Optimal Approach for Addressing Reinforcement Corrosion for Bridge Decks with Higher Strength Corrosion Resistant Reinforcement to Improve the Crack Control Performance and Provide Improved Durability

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Abstract

This paper addresses the design of bridge decks with various types of reinforcement. It is intended to show that cast-in-place decks can be cost effectively designed with corrosion resistant high strength reinforcement if current options in the AASHTO LRFD BDS are utilized. It will be shown that the analytical “strip” method can be an effective design method if the crack control criteria are revised to truly reflect the behavior of bridge decks, where the dominant cracking pattern is transverse to traffic. Thus, crack control equations should not be applied to the transverse reinforcement, implying longitudinal cracks. It will also be shown that the “Empirical Method” is a powerful tool with a strong track record for over 40 years, with more than two dozen states adopting it.

The results of this study show that it is possible to take advantage of high strength ASTM A1035 corrosion resistant steel despite its higher initial cost than ASTM A615 steel, if one considers life-cycle cost analysis. Using A615 Galvabars, with continuous machine galvanizing, per ASTM A1094, results in improvements over epoxy and hot-dip galvanized bars, but not the level of corrosion protection as A1035. Galvabars can be bent in the fabrication shop after galvanizing and even at the construction site.

Finally, adjustments to the Strip Method and to the Empirical Method are proposed to allow for wider use of these methods for high strength corrosion resistant steel, while still maintaining adequate serviceability, strength and achieving superior durability.

Introduction

This paper discusses various methods of design of cast-in-place bridge decks to satisfy strength and serviceability requirements. Two design methods are included in accordance with AASHTO LRFD BDS. The detailed analysis method, called the Strip Method, and the Empirical Method. A bridge example is used to discuss the various provisions and the results of the design. It utilizes the same features as a bridge deck designed and constructed by the Illinois Department of Transportation, District #7. This pilot bridge is located on Interstate 70 over Spring Creek Rd in Effingham County. This IDOT demonstration project provided a side-by-side comparison of Epoxy Coated Reinforcement (ECR), ASTM A775, to the more expensive ASTM A1035 (AASHTO M334) Low Carbon Chromium reinforcing rebar (LCCR). This bridge pilot was the first use of ASTM A1035 LCCR on an IDOT project. The bridge is a dual, 3 span wide flange steel girders superstructure on semi-integral abutments and multi-column piers.

The goal of the Illinois Bridge Engineers in applying ASTM A1035 at f_y = 80 ksi /550 MPa was to determine if a modest increase in the strength from 60 to 80 ksi would provide a more efficient and less costly application as compared to their typically used ECR designed bridge decks at f_y = 60 ksi/413 MPa. Since this bridge allows a side-by-side comparison, actual installed cost was collected and compared to determine the final cost premium for a 100 year service life deck when using LCCR versus ECR.

In this paper, the authors will expand the study to include ASTM A615 Grade 60 ksi/413 MPa galvanized bar using machine applied galvanizing process specified by ASTM A1094, in which the zinc is applied in a continuous process before the bars are fabricated for use in the deck. This process produces higher quality corrosion resistance than hot-dip galvanizing and also epoxy coating. Such a product is called Galvabar.

In addition, high strength ASTM A615 Grade 80 ksi/550 MPa Galvabars are used. For these bars, the ultimate strength is 100 ksi. The Power formula for the full stress strain diagram of the steel is used in the analysis, as soon to be allowed in the 2025 AASHTO approval process. Further, the high strength highly corrosion resistant ASTM A1035 bars will be used in the analysis, similar to what was used in the IDOT demonstration project. ASTM A1035 is marketed by CMC as "comparable" to stainless steel rebar (SSR) when the required durability and service life of a bridge deck is 100 years to the first major repair. For that type of steel, the full stress-strain relationship of the steel is used. It results in stress ranging from 100 ksi (690 Mpa) to 150 ksi(1034 Mpa). Thus, in this study, there will not be a capping of yield strength at 80 ksi/550 MPa as conservatively assumed in the IDOT project. It would allow for comparison of quantities and costs using the actual material properties if they are allowed to be used on actual deck projects

The Illinois Demonstration Project

The Illinois Center for Transportation, Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign published a study[(1)], that was the result of a comprehensive literature review focusing on the corrosion resistance of several alternatives to standard black reinforcing bars. The outcomes of the review were then used as the basis for developing a framework to assist bridge design engineers and IDOT officials in selecting the optimal type of reinforcement for a given bridge deck considering applicable factors influencing corrosion performance over the design service life of the structure. A short case study was used to highlight the effectiveness of the framework for two example bridges in drastically different parts of Illinois with different design parameters as well as overall implications for the most effective and economical corrosion resistant solutions. “More specifically, the framework considered six reinforcing bar types, which included standard black bars (as the control), epoxy-coated, stainless-steel, A767 galvanized, A1094 galvanized, and A1035 bars, based on the study of performance of reinforcement for Oklahoma Bridges performed by Darwin et. al.(2) While plenty of existing literature demonstrates that epoxy-coated bars have been an effective solution in combating corrosion over the past several decades, recent studies have also increased motivation and encouragement for using other bar types. According to IDOT, “enhanced corrosion performance often comes with a significant trade-off of higher initial material costs, which, for some applications, may restrict the use of higher performance corrosion-resistant bar options. Because of its relatively lower chromium content (when compared to stainless-steel bars) and its lower material cost, the use of A1035 reinforcement is likely to serve as a commendable intermediate option between standard black bars and stainless steel.” According to the study reported in this paper, ASTM A1035 CS steel appears to have higher merit than credited by IDOT.

It is expected—as shown in Figure 1, that the higher strength steel can facilitate reduced labor, repair, and construction costs when accounted for in a life-cycle analysis.

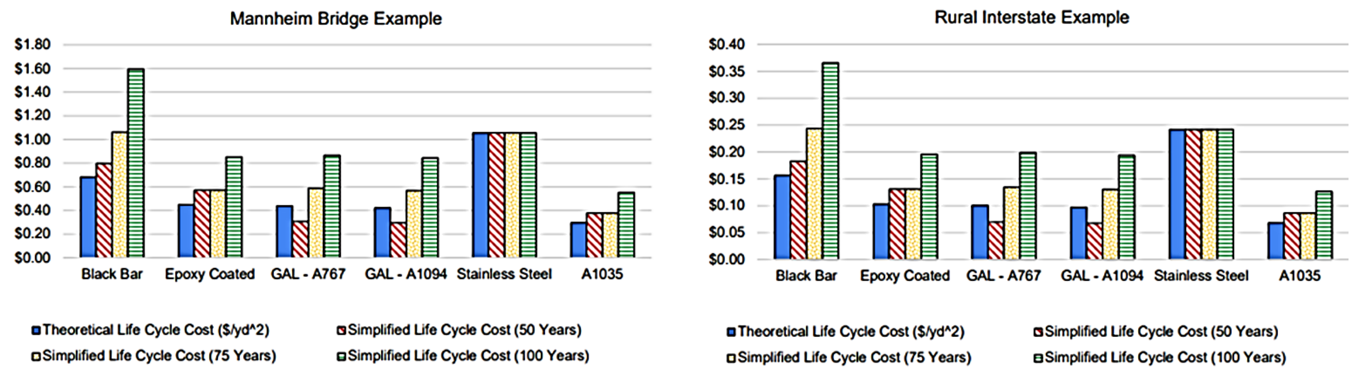

Figure 1: Graphical Representation of Life-cycle Cost Estimates (Source: ICT[1])

The study by IDOT concluded, "when considering bridge-specific design parameters, A1035 bars exhibited the lowest overall life-cycle costs in all but one case: galvanized bars using the 50-year simplified method. This is largely due to their relatively higher nominal yield strength of the A1035 bars and the corresponding effect on reducing labor and repair costs.

As a result of the above study, Illinois DOT decided to initiate a pilot project to evaluate the installed cost differences between an epoxy (A775) designed at 60 ksi/413MPa yield strength and A1035 at 80 ksi /550MPa yield strength for its two bridge decks on the Interstate 70 over Spring Creek Rd in Effingham County, see Figure 2.

Figure 2: Aerial View of I-70 over Spring Creek Rd.

(a) General View (b) Close Up

Figure 3: Illinois Demonstration Bridge using ASTM A1035 Steel

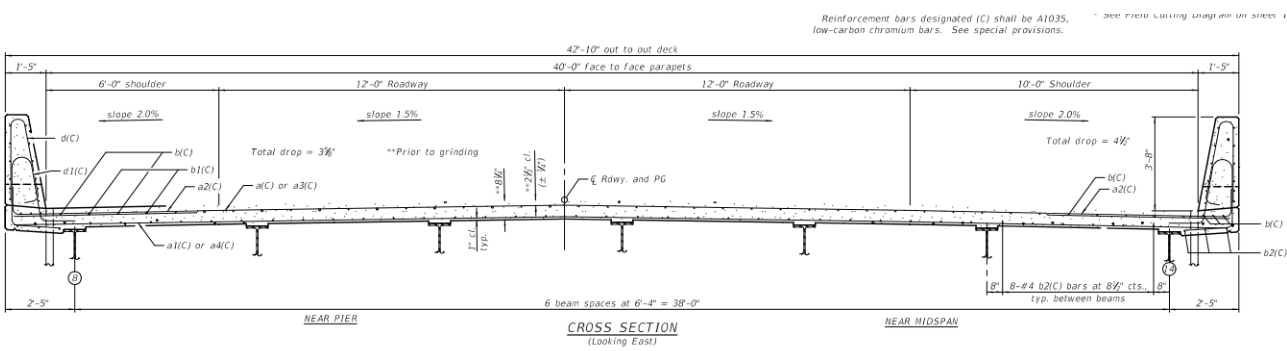

Figure 4: Cross Section of the Effingham Bridge Deck which uses Epoxy bars in WB Bridge and A1035 bars in EB Bridge

