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Design and Detailing of Glass Fiber-Reinforced Polymer-Reinforced Concrete Beams According to ACI CODE-440.11-22

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This paper aims to analyze practical considerations in the design of glass fiber-reinforced polymer-reinforced concrete (GFRP-RC) beams based on the newly adopted ACI CODE-440.11-22, addressing strength, serviceability, and detailing criteria. A beam example was taken from the ACI Reinforced Concrete Design Handbook *and redesigned using GFRP bars and stirrups to analyze the effect of changing the reinforcement type. In the first phase, the beam was designed as an over-reinforced member with high-modulus (E_f = 60,000 MPa) and low-modulus (E_f = 44,815 MPa) GFRP bars. In the second phase, a parametric study was carried out to analyze the impact of changing key design parameters—namely, bond factor k_b, concrete compressive strength f_c', and the maximum deflection limit. GFRP-RC beams require more reinforcement area compared to conventional steel-RC, which may result in bar congestion. Current Code provisions related to detailing in particular are based on conservative assumptions due to a lack of experimentation and greatly penalize the design of GFRP-RC beams. The current Code provisions for development length, bar spacing, skin reinforcement, and stress at service make GFRP-RC design challenging.*

Keywords: building code; detailing; glass fiber-reinforced polymer (GFRP) reinforcement; reinforced concrete (RC) beams; serviceability.

INTRODUCTION

A primary reason for the limited use of glass fiber-reinforced polymer (GFRP) bars in concrete structures has been the lack of engineering design standards. However, with recent developments, owners and practitioners are finding GFRP to be a viable alternative to conventional steel in reinforced concrete (RC) structures for long-term service life.[1] The improvement in material properties, available standards, and new construction strategies allow the exploitation of the full potential of this composite material[2,3] for use in concrete structures.

The Building Code ACI CODE-440.11-22 for GFRP-RC members was recently published, which represents a critical aid to practitioners interested in the use of nonmetallic reinforcement.[4] However, some provisions in ACI CODE-440.11-22[4] may be based on conservative assumptions without validation from experimental programs. These provisions make the design of GFRP-RC members difficult and may require unnecessary reinforcement. Therefore, this study is carried out to show the implications of current Code provisions that may need to be revisited and validated by experimentation.

RESEARCH SIGNIFICANCE

The ACI CODE-440.11-22[4] Building Code for GFRP-RC members is a stepping stone for the full exploitation of composites in concrete construction. However, some Code provisions are possibly unduly conservative and penalize the design. Assumptions for detailing such as development length and bar spacing make implementation difficult. This study analyzes and discusses practical considerations for GFRP-RC beam design and detailing and points out a need for some reconsideration.

METHODOLOGY

In this study, a beam from the *ACI Reinforced Concrete Design Handbook*[5,6] was selected and redesigned using GFRP reinforcement. The selected beam is part of an interior, continuous, six-bay framing, and built integrally with a 178 mm-deep slab, as shown in Fig. 1. The constituent materials selected for beam design are listed in Table 1. The concrete strength f_c' is 35 MPa, while the GFRP type is compliant with material specification ASTM D7957/D7957M-22.[7] Additionally, a new ASTM material specification is under development for a class of GFRP bars with a higher modulus of elasticity and strength; this class of GFRP bars was also considered because it represents the majority of products commercially available in the marketplace today. This study uses M29 nominal bar size for the main reinforcement in both the positive and negative moment regions, whereas for additional hooked bars, M16 and M19 sizes are used as needed. The mechanical properties of GFRP bars affecting design include guaranteed ultimate tensile strength f_{fu}, corresponding ultimate strain ε_{fu}, modulus of elasticity E_f, and modular ratio n_f. A value of 1.35 for the bond coefficient (k_b) and 0.85 for the environmental reduction factor (C_E) are adopted, as indicated in ACI CODE-440.11-22[4] Sections 24.3.2.3 and 20.2.2.3, respectively. It should be noted that the bond factor has changed from 1.35 to 1.20 in the recent publication of ACI CODE-440.11-22,[4] which is used in Phase 2. A concrete cover (c_c) of 38 mm is used, as specified in ACI CODE-440.11-22[4] Section 20.5.1.3.1.

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Table 1—Properties of GFRP reinforcement and concrete

Designation	Nominal diameter, mm	Nominal area, mm^2	Elastic modulus, MPa	Guaranteed tensile strength, MPa	Ultimate strain, %	Concrete strength, MPa	Concrete clear cover, mm
GFRP-M16*	15.8	200	60,000	907.5	0.015	35.0	38.0
GFRP-M19*	19.0	284		897.7	0.015		
GFRP-M29*	28.6	645		793.0	0.013		
GFRP-M16	15.8	200	44,815	646.7	0.014		
GFRP-M19	19.0	284		640.5	0.014		
GFRP-M29	28.6	645		565.3	0.013		

*New-generation bars with high modulus of elasticity (E_f = 60,000 MPa).

