GFRP BAR REINFORCED CONCRETE BRIDGE DECK: ASSESSMENT OF THE AASHTO DESIGN GUIDE

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ABSTRACT

In the United States, reinforced concrete (RC) bridge decks have suffered from steel corrosion due to the enormous use of deicing salts for snow/ice removals in winter seasons. For the purpose of reducing/eliminating the steel corrosion, glass fiber reinforced polymer (GFRP) bars have been extensively researched as a promising substitute to the conventional steel reinforcement. To date, several GFRP RC decks have been constructed as demonstration projects in the US and Canada. However, the high initial material costs of GFRP bars, along with the uncertainty of long-term durability of GFRP bars under sustained/fatigue loadings in service, have hindered the wide use of GFRP bars in the concrete industry. This paper aims to assess the creep and fatigue requirements for the stress levels in the GFRP bars in RC bridge decks by a parametric study per AASHTO LFRD GFRP design guide specifications. Research results showed that the current requirement for stress level in the GFRP bars in the AASHTO design guide could be over-conservative. GFRP bars could have excellent performance in the concrete structure over its service lifetime even with the existence of sustained and/or fatigue loadings.

Keywords: Concrete Deck, Durability, GFRP Bar, Sustained Load, Fatigue Load.

INTRODUCTION

Corrosion of steel reinforcing bars in RC bridge decks has resulted in severe durability problem of the bridges as constructed in the US[1]. Recently, the United States Department of Transportation rated approximately one quarter of its bridges as functionally obsolete (FO) and structurally deficient (SD)[1]. Within the bridge components, a concrete bridge deck is the element most susceptible to corrosion[2]. The heavy use of de-icing salts for snow/ice removal each winter in the US accelerates the deterioration of concrete bridge decks.

In the US, there is a total area of $3.5\times10^{8}m^{2}$ bridge deck, 86% of which was made of cast-in-place concrete deck that is facing the problem of steel corrosion[2]. The maintenance and repair of these bridge decks are quite expensive. Koch et al. (2002) conducted a study on the corrosion costs and prevention strategies in the United States for the Federal Highway Administration (*Report No. FHWA-RD-01-156*)[3]. Results[3] showed that an annual direct cost of the corrosion for highway bridges is estimated to be $8.3 billion, consisting of $3.8 billion for the annual cost to replace the structurally deficient bridges, *$2.0 billion for the maintenance of concrete bridge decks*, $2.0 billion for the maintenance of concrete substructures, and $0.5 billion for the maintenance painting of steel bridges[3]. Life-cycle analysis estimates that the indirect costs due to traffic delays and lost productivity can be ten times greater than the direct cost of corrosion[3]. In recognition of the importance to resolve the corrosion problem, several alternatives have been explored in past decades to prevent the steel corrosion in concrete bridge decks, including the use of galvanized steel reinforcement and epoxy coated steel reinforcement[4]. However, feedbacks from the field projects indicated that the above solutions were not able to successfully eliminate the steel corrosion problem[5]. In recent years, a type of composite materials, fiber reinforced polymer (FRP) reinforcement was considered as a promising substitute to the steel reinforcement due to its non-metallic characteristics[4]. Primarily, there are two types of FRP reinforcing bars: glass-FRP (GFRP) and carbon-FRP (CFRP)[4,6]. Due to the lower cost of GFRP compared to that of CFRP, the GFRP reinforcing bars are gaining more popularity in the concrete industry. Advanced GFRP composite reinforcement can help to eliminate the steel corrosion problem[4]. To date, GFRP bars have been adopted in quite a number of demonstration projects of bridges[4]. However, the wide use of GFRP bars has been hindered by the high initial material costs of GFRP bars. In addition, in the lack of real long-term durability data of GFRP bar in field concrete structures, several uncertainties related to the long-term durability issue remain unresolved, especially when under the combined effects of sustained loading and environmental attacks[4]. This paper aims to assess the stresses levels in the GFRP bars under creep and fatigue limit states, and to explore the optimum parameters for a concrete bridge deck to achieve an economic design.