Results of the IDOT Study

WB Bridge Deck Reinforcement Details: #5 Epoxy Coated Rebars have been used

Yield strength of Epoxy Coated Rebars	$f_y =$	60.00	ksi /413 MPa
In Transverse Direction			
Spacing of #5 top bars	$st_{top} =$	6.50	in
Spacing of #5 bottom bars	$st_{bot} =$	10.00	in
Area of top reinforcement	$Ast_{top} =$	0.572	in^2/ft
Area of bottom reinforcement	$Ast_{bot} =$	0.372	in^2/ft
In Longitudinal Direction			
Spacing of #5 top bars	$sl_{top} =$	12.00	in
Spacing of #5 bottom bars	$sl_{bot} =$	12.00	in
Area of top reinforcement	$Asl_{top} =$	0.310	in^2/ft
Area of bottom reinforcement	$Asl_{bot} =$	0.310	in^2/ft

EB Bridge Deck Reinforcement Details: #4 A1035 CS Rebars have been used

Yield strength of A1035 East Bound Bridge Deck Reinforcement	$f_y =$	80.00	$ksi\ /550MPa$
In Transverse Direction			
Spacing of #4 top bars	$st_{top} =$	5.00	in
Spacing of #4 bottom bars	$st_{bot} =$	8.50	in
Area of top reinforcement	$Ast_{top} =$	0.480	in^2/ft
Area of bottom reinforcement	$Ast_{bot} =$	0.282	in^2/ft
In Longitudinal Direction			
Spacing of #4 top bars	$sl_{top} =$	8.00	in
Spacing of #4 bottom bars	$sl_{bot} =$	8.50	in
Area of top reinforcement	$Asl_{top} =$	0.300	in^2/ft
Area of bottom reinforcement	$Asl_{bot} =$	0.282	in^2/ft

During the redesign of the bridge deck reinforcement with epoxy-coated rebars, it was noted that IDOT refers to the standard chart in Figure 3.2.1-1 for #5 transverse bars in the deck. For the bottom longitudinal reinforcement, 67% of the reinforcement specified for the bottom transverse design is applied here, aligning with AASHTO's current recommendations.

In the case of A1035 bars in EB Bridge, the spacing of the transverse reinforcement appears to be dictated by the crack control criteria, consistent with findings from designs using different rebar properties and cover, which are discussed in detail in Section 5.

Design Comparison Results:
West Bound Bridge – Epoxy Coated

- Conventional Design at Fy = 60 ksi / 413MPa
- Used in Superstructure & Approach Slabs
- Total Rebar Weight = 95,520 lbs / 43,327 Kg
- Area of Deck and Approach = 7620 sf / 708 m^2
- Weight of Rebar/sf = 12.5 lbs/sf of deck
- Total number of bars = 3,771

East Bound Bridge – A1035-CS

- ASTM A1035 at Fy = 80 ksi/550MPa
- Used in Superstructure & Approach
- Total Rebar Weight = 66,710 lbs/30,259 Kg
- Area of Deck and Approach = 7620 sf/708 m^2
- Weight of Rebar/sf = 8.8 lbs/sf of deck

Total number of bars = 3,848
Note that more bars are used with the A1035-CS design, but the total rebar weight is approximately 30% less compared to the Epoxy Coated design.

Cost Comparison:
West Bound Bridge – Epoxy Coated

- Low bid unit price = US$1.82/lb
- Total Cost of Superstructure & approach Slab rebar = US$173,846

East Bound Bridge – ASTM A1035-CS

- Low bid unit price = US$2.90/lb
- Approximately 59% unit price premium
- Total cost of Superstructure & Approach Slab = US$193,613
    - Additional Cost of rebar - US$19,613
    - Approximately a 11% cost premium

Use of A1035-CS designed at 80 ksi/550MPa reduced the weight of rebar by approximately 30% which partially offset the premium cost of US$20K for the A1035 material. For relatively nominal additional cost of approximately 11%, the East Bound deck has an improved service life of potentially > 100 years [(2)]. If project required stainless steel (SS) rather than A1035-CS, estimated costs would have been approximately 40% greater.

The procurement and construction of the EB bridge using A1035-CS, finished on-time with bridge opening in July of 2023.

Design According to AASHTO Specifications with Proposed Revisions

Description of the Bridge Being Used for Analysis

The superstructure and reinforcement details of the existing West Bound (WB) and East Bound (EB) bridge decks are obtained from the IDOT Effingham Bridge Plan and are listed below.

Superstructure Details

Unit Weight of Concrete	$\Upsilon =$	150.00	pcf
Compressive strength of concrete deck	$fc' =$	4.00	ksi
Skew Angle of the bridge (10°20'57")	$\alpha =$	10.35	$degree$
Weight of Future Wearing Surface	$w_{FWS} =$	50.00	psf
Spans Lengths are:	$39'\,7" - 37'\,9" - 39'\,7"$		
Width of bridge between inside faces of barriers	$Ws =$	40.00	ft
Thickness of deck slab	$t_s =$	8.25	in
Clear cover to the top rebar for Galvabars	$c_t =$	2.00	in
Clear cover to the top rebar for A1035 bars	c_t	1.5	in
Clear cover to the bottom Rebar	$c_b =$	1.00	in
No. of steel girders	$N =$	7.00	
Spacing between the CL of girders (6' 4")	$S =$	6.33	ft
Flange Width of the Girder (W24x81 beam)	$f =$	6.33	ft
Length of Overhang (2' 5")		2.42	ft

Design According to AASHTO Equivalent Strip Method

Using the IDOT Bridge Superstructure as a sample, the design of deck reinforcements has been carried out for various types of rebar. Design according to the AASHTO Equivalent Strip Method is followed in this section. One exception made in the analysis given here is the elimination of the requirement that the main transverse reinforcement spacing be limited to 1.5* total slab thickness. The second limit of 18 in. is enforced for both top and bottom bars in both directions. This is consistent with the limit stated for the Empirical Design Method. It allows for considerable relief, especially for the design example used here where the span is relatively small, and the thickness is relatively large.

Design according to the Equivalent Strip Method will be followed by design according to the Empirical Design Method in a separate section. The authors of this paper believe that the Empirical Method should be promoted and further expanded through research and demonstrations, as it has already been widely and successfully implemented for Grade 60 steel. Minor modifications to the Empirical Method will be proposed in the relevant section and in the Appendix.

Design According to AASHTO Equivalent Strip Method using A1035 Grade 100 CS bars

AASHTO M334 (ASTM A1035) bars are considered Category C for minimum cover. For A1035 CS bars, which fall under AASHTO M334 category C, the clear cover on the top of the deck can be taken as 1.5 inches, lower than the 2.0 inch cover for Category B (epoxy coated and galvanized), as per AASHTO Table 5.10.1-1.

Further assumptions made in the analysis below relate to the stress strain properties of ASTM A1035 bars. The AASHTO Concrete committee is in the process, as of the writing of this paper, of updating the flexural design provisions to include more accurate representations of the stress-strain diagrams for both concrete and steel. As a result, ASTM A1035 bars will be allowed to be used for the full curve of the stress-strain as long as the stresses and elongations meet the minimum values specified by ASTM. Therefore, for the calculations below for this type of steel, the yield strength is assumed = 100 ksi and the ultimate strength is assumed = 150 ksi. This is a deviation from the assumption made by IDOT for their demonstration bridge where the yield strength = the ultimate strength = 80 ksi.

Loads

Dead Loads

Area of overhang	$A_{overhang} =$	269.92	in^2
Area of parapet	$A_{parapet} =$	547.43	in^2
Length of Overhang (2'-5")	$L_{over} =$	2.42	ft
Self-weight of deck per sq. ft	$w_{deck} =$	103.13	psf
Weight of overhang over 2'-5"	$w_{overhang} =$	116.34	psf
Weight of parapet per linear foot of the span	$W_{parapet} =$	570.24	plf
Weight of future wearing surface	$w_{fws} =$	50.00	psf

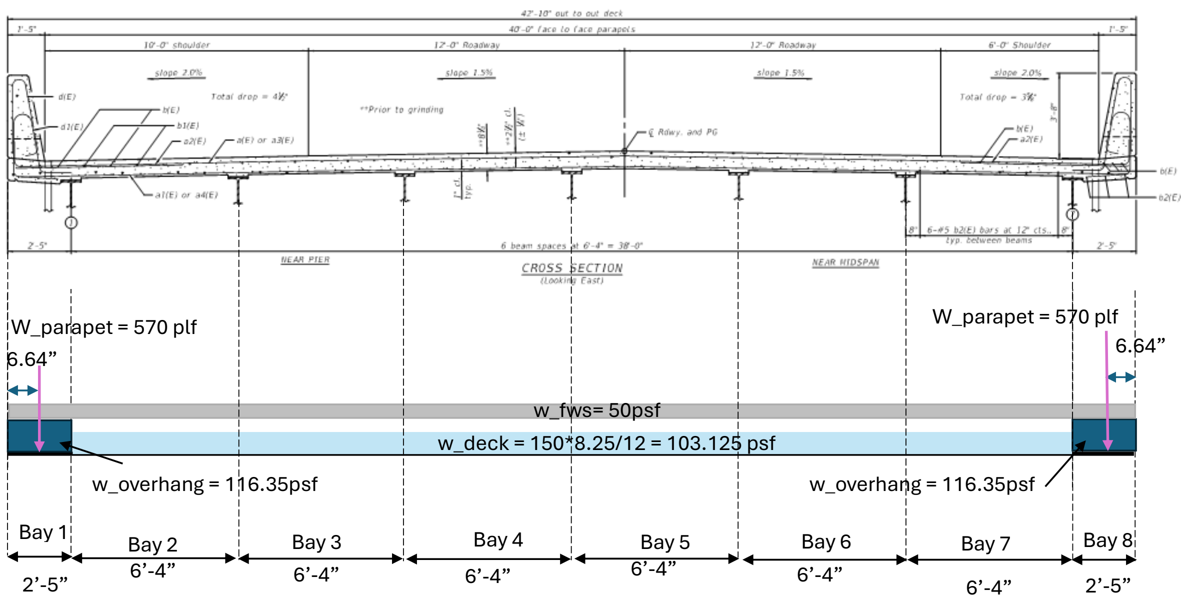

Figure 5: Dead Loads Applied to the Deck

Live Loads and Design Moment

The live load moments were obtained using AASHTO Appendix A4 table. The obtained ultimate moments are tabulated below:

Total Positive Service Moment	$Mser_{pos} =$	5.30	$Kip - ft/ft$
Total Negative Service Moment	$Mser_{neg} =$	-4.85	$Kip - ft/ft$
Total Positive Ultimate Moment	$Mu_{pos} =$	9.11	$Kip - ft/ft$
Total Negative Ultimate Moment	$Mu_{neg} =$	-9.44	$Kip - ft/ft$

Design of Bottom Transverse Reinforcement using A1035 Grade 100 CS Bars

Flexural Demand Requirement

The total slab thickness for structural design is assumed 0.25 in. less than the thickness used for weight calculations. This allows for site grinding to achieve the required roadway profile.

