AASHTO Design Specifications for GFRP-RC Bridges: 2nd Edition

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Abstract. The development of a comprehensive bridge design national standard is paramount to allow for a wider and safe deployment of Glass Fiber Reinforced Polymer (GFRP) Reinforced Concrete (RC) in the transportation infrastructure. To respond to this demand, a task force of researchers, practitioners, and transportation officials lead by the University of Miami (UM), the University of South Carolina (USC), and the Florida Department of Transportation (FDOT), has developed a draft of the second edition of the Bridge Design Guide Specifications for GFRP-RC (BDGS-GFRP), now under consideration by the American Association of State Highway and Transportation Officials (AASHTO) committee T6. This paper deals with the salient contents of the document, with specific emphasis on the design of flexural members. Compared to the first edition, changes were proposed to reflect the state-of-the-art from archival literature and harmonize the design philosophy with that of other authoritative national and international standards.

Keywords: GFRP-RC · Design · Guidelines · Bridges · Infrastructures

1 Introduction

Fiber Reinforced Polymers (FRP) bars and strands are a viable corrosion-resistant solution for Reinforced Concrete (RC) and prestressed concrete (PC) in applications were corrosion of Mild Carbon Steel (MCS) and High Strength Carbon Steel (HSCS) represents a durability and safety concern (Spadea et al. 2018). In particular, the application of Glass FRP (GFRP) bars is spreading, with a number of bridges built worldwide over the last 40 years (Bakis et al. 2002; Gooranorimi and Nanni 2017). GFRP technology is tailored for application in aggressive environments. These include: coastal areas in sub-tropical environments (Nolan et al. 2018), cold weathered regions where de-icing salts are used and freeze-thaw cycles occur (Ahmad 2003), urban and industrial areas where concrete is prone to carbonation and exposed to wet-dry cycles (Nanni et al. 2014), geotechnical applications where reinforcement is exposed to moist and contaminated soil (Mohamed and Benmokrane 2014), and applications were the

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presence of stray currents may trigger corrosion in steel reinforcement (Spagnuolo et al. 2018).

Design principles for GFRP-RC are well established (Rossini et al. 2018a) and the technology is commercially available and spreading (Ruiz et al. 2018). Guidelines and regulations have been published in North America, Europe, Russia, and China (Rossini et al. 2018b). In the United States, design principles for GFRP-RC are detailed in guidelines issued by the American Concrete Institute (ACI 2015). The deployment of GFRP-RC in buildings is regulated by the International Code Council (ICC) that maintains an Acceptance Criteria (AC) for GFRP bars (ICC 2016). The deployment of GFRP-RC in infrastructural elements is regulated by the American Association of State Highway and Transportation Officials (AASHTO). AASHTO maintains a specific document that, in its first edition, only covers the design of GFRP-RC bridge decks and open-post railings (AASHTO 2009). ASTM recently published standard specifications for GFRP bars (ASTM 2017). The document does not hold binding status by itself, but it does once referenced in national design and construction codes and standards. The document is expected to relieve the need to include a chapter covering material specifications in design guidelines as it was done in the past. In Canada, the use of GFRP bars in buildings is covered by the guidelines issued by the Canadian Standardization Association (CSA 2012). GFRP-RC deployment in infrastructures is regulated by the Canadian Highway Bridge Design Code (CHBDC) issued by CSA (2014). In Europe, guidelines for GFRP-RC design are published by the International Concrete Federation (*fib* 2007). *fib* also includes GFRP-RC in its Model Code (*fib* 2013). In Italy, guidelines for GFRP-RC design are published by the National Research Council (CNR 2007). In Russia, the deployment of GFRP in buildings is regulated by a specific addendum to the national building code (Minstroy 2018). The approach of the Russian building code is compatible with the one of the Eurocodes that, however, do not include GFRP-RC (CEN 2005a). In China, the deployment of GFRP-RC in infrastructures is regulated by national guidelines (SAC 2010).