DESIGN OF GFRP BAR REINFORCED CONCRETE DECK PER AASHTO DESIGN GUIDE SPECIFICATIONS

Over the past 20 years, the use of FRP composites in civil infrastructures has grown rapidly[7]. In civil engineering practice, the successful use of new structural material systems requires the development of design guidelines[7]. Currently, in the US, the ACI 440.1R-06[4] is being used as a guide for the design and construction of structural concrete reinforced with FRP bars. In 2009, the AASHTO published *LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings*[8] *(noted as "AASHTO design guide" hereinafter),* which is specifically for the bridge deck application. Basically, the design of GFRP reinforced concrete deck is quite similar to the steel RC deck. However, it should be noted that, in order to insure the long-term safety of GFRP reinforced concrete during its service lifetime, an environmental reduction factor, C_E, has been incorporated to account for the long-term durability of GFRP bars[4,8], as can be seen in AASHTO Eq. 2.6.1.2-1[8].

$$f_{fd} = C_E \cdot f_{fu} \qquad \text{(AASHTO 2.6.1.2-1)}$$

Where C_E is specifed as 0.8 and 0.7 for concrete element non-exposed and exposed to the earth and weather, respectively[4,8]. For GFRP reinforced concrete, both over-reinforced section ($\rho_f > \rho_{fb}$) and under-reinforced section ($\rho_f < \rho_{fb}$) are allowed in the AASHTO design guide[8], where ρ_f is the reinforcing ratio, while ρ_{fb} is the balanced ratio.

$$\rho_f = \frac{A_f}{b \cdot d}$$

$$\rho_{fb} = 0.85 \cdot \beta_1 \cdot \frac{f_c^{'}}{f_{fd}} \cdot \frac{E_f \cdot \varepsilon_{cu}}{E_f \cdot \varepsilon_{cu} + f_{fd}} \qquad \text{(AASHTO 2.7.4.2-2)}$$

When $\rho_f > \rho_{fb}$, the nominal flexural strength M_n is calculated with AASHTO Eq. 2.9.3.2.2-1[8].

$$M_n = A_f f_f (d - \frac{a}{2}) \qquad \text{(AASHTO 2.9.3.2.2-1)}$$

Where "a" is the depth of equivalent rectangular stress block, f_f is the effective strength of the GFRP bar at the strength limit state.

$$a = \frac{A_f \cdot f_f}{0.85 \cdot f_c^{'} \cdot b} \qquad \text{(AASHTO 2.9.3.2.2-2)}$$

$$f_f = (\sqrt{\frac{(E_f \varepsilon_{cu})^2}{4} + \frac{0.85 \beta_1 f_c^{'}}{\rho_f} E_f \varepsilon_{cu}} - 0.5 E_f \varepsilon_{cu}) \leq f_{fd} \qquad \text{(AASHTO 2.9.3.1-1)}$$

When $\rho_f < \rho_{fb}$, the nominal flexural strength M_n is calculated with AASHTO Eq. 2.9.3.2.2-3[8].

$$M_n = A_f f_{fd} (d - \frac{\beta_1 c_b}{2}) \qquad \text{(AASHTO 2.9.3.2.2-3)}$$

$$c_b = (\frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fd}}) \cdot d \qquad \text{(AASHTO 2.9.3.2.2-4)}$$

For the LRFD design approach, a resistance factor is required to obtain the factored flexural resistance.

$$M_r = \phi \cdot M_n \qquad \text{(AASHTO 2.9.3.2.1-1)}$$

Depending on the reinforcement ratio, different resistance factors, ϕ, are used to account for structural ductility, as shown in AASHTO Eq. 2.7.4.2-1[8].

$$\phi = \begin{cases} 0.55 & \text{for } \rho_f < \rho_{fb} \\ 0.3 + 0.25 \dfrac{\rho_f}{\rho_{fb}} & \text{for } \rho_{fb} < \rho_f < 1.4 \rho_{fb} \\ 0.65 & \text{for } \rho_f > 1.4 \rho_{fb} \end{cases} \qquad \text{(AASHTO 2.7.4.2-1)}$$

To avoid the creep rupture of GFRP bar under sustained loading, or the failure due to fatigue loading, the stress levels, f_{fs}, in the GFRP bars are limited under the fatigue and creep rupture limit state in the AASHTO design guide[8]. f_{fs} is recommended to not exceed 20% of the GFRP design strength, as shown in AASHTO Eq.2.7.3-1[8].

$$f_{fs} \leqq 0.2 f_{fd} \qquad \text{(AASHTO 2.7.3-1)}$$

Where,

$$f_{fs} = \frac{n_f d(1-k)}{I_{cr}} M_s \qquad \text{(AASHTO 2.7.3-2)}$$

$$k = (\sqrt{2\rho_f n_f + (\rho_f n_f)^2} - \rho_f n_f) \qquad \text{(AASHTO 2.7.3-3)}$$

$$I_{cr} = \frac{1}{3} d^2 k^3 + n_f A_f d^2 (1-k)^2 \qquad \text{(AASHTO 2.7.3-4)}$$

In AASHTO Eq. 2.7.3-2, M_s is the moment determined by Fatigue or Service I load combination as specified in Table 3.4.1-1 of the AASHTO LRFD Bridge Design Specification[9].

