar running longitudinally along the direction of the bridge. The electrochemical readings were interpreted using the NRC report received two years ago which gave the threshold values for corrosion of zinc in concrete. The sound beam results were the most straightforward and show a tight grouping of all of the samples that were zinc coated; however, for the corrosion current the Galfan bar performs very similarly to the black bar, having a corrosion current at least 10 times higher than those of the other zinc-coated bars. The transverse and longitudinal cracked results are more difficult to interpret but in all cases show a clear advantage of the zinc-coated bars over black bar. It was shown that the passive calcium hydroxyzincate (CHZ) passivation layer comes off in flakes when it is exposed to water and this may be a reason why there is depassivation seen in the cracked tests. The Galfan and black bars behave similarly in the transverse cracked configuration, with the Newsteel and Dubai bars behaving with an intermediate current to that of the air-wiped bar. For the longitudinally-cracked bars, the Galfan, Dubai and Newsteel samples all had corrosion currents intermediate to the air-wiped bar and the black bar. Results were then compared with the semi-stainless MMFX rebar for which Professor Hansson is also doing a study. The corrosion currents of the air-wiped bar are 10 times lower than that of MMFX after 450 days. The corrosion current of the hot dip galvanized bar in the cracked concrete samples is also an order of magnitude better than MMFX. This is a very strong result for the air-wiped (Southeast Galvanizing) bar. Even more impressive is the performance of the CGR (Dubai and Nusteel samples) in comparison with MMFX. In the sound concrete, these samples, despite their low coating weights, had a corrosion current order of magnitude less than MMFX. In the cracked concrete, the corrosion currents of the two CGR samples and MMFX was about the same. This is a very strong result and shows that the same corrosion rates can be obtained with even a thin-coated CGR bar as with the much more expensive MMFX bar. The corrosion resistance of the very thin Galfan-coated bar performed the same as the MMFX bar in the cracked concrete, having a corrosion rate of around 5 microns per year after 450 days. In the sound concrete, the air-wiped galvanizing corroded at a rate of 0.1 micron per year, whereas the MMFX corroded at 1 micron per year. The CGR from Dubai and Newsteel also corroded at the same rate as the hot dip galvanized bar in sound concrete. The longitudinal cracks specimen of the commercial CGR bars corroded about an order of magnitude less than that of the MMFX while the transverse cracked specimens behaved in a similar manner to MMFX. The reasons for this difference are currently being sorted out.

Photographs were then shown of the autopsied samples which had been cracked open to reveal the interface between the corroded rebar and the concrete. The bar was then flipped over to show its contact with the concrete in the photographs. These photographs are not very well organized in the Power Point that was obtained and need to be sorted out but show the very low corrosion rate of the galvanized rebar after 450 days. The corrosion rates using X-ray phorescence allowed for an estimation of zinc loss but again the technique was criticized and the student will need to work this out to obtain final weight-loss numbers for the autopsy bars. Despite this, it can be concluded that in sound concrete the corrosion rate of the galvanized rebar is around 0.15 microns per year, whereas in the transverse-cracked concrete it is around 1.5 microns per year. In the longitudinal-cracked concrete it is around 15 microns per year. Hot dip galvanizing was observed to provide an order of magnitude extra corrosion protection than black steel and in many cases be much better than, or at least equivalent to, the MMFX semi-stainless steel. All of these are subject to refinement and revision based upon the further work by the graduate student but are extremely encouraging for the long-term performance of galvanized rebar.

UW Data

Galvanizing vs MMFX

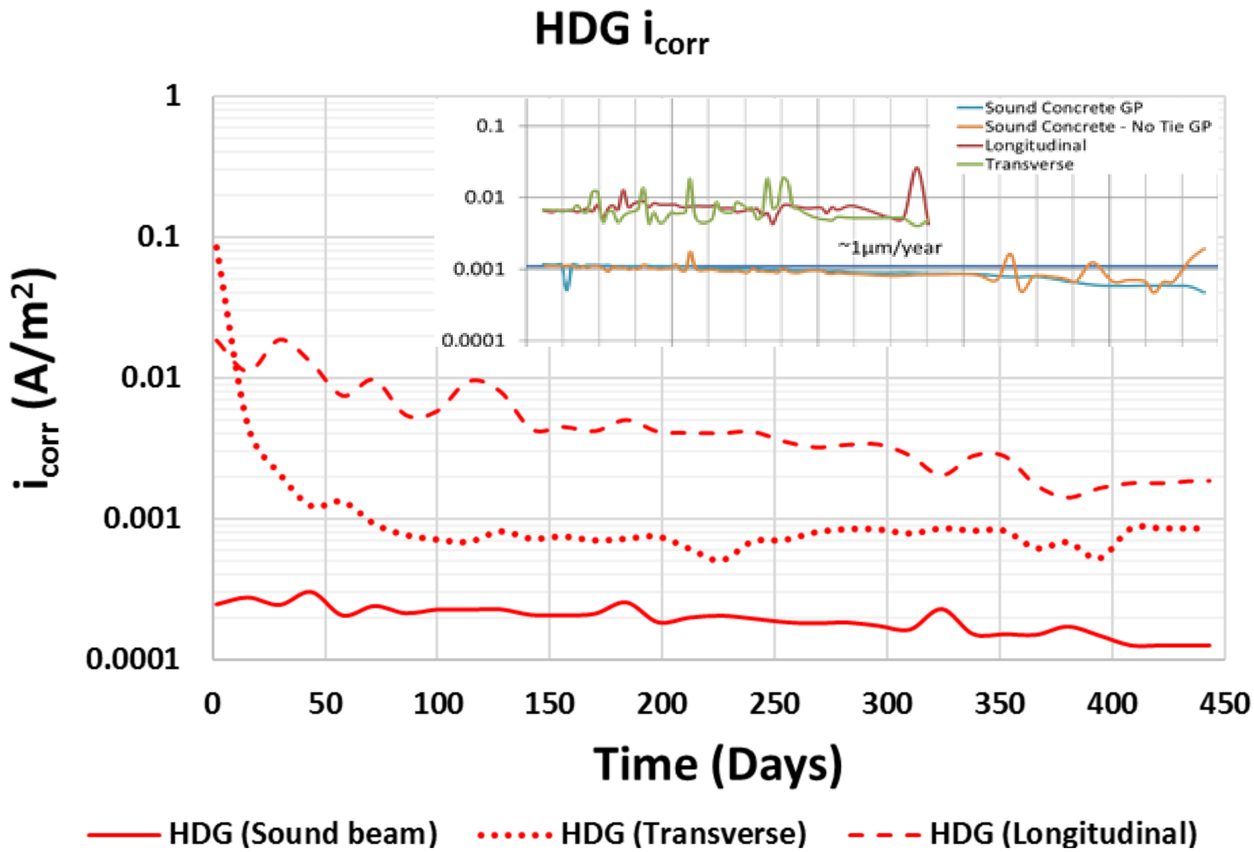
Comparison with MMFX

This is achieved by comparing:

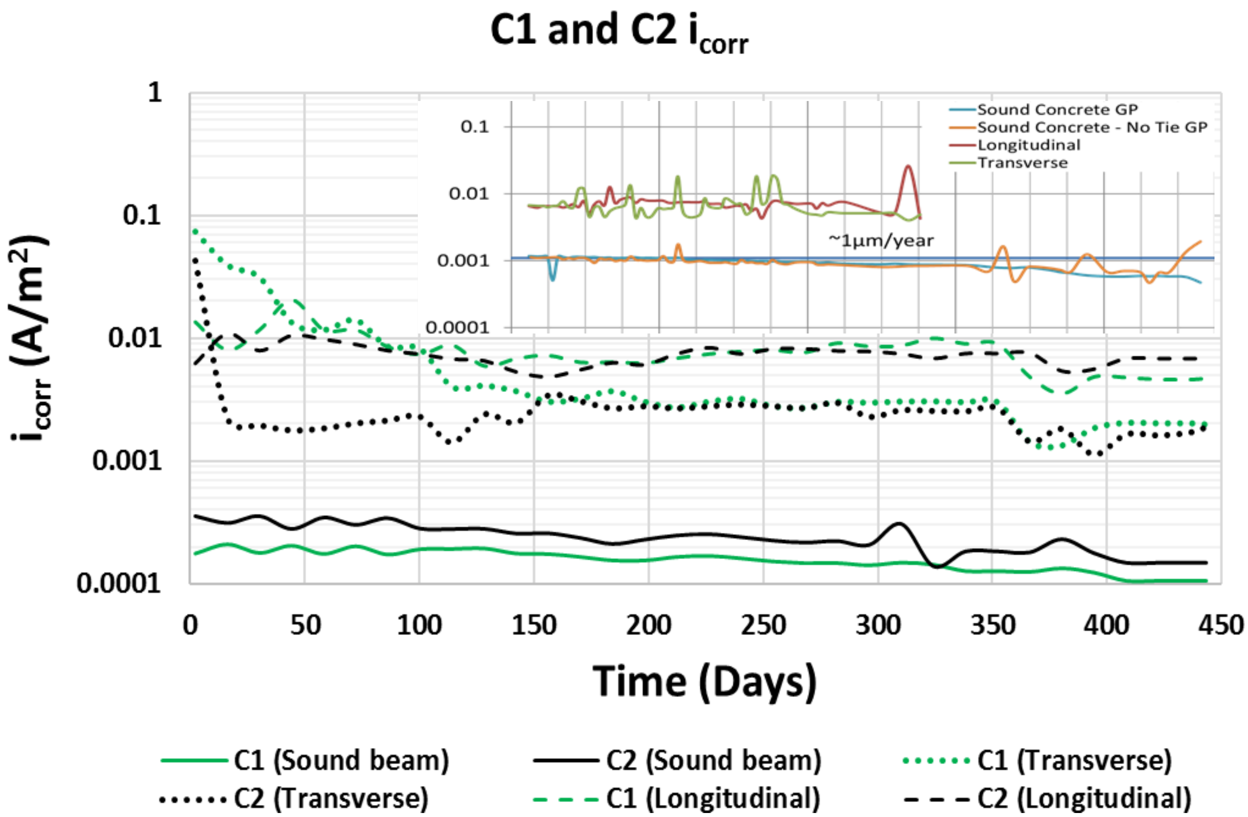
- HDG (the thickest Zn coating),
- C1 and C2 (similar coating thickness less than HDG),
- C3 (the thinnest coating similar to ‘galfan’),

with MMFX result from previous data

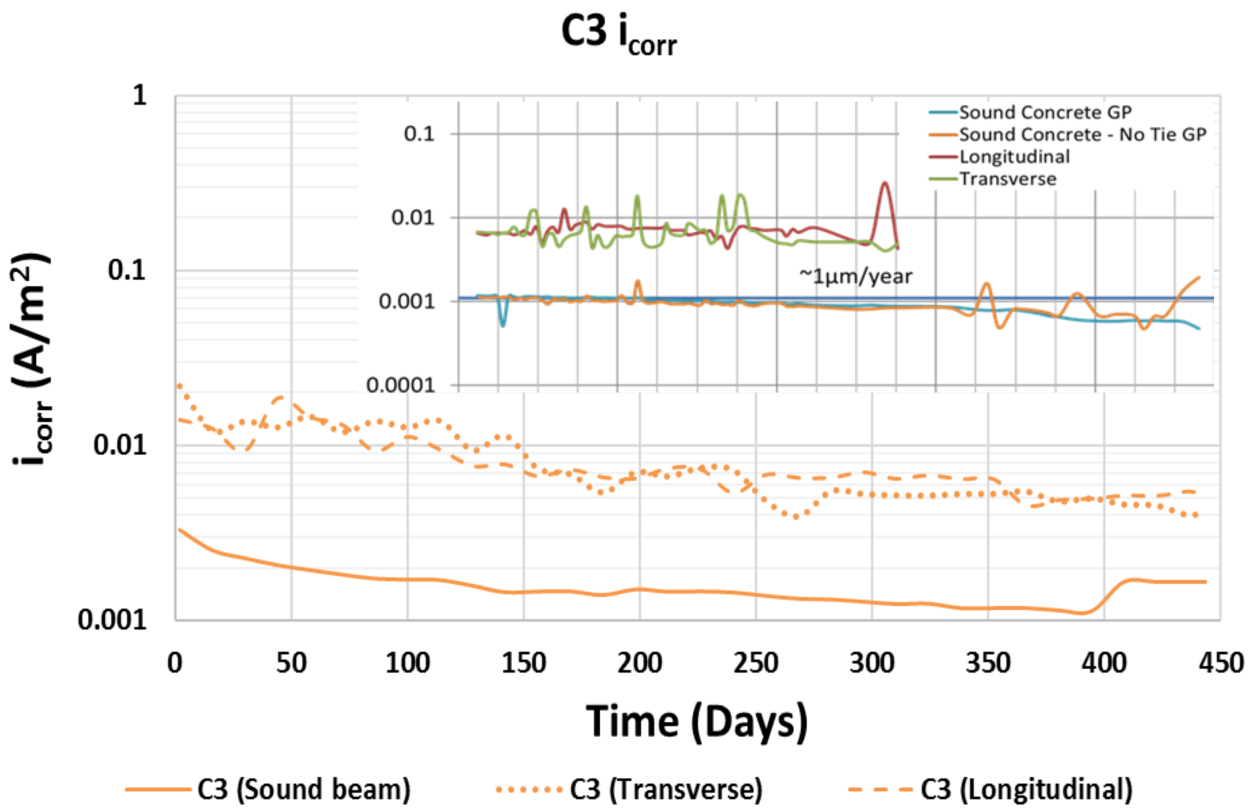
- MMFX and epoxy coated rebar (ECR) are other competitive economical ‘corrosion resistance’ rebar, and we thought it would be interesting to see how these galvanized bars compares with them.
- Although the corrosion potentials of these bars cannot be compared or interpreted with ASTM C876 or the NRC guideline for galvanized steel, comparison can be made on their corrosion current density.
- The next 3 slides compares galvanized steel with the thickest coating, HDG, those with lesser coatings, C1 and C2 (since they are both similar), and one with the thinnest coating, C3, respectively, with MMFX rebar from previous work.
- Since HDG bars cast with the MMFX from the authors work compares very well with those in the present work, a fair comparison could be made.



- It can be seen that the corrosion current density of the HDG rebar in the sound beam is about an order of magnitude lower than the MMFX at around 450 days of exposure [insert]. While the HDG is passively corroding at 0.1um/yr, the MMFX corrodes at 1um/yr.
- Similarly, after the huge initial drop, the ^{i}corr value of the longitudinally cracked HDG specimen continued to be an order of magnitude lower than the MMFX, while the transversely cracked specimen is slowly approaching an order of magnitude difference.
- This suggests that an order of magnitude more corrosion resistance can be obtained from selecting HDG rebar ahead of MMFX.
- Interestingly, the order of magnitude difference between HDG and MMFX was the same as that found between HDG and conventionally black rebar.



- Since the C1 and C2 specimen have similar coating thickness and their behaviour have shown to also be similar, both bars are compared with MMFX.
- The sound specimen of this CGR rebar behaved the same way as the HDG which was an order of magnitude better than the MMFX.
- Although the longitudinally cracked specimen of both bars also seem to be about an order of magnitude better than those of the MMFX, the transversely cracked specimens of both bars behaved in a similar manner as the MMFX [insert].



- The C3 contained the thinnest coating and performed the least of the galvanized rebar, which makes it interesting for comparison with the MMFX rebar.
- The C3 specimen, which performed the same as the black bar in the sound concrete, is an order of magnitude higher than other grades of galvanized bar in this concrete condition, but behaves the same as the MMFX rebar at 450 days.
- Similarly, the transversely- and longitudinally- cracked specimen, having the same corrosion current density, also behaved the same as the MMFX rebar with an icorr value reduced to ~5um/yr at about 450 days.
- This suggest that 30um coating of a galvanized rebar can give similar corrosion protection as MMFX.