The first generation of design guidelines and standards was issued in the late 90s and early 2000s. It succeeded in addressing the behavior of GFRP-RC structures, and the differences with respect to conventional steel RC members (Nanni 1999). However, the limited experimental database available at that time called for the introduction of relatively severe safety factors (Jawaheri and Nanni 2013).

The second generation of GFRP-RC design guidelines represents the recent state-of-the-practice. It expanded and refined the documents from the first-generation. However, little was done to address the issues that prevented one from taking full advantage of the efficiency and economical appeal of GFRP bars (Rossini et al. 2018a).

The third generation of design guidelines is currently under development and publication. It includes the 2nd edition of the AASHTO Bridge Design Guide Specifications for GFRP-RC Bridges (BDGS-GFRP) (AASHTO 2018), the first edition of the ACI Building Code Provisions for Concrete Reinforced with GFRP Bars (currently under development), and an update of the CSA Canadian Highway Bridge Design Code (CHBDC) (scheduled for development).

2 Research Significance

A draft of the second edition of AASHTO BDGS-GFRP (AASHTO 2018) was developed by a task force of researchers, practitioners, and transportation officials led by the University of Miami (UM), the University of South Carolina (USC), and the Florida Department of Transportation (FDOT). Objectives included: updating the provisions to include state-of-the-art archival literature; making the provisions more rational and address the issues preventing one from taking full advantage of the mechanical and economic appeal of GFRP bars; making the design approach consistent with the AASHTO Bridge Design Specifications for traditional construction materials (BDS); and, harmonize the design philosophy with that of other authoritative national and international standards.

3 Guidelines Integration

Consistency and clarity in standards and guidelines are paramount to allow for safe and efficient design of structural members. At the same time, standardization is crucial to leverage deployment of innovative technologies in civil engineering. Nevertheless, GFRP-RC design guidelines typically exist as separate documents with respect to MCS-RC counterparts (AASHTO 2009, 2017), or as addenda to national and local design codes (CSA 2014). Furthermore, overlapping exists, and FRP-RC/PC design guidelines have different approaches one with respect to the other and with respect to design guidelines for traditional structural materials (Rossini et al. 2018b). Differences include: the definition of the material properties to be used for design purposes; the structure of the design equations; and, the value and definition of the design parameters to be used in these equations.

The ideal setting to leverage wider deployment of GFRP-RC in substantial applications entails embedding GFRP bars as an alternative reinforcement solution in a comprehensive standard (Nolan and Nanni 2017). The approach can be expanded to include other materials, as well as PC applications (Rossini et al. 2018b).

4 Design Approach

Rossini et al. (2018b) outlined a unified design approach to FRP-RC/PC. The approach was validated on an FRP-RC/PC pedestrian bridge reinforced with Glass FRP bars, Basalt FRP bars, and Carbon FRP strands. The approach served as a framework for developing the draft of the second edition of AASHTO BDGS-GFRP and is summarized in the following with specific reference to the case of GFRP-RC.

Any mechanical problem can be defined as a system of equilibrium, compatibility and constitutive equations. Structural theories introduce assumptions to simplify the mathematical formulation of common mechanical problems, like the beam model. Classical Euler-Bernoulli assumptions hold valid in GFRP-RC bended elements, and sectional analysis can be carried out. Rigorously, the only difference with respect to MCS-RC is in the constitutive law used to model the reinforcing bars. Similarly to the design of MCS-RC, limitations in the exploitability of the materials, amount of reinforcement, and maximum strains and deflection are introduced to ensure structural assumptions are met and the desired level of safety and reliability is provided.