Besides the above stress limits, according to the AASHTO design guide, the minimum requirement for flexural tensile reinforcement, and the maximum crack width shall also be checked for the design of GFRP RC bridge deck, as shown in AASHTO Eq. 2.9.3.3-1 and Eq. 2.9.3.4-1[8].

$$A_{f,\min} \geq \max(0.16\sqrt{f_c^{'}}; 0.33) \frac{bd}{f_{fd}} \qquad \text{(AASHTO 2.9.3.3-1)}$$

$$w = 2 \frac{f_{fs}}{E_f} \beta k_b \sqrt{d_c^2 + \frac{s^2}{4}} \leqq 0.02\text{in.} \qquad \text{(AASHTO 2.9.3.4-1)}$$

Note that an allowable crack width of 0.02in. is specified as the crack control limit[8].

DESIGN ASSESSMENT OF GFRP RC DECK

Since the technology of GFRP reinforced concrete deck is relatively new to the concrete industry, a full understanding of its behavior is necessary for a safe and economic design. The cost of GFRP bar is \$3 to \$4/lb, while it is \$0.32/lb for epoxy coated rebar (http://rebar.ecn.purdue.edu/ect/links/technologies/civil/frprebar.aspx). Berg *et al.* (2006) conducted a study on the construction and cost analysis of an FRP reinforced concrete bridge deck, which showed that the material costs for the FRP reinforced deck bridge were 60% higher than that of the steel reinforced deck bridge[14]. This high initial material cost of the GFRP bar has been one of the main reasons for hindering the wide use of GFRP bar. Another issue is the uncertainty of long-term durability of GFRP bars over time when the GFRP bars are subject to the sustained stress resulted from various dead loads and live loads. This research intends to clarify the above concerns by a comprehensive parametric study per AASHTO design guide. Parameters in this study include deck thickness, girder spacing, and environmental reduction factors. Four different deck thicknesses, i.e., 7in., 8in., 9in., and 10in., and seven different girder spacings, i.e., 6ft, 7ft, 8ft, 9ft, 10ft, 11ft, and 12ft, are used in this study.

As specified in the AASHTO design guide[8], GFRP RC deck shall be designed by satisfying the strength limit state, maximum crack width, minimum reinforcement requirement, and check for the creep and fatigue rupture limit[8]. Table 1 shows the load combinations for the different design limit states.

Table 1 Load combinations for different limit states

Limit states[8]	Load combinations[9]	AASHTO Requirements[8]
Flexural resistance	1.25DC+1.5 DW +1.75LL	$M_u < \phi M_n$
Creep rupture	1.0DC+1.0 DW +LL	$f_{fs}<0.2f_{fd}$
Fatigue rupture	0.75LL	$f_{fs}<0.2f_{fd}$
Maximum crack width requirements	1.0DC+1.0 DW +1.0LL	w < 0.02 in.
Minimum requirement for reinforcement	NA	$A_{f,\min} \geq \max(0.16\sqrt{f_c'};0.33)\dfrac{bd}{f_{fd}}$

For common practice in bridge engineering, dead loads (DL) for a typical bridge deck include deck own weight, future wearing surface, and barriers. In the AASHTO design guide[8], stay-in-place formwork is recommended to be used in the GFRP RC bridge deck. Therefore, the weight of formwork is taken into account herein as well. Table 2 shows the common unit weights for the above dead loads that were used in this study.

Table 2 Unit weights for the dead loads[9]

	Concrete for deck	Barrier/each	stay-in-place formwork	future wearing surface
Unit weight	0.150 kcf	520plf	10psf	25psf

The above dead loads are assumed to be evenly distributed over the deck along its transverse direction[9,10]. As a uniformly distributed load, the positive and negative flexural moments due to the dead loads over the interior girders can be calculated with the formula "$M=wL^2/10$" [10]. Note that the deck over the exterior girders is not discussed in this study. Live load (LL) effect on the bridge deck is determined by using the AASHTO LRFD Specification Table A4-1[9], which has already incorporated the multiple presence factors and dynamic load allowance[9].

Fig.1 shows a schematic representation of the deck slab reinforcements in a transverse section. As a durable reinforcement compared to steel, a ¾'' clear concrete cover was used for both the top and bottom GFRP bars in this study on the basis of ACI440.5-08 "Specification for construction with fiber-reinforced polymer reinforcing bars[13]. The transverse bars are the main reinforcements in the bridge deck, which will be designed and investigated in this study. The top bars are designed on the basis of the maximum negative moments while the bottom bars are designed for the maximum positive moments under various limit states per AASHTO design guide[8]. In common practice, the top and bottom GFRP mats are kept identical for ease of installation[11].