VIRGINIA BRIDGE DECK SERVICE LIFE PERFORMANCE AND ASSOCIATED COSTS: INFLUENCE OF REINFORCING STEEL TYPE

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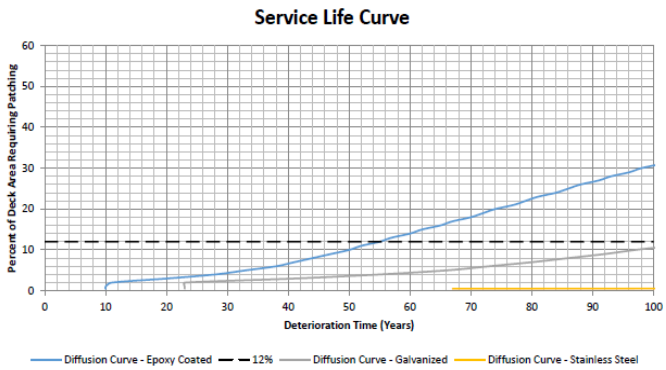
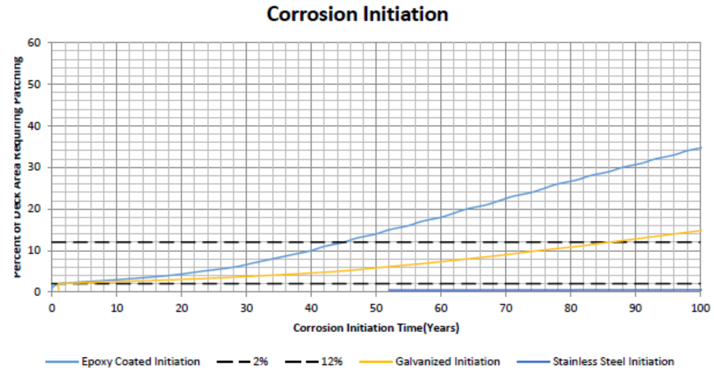
Executive Summary

This report was prepared for the American Galvanizers Association Rebar Focus Group by Prof. Richard Weyers of Virginia Tech, he is a well-recognized authority on reinforced concrete bridges and their maintenance. This report gives total present cost (TPC) and life-cycle cost (LCC) figures for the three zones of Cl exposure to which Virginia bridge decks are exposed. For these Cl exposures, surface Cl contents, corrected for background values, were calculated. The diffusion of this Cl into concrete decks and effects on service life of epoxy-coated, batch galvanized and 316 stainless steel rebar was then determined. A typical low-permeability concrete, typical of present practice in Virginia, was used. The behavior of Cl diffusion into this concrete, was then considered for initial surface crack densities of 0, 3, 6 and 12% of the deck area.

The range of critical value of Cl levels at which corrosion was initiated required extensive analysis because so many factors affect this number. The critical Cl thresholds for epoxy are identical to black rebar. The corrosion protection afforded by epoxy-coated rebar is limited to the extent of the propagation period. For black steel this is five years while for epoxy-coated rebar it is ten years. The protection period for galvanized rebar was concluded to be 4 to 5 times that of black bar. The propagation period is less than that of black bar because the corrosion of Zn then occurs in a higher-Cl environment and therefore is taken as two years. Therefore the sum of the protection and propagation periods for galvanized rebar was taken as 22 years. For 316L stainless steel, a propagation life of 15 years was used.
A Monte Carlo probability analysis then was performed, based on the distribution of values for each of the parameters described above. The results are reported as the per cent of the deck area that requires patching after the indicated number of years, with patch areas of 2, 4, 8 and 12% being reported. 12% patch area is assumed to be equal to the effective service life of the bridge. Based on these results, the stainless steel rebar had service life in excess of 100 years for all three crack conditions and all other parameters considered in this analysis.

The bridge deck service life results are presented in two ways: first, a set of graphs showing the effect of corrosion initiation time on the amount of patching that is required (percent deck area that requires patching) and second, the effect of deterioration time on the amount of patching required. On each of these graphs, shown below, the behavior of epoxy-coated, galvanized and the stainless steel rebar are for the Northern Climate Zone. All three climate zones considered in Table 4 shows the initial surface cracking conditions of 0, 3, 6 and 12% and the number of years that pass before 2, 4, 8 and 12% patching of the area of the bridge deck is required for epoxy-coated, galvanized and stainless steel rebar.





The cost analysis was carried out using the methodology contained in the Virginia Department of Transportation Manual, Chapter 35. A discount rate of 3.5% was used for the life cycle costs. An estimated uniform traffic control cost is included with the rehabilitation costs. The epoxy-coated rebar requires much more frequent patching than galvanized rebar and the bridge requires replacement after 54 years, compared with over 100 years for the galvanized rebar bridge. Total present costs do not include a discount rate but are useful to consider when the increases in funding required for construction equal the rate of inflation. Results are shown for all three climates in Tables 7, 8 and 9. It can be seen that in all three of these tables the galvanized steel rebar always has the lowest total present costs and life cycle costs, regardless of the amount of damage initially present in the bridge deck or the severity of the climate, using Cl dosing levels representative of Virginia. The difference between epoxy-coated rebar and galvanized rebar is seen to increase as Cl exposure increases and this trend will be expected to hold to higher Cl levels, typical of states with higher salt-dosing rates. In the most severe conditions shown in this study, the Southern Mountains and Northern Climate Zones with the most severe initial surface cracks present, the present cost of the epoxy-coated rebar exceeds that of stainless steel rebar, while the galvanized steel rebar still has a lower figure than stainless steel. It is expected that with higher Cl levels the galvanized steel total present costs and life-cycle costs would eventually approach that of stainless steel, while the epoxy-coated rebar total costs and life cycle costs would be far in excess of stainless steel. For the conditions present in Virginia, stainless steel rebar can really only be considered if a completely maintenance-free life is required for more than 100 years, regardless of cost.

INTRODUCTION

During the late 1950's, two events occurred which resulted in a number of rapid deteriorating bridge decks in the northern climates in the United States of America. Those events were the beginning of the construction of the Interstate Highway System and the implementation of a bare pavement policy. Bridge decks built in the later 1950's and early 1960's were designed and built with relatively small concrete cover depths of less than two inches and a relatively high concrete water to cement ratio (w/c) . The bare pavement policy resulted in a four-fold increase in the use of deicing salts, sodium and calcium chloride, during winter maintenance periods. The resultant was the rapid deterioration of steel reinforced concrete decks in less than five years after being opened to traffic. The cause is now the well-recognized chloride induced corrosion of the upper mat of plain (black) reinforcing steel, resulting in spalling of the cover concrete. The resultant potholes impaired driving safety and the need for the premature spending of maintenance funds to restore vehicular riding quality. Departments of Transportation (DOT's) response to the decrease in the time to maintenance, repair, and rehabilitation of steel reinforced concrete bridge decks in the Northern United States included:

- modification of the bridge deck surface water drainage characteristics to reduce the chloride laden water contact time with the concrete surface,
- reduction of the permeability of the concrete by decreasing the w/c , and
- increasing the concrete cover depths to a minimum of two inches and in some cases, to three inches.

Although these initial improvements extended the time to first repairs and subsequent overlaying, funding demanded further improvements which included:

- waterproof membranes with asphalt overlays
- two course construction to obtain increased concrete cover depths,
- replacing the plain reinforcing steel with galvanized or epoxy coated reinforcing steels, and
- polymer concrete overlays to further reduce the contact time of water with the concrete.

Further budget constraints and user impacts prompted the requirement for maintenance free time periods of 75 years and longer, resulting in the use of low permeability concretes and other corrosion resistance reinforcing steels (CRR).

Presently the Virginia Department of Transportation (VDOT) requirements for steel reinforced concrete bridge decks are:

- concrete cover depth of 2.50 inches, minus zero, plus 0.50 inches,
- low permeable concrete with a maximum w/c = 0.45 and a minimum of 635 lbs of cementitious material, Portland cement plus flyash or slag cement, and
- corrosion resistant reinforcing steel.

to achieve a minimum of 75 years of maintenance free service life for bridge decks in Virginia. Service life modeling techniques were developed to estimate and compare the chloride corrosion resistance methods for bridge decks. Initial models were deterministic models. Limitation of the deterministic models was recognized and full probability models were developed.

SCOPE

This report is limited to the following conditions:

- steel reinforcing concrete bridge decks within the Commonwealth of Virginia,
- bridge deck deicing salt exposure in Virginia Climate Zones, represented by three of the six zones,
- VDOT low permeable bridge deck concrete,
- zero, 3%, 6%, and 12% bridge surface cracking,
- Monte Carlo probability modeling based on Fick's Second Law of Diffusion,
- reinforcing types: epoxy coated (ECR), galvanized (GS) and 316 LN stainless steel (SS),
- and service life costs associated with maintaining bridge decks for a period of at least 75 years.