4.1 Material Properties

GFRP is a brittle composite material, elastic until failure, stronger, but less stiff with respect to MCS. The guaranteed strength (f_{fu}^*) of a GFRP bar is defined as the experimental average value minus three standard deviation (ACI 2015), corresponding to the 99.9th strength percentile. The definition is reported in Eq. 1 for clarity. The approach is more conservative with respect to the calculation of characteristic strengths for steel reinforcement and concrete, traditionally defined as the average value minus 1.64 standard deviation – 95th strength percentile – under the assumption of normal distribution (CEN 2005a).

$$f_{fu}^* = f_{fm} - 3\sigma_f \tag{1}$$

The strength of commercially available GFRP bars can vary from product to product at varying fiber content and manufacturing techniques (Emparanza et al. 2017). At the time of design, the bar manufacturer is typically not defined. Thus, the minimum guaranteed strength required for certification per ASTM D7957 (ASTM 2017) is taken as the specified tensile strength for design purposes in spite of an experimental value. The specified strength ($f_{fu}^{'}$) is always less than or equal than the guaranteed experimental strength (f_{fu}^*) of the specific batch of bars that will be deployed in construction, as shown in Eq. 2.

$$f_{fu}^{'} \leq f_{fu}^* \tag{2}$$

Tracing a straight line to limit the exploitability of different products and material systems may slow down the growth of the GFRP industry. Nevertheless, the need for standardization is paramount. A possible solution may lay in the definition of different strength grades, as traditionally done for MCS bars (ASTM 2016), steel profiles (AISC 2017; CEN 2005b), and concrete (FDOT 2018; CEN 2005a).

FRP composites are known to experience strength degradation following long-term exposure to the environment (ACI 2015; *fib* 2007). To account for the phenomenon, the design strength (f_{fd}) of the material is defined per Eq. 3 including an environmental reduction factor (C_E). The approach is in line with the principles of ACI (2015).

$$f_{fd} = C_E f_{fu}^{'} \tag{3}$$

The design strength of the material is the reference value for design calculations, both at the ultimate limit state (ULS) and service limit state (SLS). Furthermore, the strength of FRP under sustained load is reduced to avoid creep rupture (ACI 2015; *fib* 2007). Resorting to the nomenclature suggested by Rossini et al. (2018a), a creep rupture reduction factor (C_c) is applied to the design strength in order to define the design strength against creep rupture under sustained load ($f_{f,c}$) as in Eq. 4. Similarly, a fatigue reduction factor (C_f) is applied to the design strength in order to define the design strength under cyclic loading ($f_{f,f}$) as in Eq. 5.

$$f_{f,c} = C_c f_{fd} = C_E C_c f'_{fu} \tag{4}$$

$$f_{f,f} = C_f f_{fd} = C_E C_f f'_{fu} \tag{5}$$

The brittle nature of FRP reinforcement implies the possibility to either have over-reinforced flexural members that may fail because of concrete collapse in the compression zone, or under-reinforced flexural members that may fail because of reinforcement rupture in the tension zone (ACI 2015). The two failure modes are characterized by two different strength reduction factors – ϕ_c and ϕ_t respectively – defined to guarantee the same level of safety in the two cases. A flexural member can also undergo shear failure. In this case the strength reduction factor ϕ_s is aligned to values prescribed for MCS-RC in ACI (2014).

GFRP bars lack the plastic plafond typical of MCS bars. Thus, GFRP-RC flexural members do not feature ductile behavior at failure. Nevertheless, GFRP bars reach strain levels higher than the 0.005 ductility threshold set for MCS in ACI (2014). Thus, GFRP-RC flexural members feature a pseudo-ductile behavior comparable to what is required of MCS-RC to foresee upcoming failure. Figure 1a compares the mechanical behavior of GFRP and MCS bars M13. Figure 1b compares the flexural strength reduction factor proposed for GFRP bars to traditional values used for MCS (AASHTO 2017). The strength reduction factors are plotted as a function of the strain reached by the reinforcement at sectional failure. The diagram is adapted from AASHTO (2017).