Effective depth of steel $$de_{bot} = \left(t_s - 0.25in - c_b - \frac{d_b}{2}\right) = 6.75\ in$$

Using strain compatibility spreadsheet and assuming the yield strength = 100 ksi and the ultimate strength = 150 ksi.

Ultimate Moment Demand	$Mu\_pos =$	9.113	$Kip - ft/ft$
Required Area of rebar	$As\_bot\_flex =$	$\mathbf{0.125}$	in^2/ft
Factored Moment Capacity	$\Phi Mn =$	$\mathbf{9.200}$	$Kip - ft/ft$

Maximum Allowed Spacing for Concrete Deck Slab [AASHTO 5.10.3.2]

The above table shows that #4 at 19.2 in. would give the required 0.125 in^2/ft.
However, as per AASHTO 5.10.3.2, spacing of transverse rebars should be lesser than 1.5 times the thickness of member or 18 in., i.e.,

Maximum Allowable Spacing $S_{max} = \min(1.5 * (ts - 0.25), 18in) = 12\ in.$

It is clear that enforcing this clause for transverse bars unnecessarily restricts the spacing limit which hinders the potential benefit of using high strength bars. Thus, a proposed modification to discard 1.5 times the thickness of the member has been suggested.

Hence, **#4 bars @ 18 in. is allowed**.

Crack Control Requirement [AASHTO 5.6.7]

Section Modulus $Sm = 128\ in^3$

Modulus of Rupture [AASHTO 5.4.2.6]

$$fr = 0.24\sqrt{fc'} = 0.48\ ksi$$

Maximum Tensile Stress $tmax = \frac{Mu_{pos}}{Sm} = 0.854\ ksi$

Here, tmax = 0.854 ksi > 0.8 · fr = 0.384 ksi . So, crack control checks need to be performed. For this, various areas of steel per ft was considered and iterated to get the accurate crack controlled spacing requirement.

Modulus of Elasticity of steel $E_s = 29000\ ksi$

Modulus of Elasticity of concrete [AASHTO 5.4.2.4-1]

$$E_C = 12000\left(\frac{\Upsilon c}{1000}\right)^2 (fc')^{0.33} = 4266.23\ ksi$$

Modular Ratio $n = \frac{E_s}{E_C} = 6.79$

Web Width $b_w = 12\ in$

Overall thickness of the section $h = ts - 0.25\ in = 8\ in$

Sectional depth of the steel $de_{bot} = 6.75\ in$

Concrete cover from extreme fiber to the center of the extreme bar layer

$$dc = h - de_{bot} = 1.25\ in$$

Assumed area of steel required to satisfy crack control criteria

$$As_{bot} = 0.282\ in^2/\text{ft}$$

Unfactored Service Moment $Ms_{pos} = 5.3\ Kip - ft/ft$

Exposure Factor (Class 2) $\Upsilon_e = 0.75$

Reinforcement Ratio $\rho = \frac{As_{bot}}{b_w de_{bot}}$ =0.003

$n\rho = 0.024$

$k = (2 * n\rho + (n\rho)^2)^{0.5} - n\rho = 0.195$

$j = 1 - \frac{k}{3} = 0.935$

Tensile stress in steel $f_{ss} = \min\left(\frac{M_s}{A_s * j * d_e}, 0.6 f_y\right) = 35.687\ ksi$

$\beta_s = 1 + \frac{d_c}{0.7 * (h - d_c)} = 1.265$

Required crack control reinforcement spacing

$$\boldsymbol{s_{req}} = \frac{700 * \Upsilon_e}{\beta_s * f_{ss}} - 2 * d_c = \mathbf{9.134\ in}$$

Provided Spacing $\boldsymbol{s_{provided}} = \mathbf{8.5\ in}$ (OK)

Thus, even though the flexural demand permits us to use #4 @ 19 in, the proposed modification to AASHTO 5.10.3.2, requires changing the reinforcement to #4 @ 18 in. The spacing is further reduced to meet the crack control criteria, resulting in final value of #4 @ 8.5 in.

Check for minimum reinforcement [AASHTO 5.6.3.3]

For this check, the ultimate tensile strength of the reinforcement, $f_u = 150\ ksi$, is utilized.

Ratio of specified minimum yield strength to ultimate tensile strength of reinforcements

$$\Upsilon_3 = \frac{f_y}{f_u} = 0.67$$

Flexural Cracking Variability Factor $\Upsilon_1 = 1.6$

Section Modulus $S_{nc} = 128\ in^3$

Cracking Moment $M_{cr} = \Upsilon_3 * \Upsilon_1 * f_r * S_{nc} = 5.46\ kip - ft$

Check if $\varphi M_n (= 9.2\ kip - ft/ft) > \min(Mcr, 1.33 * Mu) = 5.46\ Kip - ft/ft$

Thus the #4 @ 8.5 in. is adequate.

Design of Bottom Longitudinal Reinforcement [AASHTO 9.7.3.2]

If the depth of slab is greater than 7 in and the provided cover is not less than 1 in, the area of the bottom longitudinal steel when primary reinforcement is perpendicular to traffic should be at least:

Minimum area of bottom longitudinal steel $Asl_{bot} = \min\left(\frac{220}{\sqrt{S}}, 0.67 * As_{bot}\right)$

Provided area of bottom transverse steel $Ast_{bot}, provided = 0.2 * \frac{12}{8.5} = 0.282\ in^2/ft$

And, minimum area of bottom distribution steel required per ft becomes:

$$Asl_{bot,flex} = 0.67 * 0.282 = \ 0.189\ in^2/ft$$

The equivalent spacing is: $Sl_{bot,flex} = 12.69\ in$

And the proposed modification AASHTO 5.10.3.2 allows the spacing up to 18 in. Thus, <u>#4 @ 12.5 in</u> is used.

Design of Top Transverse Reinforcement using A1035 Grade 100 CS *Bars*

Flexural Demand Requirement

Effective depth of steel $de_{top} = \left(t_s - 0.25in - c_t - \frac{d_b}{2}\right) = \left(8.25 - 0.25 - 1.5 - \frac{0.5}{2}\right) = 6.25\ in$

Ultimate Moment Demand	$Mult\_neg\ =$	9.442	$Kip - ft/ft$
Required Area of rebar	$As\_top\_req\ =$	$\mathbf{0.144}$	in^2
Factored Moment Capacity	$\Phi Mn\ =$	$\mathbf{9.500}$	$Kip - ft/ft$

Maximum Allowed Spacing for Concrete Deck Slab [AASHTO 5.10.3.2]

According to **flexural demand**, **#4 bars @ 16.5 in** bottom rebars would be sufficient. As per the proposed modification to AASHTO 5.10.3.2, the maximum spacing is 18”. Thus, #4 bars @ 16.5” is OK.

Crack Control Requirement

Section Modulus $Sm\ =\ 128\ in^3$

Modulus of Rupture [AASHTO 5.4.2.6]

$$fr = 0.24\sqrt{fc'} = 0.48\ ksi$$

Maximum Tensile Stress $tmax = \frac{Mu_{neg}}{Sm} = 0.885\ ksi$

Here, tmax = 0.885 ksi > 0.8 · fr = 0.384 ksi . So, crack control checks need to be performed. For this, various area of steel per feet was considered and iterated to get the accurate crack controlled spacing requirement.

Modulus of Elasticity of steel $E_s = 29000\ ksi$ /199,948MPa

Modulus of Elasticity of concrete [AASHTO 5.4.2.4-1]

$E_C = 12000\left(\frac{\Upsilon c}{1000}\right)^2 (fc')^{0.33} = 4266.23\ ksi$/29415.62 MPa

Modular Ratio $n = \frac{E_s}{E_C} = 6.79$

Web Width $b_w = 12\ in$

Overall thickness of the section $h = ts - 0.25\ in = 8\ in$

Sectional depth of the steel $de_{top} = 6.25\ in$

Concrete cover from extreme fiber to the center of the extreme bar layer

$dc = h - de_{top} = 1.75\ in$

Through iteration the area of steel required to satisfy crack control criteria

$As_{top} = 0.32\ in^2$/ft

This corresponds to #4 @7.5 in.

The following calculations will demonstrate that this value is acceptable.

Unfactored Service Moment $Ms_{neg} = 4.85\ Kip - ft/ft$

Exposure Factor (Class 2) $\Upsilon_e = 0.75$

Reinforcement Ratio : $\rho = \frac{As_{top}}{b_w de_{top}} = 0.004$

$n\rho = 0.029$

$k = (2 * n\rho + (n\rho)^2)^{0.5} - n\rho = 0.214$

$j = 1 - \frac{k}{3} = 0.929$

Tensile stress in steel $f_{ss} = \min\left(\frac{M_s}{A_s * j * d_e}, 0.6f_y\right) = 31.305\ ksi$

$\beta_s = 1 + \frac{d_c}{0.7*(h-d_c)} = 1.4$

The corresponding spacing is :

$s = \frac{700*\Upsilon_e}{\beta_s * f_{ss}} - 2 * d_c = \mathbf{8.5\ in}$

Which is larger than the provided spacing. Thus the #4 @7.5 in. spacing is satisfactory.

Thus, even though the flexural demand permitted us to use the spacing of 16.5 in, the crack control requirement limits the spacing of the bottom rebars to 7.5 in.