Fick's Second Law of diffusion requires four input parameters: surface chloride content which is influenced by the amount of deicing salt usage; concrete cover depth which is controlled during the construction process; chloride diffusion constant which is influenced by the type of concrete, construction methods, and environmental temperature and moisture conditions; and the chloride corrosion initiation values which are influenced by the reinforcing steel type and surface conditions.

MODELING PARAMETERS

SURFACE CHLORIDE

The Commonwealth of Virginia is composed of six climate zones (Williamson, 2007) and nine Department of Transportation Engineering Districts. The Engineering Districts are Bristol (1), Salem (2), Lynchburg (3), Richmond (4), Hampton Roads (5), Fredericksburg (6), Culpeper (7), Staunton (8), and Northern Virginia (9). The Climatic Zones are Southern Mountain (SM), Central Mountains (CM), Western Piedmont (WP), Northern (N) East Piedmont (EP), and Tidewater (TW). The Climatic Zones are in general agreement with the Engineering Districts but include portions of other engineering districts. In general terms, SM includes Bristol and a portion of Salem, CM includes portions of Salem and Staunton, WP includes Lynchburg and portions of Culpeper, N includes Northern Virginia and portions of Culpeper, EP includes Richmond and portions of Culpeper and the TW includes Hampton Roads and Fredericksburg.

Deicing salt usage in Virginia includes magnesium chloride pretreatment solutions, sodium, and calcium chloride salts.

Table 1 presents the six Virginia Climatic Zones and the average chloride spread per lane-mile over a three-year winter maintenance seasons.

Table 1. Average Chloride Spread on Virginia Roadways

Climatic Zone	kg-Cl / lane-km (lb-Cl /line-mile)
Southern Mountain (SM)	688(2,441)
Central Mountain (CM)	671(2,381)
Western Piedmont (WP)	270(781)
Northern (N)	4,369(15,501)
Eastern Piedmont (EP)	530(1,880)
Tidewater (TW)	225(798)

Deicer salt usage is not solely determined by winter precipitation-temperature conditions but is combined with roadway-population characteristics. Example, while it may be expected that the SM would have the greatest amount of deicer salt usage solely based on winter conditions, N deicer salt usage is over six times greater than the SM because of the population characteristics.

The six different Climatic Zones in Virginia can be represented by three zones, N(4,369 kg-Cl/lane-km), SM(688 kg-Cl/lane-km), and TW(225 kg-Cl/lane-km). Surface chloride values representing these three Climatic Zones were compiled from a Virginia bridge deck study which included 27 bridge decks built between 1984 and 1991 using a maximum w/c = 0.45 (Balakumaran, 2014). Surface chlorides were acid soluble chloride determined from bridge deck cores and corrected for the amount of background chloride content. Thus, representing only ingress chloride content. Where insufficient of the number data points existed for statistical reasons, 30 minimum, random values were selected within the range of values for the three Climatic Zones. The ranges for the Northern, Southern Mountains, and Tidewater Zone chloride were 17.0 to 9.4 kg/m^3, 10.8 to 7.0 kg/m, and 9.7 to 3.0 kg/m^3. The surface chloride data sets were the same for each service life analysis within each Climatic Zone.

CHLORIDE DIFFUSION CONSTANT

No Surface Cracking

For each low permeable concrete bridge deck core, background corrected acid soluble chloride content was determined as a function of depth (Balakumaran, 2014). Chloride samples were taken directly over a reinforcing bar at 6 mm depths and thus accounted for the influence of the reinforcing bar on the rate of chloride diffusion into the concrete. The distribution of chloride concentrations as a function of depth was analyzed by fitting a one-dimensional solution of Fick's Second Law of Diffusion to determine the effective diffusion coefficient over the period that the deck has been in service. The bridge decks were built between 1984 and 1991 and core samples taken in 2005. Seventy five diffusion constants ranged from 1 to 60 mm^2/yr. The median was 5 mm^2/yr. This data set was used for all analyses within each Climatic Zone.

Surface Cracking

All bridge deck surface cracks do not extend to the depth of the reinforcing steel. There is no relationship between surface crack width and depth (Balakumaran, 2014). Chloride samples were taken directly over the surface crack and followed the crack throughout its depth. Analysis showed the chloride ingress at surface cracks followed Fick's Second Law of Diffusion. Thirty-two diffusion constants were determined from cores with surface cracks. The range and median diffusion constant for crack condition were 6 to 1710 mm^2/yr and 61 mm^2/yr, respectively. The surface crack diffusion constant data set was shown to be statistically greater than the non-cracked condition (Balakumaran, 2014).

To account for the area influence of a surface crack, the length of the crack is multiplied by an influence length perpendicular and on each side of the crack by 50 mm. For the accessed conditions of 3%, 6%, and 12% cracked, non-crack diffusion constants were replaced with surface cracked diffusion constants. For the 3% crack condition two non-crack diffusion constant were replaced, the smallest and largest values of non-crack diffusion constants was replaced by the smallest and largest crack diffusion constants. The two valves represent 3% of the 75 non-crack diffusion constant data set. Likewise, five values were replaced for the 6% crack condition, two smallest, one median, and two largest. For the 12% crack condition nine valves were replaced, three smallest, median, and largest values.

The same diffusion constant data sets were used within each of the three Climatic Zones (N, SM, TW) surface chloride analyses.

COVER DEPTHS

Seventy-five cover depths were used. The range, mean, and standard deviation were 44 to 76 mm, 62 mm, and 8.9 mm, respectively. The cover depth data set is a representative subset of cover depths for the construction era of 1984 to 1991 (Balakumaran, 2014). The same cover depth data set was used in all of the service life analyses.

CHLORIDE INITIATION

The most cited chloride corrosion initiation concentrations in plain steel reinforced concrete ranged between 0.59 to 0.88 kg/m^3. These values were recognized as being lower conservative values. Subsequent research showed a large variability in the initiation values. However, the probability density function for chloride initiation of plain steel in concrete has not been generally agreed upon. Also, research studies using other than plain reinforcing steel often cite multiple values in comparison to plain steel.

To establish the chloride corrosion initiation distributions in this study, a literature search was conducted to better determine the initiation distribution for plain steel, ECR, GS, and SS. The literature search included Virginia Tech library on-line Summons electronic search, the author's personal library and personal contacts. The following present the results of the literature search.

CHLORIDE CORROSION OF STEEL IN CONCRETE

Plain Steel

It is recognized that the chloride corrosion threshold of plain steel in concrete is dependent upon a number of factors (Weyers, 2016).

Factors attributed to the variability for plain steel chloride threshold values include:

- cement type and chemical composition,
- degree of cementing materials hydration,
- environment temperature at bar depth,
- type of steel,
- electrical potential of the steel surface,
- presence of air voids and cracks at the concrete/bar interface,
- moisture content of the concrete at the bar depth,
- oxygen content of the concrete pore water at the bar surface,
- concentration of hydroxide in concrete pore water at the bar surface, and
- the cation, sodium or calcium, associated with the chloride.

The steel/concrete interface with hydroxide (pH) content of the concrete pore solution at the bar interface and the steel surface potential were considered the dominating influence factors

(Angst 2009, Alonso 2009). It is suggested to best identify critical chloride initiation, the reinforcing steel is to be ribbed, in the as-received condition, embedded in concrete or at least mortar, and the chloride introduced by a combination of capillary/diffusion mechanism (Angst, 2009).

Literature reviews for conditions similar to or equal to field conditions demonstrated that the critical chloride corrosion initiation is a distribution of values (Angst 2009, Alonso 2009). Critical low and high percent of chloride by weight of binder was shown to be 0.20 to 1.8%. For 635 lbs/cy of cementitious material (Portland plus slag or flyash plus cement) used for VDOT low permeable bridge deck concrete, the range would be 1.27 to 11.4 lbs chloride per cubic yard of concrete (pcy) (0.75 to 6.73 kg per cubic meter of concrete (kcm), (Weyers, 2016).

Concrete cores containing a single plain steel bar were taken from Virginia bridge decks. Cores were cyclic exposed to wet/dry cycles of 3.5% sodium chloride. Acid soluble chloride contents at the bar surface at corrosion initiation was estimated to range from 0.39 to 8.8 kcm(0.66 to 14.9 pcy) which is in general agreement with the above cited literature values. The distribution of values appeared to be normal distributed (Brown, 2002). However, a Weibull failure analysis showed that the corrosion initiation values laid on two distinct slopes, 0.39 to 2.6 kcm (0.66 to 4.40 pcy) and 2.9 to 8.8 kcm(4.9 to 14.9 pcy).