The different bond characteristics of GFRP bars with respect to steel reinforcement is accounted for introducing a bond reduction factor (C_b). The parameter is defined in Eq. 6 as the inverse of the bond reduction coefficient (k_b) as defined in ACI 440.1R (ACI 2015). By this definition, the bond reduction factor increases at increasing performances, consistently with the other design factors. Better bond performances enhance crack control and reduce crack width at equal load level (ACI 2015).

$$C_b = 1/k_b \tag{6}$$

4.2 Design Factors

Table 1 provides a summary of design factors as reported by international design guidelines, along with the values adopted in the second edition of AASHTO BDGS-GFRP. The flexural strength reduction factor for compression-controlled failures (ϕ_c) is raised from 0.65 to 0.75 (+15%) with respect to the first edition of AASHTO BDGS-GFRP (AASHTO 2009). The value is in line with findings of Jawaheri & Nanni (2013). The creep rupture reduction factor (C_c) is raised from 0.20 to 0.30 (+50%) with respect to the first edition of AASHTO BDGS-GFRP. The value is more reflective of the performances of ASTM-compliant GFRP bars, and about 50% of the experimental findings of Perigny et al. (2012), Sayed-Ahmed et al. (2017), and Keller et al. (2017).

Similarly, the fatigue reduction factor (C_f) is raised to 0.25 (+25%) for alignment with international standards (CNR 2007; *fib* 2013; CSA 2014). The bond reduction factor (C_b) is raised from 0.71 to 0.83 (+17%). The value is reflective of the good bond performances of GFRP bars (Gooranorimi et al. 2018) and is more conservative with respect to international guidelines (CSA 2014).

Table 1. Design parameters for GFRP-RC

	CNR	*fib*	CSA	ACI	AASHTO	
	2007	2013	2014	2015	2009	2018
ϕ_c	0.67	0.67	0.75	0.65	0.65	**0.75**
ϕ_t	0.60	0.80	0.55	0.55	0.55	**0.55**
ϕ_s	-	-	-	0.75	0.75	**0.75**
C_E	0.70	0.55$^{(1)}$	1.00	0.70	0.70	**0.70**
C_c	0.30	0.30	0.25	0.20	0.20	**0.30**
C_f	0.30	0.50	0.25	0.20	0.20	**0.25**
C_b	0.59	0.71$^{(1)}$	1.00	0.71	0.71	**0.83**

[1]from *fib* bulletin 40 (*fib* 2007).

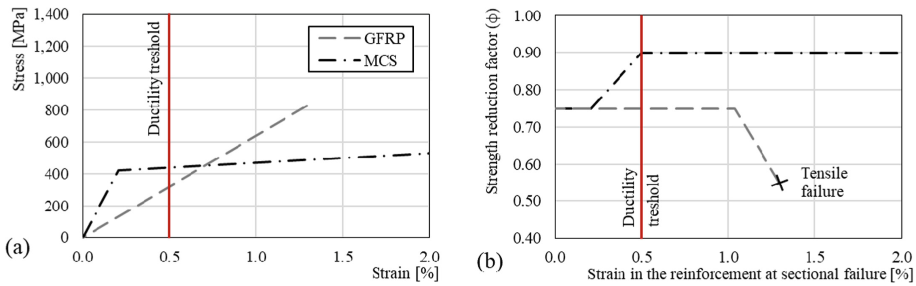

Fig. 1. Mechanical properties (a) and flexural strength reduction factors (b) for M25 bars made with GFRP and MCS.

4.3 Limit States

As for the case of MCS-RC, a GFRP-RC flexural member must be designed against a number of Ultimate Limit States (ULSs) and Service Limit States (SLSs). ULSs include compression failure of the concrete or tension failure of the GFRP bars under factored load. Furthermore, GFRP bars can experience creep rupture under sustained load, and fatigue rupture under cyclic load. These conditions are verified under service loads but represent ULSs in the sense that failure to comply may result in the catastrophic collapse of the member. SLSs include a limit on deflection (L/800 for vehicular bridges), a limit on crack width (0.7 mm), and a limit on concrete stresses under sustained load (0.45 f_c'). The relatively low stiffness of GFRP bars may result in SLSs governing the design.