Check for minimum reinforcement [AASHTO 5.7.3.3]

Ultimate Tensile of the reinforcement $f_u = 150\ ksi$

Ratio of specified minimum yield strength to ultimate tensile strength of reinforcements

$$\Upsilon_3 = \frac{f_y}{f_u} = 0.67$$

Flexural Cracking Variability Factor $\Upsilon_1 = 1.6$

Section Modulus $S_{nc} = 128\ in^3$

Cracking Moment $M_{cr} = \Upsilon_3 * \Upsilon_1 * f_r * S_{nc} = 5.461\ Kip - ft$

Check if $\varphi M_n (= 9.5\ kip - ft/ft) > \min(Mcr, 1.33 * Mu) = 5.461\ kip - ft/ft$

Yes, OK.

Design of Top Longitudinal Reinforcement [AASHTO 5.10.6]

From AASHTO temperature and shrinkage requirement commentary:

Temperature and shrinkage control reinforcement area should be greater than or equal to 18% of least cross section of concrete distributed equally around the perimeter of the component.

Thus, Area of steel required as per temperature and shrinkage criteria:

$$A_{sh} = 0.0018 * ts * \frac{12in}{ft} * 60/fy = 0.1068\ in^2/ft$$

AASHTO 5.10.6 allows for the reduction in area of shrinkage and temperature reinforcement in accordance to the ratio of yield strength of Grade 60 to the high strength reinforcement. However, a limit of 75 ksi has been placed by AASHTO. The authors propose to revise the limit to 100 ksi.

Also, this steel area should be in between 0.11 and 0.6.

$$Final\ A_{sh} = 0.11 \le A_{sh} \le 0.6 = 0.11\ in^2/ft$$

Therefore, the amount of steel required for top longitudinal reinforcement is:

$$Asl_{top} = \frac{Final\ A_{sh}}{2} = 0.055\ in^2/ft$$

The maximum spacing of the temperature and shrinkage reinforcement must be smaller of 3.0 times the deck thickness or 18.0 inches.

$$Smax_{sh} = \min(3 * ts, 18\ in) = 18\ in$$

Thus, #4 bars @ 18 inches would suffice. $Asl_{top}, provided = 0.2 * \frac{12}{18} = 0.133\ in^2/ft$

Design According to AASHTO Equivalent Strip Method using A1035 Grade 100 CS Bars, Waiving Crack Control Requirement

For design, if crack control criteria are exempted, the restrictions to reduce the spacing results in fewer bars for high strength steel. The calculation steps would be the same as given above for flexural strength and min. reinforcement. The AASHTO specified maximum spacing of 12 inches for deck slab has been ignored in this design example. A limit of 18 inches, as that for shrinkage and temperature reinforcements, and in the Empirical Design for all bars, has been adopted. This results in the following spacing of bars:

Bottom Transvers Bars: #4 $bars$ @ 18", $Ast_{bot} = 0.133\ in^2/ft$

Bottom Longitudinal Bars: #4 $bars$ @ 18", $Asl_{bot} = 0.133\ in^2/ft$

Top Transverse Bars: #4 $bars$ @ 16.5", $Ast_{top} = 0.145\ in^2/ft$

Top Longitudinal Bars: #4 $bars$ @ 18", $Ast_{top} = 0.133\ in^2/ft$

Likewise, similar calculation approaches were carried out for Galvanized Grade 80 bars with 2 in. top cover, Grade 60 stainless steel bars with 1.5" top cover and Grade 60 black bars with 2.5 in. cover. The deck reinforcement demand using GFRP bars is also carried out in Section 6 of this document. The amount of top and bottom transverse as well as longitudinal reinforcement required are summarized in Table 3 at the end of Section 6.

Design According to the AASHTO Empirical Method

The AASHTO LRFD BDS Empirical Design Method is covered in Article 9.7.2. The commentary in Article C9.7.2.1 states that, "extensive research into the behavior of concrete deck slabs discovered that the primary structural action by which these slabs resist concentrated wheel loads is not flexure, as traditionally believed, but a complex internal membrane stress state referred to as internal arching. This action is made possible by the cracking of the concrete in the positive moment region…and the resulting upward shift of the neutral axis in that portion of the slab…" as the load approaches the ultimate capacity of the span being considered. Such arching action requires that the bottom transverse reinforcement be continuous over the relatively stiff supporting stringers, or at least well anchored at the ends of each span between girder lines to ensure adequate capacity of the "tension tie" represented by the bottom transverse reinforcement. This is the reason for specifying a minimum overhang length at the exterior girder line of 5 times the slab thickness.

The Empirical Design Method was originally developed for the Ministry of Transportation of Ontario (MTO) by Hewitt and deV Batchelor (1975) and Csagoly (1979), and further verified by Holowka et al (1980), Fang (1985), and Fang et al (1990). These studies have demonstrated a factor of safety when designing with the Empirical Method of about 8.0. It is slightly lower than the factor of safety of 10.0 when designing with classical flexural design, but much higher than normally accepted levels of safety.

It has been shown that the Empirical Method works effectively for bridge decks or typical configurations, despite the lower reinforcement demands of the theoretical Strip Method. Accordingly, over twenty state DOTs have adopted this method as an acceptable alternative to the traditional flexural strength design.

The word "Empirical" in the name gives a false impression that it is less rational than the detailed flexural design method. It is the authors' opinion that the method can be employed with shorter overhangs than required if it can be shown that the bottom transverse reinforcement is adequately anchored at the ends. Adequate anchorage of the transverse reinforcement will need to be studied, reinforcement detailing developed, and full scale tested.

The criteria for the application of AASHTO Empirical Design were checked for the example bridge (Effingham County Bridge), see Table 2. It is shown that all conditions for application of the Empirical Design are met. One issue needing further investigation and revision is that the Empirical Design, while allowing for use of rebars with yield strength as much as 100 ksi, the provisions for steel areas do not seem to allow for credit for the higher strength steel than Grade 60. A proposal is given here when applying the Method to ASTM A1035 to take advantage of the higher strength.

Table 1: Criteria Checks for the use of AASHTO Empirical Method of Deck Design

Criteria	Satisfied (Yes/No)
i. Cross-frames or diaphragms are used throughout the cross-section at lines of support	Yes
ii. For cross-section involving torsionally stiff units, such as individual separated box beams, intermediate diaphragms required….	Not applicable to the example here
iii. The supporting components are made of steel and/or concrete	Yes
iv. The deck is fully cast-in-place and water cured	Yes
v. The deck is of uniform depth, except for haunches at girder flanges and other local thickening	Yes
vi. The ratio of effective length to design depth does not exceed 18.0 and is not less than 6.0	Yes (6 ≤S = 6.29' ≤ 18)
vii. Core Depth of the slab is not less than 4"	Yes (4.5" > 4")
viii. The effective length, as specified in Article 9.7.2.3, does not exceed 13.5 ft	Yes (6.29' < 13.5')
ix. The minimum depth of slab is not less than 7", excluding a sacrificial wearing surface where applicable	Yes (8" > 7")
x. There is an overhang beyond the centerline of the outside girder at least 5.0 times the depth of the slab; this condition is satisfied if the overhang is at least 3.0 times the depth of the slab and a structurally continuous concrete barrier is made composite with the overhang	Yes (29/8 = 3.625 >3, (with composite continuous barrier)
xi. The specified 28-day strength of the deck concrete is not less than 4ksi	Yes

xii. The deck is made composite with the supporting structural components	Yes

Hence, since empirical method can be used without violating any criteria for this example bridge, using the empirical method:

Design with Grade 60 steel

The minimum reinforcement for each bottom layer is:

$$As_{bot}, min = 0.27\ in^2/ft$$

With #5 bars, required spacing of bottom bars: $S_{bot}, max = 0.31 * \frac{12}{0.27} = 13.78\ in$ < 18 in

Hence, #5 bars @ 13.5 inches should be sufficient at the bottom.

And the minimum reinforcement for each top layer is:

$$As_{top}, min = 0.18\ in^2/ft$$

With #5 bars, required spacing of top bars: $S_{top}, max = \max\left(0.31 * \frac{12}{0.18} = 20.67\ in, 18\ in\right) = 18\ in$

Hence, #5 bars @ 18 inches should be sufficient at the top.

Design with Grade 100 CS Steel

A deviation in this paper when applying the Empirical Method has been introduced in order to take advantage of bars of higher strength than 60 ksi. According to the current Article 9.7.2.5 of AASHTO, no credit is given to higher strength steel than Grade 60. The reason given is that these values were determined experimentally for crack control.

In this paper, the authors propose to use a modification as follows:

Area of bottom reinforcement in each direction = $0.27*60/(0.9f_y)$ in^2/ft for steel grades of 75 ksi or higher, and 0.27 in^2/ft for lower steel grades.

Area of top reinforcement in each direction = $0.18*60/(0.9f_y)$ in^2/ft for steel grades of 75 ksi or higher, and 0.18 in^2/ft for lower steel grades.

The reason for the 0.9 factor being applied to the higher steel grades is that these grades do not have a well-defined yield. Using a 0.9 factor would place the steel stress at the straight-line zone defined by linear elasticity.

It is recognized that the proposed modification results in reduced areas of steel compared to Grade 60 steel and thus reduced stiffness and increased deflection of the slab. However, strict adherence to the service limit states provisions of Article 9.5.2 would ensure adequate stiffness of cracking reinforced concrete members. The provisions require that deflection be limited to span/800, span/1000 or span/1200 depending on the extent of pedestrian traffic. Obviously, more theoretical and experimental work would need to be performed on high strength bars to ensure adequacy for strength and serviceability. The research would need to include more specific guidance on the requirements of anchorage of the bottom transverse bars to ensure adequate formation of the arching effect.

The minimum reinforcement for each bottom layer is:

$$As_{bot}, min = 0.27 * \frac{60}{0.9fy} = 0.27 * \frac{60}{0.9*100} = 0.18\ in^2/ft$$

With #4 bars, required spacing of bottom bars: $S_{bot}, max = 0.2 * \frac{12}{0.18} = 13.33\ in$ < 18 in

Hence, #4 bars @ 13 inches should be sufficient at the bottom.

And the minimum reinforcement for each top layer is:

$$As_{top}, min = 0.18\ \frac{60}{0.9fy} = 0.18 * \frac{60}{0.9*100} = 0.12\ in^2/ft$$

With #4 bars, required spacing of top bars: $S_{top}, max = \max\left(0.2 * \frac{12}{0.12} = 20\ in, 18\ in\right) = 18\ in$
Hence, #4 bars @ 18 inches should be sufficient at the top as per the proposed modification for empirical method.

