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Economic Viability of Implementing Empirical Bridge Deck Design with Stainless Reinforcing Steel

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***Abstract*: The traditional method for designing bridge decks in the United States, otherwise known as the Equivalent Strip Method, as presented in the AASHTO LRFD Bridge Design Specifications has been in practice for a considerable amount of time. More recently the Empirical Deck Design has become more accepted and ultimately codified by being incorporated into the 2012 AASHTO LRFD Bridge Design Specifications; however, there is hesitation by many states in the US to actually implement it due, mainly, to the fact that is thought to produce more cracks when compared to the traditional design method. An increase in cracking can lead to an increase in the corrosion of reinforcing steel, and, ultimately, a decrease in expected service life. Epoxy coating is a commonly used corrosion delaying mechanism and has been around for many years. However, more recently stainless reinforcing steel has been considered due to its incredible corrosion resistance. It is postulated that, although stainless steel does have a higher initial cost, when used in conjunction with empirical design, stainless steel may be the most cost-beneficial when evaluating the ownership cost for a bridge deck over the course of the expected service life. This paper describes an investigation of economic viability of using stainless steel in bridge deck design. Results indicate that there is a high likelihood that implementing stainless steel reinforcement following the empirical bridge deck design approach will produce the lower annual costs over traditional AASHTO bridge deck design with epoxy-coated reinforcement while still resulting in a high quality, serviceable bridge deck.**

***Keywords*: Empirical bridge deck design, stainless steel, epoxy-coated reinforcing, life cycle cost analysis.**

I. INTRODUCTION

Ever since carbon steel has been used as a means of reinforcing concrete bridge decks, there has been a constant battle with corrosion-related deterioration. In 2002, the Federal Highway Administration (FHWA) released a study that stated that the United States spends an astounding amount of funds – $276 billion – directly trying to combat this issue (Balvanyos et al, FHWA, 2001). Important as it is to implement corrosion-preventive measures, equally as important is keeping costs low while still maintaining a bridge's structural integrity. A potential solution to this situation is combining stainless reinforcing steel with a design approach for bridge decks that requires minimal reinforcement.

The traditional method for designing bridge decks in the United States, otherwise known as the Equivalent Strip Method as presented in the AASHTO LRFD Bridge Design Specifications (AASHTO LRFD 2012), has been in practice for a considerable amount of time. More recently (in the past 30 years) the Empirical Deck Design – also known as the Ontario Method due to its development by the Ontario Ministry of Transportation – has become a codified and acceptable approach (Uppena, 1999). It has been referred to as a "no analysis" method, as it only requires that the deck have two isotropic reinforcing steel mats on the top and bottom that provide a set reinforcing ratio, as opposed to providing reinforcing steel based on flexural capacity requirements (AASHTO LRFD 2012). Because of the fundamental concepts behind the Empirical Design Approach, significantly less steel is required, which can potentially lead to substantial cost savings. The empirical design method is the main design method utilized in Canada and it has been incorporated in the 2012 AASHTO LRFD Bridge Design Specifications as an allowable design alternative; however, there is hesitation by many states in the US to actually implement it due to designer unfamiliarity and concerns that decks designed this way may have more cracks than bridges designed following traditional design approaches, among other reasons (Veen, 2008).

The main hesitation about using empirical design comes from the fact that the reduced amount of steel may result in more cracking compared to traditional designs (Veen, 2008). An increase in cracking can lead to an increase in corrosion of reinforcing steel, and, ultimately, a decrease in expected service life. It is widely known that corrosion is very prevalent in locations where there is a heavy use of deicing salts (Hart et al, 2009). It is further accepted that the chloride ions present in the salt permeate through the concrete – most quickly via cracks in the concrete – and penetrate to the level of the reinforcing steel, and eventually cause corrosion (Hart et al, 2009).

A common way of protecting decks from corrosion issues is to coat the reinforcing steel in epoxy. This has been proven to be an effective way to add years to the service life of a bridge deck (McDonald et al, 1998). A second, and less utilized approach, is to do away with coatings altogether, and replace traditional “black” reinforcing bars with stainless steel, as it has a much higher chloride ion threshold than its epoxy-coated counterpart (McDonald et. al., 1998). Fortunately, a review of available literature reveals no studies that indicate that initial cracking or poor initial performance is more likely for stainless steel over epoxy-coated reinforcement. There are a number of different types of stainless steel allowed for use and include: S24100 (XM-28), S31653 (316LN), S31803, and S32304 (2304) (IDOT, Materials IM 452). Although there are several differences, the different types principally correspond to differing metal compositions. Type 316LN appears to be the most commonly used stainless steel reinforcement option and due to the substantial corrosion resistance (with a yield strength of 413.7 MPa (60 ksi)). For simplicity, Type 316LN will be assumed for use in this investigation.

Stainless steel reinforcement offers a significantly higher service life than epoxy coated bars. Several studies [(Browning et al, 2011), (Bergman and Schnell, 2007), (Kahl, 2012), (Hart et al, 2009), (McDonald et al, 1998)] report a design life for stainless steel of 100 years or more; however, the increased protection comes with a much higher initial cost. Stainless steel has the potential to be 3-4 times the cost of epoxy coated bars on a per pound basis (Wenzlick, 2007).

Though the initial cost is high, the cost savings accrued over the deck’s service life due to increased corrosion protection has the potential to make stainless steel the most cost-beneficial option. Pairing the corrosion resistance of stainless steel with the decreased required amount of steel associated with empirical deck design aims to even further decrease the long-term cost of bridge decks. In the work described herein a structure was chosen for a cost-analysis of various design alternatives. The life cycle cost analysis was performed on six design alternatives – traditional, empirical, and semi-empirical deck design options with both stainless steel and epoxy-coated reinforcing. This paper details the empirical design process, variables that affect a life cycle cost, the life cycle cost analyses of the different alternatives (including a Monte Carlo based sensitivity study), and offers comparisons and conclusions from said analyses.

II. EMPIRICAL BRIDGE DECK DESIGN

The empirical deck design method was first introduced in 1979 by the Ontario Ministry of Transportation, which is why it is also often referred to as the Ontario design method (Mulert, 2001). It was a method whose goal was to reduce the amount of reinforcing steel required in bridge decks. Before this method was introduced, it was the common belief that the main way decks resisted wheel loads was through flexure, and that it would eventually fail in flexure. Studies conducted in Canada suggested that failure actually occurred as a result of local punching shear that does not affect the rest of the deck (Beal, 1983). This eventually led to the discovery of arching action (Mulert, 2001).