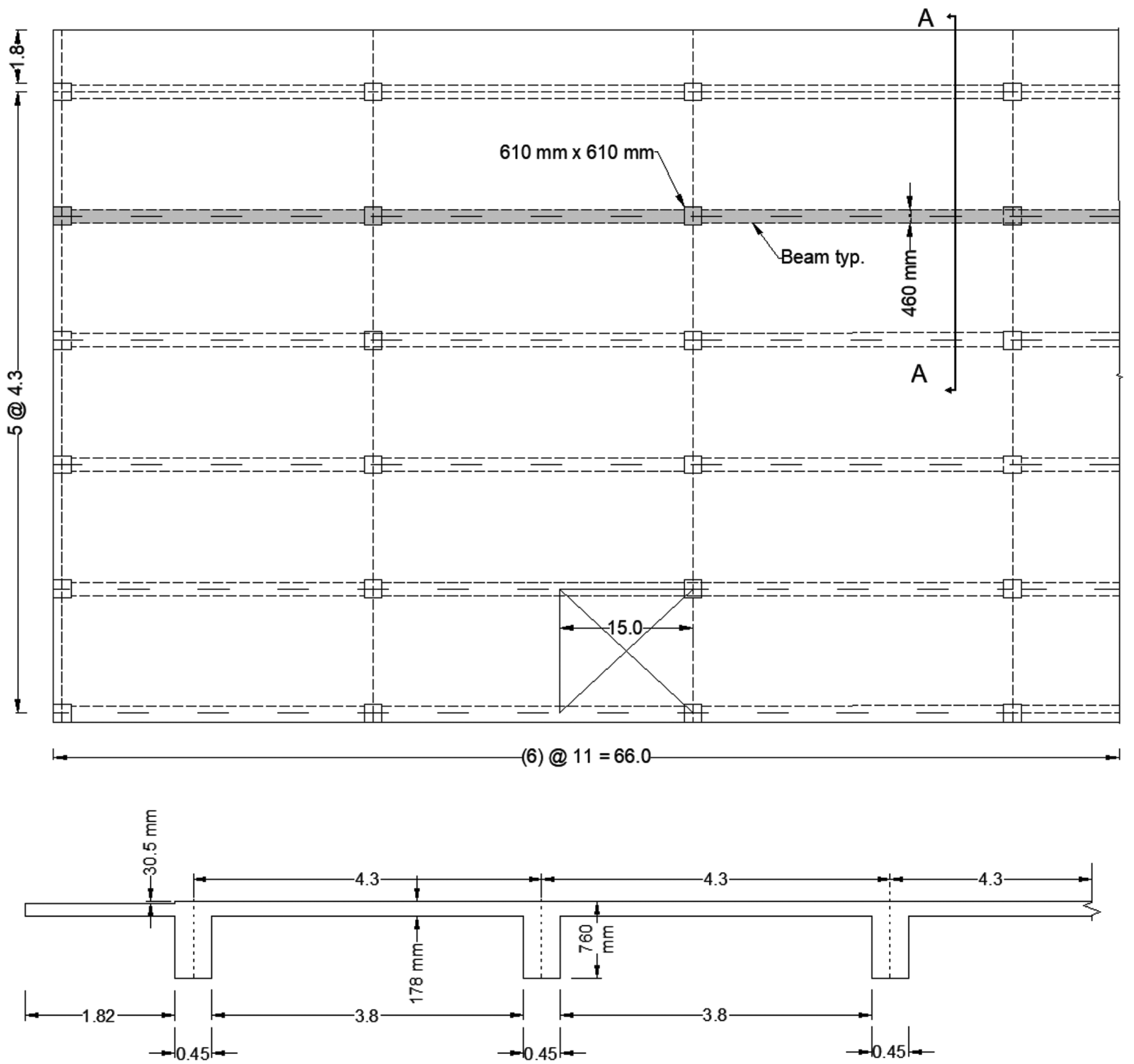


Fig. 1—Framing plan and partial section A-A showing interior beam.[6] (Note: Dimensions in meters unless otherwise indicated.)

The beam is designed as an over-reinforced member (that is, the reinforcement ratio provided exceeds the balanced reinforcement ratio) with both high- and low-modulus GFRP bars. For the former case, a parametric study is carried out by changing parameters such as the bond factor k_b, concrete compressive strength f_c', and the maximum permissible deflection limit.

Table 2—Selected moments and shear values for one-way slabs and beams (ACI CODE-440.11-22[4] Table 6.5.2)

Moment	Location	Condition	M_u	V_u
Positive	Endspan	Discontinuous end integral with support	$\frac{w_u l_n^2}{14}$	—
Negative	Interior face of exterior support	Members built integrally with supporting column	$\frac{w_u l_n^2}{16}$	$w_u l_n/2$
	Exterior face of first interior support	More than two spans	$\frac{w_u l_n^2}{10}$	$1.15(w_u l_n/2)$

ANALYSIS AND DESIGN

The beam carried a superimposed dead load of 718 N/m² and a live load of 3112 N/m², as given in the design Handbook.[6] These loads were combined as per ASCE/SEI 7-16[8] to compute the maximum factored demand. Maximum factored moments and shear forces were determined using a simplified method of analysis for continuous beams and one-way slabs as per ACI CODE-440.11-22[4] Section 6.5. The specified moment and shear values used in this example are given in Table 2 as taken directly from ACI CODE-440.11-22.[4]

For applicable factored load combinations, design strength at all sections shall satisfy the requirements of ACI CODE-440.11-22[4] Section 9.5.1.1, given as follows

$$\Phi S_n \geq U \tag{1}$$

where S_n is the nominal moment, shear, axial, or torsional strength; U is the strength of a member or cross section required to resist factored loads or related internal moments and forces; and Φ is the strength reduction factor calculated as per ACI CODE-440.11-22,[4] as given in Table 3.

The maximum spacing of GFRP reinforcement is limited, as specified by ACI CODE-440.11-22[4] Eq. (24.3.2a) and (24.3.2b), given as follows

$$S \leq \frac{0.81E_f}{f_{fs}k_b} - 2.5c_c \tag{2}$$

$$S \leq 0.66\frac{E_f}{f_{fs}k_b} - 2.5c_c \tag{3}$$

where f_{fs} is the stress at service loads.

The development length of the GFRP reinforcement is governed by Code Section 25.4.2.1, as the greater of: (a), (b), and (c), given as follows in Eq. (4) to (6)

$$l_d = \frac{d_b\left(\dfrac{f_{fr}}{0.083\sqrt{f_c}} - 340\right)}{13.6 + \dfrac{c_b}{d_b}}\omega \tag{4}$$

where f_{fr} is the tensile stress in GFRP reinforcement required to develop the full nominal section capacity, MPa; c_b is the lesser of: a) the distance from the center of a bar to the nearest concrete surface, and b) one-half the center-to-center spacing of bars being developed, mm; d_b is the nominal diameter of the bar, mm; and ω is the bar location modification factor, taken equal to 1.5 if more than 300 mm of fresh concrete is placed below the horizontal reinforcement being developed, and 1.0 for all other cases.

Table 3—Strength reduction factor Φ (ACI CODE-440.11-22[4] Section 21.2.1)

Action or structural element	Φ
Moment, axial force, or combined axial moment and axial force (Section 21.2.2)	0.55 to 0.65*
Shear	0.75

*0.65 is applicable to over-reinforced sections used in this example.

$$20d_b \tag{5}$$

$$300 \text{ mm} \tag{6}$$

There are no provisions for predetermined dimensions of beams in ACI CODE-440.11-22[4] as given in ACI 318-19 Section 9.3.1.1. Therefore, the GFRP-RC beam cross-section dimensions were determined by the trial-and-error method meeting strength and serviceability requirements. The beam cross-section dimensions are identical to those in the design Handbook,[6] and a maximum permissible deflection limit was selected in the first phase of this study as per ACI CODE-440.11-22[4] Section 24.2.2, given as follows

$$\Delta = l/240 \tag{7}$$

This limit is based on the assumption that the beam is not supporting or attached to partitions or other nonstructural elements likely to be damaged by large deflections. The aforementioned deflection limit was taken to make it analogous to the ACI 318-19[5] design taken in this study.