According to ACI (2015) the creep rupture limit state must be verified under sustained load. The AASHTO BDS for traditional construction materials lacks the explicit definition of a sustained service load. Thus, the first edition of the AASHTO BDGS-GFRP considered the entire amount of service load as sustained. The assumption is overconservative and not aligned with international bridge design guidelines (CEN 2005a). In the second edition of AASHTO BDGS-GFRP the sustained portion of the service load is set equal to the dead load (DL) plus 20% of the live load (LL) as shown in Eq. 7. The approach is in line with ACI (2015) and more conservative with respect to international guidelines that only consider DL as sustained (CEN 2005a; CNR 2007).

$$Sustained\ Load = DL + 0.20\,LL \tag{7}$$

According to ACI (2015) the fatigue rupture limit state must be verified under the sum of the sustained load plus the maximum load experience in a fatigue cycle. In translating this provision to AASHTO language, the total fatigue load is defined as the sum of the Dead Load (DL) plus the factored transient loads defined per AASHTO (2017) Fatigue load combination Fatigue I (F1). The fatigue load combination is reported in Eq. 8. The issue of load combinations for creep rupture and cyclic fatigue is also discussed by Rossini et al. (2018a).

$$Fatigue\ Load = DL + F1 \tag{8}$$

4.4 Philosophy and Applicability

The RC and PC design section of the AASHTO BDS has recently underwent a major update (Montgomery et al. 2017). The second edition of the AASHTO BDGS-GFRP is compatible with the most recent edition of the AASHTO BDS (AASHTO 2017). The GFRP counterpart reflects the structure and organization of the main document and minimizes the differences in design equations to ease application by practitioners. Differences are limited to adjusting design parameters and material properties to account for the different behavior of GFRP bars with respect to MCS. Furthermore, the second edition of the AASHTO BDGS-GFRP is meant for application along with the material specifications published by ASTM (2017). This sets the first example for the next generation of integrated GFRP-RC design, construction, and material guidelines to be consistently developed without overlapping.

The major limitation of the first edition of the AASHTO BDGS-GFRP (AASHTO 2009) laid in the limited field of application as it only covered bridge decks and open-post railings. The second edition of the AASHTO BDGS-GFRP covers all the members that compose a RC bridge. This includes decks, girders, bent caps, bulkhead caps, bearing piles, sheet piles, gravity walls, open-post railings, continuous railings, and approach slabs. It is the first regulation to cover GFRP-RC substructure, and is the most complete guideline for GFRP-RC design. Its provisions have been developed and tested on a number of structures currently built or under construction. This includes the Innovation Bridge discussed by Rossini et al. (2018b), and the Halls River Bridge discussed by Rossini et al. (2018a).

5 Parametric Analysis

In developing the draft for the second edition of the AASHTO BDGS-GFRP, parametric analysis was used as a tool to quantify the effect of the proposed variation in the design parameters. In the following, a selection of the results of the parametric analysis is discussed. The methodology adopted will be briefly summarized. For more details, reference is made to Rossini et al. (2018a).

The study focuses on the GFRP-RC pile cap of the Halls River Bridge currently under construction in Homosassa, FL (Rossini et al. 2018a) (Fig. 2). The element is deemed representative of large under-reinforced GFRP-RC members acting as pile caps in short-spanned traffic bridges. Given their exposure condition and proximity to water surfaces, these members represent typical applications for GFRP-RC and are of particular interest for FDOT.

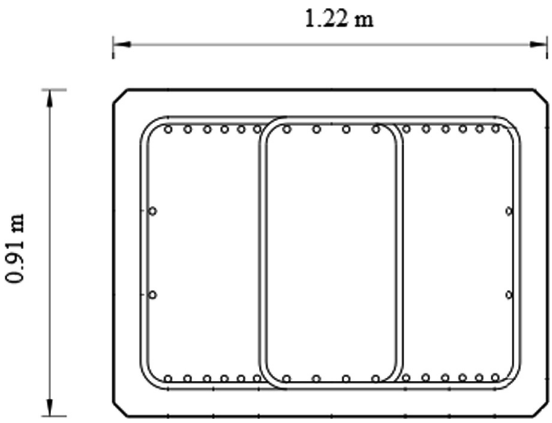

Fig. 2. Transversal section of the pile cap of the Halls River Bridge.