A comparison table is presented below for Gr. 60 Epoxy Coated bars and Gr. 100 CS Steel showing the reinforcement spacing requirement resulting from various methods: the IDOT Bridge Design Manual, AASHTO Equivalent Strip Method and Empirical Method.

Table 2: Comparison of Deck Design Results from Various Methods of Analyses

Method Used	Bottom Transverse Bars		Bottom Longitudinal Bars		Top Transverse Bars		Top Longitudinal Bars		Total Weight (lb/yd^2)	Rel-ative values in %
	Size and spacing	Weight (lb/yd^2)	Size and spacing	Weight (lb/yd^2)	Size and spacing	Weight (lb/yd^2)	Size and spacing	Weight (lb/yd^2)		
Effingham County WB Bridge Deck with Gr. 60 Epoxy Coated Rebar										
IDOT Results	#5 @ 10"	11.4	#5 @ 12"	9.5	#5 @ 6.5"	17.5	#5 @ 12"	9.5	47.9	100.0
Strip method with Crack Control	#5 @ 10.5"	10.9	#5 @ 15.5"	7.4	#5 @ 8"	14.2	#4 @ 18"	4.1	36.5	76.2
Strip method without Crack Control	#5 @ 11.5"	9.9	#5 @ 17"	6.7	#5 @ 9.5"	12.0	#4 @ 18"	4.1	32.7	68.2
Empirical Method	#5 @ 13.5"	8.4	#5 @ 13.5"	8.4	#5 @ 18"	6.3	#4 @ 18"	4.1	27.3	57.0
Effingham County EB Bridge Deck with Gr. 100 CS Steel										
IDOT Results	#4 @ 8.5"	8.6	#4 @ 8.5"	8.6	#4 @ 5"	14.7	#4 @ 8"	14.2	46.2	96.5
Strip method	#4 @ 8.5"	8.6	#4 @ 12.5"	5.9	#4 @ 7.5"	9.8	#4 @ 18"	4.1	28.4	59.3

with Crack Control										
Strip method without Crack Control	#4 @ 18"	4.1	#4 @ 18"	4.1	#4 @ 16.5"	4.5	#4 @ 18"	4.1	16.7	34.9
Empirical Method*	#4 @ 13"	5.7	#4 @ 13"	5.7	#4 @ 18"	4.1	#4 @ 18"	4.1	19.5	40.7

*With proposed modification to take advantage of higher strength of the steel.

Design Using GFRP Bars According to AASHTO LRFD Guide for GFRP-Reinforced Concrete (AASHTO GFRP-2B)

The calculation of bridge deck reinforcement requirement when GFRP bars are used has been shown below. The moment demands are obtained from the calculations shown In Section 5.1.1.1.

Clear Cover

Clear Cover to the top rebar as per AASHTO GFRP-2B, Table 6.6.2.4-1 is:

$$ct = \max(1.5 * db, 1in)$$

Irrespective of the bar size, in order to be consistent in our comparison to the corrosion resistant steel reinforcements, the same clear cover as that for Stainless steel and A1035 bars has been provided here.

Hence, $ct = 1.5\ in,\ cb = 1\ in$

Reinforcement Properties

Tensile Modulus of Elasticity of GFRP Bars: The tensile modulus ranges anywhere from 5700 ksi to 8700 ksi. Here, an average modulus of elasticity of $E_f = 6500\ ksi$ is used.

Environmental Reduction Factor (exposed to weather) [AASHTO GFRP-2B, Table 2.4.2.1-1]

$$C_E = 0.7$$

Creep reduction factor [AASHTO GFRP2, 2.5.3] $C_C = 0.3$

Bottom Transverse Reinforcement Requirements

Effective depth: $de_{bot} = 6.69\ in$

For stress block: $\alpha_1 = 0.85$, $\beta_1 = 0.85$

Here, the selected bar size and spacing of the bars are subjected to multiple iterations and a final selected values are shown for the calculation:

Selected Bar Size $Bar = Size\ 5$

Nominal Diameter $db = 0.625\ in$

Nominal Area of each bar $A_f = 0.31\ in^2$

Minimum Guaranteed Tensile Strength $f_{fu} = 95\ ksi$ [ACI 440.6-08]

Design Tensile Strength $f_{fd} = f_{fu} * C_E = 66.5\ ksi$ [AASHTO GFRP-2B, 2.4.2.1-1]

Chosen Spacing of the bottom bars $st_{bot} = 7.5\ in$

Area of the reinforcement per ft $At_{bot} = A_f * \frac{12in}{st_{bot}} = 0.496\ in\text{^}2$

Reinforcement Ratio $\rho_f = \frac{At_{bot}}{b*de_{bot}} = 0.006$

Determination of Failure Mode:

If the failure is initiated by the crushing of the concrete, the effective strength in GFRP bars is smaller than the design tensile strength.

Maximum Strain in Concrete $\epsilon_{cu} = 0.003$

Maximum Effective Tensile Stress in GFRP [AASHTO GFRP-2B, 2.6.3.1-1]

$$f_f = \sqrt{(\frac{(E_f*\epsilon_{cu})^2}{4} + \frac{0.85*\beta_1*fc'*E_f*\epsilon_{cu}}{\rho_f}} - 0.5 * E_f * \epsilon_{cu} = 86.234\ ksi > f_{fd} = 66.5\ ksi$$

Here, the calculated effective tensile stress in the GFRP bars is larger than the maximum design tensile stress. Thus, the failure mode is the rupture of GFRP bars, and the final effective tensile stress is:

$$f_f = f_{fd} = 66.5\ ksi$$

Requirement for Flexure Demand

Corresponding tensile strain $\epsilon_{fd} = \epsilon_{ft} = \frac{f_f}{E_f} = 0.01$

Stress Block $a = \beta_1 * \frac{\epsilon_{cu}}{\epsilon_{cu}+\epsilon_{fd}} * de_{bot} = 1.289\ in$ [AASHTO GFRP-2B, 2.6.3.2.2-4]

Location of Neutral Axis $c = \frac{a}{\beta_1} = 1.516\ in$

Nominal Moment Capacity $M_n = At_{bot} * f_f * \left(de_{bot} - \frac{a}{2}\right) = 16.61\ Kip - ft/ft$

Resistance Factor $\varphi = 0.55$ [AASHTO GFRP-2B, Table 2.5.5.2]

Therefore, the design flexural resistance is: $\varphi * M_n = 9.136 \frac{kipft}{ft} > M_{u,pos} = 9.113\ Kip - ft/ft$

Hence, as per flexural requirement, <u>#5 GFRP bars @ 7.5 inches</u> are required.

Requirement for Creep Rupture [AASHTO GFRP-2B, 2.5.3]

Allowable Creep Rupture Stress $f_{fcr} = C_C * f_{fd} = 19.95\ ksi$ /137.55 MPa

Modular Ratio $n_f = \frac{E_f}{Ec} = 1.524$

Ratio of depth of neutral axis to the depth of the flexural reinforcement

$$k = \sqrt{2 * \rho_f * n_f + (\rho_f * n_f)^2} - n_f * \rho_f = 0.128$$

Overall thickness of the section $h = 8\ in$

Cracked Moment of Inertia $I_{cr} = b * \frac{de_{bot}^3}{3} * k^3 + n_f * At_{bot} * (de_{bot} - k * de_{bot})^2 = 28.208\ in^4$

Creep Rupture Moment is taken as full service dead load moment and 20% of service live load moment as per AASHTO GFRP-2B, 2.5.3. The 20% factor accounts for the live load being partly available all the time and thus causes creep of the bar. It is author's opinion that this is a relatively low value and may only be valid for low volume roads.

Creep Rupture Positive Moment $Mss_{pos} = 1 * M_{DL} + 0.2 * M_{LL} = 1.347\ Kip - ft/ft$

Creep Rupture Stress $f_{fs} = n_f * de_{bot} * \frac{1-k}{I_{cr}} * Mss_{pos} = 5.091\ ksi > f_{fcr} = 19.95\ ksi$ (OK)

Based on creep rupture criteria, the spacing of #5 GFRP bars @ 7.5 inches is more than sufficient.

Minimum Flexural Requirement [AASHTO GFRP-2B, 2.6.3.3]

This criterion exists to prevent the sudden failure of the deck upon exceeding ultimate loading such that the factored flexural resistance be at least 60% greater than the cracking moment.

Modulus of Rupture $fr = 0.24 * \sqrt{fc'} = 0.48\ ksi$

1.6*Cracking Moment $1.6 * M_{cr} = 1.6 * fr * Sc = 8.192\ Kip - ft/$ft

The required minimum flexural resistance of the slab is minimum of 1.33 times the ultimate moment and cracking moment.

$$M_{min} = \min(1.6 * Mcr, 1.33Mu) = 8.192\ Kip - ft/ft < \varphi * M_n = 9.113\ Kip - ft/ft \text{ (OK)}$$

Crack Control Requirement [AASHTO GFRP-2B, 2.6.7]

The crack control equation in AASHTO GFRP-2B is based on the work of Ospina and Bakis (2007). AASHTO GFRP-2B suggests a crack width limit of 0.028 inches, which is more relaxed than traditional concrete design limits due to the corrosion-resistant properties of GFRP bars. However, since this study compares GFRP bars with equally corrosion-resistant materials, such as A1035 CS steel and stainless steel, it is reasonable to apply the same crack width limit across these materials. Therefore, to ensure uniformity and consistency, a crack width limit of 0.017 inches is used for the crack control check.

Maximum Crack Width Limit $w = 0.017\ in\ (Justified\ above)$

Calculated tensile stress in GFRP reinforcement at the service limit state:

$$f_{fs} = n_f * de_{bot} * \frac{1-k}{I_{cr}} * Ms_{pos} = 20.026\ ksi$$

Maximum allowed spacing as per crack control requirement

$$s_{max} = \min\left(\frac{1.15*C_b*E_f*w}{f_{fs}} - 2.5*cb, \frac{0.92*E_f*w}{f_{fs}}\right) = 2.767\ in < s_{provided} = 7.5\ in\ (NOT\ OK)$$

Use a bar spacing of 3 inches to satisfy crack control limit.