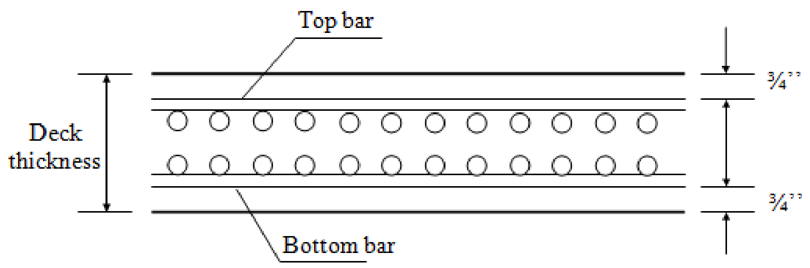


Fig. 1 Schematic illustration of deck slab reinforcements in a transverse section (Adapted from [11])

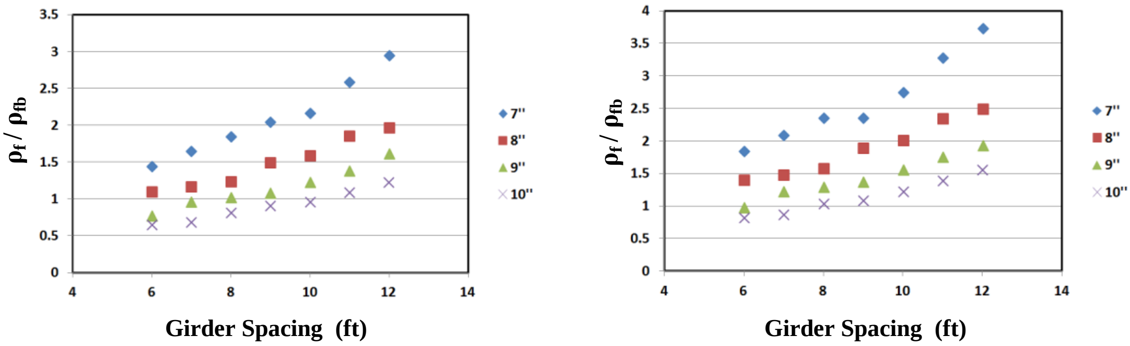
Normal weight concrete with 28 day compressive strength of 4ksi is used for the bridge deck. For the GFRP bars, the material properties are taken from the ACI 440 specifications[12], as shown in Table 3, where f_{fu} is the guarenteed tensile strength.

Table 3 GFRP Bar Materials Properties[4,12]

GFRP bar size	d_b (in.)	A_f (in.^{2})	f_{fu} (ksi)	E_f (ksi)
#4	0.500	0.20	101	5945
#5	0.625	0.31	95	5945
#6	0.750	0.44	90	5945
#7	0.875	0.60	85	5945
#8	1.000	0.79	80	5945
#9	1.128	1.00	75	5945

As specified in AASHTO Eq. 2.6.1.2-1[8], the GFRP design tensile strength, f_{fd}, is the product of C_E and f_{fu}, where C_E is the environmental reduction factor, which can be equal to 0.8 or 0.7 depending on the exposure conditions[8]. In this study, both factors of 0.7 and 0.8 were used in the parametric study.

Based on the above discussions, a design sheet was developed for the parametric study of GFRP RC decks . Totally 56 cases were explored for different deck thicknesses, girder spacings, and environmental reduction factors. Each case was designed by meeting the strength, minimum reinforcement requirement, and maximum crack width requirement. Note that over-design is intentionally avoided in this study, more specifically, e.g., the design gives a crack width as close to 0.02in. as possible if the maximum crack width governs the design, or the design gives a M_r as close to M_u as possible if the strength governs the design. The stress limit under the fatigue and creep rupture limit state is not considered as one of the design controls since the objective of this research is to identify the stress levels in the GFRP bars when all the other limit states are satisfied. Fig.2 shows the data plots for ρ_f/ρ_{fb} with respect to girder spacing and deck thickness. The legends as shown on the plots represent the deck thickness.