STRENGTH REQUIREMENTS

Flexural strength

The reinforcement area was calculated first as the greater of the area required by the ultimate factored moment demand and the area necessary to ensure that the flexural strength exceeds the cracking strength, indicated in ACI CODE-440.11-22[4] Sections 9.6.1.2(a) and (b), given as follows

$$\frac{0.41\sqrt{f_c}}{f_{fu}}b_w d \tag{8}$$

$$(2.3/f_{fu})b_w d \tag{9}$$

where b_w is the web width or diameter of the circular cross section, mm; and d is the distance from extreme compression fiber to centroid of longitudinal tension reinforcement, mm.

In this example, the factored moment was calculated for the superimposed dead load of 718 N/m² and live load of 3112 N/m² (that is, larger than the minimum 1915 N/m² of a residential load given by ASCE 7-16 Table 4.3-1[8]).

Table 4—Design of GFRP-RC beam using high-modulus bars

Location	Demand, kN·m	Area* for strength only, mm^2	Capacity*, kN·m	Provided area, mm^2, meeting all Code requirements	Provided capacity, kN·m, meeting all Code requirements
Exterior support	362	1935	466	2580	639
Midspan	413	1935	466	3226	885
Interior support	579	2580	639	3870	1064

*Reinforcement area and capacity without meeting Code provisions for strength, detailing, and serviceability.

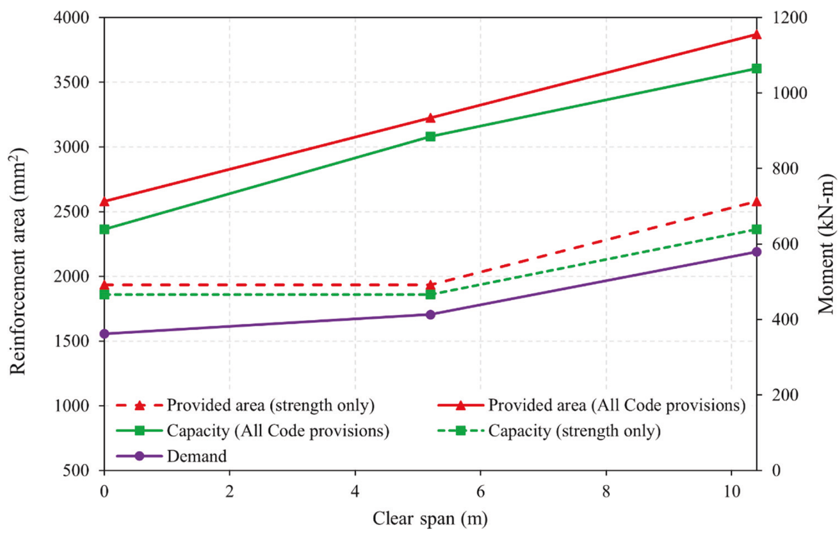


Fig. 2—Reinforcement area, demand, and capacity at three locations, with and without meeting Code provisions.

As shown in Table 4, the required reinforcement area for strength (that is, 1935, 1935, and 2580 mm^2 at exterior support, midspan, and interior support, respectively) produces a capacity large enough to satisfy the factored demand. However, Code provisions for maximum spacing, stress at service, deflection limits, and strength reduction factors penalize the design. As given in Table 4, the provided reinforcement area significantly increased (that is, 2580, 3226, and 3870 mm^2 at exterior support, midspan, and interior support, respectively) after meeting Code provisions for detailing and serviceability. The resulting capacity is 76%, 114%, and 84% higher than demand at the exterior support, midspan, and interior support, respectively. The difference between the required and provided reinforcement areas with and without meeting Code provisions, together with corresponding capacities, can be visualized in Fig. 2. Also, when satisfying Code specifications, the design changed from an under-reinforced to an over-reinforced member.

Shear strength

Separate equations are provided in ACI CODE-440.11-22[4] to avoid diagonal compression failure (Eq. (22.5.1.2), given as follows) and to limit the strain in the GFRP shear reinforcement (Section 20.2.2.6, provided later in this section)

$$V_u \leq \Phi 0.2 f_c' bd \tag{10}$$

where V_u is the factored shear force at a section, N.

The nominal shear strength of the beam was calculated as per ACI CODE-440.11-22[4] Eq. (22.5.1.1), given as

$$V_n = V_c + V_f \tag{11}$$

where V_n is the nominal shear strength, N; V_c is the nominal shear strength provided by the concrete, N; and V_f is the nominal shear strength provided by GFRP shear reinforcement, N.

The shear strength provided by concrete was calculated as the greater of two expressions from ACI CODE-440.11-22[4] Sections 22.5.5.1(a) and (b), given as follows

$$V_c = 0.42\lambda k_{cr}\sqrt{f_c'}bd \tag{12}$$

$$V_c = 0.066\lambda\sqrt{f_c'}bd \tag{13}$$

where $\lambda = \sqrt{\dfrac{2}{1 + 0.004}}$ is the size effect factor, as given in ACI CODE-440.11-22[4] Section 22.5.1.1; and k_{cr} is the ratio of depth of the elastic cracked section neutral axis to the effective depth given by Commentary Eq. (R22.5.5.1a), shown as follows

$$k_{cr,rect} = \sqrt{2\rho_f n_f + (\rho_f n_f)^2} - \rho_f n_f \tag{14}$$

where $\rho_f = A_f/b_w d$ is the reinforcement ratio; A_f is the area of GFRP longitudinal reinforcement within spacing s, mm^2; $n_f = E_f/E_c$ is the modular ratio; and E_c is the modulus of elasticity of concrete, MPa, calculated as given by Code Eq. (19.2.2.1b), given as follows

Table 5—Properties of high-modulus GFRP shear reinforcement

Designation	Nominal diameter, mm	Nominal area, mm^2	Elastic modulus, MPa	Guaranteed tensile strength, MPa	Design tensile strength, MPa	Quantity
GFRP-M13	12.7	129	60,000	574.3	490	46

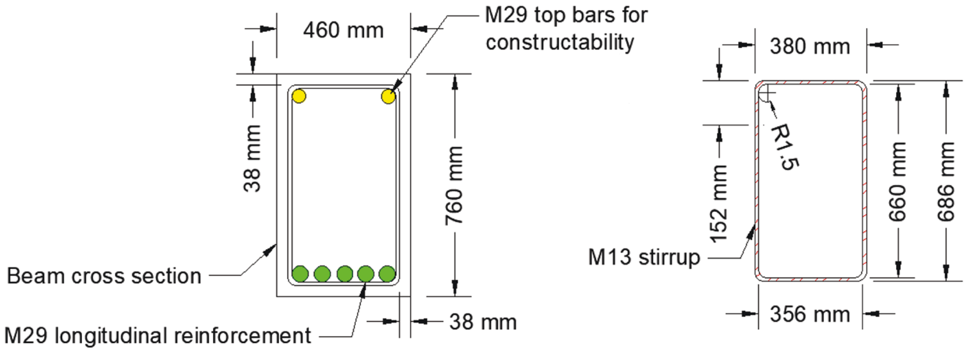


Fig. 3—Beam cross section and stirrup dimensions at midspan.

$$E_c = 4700\sqrt{f_c'} \quad (15)$$

The size effect factor was considered in the beam design because its depth exceeded 254 mm.