The 2012 AASHTO LRFD Bridge Design Specifications offers the following commentary on arching action and empirical design as a whole: “This action is made possible by the cracking of the concrete in the positive moment region of the deck slab and the resulting upward shift of the neutral axis in that portion of the slab. The action is sustained by in-plane membrane forces that develop as a result of lateral confinement provided by the surrounding concrete slab, rigid appurtenances, and supporting components acting compositely with the slab. The arching creates what can best be described as an internal compressive dome, the failure of which usually occurs as a result of overstraining around the perimeter of the wheel footprint. The resulting failure mode is that of punching shear, although the inclination of the fracture surface is much less than 45 degrees due to the presence of large in-plane compressive forces associated with arching. The arching action, however, cannot resist the full wheel load. There remains a small flexural component for which the specified minimum amount of isotropic reinforcement is more than adequate. The steel has a dual purpose: it provides for both local flexural resistance and global confinement required to develop arching effects (Fang, 1985; Holowka et al., 1980). All available test data indicate that the factor of safety of a deck designed by the flexural method specified in the 16th edition of the AASHTO Standard Specifications, working stress design, is at least 10.0. Tests indicate a comparable factor of safety of about 8.0 for an empirical design. Therefore, even the empirical design possesses an extraordinary reserve strength," (AASHTO 2012 LRFD C9.7.2.1).

Because of this arching effect, decks require only a minimal amount of steel. This steel does not have to be "designed" as with flexure, but instead, a minimum isotropic mat – which is why this method is also termed, "isotropic" deck design – of bottom and top longitudinal and transverse steel can be utilized. A typical section for an empirically designed deck may consist of No. 4 bars spaced at 305 mm (12") on center in the top and bottom layers in both the longitudinal and transverse directions (Uppena, 1999). The top of the top layer of reinforcing is placed 50.8 mm (2") from the bottom of the wearing surface, and the entire depth of the structural deck (not including the wearing surface) is 203 mm (8"). This layout provides for reinforcement ratio of 0.3% which is a reduction of 43% of the reinforcing steel used in a standard AASHTO designed deck (Uppena, 1999).

Because of the reduction in steel, material costs for bridge decks are also lessened. Additionally, construction costs have the potential to be reduced as less reinforcing steel needs to be placed, reducing time and, hence, labor costs. In several studies, it was shown that these decks performed similarly to AASHTO designed decks; however, empirically designed decks did tend to show more longitudinal cracks than traditionally designed decks. "Once cracking starts on an isotropic deck, the reduced steel is not strong enough to prevent further cracking," (Brooks, 1998). For this reason, these decks are more susceptible to corrosion of the reinforcing steel, but not so much so as to warrant them impractical.

Several states have studied empirical design since it was introduced into the LRFD Bridge Design Specifications in 1995 (Uppena, 1999). These states include: New York, Oregon, Florida, Michigan, Pennsylvania, Wisconsin, and Nebraska. A study released by the Wisconsin DOT recommended that empirical design not be used on a regular basis (Battaglia, 2012); however this same study, and others similar to it indicate that empirically designed decks do not show significantly poorer performance than a traditionally designed deck [(Uppena, 1999), (Veen, 2008), (Jansson, 2008), (Battaglia, 2012)]. Iowa (where isotropic deck design standards have been developed, but not adopted) built 14 empirical decks between 1994 and 1997. After 5 years of service, the decks were observed to be performing well with only minor cracking. These bridge decks saved the state an estimated $375,000 in steel costs (Mulert, 2001). Canada – the pioneers of the empirical deck design movement – saved an estimated $14 million over a 15 year period by implementing isotropic decks (Mulert, 2001).

Even with all the positives that come with using empirical deck design, several states are still hesitant to use it because of unfamiliarity with the method and concern that corrosion will occur sooner than with traditionally designed decks (Veen, 2008).

What is known today as the empirical deck design method, or the Ontario method, first started as a mathematical, theoretical model developed by Hewitt, et al and Batchelor, et al. They theorized that in-plane compressive forces develop as a slab cracks. Hewitt describes these forces as, "compressive force at panel edge and moment at the tension reinforcing," (Puckett et al, 1988). Together, these forces strengthen the slab to where its ultimate failure mode is punching shear instead of flexure. Hewitt's model includes boundary restraining forces, which are represented by a coefficient in the model that ranges from zero for simply supported spans up to one for completely fixed supports. The higher the boundary restraining forces, the higher the theoretical punching strength. Using Hewitt's model, a hypothetical slab would have its punching shear strength tripled going from a boundary restraining factor of 0 to 1 (Puckett et al, 1988). To test this theoretical model, they tested 27 circular slabs. They implemented a steel ring which was used to calibrate the restraining factor. Using these known restraint values, they were able to investigate the accuracy of Hewitt's theory. After testing ensued, they found that membrane forces varied linearly as concentrated loads increased and all specimens failed by punching shear, thus validating the mathematical model (Puckett et al, 1988).

The Ontario Ministry of Transportation carried out an intense series of tests on model bridges and field-tests on in-service bridges (Puckett et al, 1988). The prototype bridge testing consisted of 1/8th scale model bridges subjected to ultimate strength and dynamic load tests. They tested both orthotropically and isotropically reinforced bridges. The prototype bridges had varying reinforcing ratios. For the ultimate load tests, all specimens experienced 71 kN (16 kip) wheel loads with a 0.3 impact factor (Puckett et al, 2008). After testing had finished, it was concluded that traditional (orthotropic) designed decks had a factor of safety of 16. The factor of safety for the empirical (isotropic) decks with 0.2% reinforcing ratio was less, but still significant at 12. In addition, cracking was deemed acceptable for all bridges tested for service level loads. From these ultimate load tests, "the authors concluded that conventional reinforcing was excessive due to its high factors of safety," (Puckett et al, 1988). Dynamic tests carried out on the same specimens yielded similar results. For the same reinforcing ratio for the isotropic deck (0.2%), the design provided a factor of safety of 4 when the bridge was subjected to a minimum of 100,000 cycles at a frequency of 1-5 hz, a 71 kN (16 kip) wheel load, and an impact factor of 0.3 (Puckett et al, 1988).

The Ministry conducted tests on in-service bridges to ensure that their conclusion – that traditional designs were excessive – was valid. They placed strain gages on these bridges and measured the strains of the specimens when placed under varying loads. Data collected from these tests confirmed the conclusion that minimal reinforcing ratio was adequate and that AASHTO designed decks were very over conservative (Puckett et al, 1988).