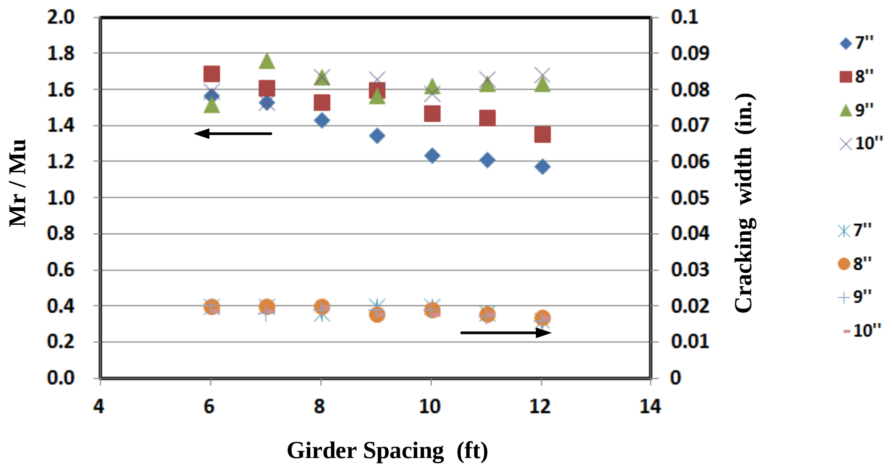


(C_E=0.7) (C_E=0.8)

Fig. 2 ρ_f / ρ_{fb} with respect to girder spacing and deck thickness

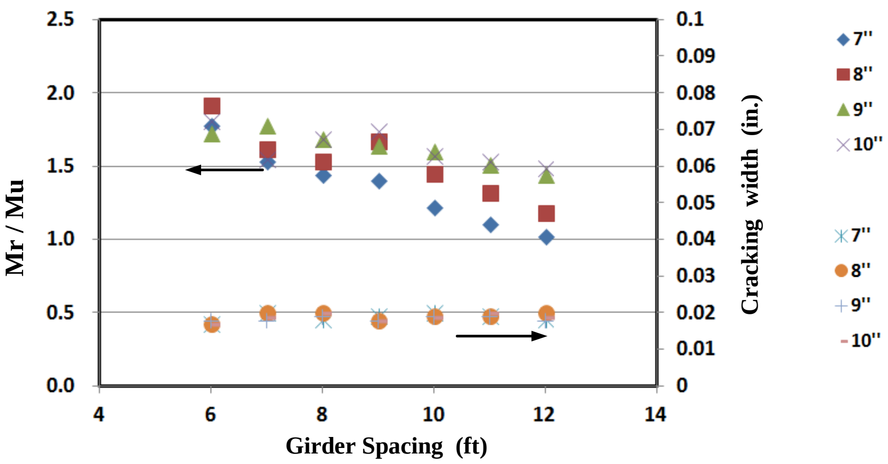
It can be seen from Fig.2, an under-reinforced section (i.e., $\rho_f / \rho_{fb} < 1.0$) can satisfy the limit requirements for a shorter span with a thicker deck, while an over-reinforced section (i.e., $\rho_f / \rho_{fb} > 1.0$) is needed for a longer span deck with a thinner deck. In addition, we can observe that, for a specific deck thickness, the ratio of ρ_f / ρ_{fb} increases as the girder spacing increases. However, for a given girder spacing, the ratio of ρ_f / ρ_{fb} decreases as the deck thickness increases. It indicated that, for a given girder spacing, a thicker deck requires less amount of GFRP bars than a thinner one, which could reduce the initial material costs of the GFRP reinforcements. However, we shall be aware that a thicker deck will increase the dead load on the deck that could result in higher sustained stresses in the GFRP bars which may be harmful to the long-term durability of the GFRP bars. Similarly, it can be seen that a shorter deck span requires less amount of GFRP bars than a longer one with a same deck thickness which could result in a saving in the GFRP material costs. However, as known, for a given bridge width, a shorter deck span requires more bridge girders that will pay off the cost savings in the GFRP reinforcements.

As stated above, the design of the GFRP RC deck may be governed by the maximum crack width or the strength limit. Fig.3 shows the plots of M_r/M_u and crack width vs. girder spacing for different deck thicknesses. The legends as shown on the plots represent the deck thickness.

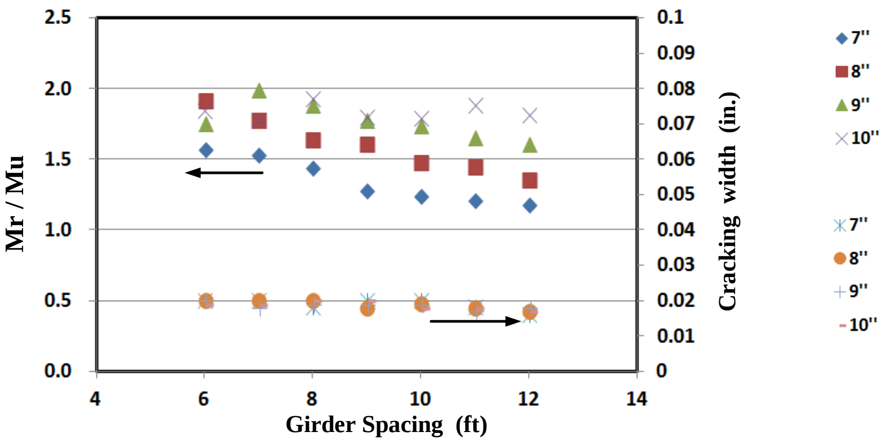


(a) bottom bar (C_E=0.7)