Considering the large number of chloride corrosion initiation values that can be associated with field structures as presented above, it is reasonable that chloride corrosion initiation distribution for field structure throughout Virginia can be associated with two distinct failure distributions from the Weibull failure analysis. A conservative approach is best used in estimating the chloride corrosion performance service life of steel reinforced concrete structural components. For this study, the lower range will be used, 0.39 to 2.6 kcm (0.66 to 4.4 pcy). The minimum, mode, and maximum for a triangular distribution will be 0.39 kcm (0.66 pcy), 0.85 kcm (1.44 pcy), and 2.6 kcm (4.4 pcy) resulting in a distribution skewed to the lower values. A skew to lower values have been illustrated in Brown's study and others (Weyers, 2016).

Epoxy Coated Reinforcing Steel

Rather than being a metallic surface which has an increased resistance to chloride in concrete, epoxy coated reinforcing steel (ECR) is an organic surface coating. The chloride corrosion protection hypothesis is that the epoxy coating acts like a barrier limiting exposure of the chloride to the steel surface. Also epoxy being a dielectric material limits the rate of corrosion. However, it has been determined that the chloride threshold for damaged epoxy-coated bars is similar to that of plain steel bars (McDonald, 1998). For field structures built with low permeable concrete and 50 mm of cover concrete, metallic coated reinforcing steel and ECR are surrounded by a high pH (11-12.5) concrete pore water for at least 20 years before the chloride diffuses to the bar depth. Where plain steel, stainless steel and galvanized steel bars passivate. The epoxy coating is subjected to degrading mechanisms. ECR samples extracted from bridge decks demonstrated the degradation of the epoxy coating in Virginia bridge decks (Weyers 1997, Weyers 1998, Pyc 2000, Brown 2005, Ramniceanu 2006, Weyers 2008 and Ramniceanu 2008).

The degradation of the epoxy coating consisted of the following:

- the epoxy coating debonds from the steel surface within about 4 years in concrete,
- the epoxy coating increases in moist content,
- ECR is electrically continuous,
- the coating is often not fully cured,
- the epoxy coating surface is cracked,
- crack widths in the epoxy coating surface is several orders of magnitude greater than the chloride ion, and
- the epoxy coating is often dented, mashed, with breaks and holidays.

It is for these field conditions for ECR that the chloride corrosion threshold can only be determined from field samples.

Sagüés reported that chloride corrosion concentration for ECR was not precisely known. But evidence suggests that the chloride corrosion initiation of ECR is at best in the same order as plain steel, 0.71 to 2.12 kcm (1.2 to 3.6 pcy) (Sagüés, 2003). Brown extracted 4-inch diameter cores from bridge decks in Virginia ranging in age from 4 to 16 years. Cores containing a single ECR #5 bar, 16 mm diameter (0.625 in.) were cyclic ponded in the laboratory with a 3% sodium chloride solution. Corrosion progress was monitored using Electrochemical Impedance Spectroscopy (EIS). The chloride corrosion initiation appeared to be a bimodal distribution (Brown, 2002). Weibull distribution, a common failure analysis for small data sets, was conducted using Brown's data (Brown, 2002). The ECR and plain steel values lied on the same straight line from 0.08 to 4.5 kcm (0.13 to 7.6 pcy). From 6.3 to 8.8 kcm (10.7 to 14.9 pcy) the ECR straight line was parallel to the plain steel straight line from 2.9 to 88 kcm (4.9 to 14.9 pcy). The ECR in the upper range is 2.9 kcm (4.9 pcy) greater than the plain steel. The difference reflects the corrosion condition variable influence for both the ECR and plain steel experienced in the field along with some influence by the epoxy coating. The results illustrate that the ECR system is not a truly reliable chloride corrosion protection method for field structure.

Balakumaran conducted a corrosion condition assessment on a 30 year old bridge deck built with a top mat of ECR in Virginia (Balakumaran, 2013). Corrosion potential, corrosion rates (3 LP) and chloride contents as a function of depth were taken at 30 locations. The deck, which was built in 1979, exhibited 14% corrosion damage. Modeling converged on 14% damage with a minimum, maximum, mode chloride contents of 0.29, 2.42 and 0.77 kcm (0.49, 4.10 and 1.30 pcy). The results illustrate that the corrosion threshold of ECR was similar to plain steel. For this study, ECR minimum, maximum and mode of the triangular distribution will be the same as plain steel, 0.39, 2.6, and 0.85 kcm (0.66, 4.4 and 1.44 pcy), respectively. Thus, the corrosion performance of ECR is limited to the extension of the corrosion propagation period. Brown estimated the extension to be 5 years, which was in agreement with a previous study of Virginia bridge decks and piles (Brown 2002, Weyers 1997). Considering plain steel propagation period to be 5 years, the total propagation period for ECR is 10 years.

Hot-Dipped Galvanized Reinforcing Steel

Table 2 presents a range of values for the chloride corrosion threshold values, from 1.5 to 3.1 times the threshold of black bar, which includes statements of at least 2.5 times black bar. The most cited value is 2.5 times black bar which will be used in this study to estimate the time to corrosion initiation for hot-dipped galvanized reinforcing steel. The minimum, maximum, and mode for hot-dipped galvanized steel for this study will be set at 0.97, 6.3 and 2.1 kcm (1.64, 10.7 and 3.5 pcy).

Table 2. Galvanized Steel Chloride Corrosion Threshold

Chloride Threshold	Method	Reference
At least 2.5 times black steel	In concrete, wet/dry cycle with NaCl	Yeomans, 1994
At least 2 to 2.5 times black steel	From laboratory and field studies	Yeomans, 2016
On average 1.58 times black steel	In concrete, wet/dry cycle-NaCl	Darwin, et. al. 2009
3.1 times black steel	In concrete, admixed with CaCl	Hegyi, et. al. 2015
1.5 to 2.5 times black steel	In chloride contaminated concrete	Bertolinli, et. al. 2013
2.0 times black steel	From laboratory and field studies	Sanchez, et. al. 2014

The corrosion protection time for hot-dipped galvanized reinforcing bar in chloride contaminated concrete is defined as the time period from corrosion initiation to dissolution of the zinc and iron-zinc layers and thus the exposure of the underlying steel. The protection period has been estimated at 4 to 5 times black bar (Yeomans 1994, Yeomans 2016, Chapter 6). Where plain steel bar period is defined as the period from corrosion initiation to cracking and spalling of 50 mm of cover concrete. This period is referred to as the corrosion propagation period which is generally agreed to be about five years (Brown 2002, Sagüés 2014). Following the dissolution of zinc layers, corrosion of the underlying steel commences, but at an accelerated rate due to the higher chloride at the bar surface. The propagation period will be less than the 5 years for black bar, estimated at 2 years. Thus for hot-dipped galvanized steel in this study the protection period plus the propagation period is estimated at a conservative time period of 22 years.

Stainless Steel

Table 3 presents the results chloride corrosion initiation of 316 LN in mortar and concrete. Considering a cementatious material (Portland cement plus slag or flyash) of 635 lbs/cy minimum as specified by the Virginia Department of Transportation for bridge deck concrete, the range of chloride corrosion threshold values would be 9.73 to 30 kcm (16.5 to 50 pcy) as shown in Table 3. The most cited value is about 3.5% of cementitious material or 13 kcm (22 pcy) for 635 lbs of cementitious material which is selected as the mode values. The minimum and maximum values are to be conservative values of 9.4 and 18.8 kcm (16 to 32 pcy). The resulting triangular distribution would approximate a normal distribution.