Each parametric curve is constructed by calculating the minimum number of M25 GFRP bars that satisfies the specific design requirement: moment capacity, minimum reinforcement, creep rupture, fatigue, and crack width limits. The design for positive moment capacity based on ACI (2015) and AASHTO (2009) resulted in 16 M25 bars with a guaranteed strength of 550 MPa and an elastic modulus of 45 GPa, for a total area of 8084 mm^2 to resist a factored moment demand of 575 kN-m. The design for positive moment capacity based on the second edition of AASHTO BDGS-GFRP resulted in 9 M25 bars, for a total area of 4547 mm^2. This corresponds to a reduction of 40% with respect to the first edition (Rossini et al. 2018b).

Figure 3 shows the influence of the variation of a selection of parameters on the required amount of reinforcement. The design demand is represented in terms of required number of longitudinal M25 bars. For each diagram, design equations are plotted as a function of the selected range. The remaining parameters are set constant and equal to values recommended in the first and second edition of the AASHTO BDGS-GFRP respectively. The results presented in Fig. 3 are case-dependent, but the trend of the curves is indicative in general.

Comparing Fig. 3a and Fig. 3b shows how the rationalization of the sustained and cyclic load demand discussed in Sect. 4.3 reduces the influence of the cyclic fatigue and creep rupture requirements from governing to negligible. Relaxation of the creep rupture reduction factor (C_c) from 0.20 to 0.30, and the fatigue reduction factor (C_f) from 0.20 to 0.25 contributes to this outcome, but the effect is limited as shown in Fig. 3b.

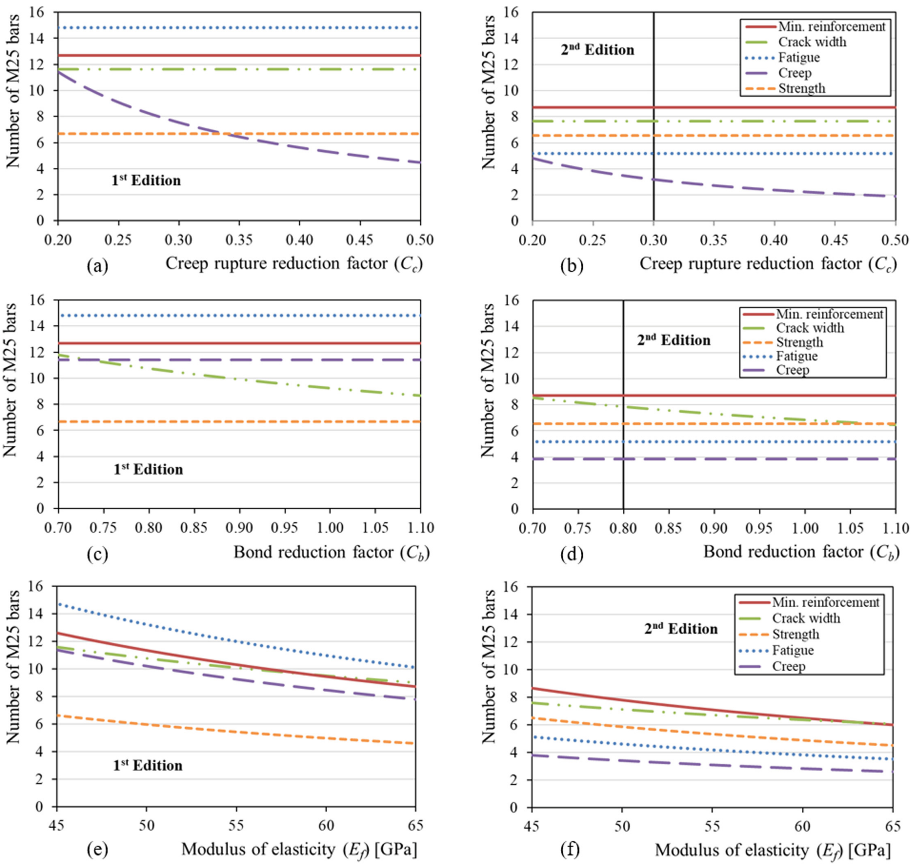

Fig. 3. Required number of M25 bars as a function of the variation of the design parameters using the equations of AASHTO BDGS-GFRP 1st edition (left) and AASHTO BDGS-GFRP 2nd edition (right).