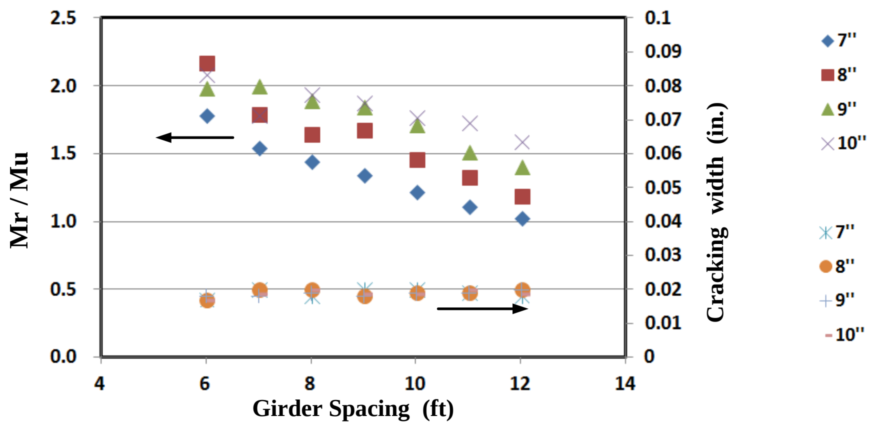
Fig. 3 M_r/M_u and crack width vs. girder spacing and deck thickness



(b) top bar (C_E=0.7)



(c) bottom bar (C_E=0.8)

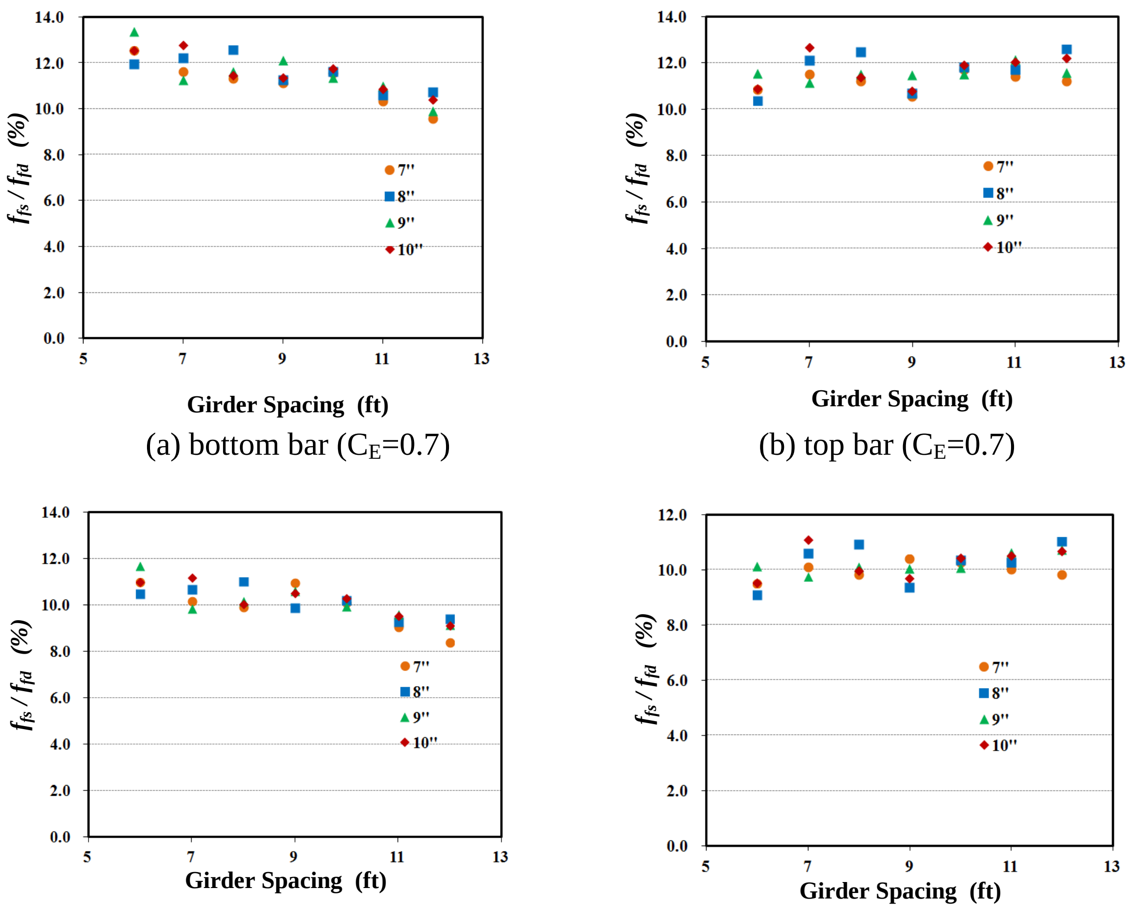


(d) top bar (C_E=0.8)