Table 3. 316 LN Steel Chloride Corrosion Threshold

Chloride Threshold	Method	Reference
>5 to > 8% by wt. of cement	Admixed in concrete or mortar	Hansson, 2016
3.5% by wt. of cement	Ponding of concrete	Hansson, 2016
3.5 to 8% by wt. of cement	Concrete structures in salt laden environments	Pietro, 2004
2.6 to 3.5% by wt. cementitious material	Ponding of mortar	Islam, 2013
12.1 kcm	Ponding of concrete	Clemena, 2002
8.3 to 12.8 kcm	Chloride into mortar, potential gradient	Trejo, 2004
10 times plain steel	Chloride ingress, concrete laboratory	Sanchez, et. al. 2014

Considering the critical corrosion rate of black bar and 316 LN stainless steel bar to be equal to cause cracking and spalling of the cover concrete and the average corrosion rate of 316 LN after corrosion initiation equal to one-third of black steel (Hartt 2012), the corrosion propagation period for a 50 mm concrete cover will be three times greater than the black bar period of 5 years, or a 15 year propagation period for 316 LN rebar.

Summary Chloride Corrosion Initiation and Propagation

The following summarizes the triangular distributions and the corrosion propagation periods used in this report, presented in the preceding sections for ECR, GS, and 316 LN SS.

Bar Type	Minimum kcm (pcy)	Maximum kcm (pcy)	Mode kcm (pcy)	Propagation yrs
ECR	0.39 (0.66)	2.6 (4.4)	0.85 (1.44)	10
GS	0.97 (1.64)	6.3 (10.7)	2.1 (3.6)	22
316 LN SS	9.4 (16)	18.8 (32)	13 (22)	15

MODELING RESULTS

Figures 1 and 2 presents atypical output results of the probability corrosion initiation and service life performance for the Northern Climatic Zone for surface cracking influencing 6% of the deck area. Figure 1 presents the corrosion initiation time in years as percent of deck area. Figure 2 presents service life prediction as the percent deck deterioration as function of time in years. Deck deterioration is the sum total of the corrosion initiation plus the corrosion propagation time period. As shown ECR and GS service life at 12% damage are 55 and 108 years, respectively. 316 LN SS is far in excess of 100 years. The 2% ECR corrosion initiation occurs at 1 year and first deck patching would occur at 11 years. An additional 10% patch would be required between 11 and 55 years, at which time the deck would need to be overlaid. For GS, first patching would be needed at 23 years at 2% deterioration and an additional 6% patching to reach the 75 year period.

For all conditions, 316 LN SS in Virginia Bridge Decks, the maintenance free service life is in excess of 100 years.

Table 4 summarizes the corrosion performance service life of Virginia Bridge Decks in the Tidewater, Southern Mountains, and Northern Climatic Zones for 0%, 3%, 6%, and 12% surface cracking conditions. As previously stated, 316 LN SS has a maintenance free service life in excess of 100 years for all three Climatic Zones and the three surface crack conditions. There is little difference in corrosion protection performance between 0% and 3% cracking for ECR and GS in all three Climatic Zones. These results are in agreement with previously reported analysis (Balakumaran, 2014).

Percentile (%)	Time to 2% Initiation (Years) Epoxy Coated	Galvanized	Stainless Steel
0.5	0	1	52
1	0	1	475
2	1	1	1192
4	18	33	2337
8	34	64	4868
12	45	86	8537
16	55	106	9999
48	136	282	9999

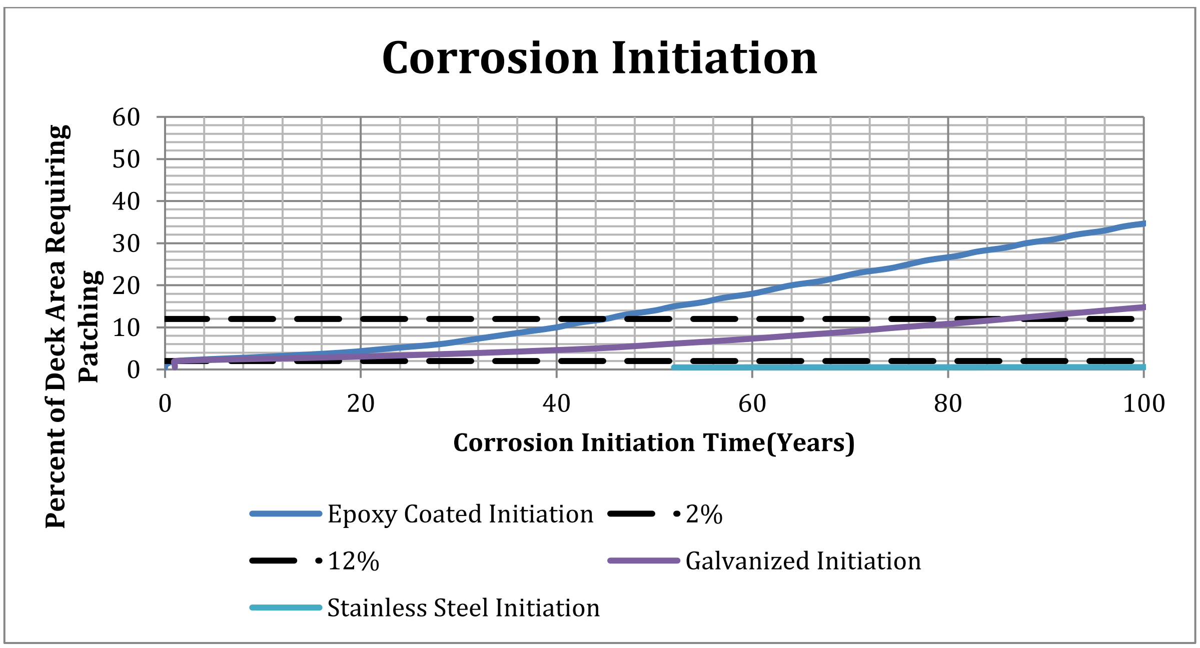


Figure 1. Virginia Northern Climate Zone, Deck Surface Cracking Influence Area 6%, Corrosion Initiation

Estimated Time Due to Diffusion			
	Epoxy Coated	Galvanized	Stainless Steel
Time to 2% Initiation (years)	1	1	1192
Time to Crack Concrete (Years)	10	22	15
Time for Diffusion (2% to 12%) (Years)	44	85	7345
Total Time for 12% Damage (Years)	55	108	8552

Figure 2. Virginia Northern Climate Zone, Deck Surface Cracking Influence Area 6%, Service Life

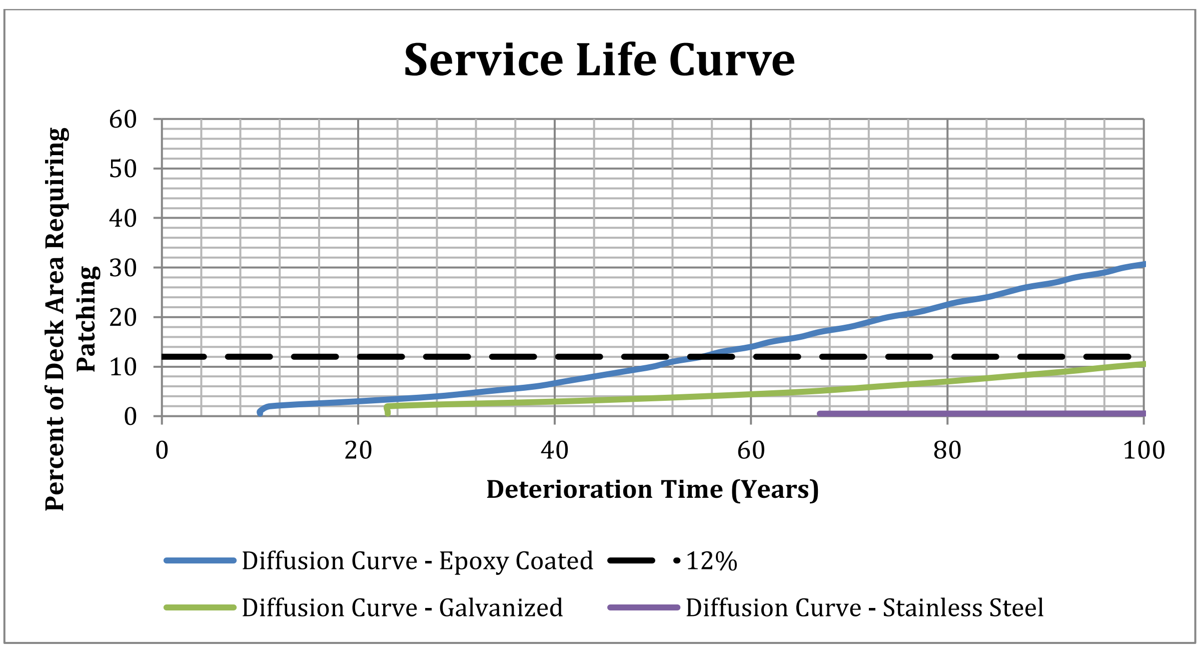


Table 4. Corrosion Resistance Performance of Virginia Bridge Decks

Virginia Climatic Zone: Tidewater

Damage	ECR	GS	SS
No Surface Cracking			
2%	44	>100	>100
4%	54		
8%	72		
12%, EFSL	88		
3% Surface Cracking			
2%	38	83	>100
4%	50	>100	
8%	69		
12%, EFSL	87		
6% Surface Cracking			
2%	11	31	>100
4%	38	95	
8%	62	>100	
12%, EFSL	80		
12% Surface Cracking			
2%	11	25	>100
4%	15	51	
8%	44	>100	
12%, EFSL	65		