The ultimate factored shear force exceeded the concrete strength and the beam required shear reinforcement. Shear strength provided by the GFRP reinforcement was calculated as given in Code Eq. (22.5.8.5.3)

$$V_f = A_{fv}f_{ft}(d/s) \quad (16)$$

where A_{fv} is the area of shear reinforcement calculated as given in the Commentary Eq. (R22.5.8.5), given as follows

$$\frac{A_{fv}}{s} = \frac{V_u - \Phi V_c}{\Phi f_{ft} d} \quad (17)$$

where f_{ft} is the permissible stress in the GFRP shear reinforcement. The design tensile strength of GFRP transverse reinforcement is controlled by the strength of the bent portion of the bar and by a strain limit of 0.005, as given by Code Section 20.2.2.6

$$f_{ft} \leq (f_{fb}, 0.005E_f) \quad (18)$$

where $f_{fb} = C_E f_{fb}^*$ is the design tensile strength of the bent portion of GFRP reinforcement; and f_{fb} is the guaranteed ultimate tensile strength of the bent portion of the bar. Its minimum value is taken as specified in ASTM D7957/D7957M[7] by dividing the ultimate guaranteed tensile force of the bent portion of the bar by the nominal cross-sectional area of the bar.

The maximum spacing between legs of shear reinforcement was calculated as the least of the maximum spacing limitations given by the Code and its Commentary in Sections R22.5.8.5.3, 9.6.3.4, and 9.7.6.2.2.

$$S_{max} = \frac{A_{fv}\Phi f_{ft} d}{V_u - \Phi V_c} \quad (19)$$

Following the example in the Design Handbook,[6] torsion effects were not considered; therefore, maximum spacing was limited, as given in Code Sections 9.6.3.4(a) and (b), shown as follows

$$S_{max} = \frac{A_{fv}f_{ft}}{0.062\sqrt{fc}b} \quad (20)$$

$$S_{max} = A_{fv}f_{ft}/0.35b \quad (21)$$

The final limit for the spacing between the legs of shear reinforcement is given in Code Section 9.7.6.2.2, shown as follows

$$S_{max} = \min((d/2), 610 \text{ mm}) \quad (22)$$

A lower value of V_c and a 40% reduction in the strength at the bend of GFRP transverse reinforcement[7] significantly affect shear design, and members using GFRP shear reinforcement require more and larger-diameter stirrups than for the case of steel stirrups. In this example, the beam designed with GFRP required 46 M13 GFRP stirrups, whereas the same beam required 35 M10 steel stirrups. The properties of GFRP shear reinforcement are listed in Table 5. For anchorage, continuous closed stirrups were used as defined in Code Section 25.7.1.3. The radius of the bend for an M13 stirrup used was 38 mm, as per Table 4 in ASTM D7957/D7957M.[7] The stirrup size, its dimensions, and a beam cross section at a typical location are shown in Fig. 3. The shear demand due to factored loads on the beam and shear strength provided by concrete and shear reinforcement can be visualized in shear demand and capacity envelopes given in Fig. 4 together with the stirrup number, size, and spacing varying along the beam length.

DETAILING AND SERVICEABILITY REQUIREMENTS

Design of beam using high-modulus GFRP bars (E_f = 60,000 MPa)

The beam cross section (that is, 460 x 760 mm) was designed as compression-controlled using dimensions identical to the steel-RC beam in the Handbook.[6] The amount of

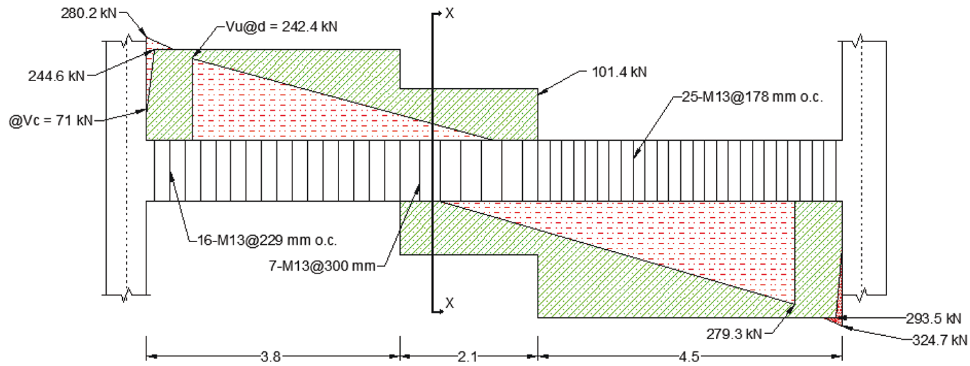


Fig. 4—Shear demand and capacity envelopes.

Table 6—Design of GFRP-RC beam using high-modulus bars (60,000 MPa)

Location	Demand, kN·m	Area for strength only, mm^2	Provided area, mm^2, meeting all Code requirements	Provided capacity, kN·m, meeting all Code requirements	Development length, mm
Exterior support	362	2348	2580	639	2769
Midspan	413	2348	3226	885	1626
Interior support	579	2348	3870	1064	1423

GFRP reinforcement to satisfy strength requirements is indicated as the "required area," whereas the larger amount of GFRP reinforcement needed to satisfy serviceability requirements (that is, deflection control) and detailing requirements (that is, maximum bar spacing) is indicated as the "provided area." The difference between the required and provided areas of reinforcement to meet serviceability and detailing requirements can be observed in Table 6, developed using high-modulus (E_f = 60,000 MPa) GFRP reinforcement.

The required reinforcement area for the negative moment at the exterior support to meet strength requirements is 2348 mm^2, while the provided area increased to 2580 mm^2 to meet the detailing requirements as well. However, the developed capacity at the face of the column is 132 kN·m, lower than the demand of 362 kN·m, as GFRP bars are not fully developed at this location. For full capacity, the GFRP reinforcement needed a development length of 2769 mm, while only 2572 mm is available at the face of the support. Because long M29 bars cannot terminate with a hook, three M19 hooked bars were used to satisfy the demand, thus increasing the provided area to 3420 mm^2 while creating some congestion at this location.

Similarly, at the interior support, the required negative-moment reinforcement area is 2348 mm^2, whereas the provided area is 3870 mm^2, an increase in reinforcement area of 1522 mm^2 over that required for strength. This increase in reinforcement area at the interior support is due to the need of meeting the maximum spacing limitation of the Code, governed by Eq. (24.3.2a) and (24.3.2b).