Fig. 3 M_r/M_u and crack width vs. girder spacing and deck thickness (Continued)

It can be seen from Fig.3, the maximum crack width for all cases are smaller or equal to 0.02in.. All the cases have a M_r/M_u ratio greater than 1.0. In general, the case with a thicker deck give a higher ratio of M_r/M_u, while a longer deck span result in a lower ratio of M_r/M_u. It should be pointed out that the case with a 12ft span and 7in. thick deck has a M_r/M_u ratio right above 1.0, of which the design is controlled by the strength limit. The majority of the cases are controlled by the maximum crack width.

To avoid any failure due to fatigue loading, the stress levels in the GFRP bars are limited not to exceed 20% of the GFRP design strength in the AASHTO design guide[8]. Fig. 4 shows the fatigue stress levels (i.e., f_{fs}/f_{fd}) in the GFRP bars by the parametric study. It can be seen that, the stress level in the GFRP bars ranges from approximately 9% to 14% when C_E=0.7 is used, while it ranges from 8% to 12% when C_E is equal to 0.8. It can be seen that, the stress levels in the GFRP bars are less than 20% of the GFRP design strength for all the cases being investigated herein. This observation is in good agreement with the section 2.7.3 of the AASHTO design guide[8], which states "*Fatigue need not to be investigated for concret slab in multigirder applications*"[8].



(a) bottom bar (C_E=0.7)

(b) top bar (C_E=0.7)

(c) bottom bar (C_E=0.8)

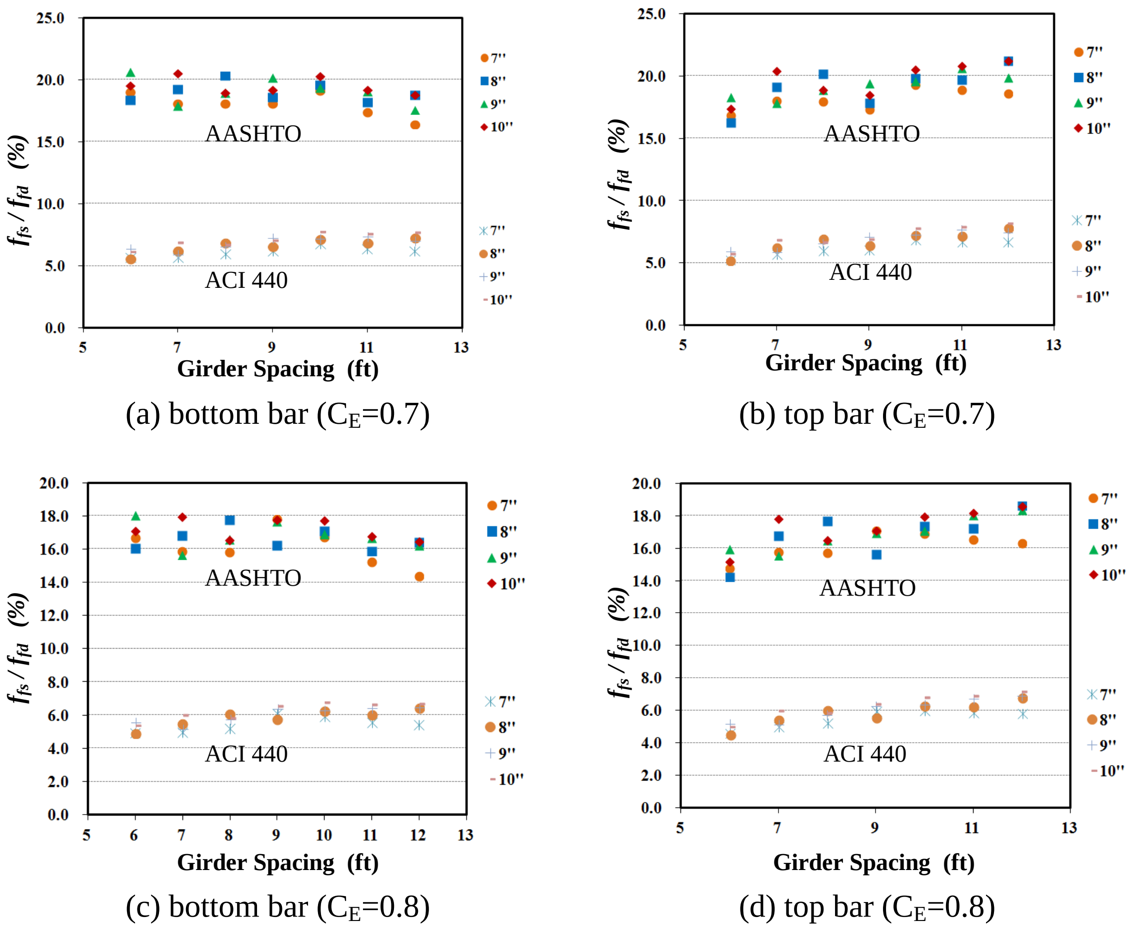
(d) top bar (C_E=0.8)