To further confirm their results, the Ontario Ministry of Transportation built a full-scale test bridge – the Conestogo Bridge – of their own, with several different reinforcing patterns in different panels of the deck. During testing, the deck panel with 0.2% isotropic reinforcing showed excessive cracking upon removal of the load, but the 0.3% isotropic panel showed only fine cracks that disappeared upon removal of the load, leading to the ultimate recommendation of 0.3% for bottom reinforcing. In addition, this full-scale test confirmed the presence of arching action (Puckett et al, 1988).

The empirical deck design can be applied to nearly the entire deck, save the overhangs and in the negative moment regions over the piers where traditional – flexural – design will control. In addition, this design method may not be applied to decks constructed using stay-in-place forms. Other design requirements are listed in section 9.7.2.4 of the 2012 AASHTO LRFD Bridge Design Specifications. The reinforcing must also meet certain criteria, codified in section 9.7.2.5 of 2012 AASHTO LRFD specifications. Most important to note in this section is the requirement of minimum reinforcing steel. The steel provided in the bottom layers must be at least 580 mm^2/m (0.27 in^2/ft) and the reinforcing steel provided in the top layers must be at least 387 mm^2/m (0.18 in^2/ft). These minimum areas of reinforcing correspond 0.3% and 0.2% reinforcing ratios, respectively (AASHTO LRFD 2012, 9.7.2.5).

As the leaders in this design method, it is important to consider the Canadian Bridge Design Code literature on empirical deck design, and to see how it compares to the specifications in the US.The 2014 Canadian Code specifies that the deck must be supported on girders of steel or concrete. The method does not apply to overhang locations nor longitudinal negative moments, and requirements for slabs pertaining to flexure and axial loads, shear and torsion, crack control, and deformation shall be waived.

The Canadian code requirements are very similar to AASHTO; the main difference being that the deck may be cast in place, partially precast, or fully precast in Canada, whereas AASHTO only allows for empirical to be used with cast-in-place decks. General requirements for empirical bridge deck design can be found in the 2014 Canadian Highway Bridge Design Code Specifications section 8.18.4.1. Provisions for cast-in-place, partial-depth precast decks, and full-depth precast decks can be found in in sections 8.18.4.2, 8.18.4.3 and 8.18.4.4, respectively.

III. CASE STUDY

The bridge that has been used as a case study in this work is the bridge on Southbound US 65 over the Des Moines River South Overflow, southeast of Des Moines, Iowa. Figure 1 shows an image of the bridge. It is a 142 m x 12.2 m (466'-0 x 40'-0) pre-tensioned, pre-stressed concrete girder bridge with an average annual daily traffic count of 25,000 vehicles per day. This structure meets all criteria to be able to use empirical design as required in the 2012 AASHTO LRFD Design Specifications. The bridge has a 203.2 mm (8") cast-in-place, water cured deck with 63.5 mm (2 ½") top cover and 25.4 mm (1") bottom cover. The deck is supported on, and composite with, five girders that are spaced at 2.75 m (9'-0 ½") on center with a 1.1 m (3'-7") overhang on either side. Concrete diaphragms are provided at the piers and abutments, satisfying the requirement for cross frames or diaphragms at lines of support.

The deck was originally designed using traditional methods; with a final total of 65,677 kg (144,793 lbs) of epoxy-coated reinforcing steel in the deck. This 65,677 kg is comprised of (1) the reinforcement in the overhangs, as well as the longitudinal reinforcement over the piers which remained constant regardless of deck design method, as flexure does control the design in those areas and (2) the primary reinforcement which is subject to change with the empirical (and semi-empirical methods). The quantity of steel that remains unchanged, regardless of deck design method (ie, part 1 of the total deck steel), amounts to 17,113 kg (37,727 lbs). The primary deck reinforcing (part 2 of the total deck reinforcement) totals 48,564 kg (107,066 lbs).

Fig.1: US 65 Des Moines River South Overflow Bridge

Empirical Design of Deck:

As discussed previously, the use of the empirical deck design requires no structural analysis. As long as the structure meets the requirements presented in the 2012 AASHTO LRFD code, the deck may be reinforced with minimal steel. This case study structure was redesigned using #4 bars at 305 mm (12") on center for the top mat (both longitudinal and transverse direction) and #5 bars at 305 mm (12") on center for the bottom mat (both longitudinal and transverse directions). This spacing and bar size corresponds to 430 mm^2/m width (0.20 in^2/ft) for the #4's and 667 mm^2/m width (0.31 in^2/ft) for the #5's, as well as provides more than the minimum requirement of 387 mm^2/m width (0.18 in^2/ft) and 580 mm^2/m width (0.27 in^2/ft), respectively (the minimum area of steel to be provided coming from the recommended reinforcing ratios of 0.002 for the top mat and 0.003 for the bottom mat (AASHTO LRFD 2012)). After implementing the empirical design, the primary reinforcement was reduced to 28,656 kg (63,176 lbs). Adding this to the 17,113 kg (37,727 lbs) of unchanged reinforcing in the overhangs and negative moment areas gives a total weight of steel equal to 45,769 kg (100,903 lbs). Using the empirical approach for this structure reduces the total amount of required reinforcing steel by approximately 30%.

Semi-Empirical Design of Deck:

Similar to empirical design, semi-empirical designs aim to reduce the total amount of reinforcing steel; however, it is a hybrid combination of traditional and empirical design methods. This design was completed following the example from the Texas Department of Transportation (Holt, 2014). In their semi-empirical deck design, they leave the top mat of reinforcing as is, and reduce the bottom mat to that resulting from an empirical design. The reinforcement ratio used in the bottom mat is 0.003, which corresponds to 580 mm^2/m width (0.27 in^2/ft). The necessary reinforcement is supplied by #5 bars at 305 mm (12") on center, which corresponds to an area of steel provided of 667 mm^2/m width (0.31 in^2/ft). In the structure being analyzed, the top mat of reinforcing tallies 41,395 kg (91,261 lbs). The bottom mat, before redesign is 24,282 kg (53,532 lbs) of steel. After implementation of the #5's at 305 mm (12") in the bottom mat, the total steel in the bottom of the deck was reduced to 17,468 kg (38,511 lbs). This results in a new total steel weight of 58,864 kg (129,772 lbs), which is approximately a 10% reduction from the traditional design of 65,677 kg (144,793 lbs) of deck reinforcing.