The required positive-moment reinforcement area at midspan was 2348 mm^2, but the provided area has to increase to 3226 mm^2 to satisfy Code provisions for maximum spacing limits. ACI CODE-440.11-22[4] Section 9.7.3.8.2 requires that one-fourth of the maximum positive-moment reinforcement be extended along the beam bottom into the support. Therefore, two M29 bars were extended into the column. Also, Code Section 9.7.7.4 requires that longitudinal integrity reinforcement at noncontinuous supports be anchored to develop f_{fu} (ultimate guaranteed tensile strength) at the face of the support. To develop a full capacity of 872 kN, GFRP bars required a development length of 2007 mm, with only 572 mm available with a corresponding force of 250 kN. Therefore, three M19 GFRP hooked bars were used to enhance the capacity at the face of the edge column to 898 kN. The required area (one-fourth of positive reinforcement) was 806 mm^2, whereas the provided area at the face of the column increased to 2142 mm^2.

The longitudinal skin reinforcement was provided as required by Code Section 9.7.2.3 (that is, skin reinforcement should be uniformly distributed on both side faces for beams exceeding 458 mm to a distance of $h/2$ from the tension face). The Code provisions in Section 24.3.2 limit the maximum spacing of skin reinforcement; therefore, four M10 GFRP bars were used in this beam at 114 mm center-to-center spacing on each face.

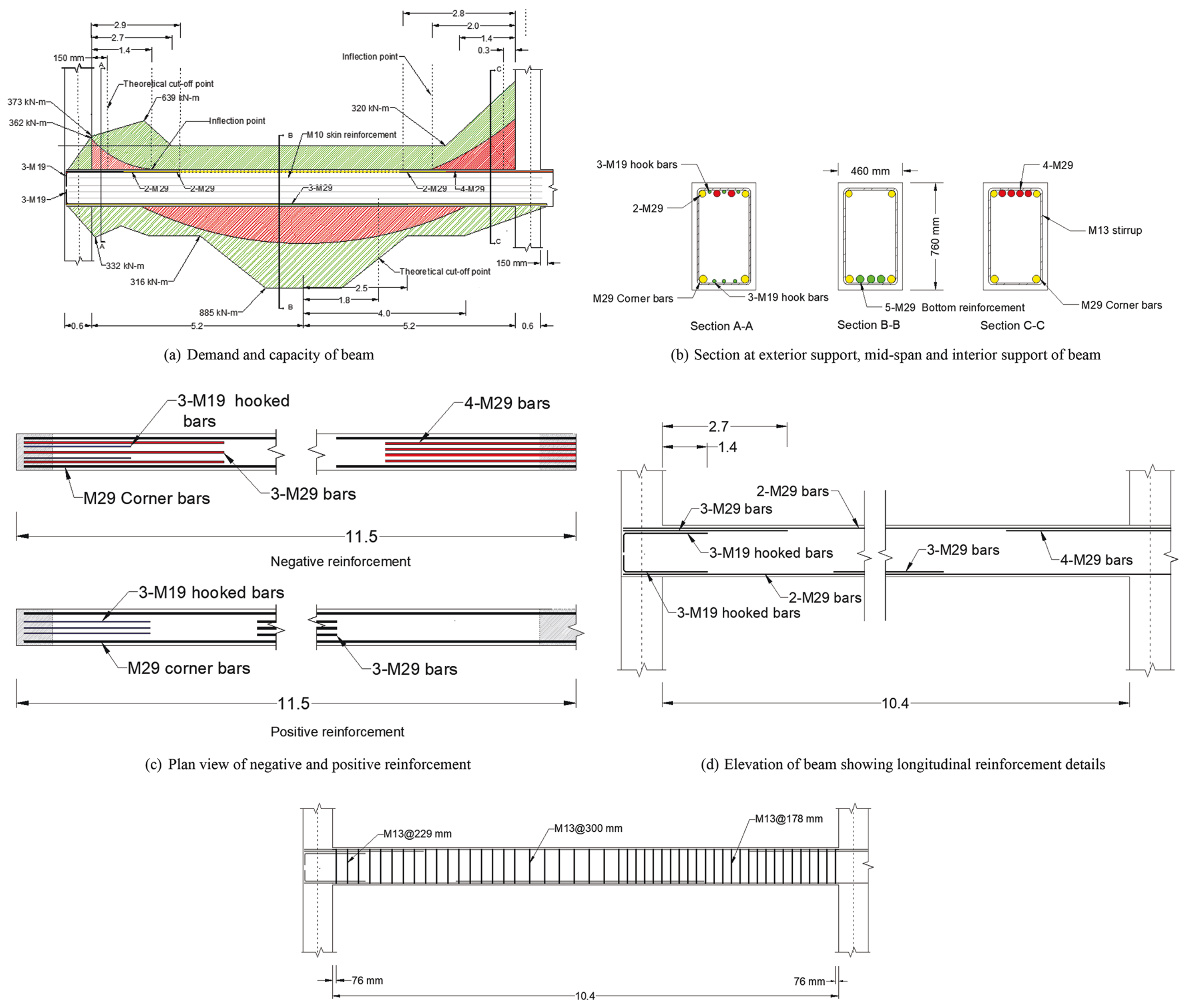
The required and provided area of reinforcement at the exterior support, midspan, and interior support and corresponding development length values are listed in Table 6. Figure 5(a) shows the detailing of the reinforcement, theoretical cutoff points, and inflection points for three different sections. Demand and capacity along the length of the beam are shown, with the latter being much larger than demand because of the Code provisions for detailing. Figures 5(a) to (e) present the reinforcement details at three locations, the plan view of positive and negative reinforcement, the elevation of the beam showing longitudinal reinforcement details, and the elevation of the beam showing shear reinforcement.

Design of beam using low-modulus GFRP bars (E_f = 44,815 MPa)

By making explicit reference to ASTM D7957/D7957M,[7] ACI CODE-440.11-22[4] is currently based on low-modulus

Table 7—Design of GFRP-RC beam using currently specified low-modulus bars (E_f = 44,815 MPa)

Location	Demand, kN·m	Area for strength only, mm^2	Provided area, mm^2, meeting all Code requirements	Provided capacity, kN·m, meeting all Code requirements	Development length, mm
Exterior support	362	3420	3870	690	1752
Midspan	413	3420	3870	885	1168
Interior support	579	3420	6452	1063	990



(a) Demand and capacity of beam

(b) Section at exterior support, mid-span and interior support of beam

(c) Plan view of negative and positive reinforcement

(d) Elevation of beam showing longitudinal reinforcement details

(e) Elevation of beam showing shear reinforcement

Fig. 5—Demand and capacity envelopes, beam dimensions, and reinforcement details. (Note: Units in meters unless otherwise indicated.)

