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Implications of ACI CODE-440.11 Code Provisions on Design of Glass Fiber-Reinforced Polymer-Reinforced Concrete Footings

by Zahid Hussain and Antonio Nanni

The first edition of ACI CODE-440.11 was published in September 2022, where some code provisions were either based on limited research or only analytically developed. Therefore, some code provisions, notably shear and development length in footings, are difficult to implement. This study, through a design example, aims at a better understanding of the implications of code provisions in ACI CODE-440.11-22 and compares them with ones in CSA S806-12, thereby highlighting a need for reconsiderations. An example of the footing originally designed with steel reinforcement was taken from the ACI Reinforced Concrete Design Handbook *and redesigned with GFRP reinforcement as per ACI CODE-440.11-22 and CSA S806-12. A footing designed as per ACI CODE-440.11-22 requires a thicker concrete cross section to satisfy shear requirements; however, when designed as per CSA S806-12, the required thickness becomes closer to that of the steel-reinforced concrete (RC) footing. The development length required for a glass fiber-reinforced polymer-reinforced concrete (GFRP-RC) cross section designed as per ACI CODE-440.11-22 was 13% and 92% greater than that designed as per CSA S806-12 and ACI 318-19, respectively. Also, the reinforcement area required to meet detailing requirements is 170% higher than that for steel-RC cross section. Based on the outcomes of this study, there appears to be a need for reconsideration of some code provisions in ACI CODE-440.11-22 to make GFRP reinforcement a viable option for RC members.*

Keywords: building code; footing; glass fiber-reinforced polymer (GFRP) reinforcement; reinforced concrete; shear.

INTRODUCTION

ACI CODE-440.11-22[1] is a milestone for practitioners interested in the use of nonmetallic reinforcement for concrete structures, even though some provisions make the design difficult and the implementation challenging. For example, the current code requirements for shear in ACI CODE-440.11-22[1] were derived based on the neutral axis depth of the cracked cross section, differently from ACI 318-19.[2] The equations are further dependent on the axial stiffness of glass fiber-reinforced polymer (GFRP) reinforcement. Because GFRP reinforcement has lower stiffness than steel, the shear design of GFRP-reinforced concrete (RC) members requires deeper cross sections, making execution difficult, particularly for shallow foundations.

ACI CODE-440.11-22[1] conservatively ignores some of the beneficial effects on the shear capacity of GFRP-RC members, which are otherwise addressed in Canadian Standard Association (CSA) S806-12.[3] For example, in calculating one-way shear resistance provided by concrete, CSA S806-12[3] considers the arching effect. Also, one-way and two-way shear strength are both dependent on the longitudinal reinforcement ratio, whereas ACI CODE-440.11-22[1] uses the axial stiffness of GFRP reinforcement in calculating the neutral axis depth for a cross section.

It appears that implementation of shear and development length provisions in ACI CODE-440.11-22 would be difficult due to some assumptions made during their development. Therefore, this study was carried out to show the implications of code provisions in ACI CODE-440.11-22[1] on the design of GFRP-RC members (a square footing) by providing a comparison with CSA S806-12[3] and ACI 318-19,[2] highlighting the conservatism in ACI CODE-440.11-22[1] code provisions.

RESEARCH SIGNIFICANCE

The significance of this research lies in the critical examination and evaluation of certain provisions within ACI 440.11-22 pertaining to GFRP reinforcement. A substantial portion of these provisions has been formulated either through analytical methodologies or with reliance on limited research. The undue conservativeness of these provisions poses implementation challenges in the design process and complicates practical implementation of GFRP reinforcement as a suitable substitute for metallic reinforcement. Therefore, this study serves the imperative purpose of identifying and elucidating specific provisions that warrant reconsideration in light of recent advancements in research.