Table 4. Corrosion Resistance Performance of Virginia Bridge Decks (Continued)

Virginia Climatic Zone: Southern Mountains

Damage	ECR	GS	SS
No Surface Cracking			
2%	38	81	>100
4%	46	99	
8%	58	>100	
12%, EFSL	71		
3% Surface Cracking			
2%	34	68	>100
4%	44	92	
8%	58	>100	
12%, EFSL	71		
6% Surface Cracking			
2%	11	25	>100
4%	32	69	
8%	51	>100	
12%, EFSL	65		
12% Surface Cracking			
2%	11	24	>100
4%	17	36	
8%	37	85	
12%, EFSL	55	>100	

Table 4. Corrosion Resistance Performance of Virginia Bridge Decks (Continued)

Virginia Climatic Zone: Northern

Damage	ECR	GS	SS
No Surface Cracking			
2%	33	63	>100
4%	39	76	
8%	48	95	
12%, EFSL	59	>100	
3% Surface Cracking			
2%	29	55	>100
4%	37	71	
8%	49	96	
12%, EFSL	59	>100	
6% Surface Cracking			
2%	11	23	>100
4%	28	55	
8%	44	86	
12%, EFSL	54	>100	
12% Surface Cracking			
2%	11	23	>100
4%	15	31	
8%	30	62	
12%, EFSL	46	89	

For all conditions, Climatic and cracking, GS estimated maintenance free time periods and time to overlay at 12% damage (end of functional service life, EFSL) are significantly greater than ECR.

Within the TW zone at 6% and 12% cracking GS would require first patching at 31 and 25 years, respectively. GS would require less patching than ECR, at 6% surface cracking 4% patching in 95 years and 4% patching in 51 years at 12% surface cracking, see Table 4. Whereas, ECR at 6% and 12% surface cracking patching of 2% of the deck surface area would occur at 11 years. The ECR 12% cracking condition in the TW zone would need to be overlaid at an age of 65 years, but the ECR 6% surface cracking would only exhibit greater than 8% patching needs in 75 years, see Table 4.

Comparison between the Climate Zone, shows that as deicing salt increases, the corrosion resistance performance of both the ECR and GS decreases. For the SM, ECR would require overlaying for all four bridge deck surface conditions, 71 years at 0% and 3% surface cracking, 66 years at 6% surface cracking and 55 years at 12% surface cracking. The GS decks would not need to be overlaid within 75 year service life period. The time at which first patching would take place for GS decks is greater than ECR decks, 29 years and 13 years greater for 6% and 12% surface cracking, respectively.

The Northern Climate results are similar to the SM results, GS decks would outperform ECR decks. ECR decks would experience patching needs early, 11 years at 6% and 12% surface cracking. Whereas, GS decks first patching would occur at an estimated 23 years, but would not need to be overlaid within the 75 year service life. For the ECR decks, they would have to be overlaid at 59 years, 54 years, and 46 years for the 3%, 6% and 12% surface cracking conditions.

COST ANALYSIS

VDOT Chapter 35 includes guidelines for life cycle cost analysis (VDOT, Chapter 32, 2012). The criteria to be followed shall be used when applicable. The criteria included are new and rigid overlays and type B patching. Type B patching is defined as a removal depth to below the upper mat of reinforcing steel. The criteria used in this cost analysis for new/replacement decks are the factors determined previously for the three climatic zones, degree of surface cracking and EFSL at 12% deterioration. Twelve percent deterioration value was previously estimated during the SHRP Program (Weyers, 1993). For rigid overlays, in this case, latex modified concrete, very early strength (VDOT LMC-VE) is used. The VDOT Chapter 32 criteria was used for the LMC-VE overlay, 2% patching at 10 years and 2% patching every 2 years thereafter until 20 years with a presumed life of 25 years.

Life cycle cost analysis (LCCA) calculation used the Present Worth methodology as illustrated in the U.S. Department of Transportation Primer (Life Cycle Cost Analysis Primer, 2002). The primer states "adjusting for inflation and discounting are entirely separate concerns, and they should not be confused by attempting to calculate both at once". Nominal or market interest rates typically range between 3 to 5 percent (Primer, 2002). Others have estimated the real discount range as 4 to 6 percent (Cady, 1988). Thus, a real interest rate of 3.5% was used in the LCCA in this study.

The primary reason for excluding an inflation factor is the highly variable nature of inflation. The Consumer Price Index (CPI) follows the same general inflationary rates as the engineering indexes, Engineering News Record (ENR), FHWA Composite Construction Cost Index (CCCI) and the FHWA Structures Construction Cost Index (SCCI) (Williamson, 2007). As an example of the variability of inflation, the CPI was relatively uniform at 8% between

1973 and 1983, 3% between 1983 and 2008, a deflation rate of 0.4% in 2009, and about 1.5% from 2010 through 2016.

For the condition, which will be used in the comparison of Total Cost (TC), where the inflation rate and the increase in funding are equal, the dollar values (purchasing power) are equal throughout the study periods (Cady, 1988).

PRICES

Factors that are common amount comparison construction methods need not be included in the TC and LLCA Methods. For new/replacement bridge decks in this study for the influence of epoxy coated reinforcing steel (ECR), hot-dipped galvanized reinforcing steel (GS), and solid stainless reinforcing steel (Class III) cost of the rebar only need to be included in the cost analyses. However, the VDOT bridge deck concrete Class A4/A4 low shrinkage concrete is a low permeable concrete and thus was included and was used in the service life performance modeling because of its enhanced concrete characteristics. Other cost factors also need to be considered, user costs and traffic control, both of which are unique to individual construction sites. Thus, user costs are not included but an estimated uniform traffic control cost is included in the Type B patching and LMC-VE overlay prices (Williamson, 2007). The prices used in this study are presented in Table 5 and are for the year 2016. For price determined in years other than 2016, the USDOT/FHWA National Highway Construction Cost Index (NHCCI) was used to bring the price forward to 2016. NHCCI is replacing the other previously cited USDOT/FHWA indexes.

Table 5. Bridge Deck Construction and Rehabilitation Prices, 2016

In-Place Unit Price, $/lb.

Rebar Type	ECR	GS	Class III
	1.15	1.15	3.20

In-Place VDOT Class 4A/Class 4A, Low Shrinkage, $900.00/cy

*Initial Bridge Deck Construction Price, $

Rebar Type	ECR	GS	Class III
	$239,670	$243,220	$331,080

LMC-VE Overlay, Type B Patch, Traffic Control Prices

In-Place LMC-VE,	$13.35/sf
Milling	$ 2.55/sf
Grooving	$ 0.60/sf
	$16.50/sf
Type B patching,	$55.00/sf
Traffic Control,	$1.70/sf

*Based on average deck thickness 8.5 in., 4.172 ft. #5 bar plus 1.336 ft. #4 bar/sf of deck surface, and average bridge deck of 40 ft. by 200 ft. (8000 sf).

Reinforcing Steel

On August 22, 2012, VDOT issued the Structure and Bridge Division Instructional and Information Memorandum, Number 11M-S&B-81.5. In part the memorandum states “Based on research completed at the Virginia Transportation Research Council (VTRC) and at Virginia Tech, the Structure and Bridge Division in consultation with FHWA has decided that to achieve a 75-year or longer life of our bridges, we should discontinue the use of epoxy coated bars and galvanized bars.” While many types of corrosion resistant reinforcing (CRR) steels had been studied, the decision was made to use the following three types of deformed bars:

1. Low Carbon/Chromium reinforcing steel bars conform to ASTM A1035/A1035M, (MMFX-2 and possibly others).
2. Stainless reinforcing steel clad bars conforming to AASHTO Designation MP13M/MP 13-04.
3. Solid stainless steel reinforcing bars conforming to ASTM 955/A955M-UNS Designations S24000, S24100, S30400, S31603, S31653, S31803, S32101.