IV. PARAMETERS INFLUENCING LIFE CYCLE COST ANALYSIS

The life cycle cost of any item is dependent upon several variables. To accurately capture the cost over the life of a bridge deck, the following parameters must be considered: service life of each alternative being evaluated, material and other initial costs, operation and maintenance costs, user delay costs, and interest rate. Detailed descriptions of each parameter are described below.

Service Life of Epoxy-Coated Reinforced and Stainless Steel Reinforced Decks:

To determine a theoretical service life and time to corrosion and corrosion related damages, researchers make use of physical tests and observation on bridge deck samples. Obtaining deck samples was beyond the scope of this study; therefore, average time to corrosion initiation and corrosion related damage for both epoxy coated reinforcement and stainless steel reinforcement have been obtained through a literature review and sources in the bridge maintenance group at the Iowa Department of Transportation (IDOT).

Browning et al conducted a study on a series of corrosion protection systems, including stainless steel. From their extrapolated data, they determined the time to first repair on a stainless steel deck would be 200 or more years (Browning et al, 2012). A study published by the FHWA estimates that stainless steel decks can provide at least 100 years if service life (McDonald et al, 1998). A report from the Michigan Department of Transportation cites an estimated average of 100 years for the service life of stainless steel decks (Kahl, 2012). Similarly, a study conducted by Hartt et al stated that solid stainless steel bars would enable a bridge deck to reach its expected design life of 75-100 years (Hartt et al, 2009). Based on the estimates provided by this literature review, it was determined that stainless steel decks will provide an average of 100 years of corrosion-free service life with a standard deviation of approximately 25 years.

Likewise, a literature review revealed several service life estimations for epoxy-coated decks. One study showed that decks with epoxy-coated rebar could provide 100 years of service life with sufficient concrete cover (50.8 mm; 2”) and water-to-cement ratios (0.45) (Brown et al., 2007). Another report estimates 40 years of life (Bergmann, 2007). A report out of Virginia estimates than an epoxy-coated reinforced deck could reach a 75-year design life (Dillard et al, 2000). Fanous et al conducted studies on deck samples in Iowa, and predicted a service life of 50 years (Fanous and Wu, 2000). A fifth report examined bridge deck surface ratings from several in service bridge decks; they estimated a service life of 70 years for epoxy-coated reinforcing steel decks (Boatman, 2010). A life cycle cost analysis conducted by researchers out of Michigan used a value of 60 years for epoxy-coated decks (Kahl, 2012). A study conducted by the FHWA estimates that without quality control of the deck, it could last 54 years (McDonald et al, 1998). The results of this literature review estimate an average service life of approximately 65 years with a standard deviation of 20 years; however, bridge maintenance officials at the IDOT predict that bridges in Iowa with epoxy coated bars can offer an 85 year service life; a value they believe to best approximate the life of Iowa epoxy-coated bridge decks when exposed to current weather conditions and deicing practices (Carter, M., Neubauer, S., Port., G. Personal Communication, April, 2015). 85 years is within one standard deviation of the calculated average; therefore, an 85-year service life for epoxy-coated rebar decks is implemented in the “base case” study as well as in the sensitivity study.

Material and Initial Costs:

Three material costs were incorporated into this analysis: concrete, epoxy-coated reinforcing, and stainless steel. Because the structure being analyzed is already constructed, the base bid prices used here were taken directly from the winning contractor bid (BidExpress, 2015). The winning bid quoted prices for concrete, epoxy-coated rebar, and stainless steel at $549.34/m^3 ($420.00/cy), $1.98/kg ($0.90/lb), and $6.56/kg ($2.98/lb), respectively; the bid price for the stainless reinforcing steel coming from its use as barrier rail reinforcement. Using these item prices, initial costs for each alternative could be estimated. See Table 1 for the calculation of total costs per item and total initial cost for each design option.

TABLE I: MATERIAL AND INITIAL COSTS FOR EACH DESIGN ALTERNATIVE

	Traditionally Designed with ECR[a]	**Traditionally Designed with SS[b]**	**Empirically Designed with ECR**	**Empirically Designed with SS**	**Semi-Emp. Designed with ECR**	**Semi-Emp. Designed with SS**
Required Deck Concrete (m^3)[c]	469.9	469.9	469.9	469.9	469.9	469.9
Bid Price for Concrete ($/m^3)	$549.34	$549.34	$549.34	$549.34	$549.34	$549.34
Cost of Concrete	$258,132.00	$258,132.00	$258,132.00	$258,132.00	$258,132.00	$258,132.00
Required Deck Reinforcing Steel (kg)	65,677	65,677	45,769	45,769	58,864	58,864
Bid Price for Rebar ($/kg)	$1.98	$6.56	$1.98	$6.56	$1.98	$6.56
Cost of Rebar	$130,313.70	$431,483.14	$90,812.70	$300,690.94	$116,794.80	$386,720.56
Remainder of Bridge Costs	$6,447,190.00	$6,447,190.00	$6,447,190.00	$6,447,190.00	$6,447,190.00	$6,447,190.00
Total Initial Cost	$6,835,635.70	$7,136,805.14	$6,796,134.70	$7,006,012.94	$6,822,116.80	$7,092,042.56

[a]ECR: Epoxy-Coated Reinforcing

[b]SS: Stainless Steel

[c]Based on gross area of the deck.

Operation and Maintenance Costs:

Bridge decks are under constant wear and tear throughout their lives. As such, they require constant attention to ensure they reach the maximum useful and practical life. Many of these activities related to deck preservation are listed in Table 2. Table 2 presents assumptions made in this work for the time of occurrence, the extent of the event, and the unit cost of each event. It should be noted that most – excluding PCC deck patching – of these activities are related to corrosion. Because of this, all of these operation and maintenance costs, except for PCC deck patching at 60 years, are only applied to the three epoxy-coated deck design alternatives because it has been assumed that no corrosion-related deterioration occurs in the stainless steel bridge deck. The PCC patching activity is applied to every design at 60 years.

Although the "extent of event" provided is an estimate, it is consistent with a report by the FHWA which states that at approximately 40 years, 10% delamination will have occurred, and approximately 7 years later, another 10% delamination (McDonald et al, 1998). Similarly, for "age at occurrence" predictions, Dillard et al reports that overlays may take place at 5 year intervals starting at year 45 (Dillard et al, 2000). A second report states that epoxy-coated reinforced decks can expect a time of approximately 50 years to the first repair, which is in close proximity to the time presented in Table 2 (Browning et al, 2012).