(E_f = 44,815 MPa) GFRP bars, despite the availability of new-generation high-modulus bars. Therefore, this study also investigated the use of the currently specified bars to evaluate the effect of lower elastic modulus on design and detailing. As old-generation bars have lower strength and stiffness values, the minimum required reinforcement area increased from 2348 mm^2, using high-modulus GFRP bars as shown in Table 6, to 3420 mm^2 with old-generation bars, as given in Table 7. It can be observed that the provided area at the exterior support is 3870 mm^2 greater than the required 3420 mm^2; however, it produced a capacity of 225 kN·m at the face of the column against a demand of 362 kN·m due to higher development length values. To enhance the capacity at the face of the exterior column, three M19 bars were used, increasing the provided area to 4710 mm^2 and the capacity to 398 kN·m at the face of the column.

Similarly, the interior support required reinforcement area for strength was 3420 mm^2. However, the bond stresses were

Table 8—GFRP-RC beam using high-modulus bars with different k_b values

Location	Area for strength only, mm^2	k_b = 1.35		k_b = 1.20		k_b = 1.05	
		Required area, mm^2, without meeting serviceability	Provided area, mm^2, meeting serviceability	Required area, mm^2, without meeting serviceability	Provided area, mm^2, meeting serviceability	Required area, mm^2, without meeting serviceability	Provided area, mm^2, meeting serviceability
Exterior support	1935	2580	2580	2580	2580	2580	2580
Midspan	1935	2580	3226	2580	3226	2580	3226
Interior support	2580	3870	3870	3226	3226	3226	3226

higher than the maximum specified limit in ACI CODE-440.11-22[4] Section 24.3.2.2, calculated as follows.

$$f_{fs} \leq 0.36E_f/d_cB_{cr}k_b \quad (23)$$

where d_c is the thickness of concrete cover measured from the extreme tension fiber to the center of the bar location closest thereto, mm; and B_{cr} is the ratio of the distance from the elastic cracked section neutral axis to the extreme tension fiber to the distance from the elastic cracked section neutral axis to the centroid of tensile reinforcement.

To satisfy the maximum allowed stresses at service, the provided area has to increase from 3420 to 6452 mm^2. An increase in the provided area of 3032 mm^2 (beyond the required area) makes detailing difficult. The minimum clear spacing between parallel reinforcement in a horizontal layer is specified in Code Section 25.2.1 as the least of 25.4 mm, the diameter of the bar, and four-thirds the diameter of the aggregate. To avoid violation of this limit, negative reinforcement was placed in two layers at the interior support.

The required reinforcement area at midspan was 3420 mm^2 and the provided area is 3870 mm^2, sufficient to satisfy Code requirements. Additionally, one-fourth of the positive reinforcement should extend into the support (ACI CODE-440.11-22[4] Section 9.7.3.8.2); therefore, two M29 bars were extended to both the exterior and interior supports. Code Section 9.7.7.4 states that this reinforcement should be anchored to generate f_{fu} (that is, a force equal to 618 kN) at the face of the column. To develop f_{fu}, a development length of 1270 mm was required for two M29 bars. However, with the available development length, the developed force was only 276 kN. Therefore, three M19 hooked bars were used to increase the capacity greater than f_{fu}. The required and provided areas of reinforcement and corresponding development length values at three locations are provided in Table 7. It also shows demand and capacity values for the exterior support, midspan, and interior support.

Design of beam with low-modulus GFRP bars (E_f = 44,815 MPa) when h = 660 mm

The current steel-RC beam in the Handbook[6] uses a height of 760 mm based on ACI 318-19 Section 9.3.1.1, thus automatically meeting serviceability requirements. This height value is conservative because the actual height required for deflection control is 660 mm when performing deflection calculations for steel-RC beams.

To maintain the same beam height (that is, 660 mm) when using GFRP reinforcement, 21 M29 bars would be required at midspan, which is obviously not realistic.

PARAMETRIC STUDY

In the second phase of this project, a parametric study was carried out using high-modulus GFRP bars (E_f = 60,000 MPa) by changing the values of bond factor k_b, concrete compressive strength f_c', and deflection limits while maintaining the beam cross-section dimensions equal to 460 x 760 mm.

Design of beam using high-modulus GFRP bars (E_f = 60,000 MPa) with different k_b values

The Code provisions for maximum GFRP bar spacing and stress at service loads were found to be critical design limitations. These provisions are controlled by the bond factor k_b, which was originally 1.35 (as used in Phase 1) and changed to 1.20 in the recent publication of ACI CODE-440.11-22.[4] To better understand its implications, two different k_b values (that is, 1.20 and 1.05) other than 1.35 used in Phase 1 were considered using f_c' = 35 MPa and a deflection limit of l/240, allowing the member to be under-reinforced.

It was found that changing the bond factor from 1.35 to 1.20 had a beneficial effect on the maximum allowable stress limit at service, which increased by 12.5%, and the maximum spacing limit, which also increased by 21.5% at three critical locations in the beam. Though the provided reinforcement area remains the same at the exterior support, the safety margin significantly improved. The effect of a lower bond factor k_b was more apparent at the interior support; here, the required area decreased by 17% while satisfying Code provisions for bond stresses and maximum spacing limitations, as shown in Table 8. It is worth noting that when designing beams using high-modulus GFRP bars, an additional reinforcement area equal to 1522 mm^2 was required to satisfy the maximum service stress limit at the interior support. Because k_b is directly related to the service stress limit, lowering its value showed beneficial effects, as shown in Table 8.

Similarly, reducing k_b to 1.05 increases the maximum allowable stress limit at service by 28.5% and maximum spacing by 49% compared to when using k_b = 1.35. Similar to the case of k_b = 1.20, the reduction in reinforcement areas by 17% was observed at the interior support.