The above steel reinforcing bars have been placed in the following classes for use in the indicated roadways.

Class I	Low-Carbon/Chromium, Solid Stainless S32101: Rural collector and local streets and urban collector and local streets.
Class II	Solid stainless S24100 and stainless steel clad: Rural minor arterial and urban collector street.
Class III	Solid stainless S24000, S30400, S31603, S31653, S31803, S32304, Freeway, Rural Principal Arterial, Urban Principal Arterial.

The price shown in Table 5 for Class III, which includes 316 LN, is the weighted average low bid in the VDOT Bid Tabulations for the entire year 2016.

The price for ECR, which was discontinued by VDOT, is the weighted average minimum from West Virginia Transportation Department, a state in near location to Virginia, bid tabulations for the years 2014 and 2015. The year 2016 was not available on-line.

States in close proximity to Virginia do not extensively use galvanized rebars. Thus, the price for GS in Table 4 was taken from the literature (Berke 2012, White 2005). These citations were confirmed by the following: cost to galvanize rebar is $0.11 to $0.17/lb from survey of galvanizer's plus a weighted average low bid price, North Carolina bid tabulations 2016 for black bar $1.00/lb. Thus approximate GS range $1.11 to $1.17/lb. Galvanized steel rebar of $0.68 to $0.73/lb with ECR being near midrange at $0.70 for material cost only (Triandafilou, 2012).

The Class 4A/Class 4A Low Shrinkage concrete, LMC-VE overlay and Type B patch is also the weighted average low bid price from VDOT Bid Tabulations of the entire year 2016.

LCCA and Total Present Cost

Table 6 presents the cash flow values for 6% surface cracking in the Northern Climate Zone of Virginia for the costs at years from the service life performance analysis. As illustrated, the ECR built deck requires significantly more patching than the GS built deck and also require to be overlaid at year 54. Whereas the GS built deck would only require to be patched. The Class III deck would be maintenance free at 75 years and beyond 100 years.

Tables 7, 8, and 9 summarizes the TC and LCCA analyses for the 3%, 6%, and 12% deck surface cracking for three Virginia Climatic Zones: Tidewater, Southern Mountains, and Northern Zone.

Table 6. Cash Flow, 6% Cracking Northern Climate

Percent Deterioration	At Year	Activity	Cost $	Factor 3.5%	LCCA $
Epoxy Coated Rebar, Initial Construction					239,670
2%	11	Patch	9,120	0.6849	6,250
4%	28	Patch	9,120	0.3816	3,480
8%	44	Patch	18,240	0.2201	4,010
12%	54	Overlay	145,600	0.1546	22,720
2%	64	Patch	9,120	0.1106	1,010
4%	66	Patch	9,120	0.1032	940
6%	68	Patch	9,120	0.0963	880
8%	70	Patch	9,120	0.0899	820
10%	72	Patch	9,120	0.0840	770
12%	74	Patch	9,120	0.0784	720
Total Costs			476,470		281,270
Galvanized Rebar, Initial Construction					243,220
2%	23	Patch	9,120	0.4533	4,130
4%	55	Patch	9,120	0.1508	1,370
6%	75	Patch	9,120	0.0757	690
Total Costs			270,580		249,410
Class III Stainless Rebar, Initial Construction			331,800		331,800

Table 7. Present Cost and Life Cycle Cost – Tidewater Climate Zone

Rebar Type	Present Cost $	Difference $	Difference %	LCCA $	Difference $	Difference %
3% Deck Cracking						
ECR	267,030	--	--	244,620	--	--
GS	243,220	23,810	9	243,220	1440	1
Class III	331,080	64,050	-24	331,080	85,720	-35
6% Deck Cracking						
ECR	276,150	--	--	250,550	--	--
GS	252,340	23,810	9	246,360	4190	2
Class III	331,080	54,930	-20	331,080	80,530	-32
12% Deck Cracking						
ECR	294,390	--	--	257,330	--	--
GS	261,570	32,820	11	248,790	8540	3

Class III	331,080	36,690	-12	331,080	73,750	-29

Table 8. Present Cost and Life Cycle Cost – Southern Mountains Climate Zone

Rebar Type	Present Cost $	Difference $	Difference %	LCCA $	Difference $	Difference %
3% Deck Cracking						
ECR	294,390	--	--	248,080	--	--
GS	252,340	42,050	14	244,095	3980	2
Class III	331,080	36,690	-12	331,080	83,000	-33
6% Deck Cracking						
ECR	312,630	--	--	258,180	--	--
GS	263,740	48,890	16	248,100	10,080	4
Class III	331,080	18,450	-6	331,080	72,900	-28
12% Deck Cracking						
ECR	476,480	--	--	287,330	--	--
GS	275,140	201,340	42	249,240	38,090	13
Class III	331,080	145,400	30	331,080	47,750	-15

Table 9. Present Cost and Life Cycle Cost – Northern Climate Zone

Rebar Type	Present Cost $	Difference $	Difference %	LCCA $	Difference $	Difference %
3% Deck Cracking						
ECR	458,230	--	--	271,170	--	--
GS	261,684	196,500	43	245,400	25,770	9
Class III	331,080	127,150	28	331,080	59,910	-22
6% Deck Cracking						
ECR	476,470	--	--	281,260	--	--
GS	270,580	208,890	43	249,410	31,850	11
Class III	331,080	145,390	30	331,080	49,820	-17
12% Deck Cracking						
ECR	502,300	--	--	296,920	--	--
GS	288,820	213,480	42	253,340	43,580	15
Class III	331,080	171,220	34	331,080	34,160	-11

For the study criteria of a bridge deck with low permeable concrete, design cover depth of 2.5 inches, and chloride surface concentrations, hot-dipped galvanized reinforcing steel has the lowest life cycle cost for all combinations of deck cracking and environmental climate zones. Class III solid stainless as represented by 316 LN is the most costly alternative based on life cycle costs, but provides a maintenance free condition for service lives of greater than 75 years. Epoxy coated reinforcing steel would require the greatest amount of maintenance over a 75 year service life compared to galvanized reinforcing steel. The cost for same cracking-chloride exposure conditions, the difference is small based on life cycle costs except for the SM at 12% surface cracking and in all three surface cracking conditions for the N Climatic Zone.

Life cycle cost analysis is an investment scheme that provides transportation agencies the ability to enhance its stewardship of the public's investment in transportation facilities. However, the lowest life cycle cost may not be the best solution when considering budget constraints. Total present cost can be considered when the rate of funding increase is equal to the rate of inflation. As previously stated, Table 7, 8, and 9, present the total present cost (TPC) along with the LCCA. As shown, galvanized reinforcing steel would provide the lowest TPC for the criteria used in this report. However the TPC difference between ECR and GS increase significantly with increasing chloride exposure as represented by the Tidewater, Southern Mountain and Northern Climatic Zones in Virginia. Also for ECR versus Class III, the TPC of ECR approaches and exceeds the TPC of Class III as the degree of cracking and chloride exposure condition increase. ECR TPC exceeds Class III in the Northern Climate Zone and for the 12% cracking in Southern Mountain Climate Zone. These results reflect the influence of increasing surface cracking and increasing chloride exposure conditions.

CONCLUSIONS

For the study criteria of low permeable bridge deck concrete, design cover depth of 2.5, and the climatic chloride exposures, the following conclusions are warranted.

- The probability chloride diffusion-based model demonstrates the influence of bridge deck surface cracking conditions and chloride exposure conditions.
- Comparing ECR, GS, Class III solid stainless steel, GS has the lowest LCC and TPC.
- GS performs better than ECR. The difference between ECR and GS increases as Cl exposure increases. This trend will be expected to hold to higher Cl levels, typical of states with higher salt-dosing rates.
- The service life of GS decks is shown to be 100 years. This is in comparison to ECR deck life of 55 years and Class III solid stainless steel of 100+ years.
- Class III stainless steel becomes a more favorable alternative for the condition of increased bridge deck surface cracking and increase chloride exposure.

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