TABLE II: SCHEDULE OF EVENTS AND COSTS FOR LIFE CYCLE COST ANALYSIS FOR EPOXY-COATED REINFORCED DECKS

Event	Age at Occurrence	Extent of Event (as a percentage of deck area)[b]	Unit Cost in 2015 Dollars [$/m^2($/SF)]
Original Construction	0	--	--
1st PCC Overlay	40	100%	$484.38 ($45.00)
Epoxy Injection	50	10%	$65.58 ($6.00)
Epoxy Injection	55	10%	$65.58 ($6.00)
PCC Deck Patching	60	7%	$645.83 ($60.00)
2nd PCC Overlay	70	100%	$538.20 ($50.00)
Epoxy Injection	78	10%	$65.58 ($6.00)
Replace Bridge	85	100%	$538.20 ($50.00)[a]

Reference: Carter, M., Neubauer, S., Port., G.(Personal Communication, April, 2015)

[a]Includes $102.26/m^2 ($9.50/SF) cost for rebar

[b]Values shown are for a traditionally designed and semi-empirical deck. Due to slight increases in cracking, empirically designed deck LCCA uses 11% and 8% for epoxy injection and PCC deck patching, respectively.

User Delay Costs:

The structure being analyzed is a US Highway bridge, and as such, any construction that occurs must be completed with at least one lane staying operational, and much of the work is completed at night. Because there is no detour, user delay costs must be considered; however, the Iowa Department of Transportation does not calculate user delay costs directly, as many of the parameters required to accurately estimate these costs are not readily available and take a great amount of time to calculate. Instead, the Iowa DOT replaces these costs with any additional costs that may be accrued due to the need for temporary construction equipment such as temporary barrier rails, flood lights, etc. In addition to temporary construction equipment costs, traffic control costs (signs, flaggers, etc.) must be included in these delay costs. Utilizing BidExpress (BidX), staging costs and traffic control costs were estimated to be $19,440 per repair. These costs are included when a replacement or repair takes place, and are reflected in the costs presented in Table 2.

Interest Rate:

Two "types" of interest rates exist: nominal and real. The nominal rate refers to the interest rate neglecting the effect of inflation. Real interest rate is the rate that "the lender or investor receives after inflation is factored in," (Cussen). So, the real interest rate is, essentially, the inflation rate subtracted from the nominal rate. The real interest rate is used in this study. In Appendix C of Circular A-94 developed by the White House Office of Management and Budget, there is a 30-year outlook on real interest rates (White House Office of Management and Budget, 2015). The authors state that for an analysis period greater than 30 years, the 30-year rate shall be used. An average of the 30-year rate was calculated using the provided rates from 1979 to 2015 – said average was calculated to be 3.95% (White House Office of Management and Budget, 2015).

Base Life Cycle Cost Analysis Results and Discussion

The design alternatives were first analyzed using the specific values listed in the previous section. This is the "base" case and represents engineering-based best estimates. A sensitivity study was then conducted using a Monte Carlo simulation consisting of 1,000,000 trials. Variables considered in the Monte Carlo analysis and the respective results are given following presentation of the "base" case results. The life cycle costs have been analyzed using annual costs which are presented in Table 3. The initial cost for a traditional deck is roughly $170,000 lower than that of a stainless steel empirical deck; however, the annual cost for the former ends up being $4,400 more per year over the course of its service life. In addition to the empirically designed bridge deck with stainless steel being less in annual costs than traditional with epoxy-coated rebar, Table 3 also shows the same result for a semi-empirically designed deck. A semi-empirically designed deck is approximately $256,000 more initially, but annually, it is approximately $900 less than the traditionally designed deck with epoxy-coated reinforcement.

TABLE III: ANNUAL COST FOR EACH ALTERNATIVE OVER THEIR SERVICE LIVES (BASE CASE)

Design Alternative	Annual Cost	Cost Difference from Lowest Cost Alternative ($/yr)
Traditionally Designed with SS	$288,193.73	$5,275.90
Traditionally Designed with ECR	$287,292.81	$4,374.99
Empirically Designed with SS	$282,917.83	--
Empirically Designed with ECR	$284,278.26	$1,360.43
Semi-Empirically Designed with SS	$286,388.09	$3,470.27
Semi-Empirically Designed with ECR	$285,321.83	$2,404.01

V. SENSITIVITY STUDY

Because the parameters that influence the life cycle cost of a bridge deck are known to fluctuate, a sensitivity study was completed using a Monte Carlo simulation. This analysis procedure enables the user to input several parameters – the values of which are known to be variable – which can be defined by different distributions. A Monte Carlo analysis then takes those input distributions and parameters and outputs a range of possible outcomes as well as how likely those possibilities are to occur (Palisade Corporation 2015).

Varying Parameters:

The variables that were studied here include interest rate; bid price of concrete, epoxy-coated steel, and stainless steel; and service life of an epoxy-coating reinforced deck and service life of a stainless steel reinforced deck. All parameters were defined to have normal distributions with average, μ, and standard deviation, σ. Table 4 lists these variables along with the respective average and standard deviation estimated from available information presented earlier. For the sensitivity study, all other values not listed in this section were kept constant. The "age at occurrence" values (listed in Table 2) were assumed to be a function of service life, rather than constant values. Because service life for the alternatives changes, the time to repairs and replacement change as well

TABLE IV: MONTE CARLO SIMULATION DISTRIBUTION INPUTS

Parameter	Average Value, μ	Standard Deviation, σ
Interest Rate	3.95%[a]	1.61%[a]
Concrete Bid Price	$549.34/m^3 ($420.00/cy)[b]	$65.40/m^3 ($50/cy)[b]
Epoxy-Coated Rebar Bid Price	$1.98/kg ($0.90/lb)[b]	$0.33/kg ($0.15/lb)[b]
Stainless Steel Bid Price	$6.56/kg ($2.98/lb)[b]	$0.77/kg ($0.35/lb)[b]
Service Life Epoxy-Coating Reinforced Deck	85 years[c]	20 years[c]
Service Life Stainless Steel Reinforced Deck	100 years[d]	25 years[d]

[a]White House Office of Management and Budget Circular A-94, Appendix C. (2015).