It is worth noting that provided reinforcement areas are similar for three k_b values (1.35, 1.20, and 1.05) at the

Table 9—GFRP-RC beam using high-modulus bars with different concrete strengths

Location	Reinforcement area for strength only, mm²	f_c' = 20 MPa		f_c' = 35 MPa		f_c' = 50 MPa	
		Reinforcement area provided, mm²		Reinforcement area provided, mm²		Reinforcement area provided, mm²	
		Without serviceability	With serviceability	Without serviceability	With serviceability	Without serviceability	With serviceability
Exterior support	1935	2580	2580	2580	2580	2580	2580
Midspan	1935	3226	4516	3226	3226	3226	3226
Interior support	2580	3870	3870	3870	3870	3870	3870

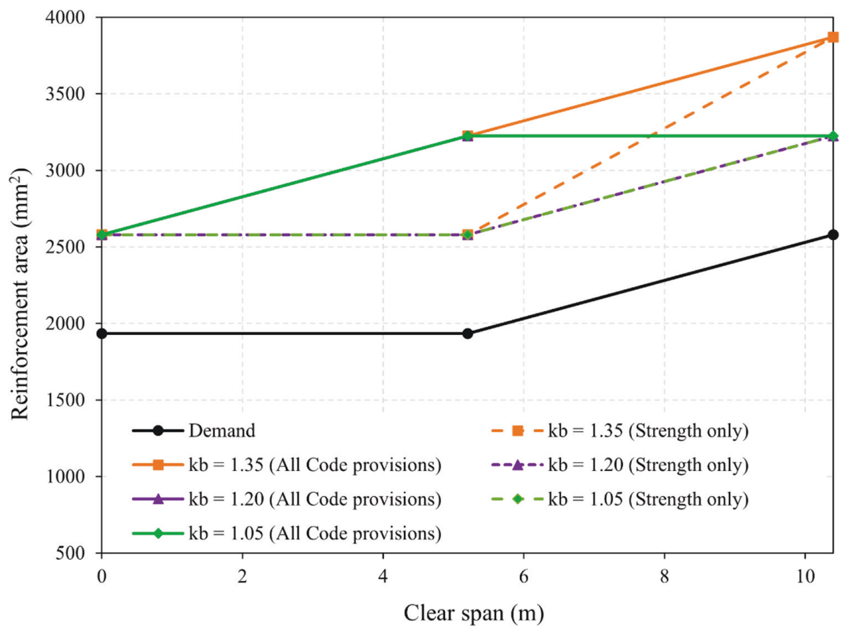


Fig. 6—Reinforcement area with different k_b *values with and without meeting serviceability requirements (lines overlap).*

exterior support and midspan, while reduction is observed at the interior support. This is because lowering k_b increased the maximum allowable spacing and stress at service limits but not enough to reduce the reinforcement area significantly. A lower k_b may not considerably affect the Code provisions for maximum spacing and stress at service. Therefore, the whole equations, especially the value of the coefficients, may need to be revisited with experimentation.

The effect of different k_b values on reinforcement requirements for stress at service and maximum spacing limits with and without meeting Code provisions is depicted in Fig. 6; the difference in the provided area is noticeable at the interior support. It should be noted that reducing the k_b value from 1.35 to 1.20 and 1.05 had some beneficial effects, which were reversed by serviceability requirements. Therefore, the lines in Fig. 6 overlap.

Design of beam using high-modulus GFRP bars (E_f = 60,000 MPa) with different f_c' values

Two different values of concrete strength were used (that is, f_c' = 20 and 50 MPa) to visualize the effects on the design of GFRP-RC members for k_b = 1.35 and a deflection limit of l/240. When the concrete compressive strength becomes 20 MPa, the design is penalized by the maximum spacing provisions of the Code, governed by stresses at service loads and k_b. Hence, as shown in Table 9, the required reinforcement area at the exterior support is 1935 mm², whereas the provided area is 2580 mm². Similarly, at the interior support, to avoid violation of the maximum spacing provisions of the Code, the reinforcement area has to increase to 3870 mm² against the minimum required 2580 mm².

The reduction in concrete strength significantly affects the serviceability requirements. As shown in Table 9, the required reinforcement area at midspan is 1935 mm²; however, to satisfy detailing constraints, it should increase to 3226 mm². Finally, this specification is aggravated by serviceability, requiring a reinforcement area of 4516 mm².

When concrete compressive strength is increased to 50 MPa, serviceability requirements were easily satisfied. As shown in Table 9, there is no difference between provided reinforcement areas with and without serviceability, implying that concrete strength has profound effects on the deflection of GFRP-RC member design, given detailing requirements are satisfied. Additionally, gains achieved by increasing the compressive strength of concrete from 35 to 50 MPa are reversed by Code provisions for maximum spacing and stress at service loads.

As shown in Fig. 7, with a concrete strength of 20 MPa, the reinforcement area at midspan is higher, indicating that GFRP-RC members are prone to more deflection at low

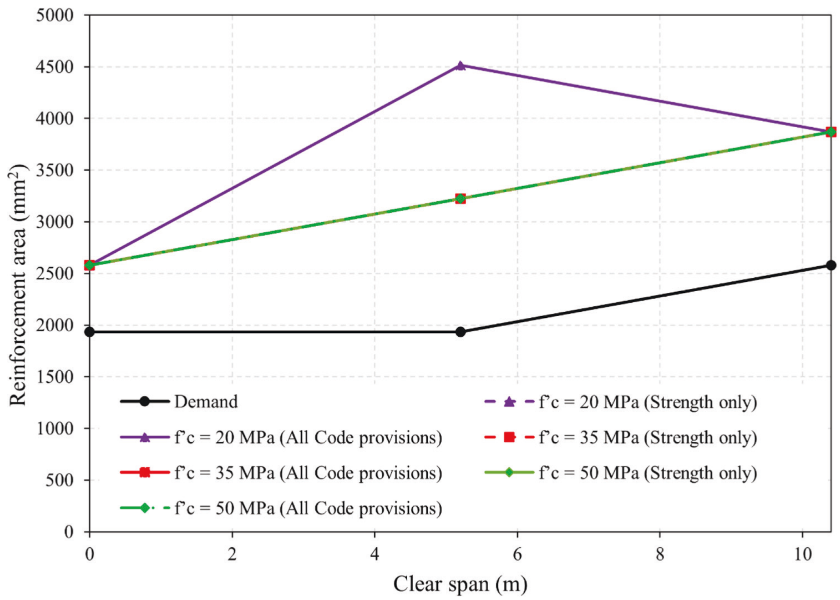


Fig. 7—Reinforcement area with different f_c' *values with and without meeting serviceability requirements (lines overlap).*

Table 10—GFRP-RC beam using high-modulus bars with different deflection limits

Location	Reinforcement area required for strength only, mm²	Reinforcement area provided, mm² $\Delta = l/240$	Reinforcement area provided, mm² $\Delta = l/360$	Reinforcement area provided, mm² $\Delta = l/480$
Exterior support	1935	2580	2580	2580
Midspan	1935	3226	6452	12,258
Interior support	2580	3870	3870	3870

concrete strengths. Increasing concrete strengths significantly decreases the reinforcement area used to meet serviceability requirements. It should be noted that some curves representing identical values in Fig. 7 overlap, hence the difference in using different concrete strengths may not be visible in the figure.