Fig. 4 Stress level in GFRP bar for fatigue limit state

Another concern for GFRP RC deck is the sustained stress in the GFRP bars which could affect their long-term performance. Fig. 5 shows the stress level (i.e., f_{fs}/f_{fd}) in the GFRP bars under the creep limit state where service I load combination (i.e., 1.0DC+1.0 DW +LL) is used[8,9]. It can be seen that, the sustained stress level ranges from approximately

15% to 21% and 14% to 19% when C_E is equal to 0.7 and 0.8, respectively. As can be seen, all the cases can meet the creep rupture limit when C_E=0.8. However, when C_E=0.7 is used, a few cases slightly exceed the creep rupture limit.

The ACI 440.1R-06, another design guide for structural concrete reinforced with FRP bars in the US, specified a different load combination (i.e., DL+0.2LL) for calculating the stress level in the GFRP bars under the creep limit state. In this study, the sustained stresses in the GFRP bars are also investigated in accordance with the ACI 440 design guide, from which the results are shown in Fig. 5. It can be seen that, the sustained stress level ranges from approximately 5-8% and 4-7% for C_E=0.7 and C_E=0.8, respectively, which is much lower than that from the AASHTO design guide[8].



(a) bottom bar (C_E=0.7) (b) top bar (C_E=0.7)

(c) bottom bar (C_E=0.8) (d) top bar (C_E=0.8)

Fig. 5 Stress level in GFRP bar for creep limit state

Benmokrane *et al.* (2006) designed and tested a demonstration project of concrete bridge deck in Vermont State in the United States[11]. The bridge deck was designed with GFRP bars with a thickness of 9in.. The bridge girder spacing is 7.75ft. The concrete specified design strength was 4ksi in compression at 28 days, while the GFRP bar has an E-modulus of 5802ksi and a guaranteed tensile strength of 76.1ksi. The transverse bars for the top and bottom GFRP mats were #6@4in.. The top and bottom concrete clear cover are 2.5in. and 1.5in., respectively. An environmental reduction factor C_E of 0.7 is used in this project. After the bridge was constructed, a load test was conducted to investigate the stresses in GFRP reinforcements under different test trucks. Results[11] indicated that the stress levels in the GFRPs bar under test trucks were no more than 0.2% of the GFRP design strength[11]. The authors extrapolated the tested values from the test trucks to the LRFD design truck resulting in a stress level less than 1% of the GFRP design strength[11]. This stress range (i.e., 1%) as obtained from the above field test is much lower than the stress range under the fatigue limit state (i.e. 9-14%) as determined by the AASHTO design guide where a loading of 0.75LL (live load only) is used for calculating the stress in the GFRP bar. Thus, it can be seen that the stress limit under the creep and fatigue limit state in the AASHTO design guide could be over-conservative.

CONCLUSIONS

This paper investigated the stress levels in the GFRP bars under creep and fatigue limit state by a parametric study per AASHTO LFRD GFRP design guide specifications. Based on the study in this paper, the following conclusions can be made:

- For most cases, the maximum crack width controls the design of GFRP RC bridge deck;
- For a shorter span deck, an under-reinforced section could satisfy the design requirements; For a longer span deck, over-reinforced section is required to meet the design requirements;
- The stress level in GFRP bars ranges from approximately 8% to 14% under the fatigue limit state per AASHTO design guide;
- The stress level in GFRP bars ranges from approximately 14% to 21% under the creep limit state per AASHTO design guide;
- The stress level in GFRP bars ranges from approximately 4-8% under the creep limit state per ACI440.1R-06 design guide;
- The stress level requirement for GFRP bars under the creep and fatigue limit state in the AASHTO design guide could be over-conservative.

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