[b]BidExpress 2015

[c](Bergmann and Schnell, 2001), (Kahl, 2012), (McDonald et al, 1998), (Boatman, 2010), (Fanous and Wu, 2000), (Dillard et al, 2000), (Brown et al, 2007), (Carter, M., Neubauer, S., Port., G.(Personal Communication, April, 2015))

[d](Browning et al, 2011), (Bergman and Schnell, 2007), (Kahl, 2012), (Hart et al, 2009), (McDonald et al, 1998)

Simulation Results and Discussion:

The number of iterations chosen for the Monte Carlo-based sensitivity study was 1,000,000. Comparisons were made for empirically designed stainless steel decks versus traditionally designed epoxy-coated reinforcing decks and for semi-empirically designed stainless steel decks versus traditionally designed epoxy-coated reinforcing decks. Figure 2 shows the probability of having at least a "break even" situation (i.e., $0 in annual cost savings) for both empirical stainless steel deck versus traditional epoxy-coated reinforcing deck as well as semi-empirical stainless steel deck versus traditional epoxy-coated reinforcing deck. Of the 1,000,000 cases generated by the simulation, there is a 67% probability that the empirical stainless steel reinforced deck will be at least equal to or less expensive than the traditionally designed epoxy-coated option. Conversely, there is a 33% probability that the stainless steel option will be at least equal to or more expensive than the epoxy-coated alternative. The probabilities for the semi-empirical stainless steel deck are 50.2% and 49.8%, respectively.

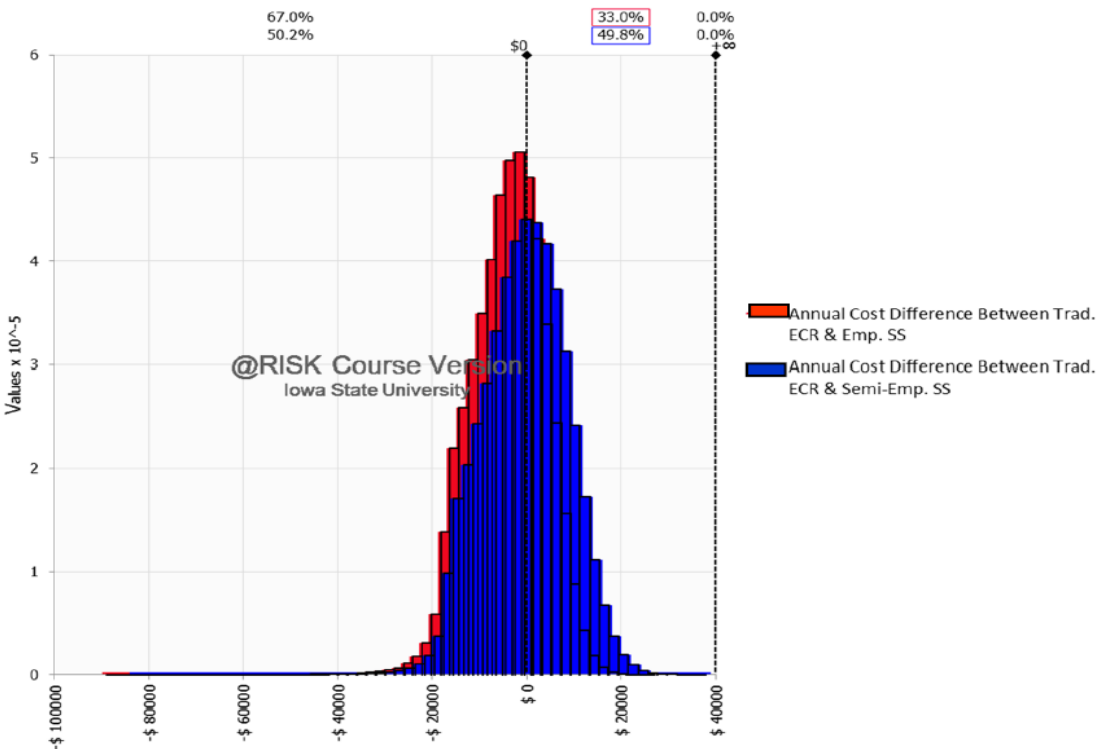

Fig.2: Annual Cost Difference Between Empirical SS and Traditional ECR & Semi-Empirical and Traditional ECR - Probability of at Least $0 in Cost Savings Shown

In addition to the break-even case, Table 5 provides a few notable probability scenarios. Of the cases generated, there will be an X% probability that the cost savings will be that dollar amount shown in the table or better for each stainless steel alternative. Provided are 25%, 50% (the median cost savings) and 75% probability scenarios A negative value indicates that the stainless steel options are less expensive annually when compared to the traditional epoxy-coated option, while a positive dollar amount indicates that the epoxy-coated alternative is more cost-beneficial. For example, of the cases generated by the simulation, there is a 50% probability that the empirical stainless steel option will have at least $3,378 per year in cost savings over the traditional option and a 50% probability that the semi-empirical deck will be up to $36 more per year than the traditional epoxy-coated option. The "base" case presented earlier falls within the 50% and 75% probability ranges, which indicates that it is more likely to produce cost savings in favor of the empirical stainless steel option than in favor of the traditionally designed epoxy-coated reinforcing deck.

TABLE V: ANNUAL COST SAVINGS FOR EMPIRICAL SS AND SEMI-EMPIRICAL SS DECKS COMPARED TO TRADITIONAL ECR DECKS

Probability Scenario	Cost Savings of Empirical SS over Traditional ECR[a]	Cost Savings of Semi-Empirical SS over Traditional ECR[a]
25%	-$9,126/yr	-$6,337/yr
50%	-$3,378/yr	$36/yr
75%	$1,690/yr	$5,840/yr

[a]A positive value indicates that the traditionally designed deck with ECR is less expensive.

Table 6 provides various statistics for both comparisons: maximum and minimum annual cost savings, mean, median, and mode cost savings, and the standard deviation. Again, a negative dollar amount indicates that the stainless steel option is less expensive annually than the traditional epoxy-coated deck. Though the maximum cost savings for each stainless steel alternative is significant, these dollar amounts should not be expected, as they fall on the very tail end of the normal distribution curves; this follows for the minimum cost savings as well.

Table 6 shows that the mean cost savings for both stainless steel decks favors the less expensive annual costs when compared to the epoxy-coated option with values of $3,833 and $310 for the empirical and semi-empirical, respectively; however, it should be noted that there is a very large standard deviation from those means. This is because of the large fluctuation in the input parameters. Figures 3 and 4 depict the sensitivity of each outcome to the inputs; inputs are ordered from top to bottom from highest impact on output to the lowest. It is evident that both design alternative comparisons are highly sensitive to the interest rate. Similarly, both comparisons are very dependent upon the service life of the epoxy-coated, traditionally designed deck.