Along with fulfilling the crack control spacing requirements, the cover used in the crack control criteria need to meet the maximum specified cover requirement as below:

Ratio of the distance from neutral axis (NA) to extreme tension fiber and distance form NA to center of tensile reinforcement $\xi = \frac{ts-k*de_{bot}}{de_{bot}-k*de_{bot}} = 1.268$

Maximum Effective Cover [AASHTO GFRP-2B, 2.6.7-2] $dc_{max} = (C_b * E_f * w)/(2 * f_{fs} * \xi) = 11.7\ in > cb + db/2 = 1.3125\ in\ (OK)$

Maximum Clear Cover [AASHTO GFRP-2B, 2.6.7-1] $c_c, max = 2 + \frac{db}{2} = 2.3125 > cb = 1"\ (OK)$

As with steel reinforcement, the crack control significantly governs the reinforcement spacing for GFRP bars too. However, since most cracks in girder-supported decks occur in the transverse direction, the transverse bars do not need to meet the crack control spacing requirements, as they do not play a primary role in limiting these cracks.

Therefore, the authors propose that the crack control criteria for transverse bars be removed, regardless of the type of reinforcement.

Maximum Spacing of Reinforcing Bars [AASHTO GFRP-2B, 2.9.3.2]

The spacing of reinforcing bars are limited to lesser of 1.5 times of thickness of the member or 18 inches.

Maximum allowed spacing $S_{max} = \min(1.5 * ts, 18) = 12\ in$

Thus, the summarized bottom transverse reinforcement requirements are:

Ultimate Flexural Requirement [AASHTO GFRP-2B, 2.6.3]: #5 @ 7.5”

Creep Requirement [AASHTO GFRP-2B, 2.5.3]: #5 @ 30”

Crack Control Requirement [AASHTO GFRP-2B, 2.6.7]: #5 @ 3”

Maximum Spacing Requirement [AASHTO GFRP-2B, 2.9.3.2]: #5 @ 12”

Final reinforcement provided: #5 @ 7.5” when the crack control criteria are waived.

<u>Bottom Longitudinal Reinforcement Requirement [AASHTO GFRP-2B, 2.10.2.1-1]</u>

As per AASHTO GFRP-2B, 2.10.2.1-1, the amount of distribution reinforcement can be taken as the percent of main reinforcement as:

Minimum amount of bottom distribution reinforcement required:

$$Asl_{bot} = \min\left(\frac{100}{\sqrt{S}}, 50\%\ of\ Ast_{bot}\right) = 0.248in^2$$

And the equivalent spacing of bottom longitudinal steel per ft is:

$$Sl_{bot_{req}} = A_f * \frac{12in}{Asl_{bot}} = 15\ in$$

The provided spacing is restricted to 12 inches by AASHTO GFRP-2B, 2.9.3.2.
Hence, the bottom longitudinal provided is: #5 bars @ 12 inches.

Top Transverse Reinforcement Requirements

Following the method for calculating bottom transverse reinforcement outlined in Section 6.3, the top transverse reinforcement has also been designed, and the corresponding spacings have been determined:

Ultimate Flexural Requirement [AASHTO GFRP-2B, 2.6.3]: #5 @ 6.5”
Creep Requirement [AASHTO GFRP-2B, 2.5.3]: #5 @ 25”
Crack Control Requirement [AASHTO GFRP-2B, 2.6.7]: #5 @ 2”
Maximum Spacing Requirement [AASHTO GFRP-2B, 2.9.3.2]: #5 @ 12”
Final reinforcement provided: #5 @ 6.5” when the crack control criteria are waived.

Top Longitudinal Reinforcement Requirements [AASHTO GFRP-2B, 2.9.6]

Minimum Reinforcement Ratio : $\rho_{sh} = 0.0014 \leq \frac{3132}{E_f * f_{fd}} \leq 0.0036 = 0.0036$

Required Area of shrinkage and temperature reinforcement is:

$$Asl_{top} = \frac{\rho_{sh}}{2} * b * ts = 0.178\ in^2$$

Required equivalent spacing $sl_{top} = Asl_{top} * \frac{12}{A_f} = 6.9\ in$

Also, the maximum spacing of shrinkage and temperature reinforcement should be less than the minimum of 3 times the slab thickness or 12”.
Hence, the top longitudinal provided is: #5 bars @ 7 inches.

The summarized table with the required reinforcement demands using various rebars and following various design methods are tabulated below. Even though the unit weight of GFRP bars is 5 times lower than the density of steel, the total weight of the reinforcement using GFRP bars is nearly only 15% and 30% lower than using A1035 CS steel, with and without crack control provisions. This is due to a significantly larger reinforcement demands, resulting from lower modulus of elasticity of GFRP bars.

Table 3: Deck Reinforcement According to the Equivalent Strip Method for Variuos Bar Types

Bars Type	Bottom Transverse Bars		Bottom Longitudinal Bars		Top Transverse Bars		Top Longitudinal Bars		Total Weight of steel (lb/yd^2)	Rel-ative values in %
	Size and spacing	Area (in^2/ft)	Size and spacing	Area (in^2/ft)	Size and spacing	Area (in^2/ft)	Size and spacing	Area (in^2/ft)		
With Current AASHTO Crack Control Provisions Enforced**										

Black Bar Gr. 60	#5 @ 10.5"	0.354	#5 @ 15.5"	0.240	#5 @ 7"	0.531	#4 @ 18"	0.133	38.6	**100.0**
Galvabar Gr. 60	#5 @ 10.5"	0.354	#5 @ 15.5"	0.240	#5 @ 8"	0.465	#4 @ 18"	0.133	36.5	94.7
Stainless Steel Gr. 60	#5 @ 10.5"	0.354	#5 @ 15.5"	0.240	#5 @ 9.5"	0.392	#4 @ 18"	0.133	34.3	88.9
Galvabar Gr. 80	#4 @ 8.5"	0.282	#4 @ 12.5"	0.192	#4 @ 6"	0.400	#4 @ 18"	0.133	30.9	80.0
A1035 CS Steel Gr. 100	#4 @ 8.5"	0.282	#4 @ 12.5"	0.192	#4 @ 7.5"	0.320	#4 @ 18"	0.133	28.4	73.7
D8505 GFRP Bars***	#5 @ 3"	1.24	#5 @ 12"	0.31	#5 @ 2"	1.86	#5 @ 7"	0.531	24.6	63.9
With Crack Control Provisions Waived**										
Black Bar Gr. 60	#5 @ 11.5"	0.323	#5 @ 17"	0.219	#5 @ 8.5"	0.438	#4 @ 18"	0.133	34.1	88.4
Galvabar Gr. 60	#5 @ 11.5"	0.323	#5 @ 17"	0.219	#5 @ 9.5"	0.392	#4 @ 18"	0.133	32.7	84.8
Stainless Steel Gr. 60	#5 @ 11.5"	0.323	#5 @ 17"	0.219	#5 @ 10"	0.372	#4 @ 18"	0.133	32.1	83.2
Galvabar Gr. 80	#4 @ 12.5"	0.192	#4 @ 18"	0.133	#4 @ 9.5"	0.253	#4 @ 18"	0.133	21.8	56.5
A1035 CS Steel Gr. 100	#4 @ 18"	0.133	#4 @ 18"	0.133	#4 @ 16.5"	0.145	#4 @ 18"	0.133	16.7	43.3
D8505 GFRP Bars***	#5 @ 7.5"	0.496	#5 @ 12"	0.31	#5 @ 6.5"	0.572	#5 @ 7”	0.531	11.9	31

**With the proposed modification of AASHTO provisions to allow maximum bar spacing to increase from 12 in. to 18 in.
***The weight of GFRP is converted to an equivalent weight of the steel for comparison

Design using Empirical method for GFRP Bars [AASHTO GFRP2, 3.7.2.5]

All criteria for applying the empirical method to GFRP bars are similar to those for using steel reinforcements, with one exception: the slab’s core depth must be at least 3.5 inches, which is met in this case. Therefore, the empirical method is applicable here as well.

Size of bars using Empirical Method: $Bar\ Size \geq No. 5\ or\ higher$

The minimum reinforcement for bottom transverse layer is:

$$Ast_{bot}, min = \frac{870 * de_{bot}}{E_f} = 870 * \frac{6.688}{6500} = 0.9\ in^2/ft$$

With #5 bars, required spacing of bottom bars: $S_{bot}, max = 0.31 * \frac{12}{0.9} = 4.13\ in$ < 12 in

Hence, #5 bars @ 4 inches is required as per Empirical method at the bottom.

And the minimum reinforcement required for the other 3 layers are:
Top transverse reinforcement $Ast_{top}, min = 0.0035 * 12 * de_{top} = 0.26 in^2/ft$
With #5 bars, required spacing of bottom bars: $S_{bot}, max = 0.31 * \frac{12}{0.26} = 14.3\ in$ > 12 in

Since the limit of 12 in. has been placed for the reinforcement spacing, in all other directions, #5 bars at 12 inches are provided for the other 3 directions.

Conclusion: Finalized bar layout is: #5 @ 4” for bottom transverse bars and #5 @ 12” for the other 3 layers.

The final comparison chart for deck reinforcement in terms of relative area per sq. yard is shown in Figure 6 below.