Design of beam using high-modulus GFRP bars (E_f = 60,000 MPa) changing deflection limit

In ACI CODE-440.11-22[4] Section 24.2.2, two more stringent deflection limits (that is, $l/360$ and $l/480$) other than the one used in Phase 1 ($l/240$) are provided. As GFRP-RC members are sensitive to serviceability requirements, a design attempt is made on these two limits using $f_c' = 35$ MPa and $k_b = 1.35$.

For the more stringent deflection limits, reinforcement is increased at midspan to satisfy serviceability. As seen in Table 10, provided areas of reinforcement at supports remain the same for any deflection limit. However, the provided area at midspan has to increase to 6451 mm² to satisfy serviceability to meet $l/360$. When the deflection limit was set to $l/480$, the provided reinforcement area increased to 12,258 mm², making it impossible to construct. If the limit of $l/480$ has to be met, the beam cross-section dimensions must change.

OBSERVATIONS

Development length

The development length equation in ACI CODE-440.11-22[4] results in very large values, and this, coupled with the inability to make a hook at the end of long longitudinal bars, makes design challenging and costly. There have been improvements in composite material properties as well as surface deformations since the Code equation was developed. Additionally, the current equation is based on the test data obtained more than two decades ago[9] and the bars used in those tests are no longer available today. Therefore, it is necessary to reassess and update the development length equation for GFRP bars to incorporate the improvements in the material properties and develop a more representative equation for development length.

Maximum spacing limit

The maximum spacing limit is governed by Code Section 24.3.2 to control cracking, developed by Ospina and Bakis in 2007, based on the modifications to the work done by Frosch in 1999 for steel-RC.[10,11] This limit is governed by the bond factor k_b and stress at service loads. Stress at service loads is also dependent on the bond factor. Reinforcement spacing limitations greatly penalize the design, and the resulting capacity becomes typically very large compared to demand. This additional reinforcement not only results in extra cost but in detailing difficulties as well. There have been improvements in GFRP material properties, warranting reconsideration of these provisions.

Skin reinforcement

To control web cracking, provisions for GFRP skin reinforcement are given in Code Section 9.7.2.3.[4] These provisions are based on the physical model developed for steel-RC members for skin reinforcement.[12] Additionally, the provisions for steel-RC are applicable to member depths greater than 760 mm; however, for GFRP-RC members, skin reinforcement needs to be provided for depths greater than

460 mm. Further, Code provisions require skin reinforcement to be placed at a maximum spacing as given in Code Section 24.3.2[4] with an overall outcome that appears unreasonable. Because there has been no experimentation dedicated to GFRP skin reinforcement, the current maximum spacing requirements need to be reassessed.

CONCLUSIONS AND RECOMMENDATIONS

In this study, a beam example was taken from the *ACI Reinforced Concrete Design Handbook*[6] and redesigned with glass fiber-reinforced polymer (GFRP) reinforcement to show the implication of some ACI CODE-440.11-22[4] provisions. This study considered both new-generation (E_f = 60,000 MPa) and old-generation (E_f = 44,815 MPa) bars compliant with ASTM D7957/D7957M-22,[7] as currently specified by the Code. Using the same beam cross section as steel-reinforced concrete (RC), the concrete strength f_c' used was equal to 35 MPa, and the bond coefficient k_b = 1.35. An assumption about the maximum permissible deflection limit of l/240 was also made. Later, a parametric study was carried out to analyze the effects of changing the values of k_b, f_c', and the maximum permissible deflection limit.

Based on the outcomes of this study in the design and detailing, the following conclusions were drawn:

- Design of beams reinforced with GFRP is generally governed by Code serviceability (that is, deflection control) and detailing (that is, maximum reinforcement spacing) requirements.
- Given that the elastic modulus of GFRP bars is lower than that of steel, more reinforcement area is needed to satisfy deflection limits.
- Code provisions for maximum spacing and allowable stress limit at service loads are governed by the bond factor k_b. Changing k_b from the current Code value to lower ones (that is, 1.20 or 1.05) increases the maximum allowable limits for stress at service and bar spacing but does not significantly reduce reinforcement requirements.
- Increasing concrete compressive strength to 50 MPa significantly reduced the deflection of the GFRP-RC member. However, gains achieved by increasing compressive strength are nullified by Code provisions for maximum spacing and stress at service.
- The maximum permissible deflection limits in the Code other than l/240 (that is, l/360 and l/480) are difficult to accomplish with GFRP reinforcement using cross-section dimensions typical of steel-RC.
- The number of skin reinforcement bars is governed by Code maximum spacing provisions, which are found to penalize design.
- The current development length equation results in very large values, causing detailing difficulties and bar congestion, especially at the exterior support.
- Recent developments in the manufacturing of GFRP bars and an increased modulus of elasticity from 44,815 to 60,000 MPa has a positive impact on design.
- Experimental investigations aimed at reassessing Code limits for development length, maximum spacing, and stress at service loads by incorporating the improvements in material properties are needed.
- The shear design of the GFRP-RC beam is affected by a reduction in concrete contribution, V_c, and strength at the bent portion of the GFRP stirrups. Hence, more shear reinforcement than its steel counterpart is required.

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NOTATION

A_{fv}	=	area of shear reinforcement, mm^2
b_w	=	web width or diameter of circular cross section, mm
c_b	=	lesser of: a) distance from center of bar to nearest concrete surface; and b) one-half center-to-center spacing of bars being developed, mm
c_c	=	concrete cover, mm
d	=	distance from extreme compression fiber to centroid of longitudinal tension reinforcement, mm
d_b	=	nominal diameter of bar, mm
E_c	=	modulus of elasticity of concrete, MPa
E_f	=	modulus of elasticity of GFRP reinforcement, MPa
f_c'	=	compressive strength of concrete at 28 days
f_{fb}	=	guaranteed ultimate tensile strength of bent portion of bar
f_{fr}	=	tensile stress in GFRP reinforcement required to develop full nominal section capacity, MPa
f_{fs}	=	stress at service loads
k_b	=	bond-dependent coefficient
k_{cr}	=	ratio of depth of elastic cracked section neutral axis to effective depth
l_n	=	length of clear span measured between face-to-face of supports, m
M_u	=	ultimate factored moment at section, kN·m
S_{max}	=	maximum allowed spacing, mm
S_n	=	nominal moment, shear, axial or torsional strength
U	=	strength of member or cross section required to resist factored loads or related internal moments and forces
V_c	=	nominal shear strength provided by concrete, N
V_f	=	nominal shear strength provided by GFRP shear reinforcement, N
V_n	=	nominal shear strength, N
V_u	=	factored shear force at section, N
W_u	=	ultimate factored load, kN/m
Δ	=	maximum permissible deflection
ε_f	=	strain in GFRP flexural reinforcement
Φ	=	strength reduction factor
ω	=	bar location modification factor

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