The presented data clearly indicates that there is indeed a chance for both the semi-empirical and empirical stainless steel options to be more expensive than a traditional deck; however, especially in the case of the empirical stainless steel deck, that likelihood is relatively low – less than 35% probability. For this reason, reinforcing a deck in an empirical fashion in conjunction with stainless steel is an option that should be seriously considered.

TABLE VI: STATISTICS FOR ANNUAL COSTS OF TRADITIONAL ECR DECKS COMPARED TO BOTH SEMI-EMPIRICAL AND EMPIRICAL SS DECKS

Statistic	Empirical SS Deck Compared to Traditional ECR Deck[a]	Semi-Empirical SS Deck Compared to Traditional ECR Deck[a]
Maximum Annual Cost Savings	-$89,025.05/yr	-$83,247.82/yr
Minimum Annual Cost Savings	$27,040.80/yr	$38,009.54/yr
Mean Annual Cost Savings	-$3,833.39/yr	-$309.77/yr
Mode of Annual Cost Savings	-$2,092.39/yr	$2,120.10/yr
Median of Annual Cost Savings	-$3,378.16/yr	-$35.72/yr
Standard Deviation of Annual Cost Savings	$7,717.90	$8,693.86

[a]A positive value indicates that the traditionally designed deck with ECR is less expensive.

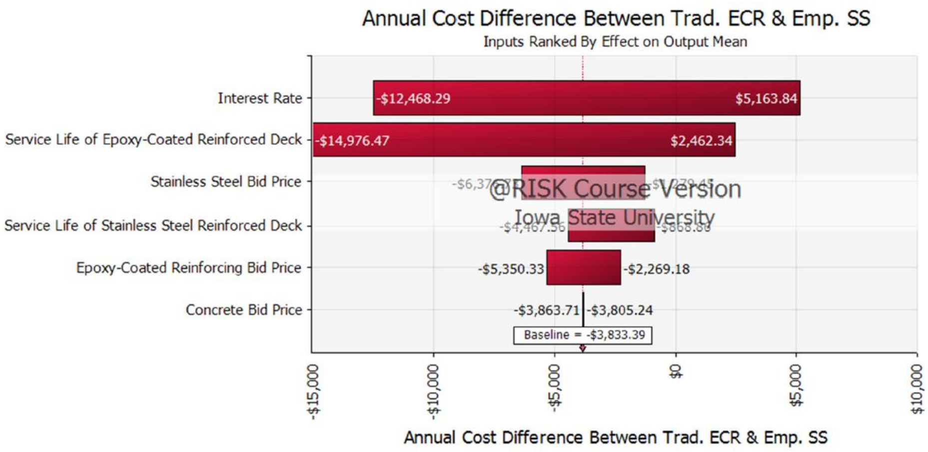

Fig.3: Sensitivity of Output to All Inputs - Empirical Stainless Steel vs. Traditional Epoxy-Coated Reinforcing

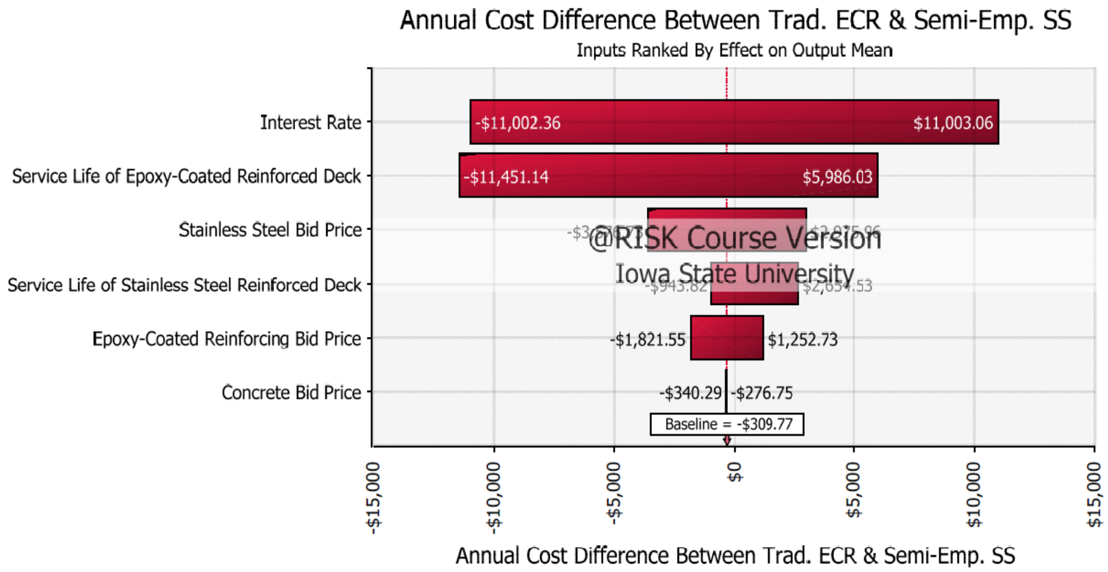

Fig.4: Sensitivity of Output to All Inputs - Semi-Empirical Stainless Steel vs. Traditional Epoxy-Coated Reinforcing

VI. SUMMARY AND CONCLUSIONS

The empirical design method (also known as the Ontario Method) is a method for designing bridge decks that came about in the last 30-35 years. The goal with implementing this design method is to significantly reduce the amount of reinforcing steel in a deck by utilizing known arching action. This minimal amount of reinforcing is provided by placing two mats of isotropic reinforcing in both the top and the bottom of the deck (Mulert, 2001). The 2012 AASHTO LRFD Bridge Specifications allow the use of this method, and specify many requirements that must be met to be able to use this often termed “no analysis” method; however, there is hesitation to use it because of unfamiliarity and, more importantly, it is thought to increase longitudinal cracking – and hence, a shorter service life – when compared to decks designed using the equivalent strip method (traditional method) (Veen, 2008).

To increase the durability of decks by way of corrosion resistance, stainless steel can be implemented. Currently, this is not a very popular option as stainless steel is initially much more expensive than its epoxy-coated counterpart. Yet to be implemented is a combination of stainless steel reinforcing with empirical design. The goal of this is to reduce the amount of reinforcing needed, and therefore, initial cost, while still maintaining a corrosion and relatively crack free deck.