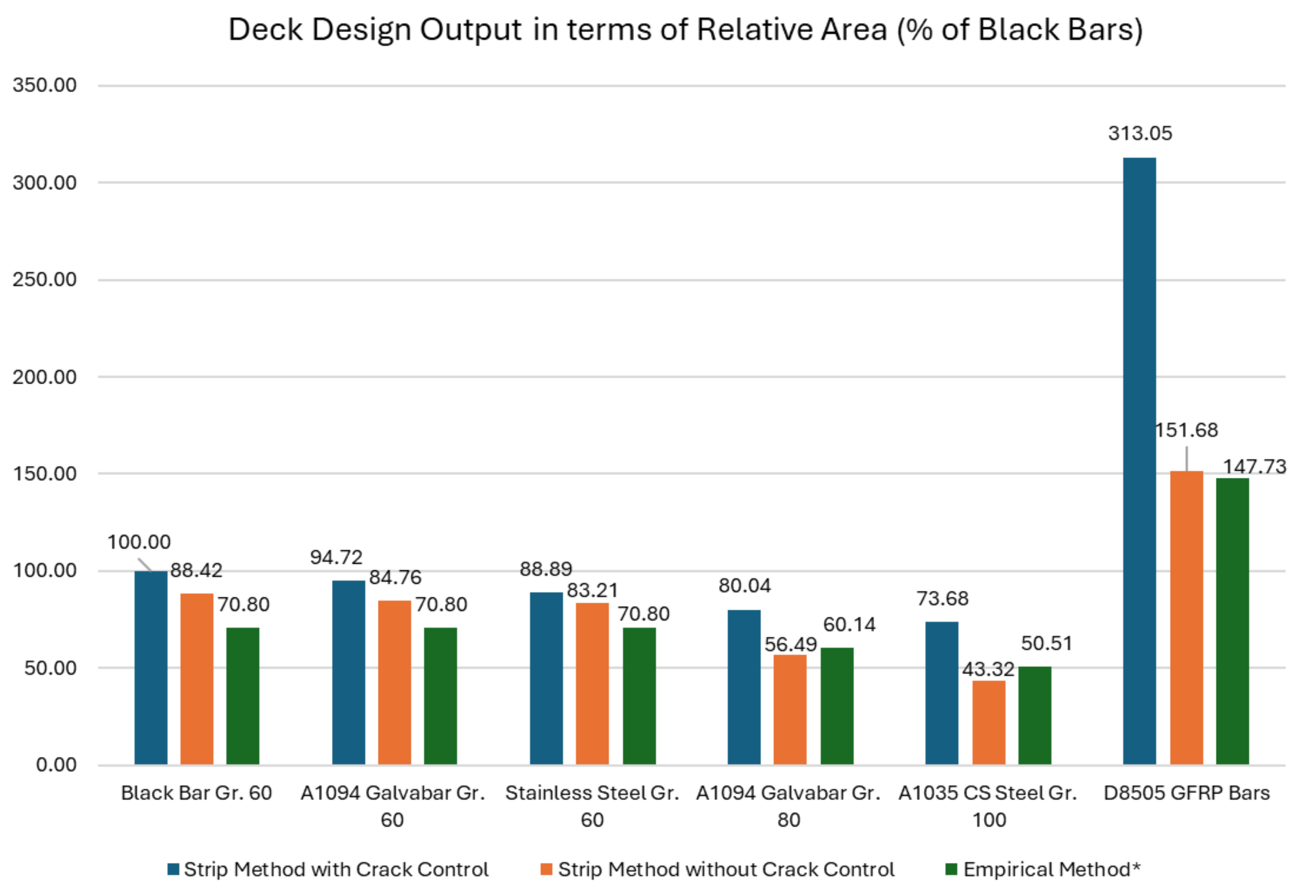

Figure 6: Deck design output using various rebars in relation to % area/sq. yd of black bars

Life Cycle Cost Analysis

Initial Deck Cost Analysis

The IDOT research on life cycle cost analysis of various rebars is based on a study by Darwin et al., which evaluated the performance of deck reinforcement in Oklahoma Bridges. That study assumes a constant steel quantity of 64.9 lb/yd² for different types of steel. However, from our comparison table, it is clear to us that this assumption is not valid, as using high-strength steel allows for a significant reduction in the amount of reinforcement. Thus, for the cost analysis, the respective area of steel per square yard tabulated in Table 1 has been used. Additionally, actual costs of various reinforcement types provided by MMFX were used to determine the initial cost of rebars while the concrete cost of $120.1 per square yard, as is suggested in Darwin's paper, is assumed. Based on these values, the initial cost of decks using various rebars are shown in Table 4.

The cost of #5 GFRP bar is given as $0.7/linear foot as per the market price. This amounts to $3.26/lb, which is used as the initial cost in Table 4.

Table 1: Initial Cost Comparison for Various Rebars using Different Methods of Deck Design

Bar Types	Total Weight (lb/yd²)	Unit In-place Cost of rebars ($/lb)	In Place Initial Cost of rebars ($/yd²)	Normalized Initial Cost of Rebars only	In place Initial cost of concrete + rebars (($/yd²)	Normalized Initial Cost
Equivalent Strip Method with Crack Control						
Black Bar Gr. 60	38.6	0.49	18.89	1.00	138.99	1.00
A 615 Galvabar Gr. 60	36.5	1.13	41.27	2.18	161.37	1.16
Stainless Steel Gr. 60	34.3	2.66	91.17	**4.83**	211.27	**1.52**
A615 Galvabar Gr. 80	30.9	1.18	36.29	1.92	156.39	1.13
A1035 CS Steel Gr. 100	28.4	2.03	57.67	3.05	177.77	1.28
D8505 GFRP Bars	24.63	3.26	80.31	4.25	200.41	1.44
Equivalent Strip Method without Crack Control						
Black Bar Gr. 60	34.1	0.49	16.71	0.88	136.81	0.98
A 615 Galvabar Gr. 60	32.7	1.13	36.93	1.95	157.03	1.13
Stainless Steel Gr. 60	32.1	2.66	85.34	4.52	205.44	1.48
A615 Galvabar Gr. 80	21.8	1.18	25.62	1.36	145.72	1.05
A1035 CS Steel Gr. 100	16.7	2.03	33.91	1.79	154.01	1.11

D8505 GFRP Bars	11.94	3.26	38.91	2.06	159.01	1.14
Empirical method*						
Black Bar Gr. 60	27.3	0.49	13.37	0.71	133.47	0.96
A 615 Galvabar Gr. 60	27.3	1.13	30.85	1.63	150.95	1.09
Stainless Steel Gr. 60	27.3	2.66	72.62	3.84	192.72	1.39
A615 Galvabar Gr. 80	23.2	1.18	27.27	1.44	147.37	1.06
A1035 CS Steel Gr. 100	19.5	2.03	39.53	2.09	159.63	1.15
D8505 GFRP Bars	11.63	3.26	37.9	2.01	158	1.14

*With proposed modification to take advantage of high strength steel.

If the initial cost is normalized relative to the cost of black bars, including concrete, stainless steel is 1.52 times more expensive than black bars. However, when looking only at the steel costs, the table shows stainless steel to be 4.83 times more expensive. The ICT study on bridge deck replacement suggests that using stainless steel increases the initial deck cost to 3.93 times that of black bars, including concrete costs. This figure is notably higher than our calculated 1.52, which we could not substantiate. For the life cycle cost calculation, the price of stainless steel from the ICT study is used.

The comparison bar chart for the initial cost of deck using various types of rebars are shown in Figure 7. When crack control criteria are imposed, the cost of decks using GFRP bars are almost the same as the cost of decks using stainless steel, since the lower modulus of elasticity of GFRP bars demand narrower spacing and the cost of GFRP bars are comparable to the cost of stainless steel. However, without crack control requirement, the cost of deck using GFRP bars is as low as that of A1035 CS steel.

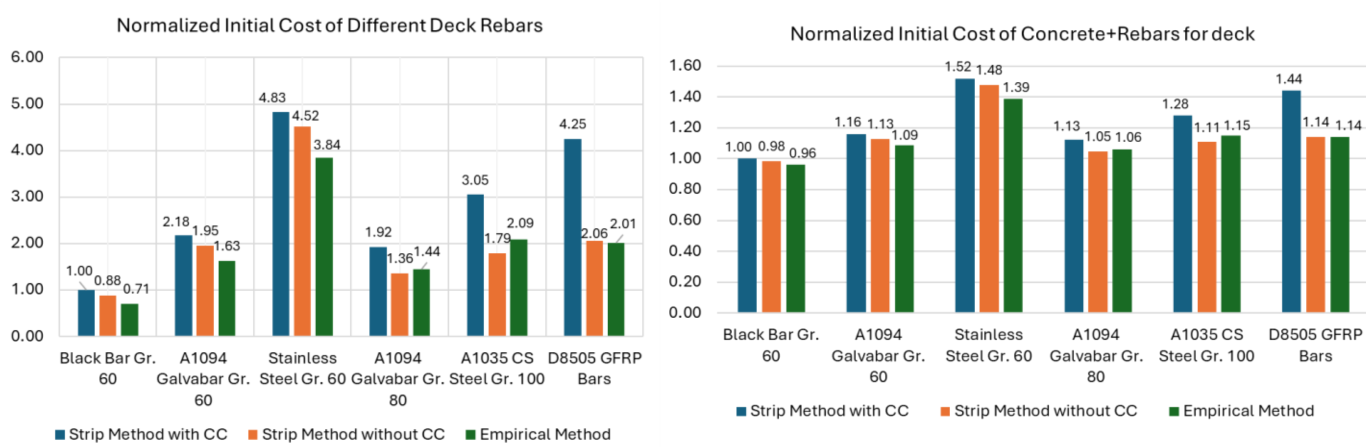

Figure 7: Normalized Initial Cost of Different rebars (left) and Normalized Initial Cost of Deck (Concrete + Rebar) using Various Rebars (right).

Table 2: Comparison of Normalized Initial Cost of Deck between ICT Study and Our Approach

Bar Type	Normalized Initial Cost

	IDOT Study	Equivalent Strip Method with Crack Control (CC)
Black Bar	US$ 1.00	US $ 1.00
Galvanized Bar	US$ 1.15	US $ 1.16
Stainless Steel	**US$ 3.97**	**US $ 1.52**
A 1035	US$ 1.27	US $ 1.28

Table 5 highlights the notably high initial cost for stainless steel reported by ICT. Other bar costs align more closely with our results when using the equivalent strip method with crack control. If the crack control criteria are waived, there is potential for a significant reduction in the initial cost for high-strength steel, as shown in Figure 5.