Experimental results suggest that further margin for improvement exists, but additional research is required. The prioritization of research into creep rupture and cyclic fatigue endurance limits is suggested by the limited database available and by mechanical considerations discussed by Rossini et al. (2018a).

The crack width requirement governs over the strength requirement as show in Fig. 3c and Fig. 3d. A relaxation of the bond reduction factor (C_b) from 0.71 to 0.83, along with a relaxation of the crack width limit (w) from 0.5 mm to 0.7 mm, and a relaxation of the minimum concrete clear cover (c_c) from 51 mm to 38 mm, reduces the required amount of reinforcement to fulfill SLSs by about 30%.

The design of large section MCS-RC members is typically governed by minimum reinforcement considerations. This is not the case for GFRP-RC members designed according to the first edition of AASHTO BDGS-GFRP. Conversely, the minimum reinforcement requirement governs the design of GFRP-RC large section members according to the draft of the second edition of AASHTO BDGS-GFRP. This follows the rationalization of the sustained and cyclic load demand discussed in Sect. 4.3. Furthermore, the minimum reinforcement requirement has been aligned to the formulation adopted in AASHTO BDS (2017). This approach ensures a minimum level of strength and ductility to the system as a function of the mechanical properties of the reinforcement, and offsets overconservativeness in some cases. Details are discussed by Rossini et al. (2018a, b).

Figure 3e and Fig. 3f show how improving the stiffness (E_f) – and therefore the strength (f^*_{fu}) – of GFRP bars amplifies the benefits of the proposed refinements in design limits. However, the increment in the elastic modulus should not come from a mere increase of the effective cross-sectional area compared to the nominal design area, but rather a combination of increased fiber ratio, improved material properties, and superior manufacturing quality control.

An improved quality control of the product may help refining most of the design parameters discussed but needs to be reflected in more performing material specifications (ASTM 2017) before designers can take full advantage of it.

6 Conclusions

In this study the salient contents of the draft of the second editions of the AASHTO BDGS-GFRP are discussed along with the conceptual framework functional to the development of the document. Specific emphasis is devoted to flexural members. The differences with respect to the first edition are quantified resorting to parametric analysis.

The second edition of AASHTO BDGS-GFRP (AASHTO 2018) aims to provide a rational and consistent framework for the design of GFRP-RC bridge structures. This is expected to raise awareness and leverage wider deployment of non-corrosive reinforcement solutions in infrastructures. Furthermore, the definition of a consistent regulatory framework is expected to help define and prioritize Research and Development (R&D) areas – at the academic, private, and regulatory level – to make the technology more efficient, economical and environmentally appealing (Rossini et al. 2018a). Specific features of the document include:

1. Design parameters and procedures have been updated to reflect advancements in the state-of-the-art. This includes refinement of the strength reduction factor for compression-controlled failures (ϕ_c); refinement of the creep rupture reduction factor (C_c), fatigue reduction factor (C_f), and bond reduction factor (C_b).
2. Design demands and limit states have been made more rational and consistent with national and international guidelines (CEN 2005a; CNR 2007; ACI 2015; AASHTO 2017). This offsets overconservativeness in creep rupture and cyclic fatigue demands.
3. Design equations have been updated to align the document to the most recent edition of the AASHTO Bridge Design Specifications (AASHTO 2017). This creates a familiar environment for the practitioners approaching GFRP-RC for the first time and resolve some inconsistencies.
4. The document is expanded to include all the reinforced concrete components of a bridge structure. The first edition only included bridge decks and open-post railings. It is the first guideline to include provisions for GFRP-RC substructures.
5. The document is structured to automatically benefit from any refinement in material specifications issued by ASTM (2017). This would not be the case if an additional material specification chapter was to be introduced as done in the first edition.

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