This study looked at life cycle cost analyses of six different deck design options (hypothetically implemented in a bridge southeast of Des Moines, Iowa): traditionally designed decks with (1) stainless steel and (2) epoxy-coated rebar, empirically designed decks with (3) stainless steel and (4) epoxy-coated rebar, and semi-empirically design decks with (5) stainless steel and (6) epoxy-coated reinforcing. A sensitivity study was conducted using a Monte Carlo simulation. The presented data clearly indicated that there is indeed a chance for both the semi-empirical and empirical stainless steel options to be more expensive than a traditional deck; however, especially in the case of the empirical stainless steel deck, that likelihood is relatively low – less than 35% probability.

Stainless steel reinforcing has, undoubtedly, a much higher initial cost than epoxy-coated rebar; however, this investigation into the economic benefit of combining stainless steel and empirical deck design shows that implementing said combination is an option that should be seriously considered in future bridge projects.

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REFERENCES

[1] American Association of State Highway and Transportation Officials (AASHTO). *LRFD Bridge Design Specifications.* 6th Edition, 2012.

[2] Balvanyos, L., Lave, L., Thompson, N.G., Yunovich, M. (2001). *Corrosion Cost and Preventive Strategies in the United States, Appendix D – Highway Bridges*. Federal Highway Administration. FHWA-RD-01-157.

[3] Battaglia, I. (2012). *Evaluation of the Ontario Empirical Method for Design of Composite Bridge Deck Slab Reinforcement.* Wisconsin Department of Transportation. Report No. WI-03-12.

[4] Beal, D. (1983). *Reinforcement for Concrete Bridge Decks*. (Report No. FHWA/NY/RR-83/105).

[5] Bergmann, M.P. and Schnell, R.E. (2001). *Improving Tomorrow's Infrastructure: Extending the Life of Concrete Structures with Solid Stainless Steel Reinforcing Bar.* Print.

[6] BidExpress. (2015).

[7] Boatman, B. (2010). *Epoxy Coated Rebar Bridge Decks: Expected Service Life*. Michigan Department of Transportation.

[8] Brooks, E. (1998). *Concrete Bridge Becks with Isotropic Reinforcing.* (Report No. OR-EF-99-06).

[9] Brown, M., Sprinkel, M., Weyers, R.E., Williamson, G. (2007). *Bridge Deck Service Life Prediction and Costs.* Report No. VTRC 08-CR4.

[10] Browning, J., Darwin, D., Locke, C., O'Reilly, M. (2012). *Evaluation of Multiple Corrosion Protection Systems for Reinforced Concrete Bridge Decks*. Report No. 100.

[11] Canadian Standards Association. *Canadian Highway Bridge Design Code.* 6th Edition. 2014.

[12] Cussen, M. "Undestanding Interest Rates: Nominal, Real and Effective." <www.investopedia.com>

[13] Google Maps. (2015).

[14] Dillard, J., Mokarem, D., Pyc, W., Sprinkel, M., Weyers, R.E., Weyers, R.M., Zemajtis, J. (2000). *Field Performance of Epoxy-Coated Reinforcing Steel in Virginia Bridge Decks*. Virginia Transportation Research Council. VTRC 00-R16.

[15] Fang, K.I. (1985). *Behavior of Ontario-Type Bridge Deck on Steel Girders*. Ph.D. dissertation University of Texas, Austin, TX.

[16] Hartt, W., Himiob, R., Marino, F., Paredes, M., Powers, R., Simmons, R., Virmani, Y., Yu, H. (2009). *Corrosion Resistant Alloys for Reinforced Concrete.* Report No. HRT-09-020.

[17] Hewitt, B.E., and Batchelor, B. (1975). "Punching Shear Strength of Restraint Slabs." *Journal of the Structural Division*. American Society of Civil Engineers, New York, NY, Vol. 109. No. ST9, pp 1837-1853.

[18] Holowka, M., Dorton, R.A., Csagoly, P.F. (1980). *Punching Shear Strength of Restrained Circular Slabs*. Ministry of Transportation and Communication, Downsview, Ontario, Canada.

[19] Holt, J. (2014). *Reducing Steel in Bridge Decks*. Concrete Bridge Views, Issue 78.

[20] Fanous, F., Wu, H. (2000). *Service Life or Iowa Bridge Decks Reinforced with Epoxy-Coated Bars*. Mid-Continent Transportation Symposium Proceedings.

[21] Iowa Depatment of Transportation (2015). *Standard Specifications for Highway and Bridge Construction.* Materials IM 452.

[22] Iowa Department of Transportation. (2015). Structure Inventory and Inspection Management System (SIIMS).

[23] Jansson, P. (2008). *Performance Evaluation of Isotropic Bridge Decks.* Report 91-F-0170.

[24] Kahl, S. (2012). *Stainless and Stainless-Clad Reinforcement for Highway Bridge Use*. Michigan Department of Transportation. Report No. RC-1560.

[25] McDonald, D.B., Pfeifer, D.W., Sherman, M.R. (1998). *Corrosion Evaluation of Epoxy-Coated, Metallic Clad, and Solid Metallic Reinforcing Bars in Concrete.* Report No. FHWA-RD-98-153.

[26] Mulert, B. (2001). *Empirical Deck Design in Concrete Bridge Decks*.

[27] Puckett, J.A, Lohrer, J.D., Naiknavare, R.D. (1988). *Evaluation of Bridge Deck Utilizing Ontario Bridge Deck Design Method.*

[28] Microsoft Excel. (2014).

[29] Palisade Corporation. (2014). @Risk for Excel Version 6.3.0.

[30] Palisade Corporation. (2015). “Monte Carlo Simulation.” <http://www.palisade.com/risk/monte_carlo_simulation.asp>

[31] Smith, J.L. and Virmani, Y.P. (1996). *Performance of Epoxy Coated Rebars in Bridge Decks*. Federal Highway Administration. Report FHWA-RD-96-092.

[32] Uppena, T. (1999). *Empirical Deck Design in Concrete Bridge Decks*.

[33] Veen, N. (2008). *The Suitability of Iowa’s Empirical Bridge Decks*.

[34] Wenzlick, J. (2007). *Evaluation of Stainless Steel Reinforcement in Bridge Decks*. Missouri Department of Transportation. Report No. RI00-027

[35] The White House Office of Management and Budget, OMB Circular No. A-94, Appendix C. *Discount Rates for Cost-Effectiveness, Lease Purchase, and Related Analyses.* Washington, D.C., December 2014.