Life Cycle Cost

To calculate the life cycle cost of a deck using different types of rebar, future material repair costs were adopted from Darwin et al., using the averages for 2.5-inch and 3-inch cover decks. For A615 Galvabars of Gr.60 and Gr.80, repair events and future repair costs are assumed to match those of A767 Galvanized Bars, as derived from Darwin. However, the repair event numbers for various rebars have been modified as we see fit. A1035 CS steel is guaranteed to have a life span of 100 years, close to stainless steel. So, no repair event is assigned for A1035 CS steel in a 50- or 100-years life. And the number of repairs for black bars is reduced to 4, as opposed to 5 used by Darwin. By making the necessary changes to the repair events, a simplified life cycle cost method has been utilized to calculate the life cycle cost. Since the exact average time to repair couldn't be identified, the present value method hasn't been utilized here. The equation for simplified life cycle cost (2) is shown below:

$$SLCC = Initial\ Cost + fN_R \qquad \text{-Equation 1}$$

where: $SLCC = Simplified\ life\ cycle\ cost\ at\ the\ end\ of\ the\ design\ service\ life$
$f = Unit\ Future\ Material\ Repair\ Cost$

$N_R = Number\ of\ repairs\ needed\ within\ the\ design\ service\ life$

To determine the life span of GFRP embedded concrete decks, the durability of GFRP bars embedded in concrete under different environmental conditions with a life-cycle assessment model calibrated with real-world data over extended periods would be necessary. The work by Jia et. al.[8] highlights that GFRP bars demonstrate a substantial tensile strength degradation (by up to 50%) under accelerated environmental conditions, such as exposure to humidity, saline solutions, and varying water-to-cement ratios.

Thus, for the purpose of this study, one repair event has been assigned to the deck with GFRP bars over a 50-year as well as a 100-year service life of deck.
The calculated life cycle costs for 50 years and 100 years life cycle are tabulated below using constant future repair cost:

Bar Types			50 years Life Cycle	100 years Life Cycle

	Total Initial Cost (US$/yd^2)	Future Material Repair Cost (US$/yd^2)	Repair Event Number in 50 yr. Life Cycle	SLCC (US$/yd^2)	Normalized Cost (wrt Initial cost of Black Bar)	Repair Event Number in 100 yr. Life Cycle	SLCC (US$/yd^2)	Normalized Cost
Equivalent Strip Method with Crack Control								
Black Bar Gr. 60	138.99	532.84	2	1204.68	8.67	4	2270.36	16.33
A 615 Galvabar Gr. 60	161.37	564.81	1	726.18	5.22	2	1291.00	9.29
Stainless Steel Gr. 60	551.8	0.00	0	551.8	3.97	0	551.8	3.97
A615 Galvabar Gr. 80	156.39	564.81	1	721.20	5.19	2	1286.01	9.25
A1035 CS Steel Gr. 100	177.77	0.00	0	177.77	1.28	0	177.77	1.28
D8505 GFRP Bars	200.41	564.81	1	765.22	5.51	1	765.22	5.51
Equivalent Strip Method without Crack Control								
Black Bar Gr. 60	136.81	532.84	2	1202.49	8.65	4	2268.17	16.32
A 615 Galvabar Gr. 60	157.03	564.81	1	721.84	5.19	2	1286.66	9.26
Stainless Steel Gr. 60	205.44	0.00	0	205.44	1.48	0	205.44	1.48
A615 Galvabar Gr. 80	145.72	564.81	1	710.53	5.11	2	1275.34	9.18
A1035 CS Steel Gr. 100	154.01	586.13	0	154.01	1.11	0	154.01	1.11
D8505 GFRP Bars	159.01	564.81	1	723.82	5.21	1	723.82	5.21
Empirical method								
Black Bar Gr. 60	133.47	532.84	2	1199.15	8.63	4	2264.84	16.29

A 615 Galvabar Gr. 60	150.95	564.81	1	715.76	5.15	2	1280.57	9.21
Stainless Steel Gr. 60	192.72	0.00	0	192.72	1.39	0	192.72	1.39
A615 Galvabar Gr. 80	147.37	564.81	1	712.18	5.12	2	1276.99	9.19
A1035 CS Steel Gr. 100	159.63	586.13	0	159.63	1.15	0	159.63	1.15
D8505 GFRP Bars	158	564.81	1	722.81	5.2	1	722.81	5.2

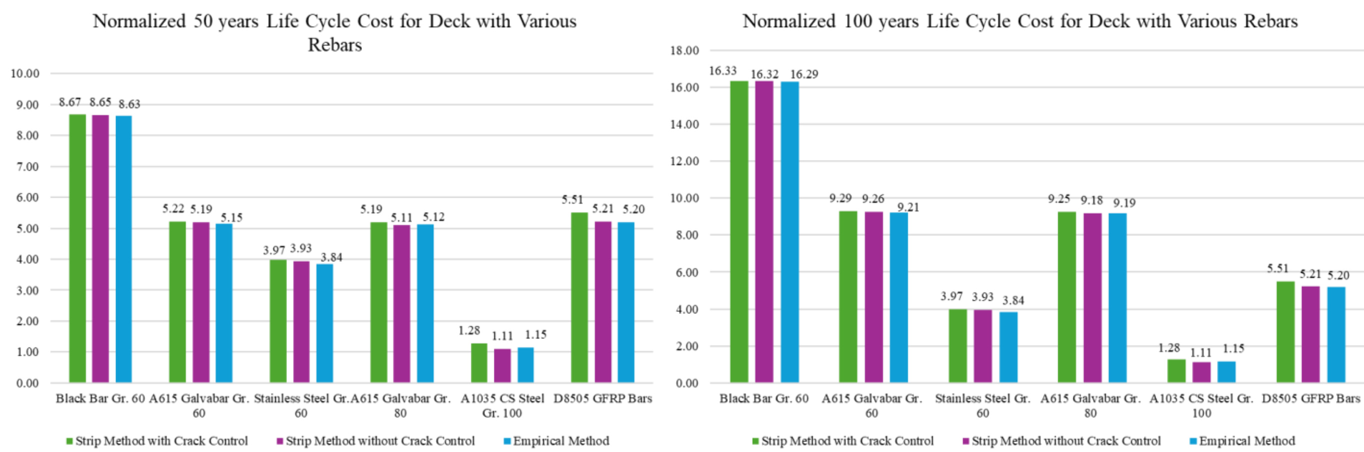

Figure 8: Life Cycle Cost Comparison

The future cost of repair/replacing a deck, obtained from Darwin et. al., is almost 4 times the initial cost. Thus, over 50 years, black bars may cost 8 times more, and over 100 years, up to 16 times more. In contrast, stainless steel and A1035 CS Steel require no repairs or replacements, keeping their life cycle cost the same as the initial cost. Therefore, A1035 Gr. 100 bars offer the cheapest design, due to their lower upfront cost compared to stainless steel. Similarly, revising the AASHTO Equivalent Strip method and the Empirical method can allow for utilizing the true benefits of high-strength steels in decks without compromising the actual structural or functional requirements as shown by comparison charts and graphs.

Conclusions

1. The Illinois DOT study was an important development in advancing the use of high strength corrosion resistant steel.
2. It will be interesting to observe the deck behavior with Epoxy Coated Gr. 60 steel as compared to a reduced amount of corrosion resistant high strength ASTM A1035 steel.

3. The study by IDOT showed that Gr. 60 Galvabar has roughly the same cost as Epoxy bars while this product has high quality and more user-friendly in creating bends.
4. The study presented here demonstrates that the black bars have the highest life cycle cost while the A1035 bars have the lowest cost.
5. It is recommended that strip method be replaced with the empirical method as much as possible.
6. Crack control equations used for transverse reinforcement give a false impression that the cracking is generally in the longitudinal direction.
7. More research is needed to justify using maximum bar spacing of 18 in.
8. More research is needed to provide documentation to support modifying the steel areas in the Empirical design to take advantage of higher strength steels.
9. More research is needed to provide more precise provisions than overhang length for anchorage of the bottom reinforcement to ensure arching action in the Empirical method.
10. If crack control criteria are ignored for GFRP bars, the initial cost of deck using GFRP bars is almost the same as A1035 steel despite the expensive cost of GFRP bar itself. However, the crack control criteria almost always govern the spacing of GFRP bars, making them nearly as expensive as stainless steel.
11. More research and track record are needed to confirm some of the criteria used in this paper for GFRP, especially the issue of durability in bridge decks.

References

1. Gombeda, M. J., Lallas, Z. N., and Rivera, E., Jr. (2022). Optimal approach for addressing reinforcement corrosion for concrete bridge decks in Illinois—Phase II. Illinois Center for Transportation, Civil Engineering Studies, Illinois Institute of Technology, FHWA-ICT-23-004, ICT Project R27-SP52. https://doi.org/10.36501/0197-9191/23-005.
2. Darwin, D., O' Reilly, M., Grayli, P. V., & Hartell, J. A. (2020). Evaluating the performance of existing reinforcement for Oklahoma bridges (Report No. FHWA-OK-20-06). Oklahoma Department of Transportation.
3. Hewitt, B. E., and B. deV Batchelor. "Punching Shear Strength of Restraint Slabs," Journal of the Structural Division. American Society of Civil Engineers, New York, NY, 1975, Vol. 101, No. ST9, pp. 1837–1853.
4. Csagoly, P. F. Design of Thin Concrete Deck Slabs by the Ontario Highway Bridge Design Code. Ministry of Transportation of Ontario, Downsville, Ontario, Canada, 1979.
5. Holowka, M., R. A. Dorton, and P. F. Csagoly. Punching Shear Strength of Restrained Circular Slabs. Ministry of Transportation and Communication, Downsview, Ontario, Canada, 1980.Fang (1985).
6. Fang, K. I., J. Worley, N. H. Burns, and R. E. Klingner. "Behavior of Isotropic Reinforced Concrete Bridge Decks on Steel Girders," Journal of Structural Engineering, American Society of Civil Engineers, New York, NY, Vol. 116, No. 3, March 1990, pp. 659–678.
7. Ospina, C. E., and Bakis, C. E. "Indirect flexural crack control of concrete beams and one-way slabs reinforced with FRP bars." Proceedings of the 8th International Symposium on Fiber Reinforced Polymer Reinforcement for Concrete Structures (FRPRCS-8), T.C. Triantafillou, ed., University of Patras, Greece, 2007 (CD_ROM).

8. Jia, D., Guo, Q., Mao, J., Lv, J., and Yang, Z. (2020). "Durability of glass fibre-reinforced polymer (GFRP) bars embedded in concrete under various environments. I: Experiments and analysis." *Composite Structures*, 234, 111687. https://doi.org/10.1016/j.compstruct.2019.111687