MATERIALS AND METHODS

The analysis and comparison of code provisions in ACI CODE-440.11-22,[1] CSA S806-12,[3] and ACI 318-19[2] was carried out using a footing example taken from the *ACI Reinforced Concrete Design Handbook, A Companion to ACI 318-19*.[4] The selected design example (originally for steel-RC) was redesigned using GFRP reinforcement as per provisions in ACI CODE-440.11-22[1] and CSA S806-12.[3] The footing supports the load from a square interior column, as shown in Fig. 1. The constituent materials selected for the footing design are shown in Table 1. The concrete strength, f_c', is 28 MPa while the GFRP reinforcement is compliant with the material specification ASTM D7957/D7957M.[5] The mechanical properties of GFRP bars affecting design include

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Table 1—Properties of GFRP reinforcement, concrete, and soil

Designation		Nominal diameter, mm	Nominal area, mm²	Elastic modulus, MPa	Guaranteed tensile strength, MPa	Ultimate strain, %	Concrete strength, MPa	Concrete clear cover, mm	$q_{(D+L)}$, kN/m²
GFRP reinforcement	ASTM D7957	28.6	645	44,816	565	1.2	—	—	—
	ASTM D8505			60,000	793	1.3	—	—	—
Concrete		—	—	24,870	—	0.0035 (CSA) 0.003 (ACI)	28.0	76.0	—
Soil bearing capacity		—	—	—	—	—	—	—	268

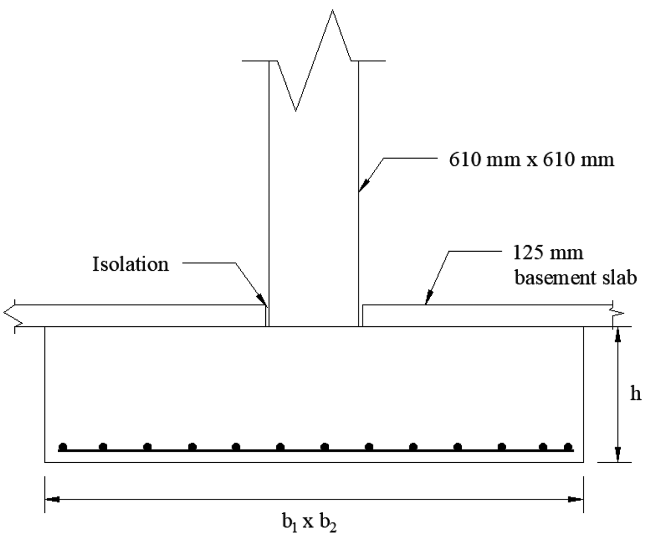

Fig. 1—Square footing with square column. (Reproduced from ACI Reinforced Concrete Design Handbook.[4]*)*

guaranteed ultimate tensile strength (f_{fu}), corresponding ultimate strain (ε_{fu}), modulus of elasticity (E_f), and modular ratio (n_f). A value of 1.20 for the bond coefficient, k_b, was selected as per ACI CODE-440.11-22[1] and CSA S806-12[3] Sections 24.3.2.3 and 8.3.1.1, respectively. Similarly, a value of 0.85 was adopted for the environmental reduction factor, C_E, as indicated in ACI CODE-440.11-22,[1] Section 20.2.2.3. A concrete cover, c_c, of 76 mm is used as specified in ACI CODE-440.11-22[1] and CSA S806-12[3] in Sections 20.5.1.3.1 and 8.3, respectively. The admissible soil bearing capacity considered for the dead and live loads was 268 kN/m², as given in the *ACI Reinforced Concrete Design Handbook*.[4] Table 1 also presents the properties of new-generation GFRP bars with high elastic modulus and strength, which are currently not specified in ACI CODE-440.11-22.

The square footing carried an axial dead load equal to 2407 kN, plus a live load of 863 kN. These loads were combined as per ASCE 7-16[6] to compute the maximum factored demand. First, the square footing is designed as per ACI CODE-440.11-22[1] and CSA S806-12.[3] Later, a comparison based on the design of this footing following the provisions of three building codes (that is, ACI CODE-440.11-22,[1] CSA S806-12,[3] and ACI 318-19[2]) is presented. Also, a discussion about the development and implications of shear and development length equations in ACI CODE-440.11-22[1] is provided.

Table 2—Strength reduction factor Φ (ACI CODE-440.11-22, 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

*Applicable to over-reinforced sections.

Code provisions

ACI CODE-440.11-22 code requirements—For applicable factored load combinations, design strength at all sections shall satisfy the requirements of ACI CODE-440.11-22,[1] Sections 7.5.1.1 and 8.5.1.1, as given here

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

where S_n is nominal moment, shear, axial or torsional strength; U is shear, moment, torsional moment, or axial force resulting from the factored loads; and Φ is strength reduction factor calculated as per ACI CODE-440.11-22,[1] as given in Table 2.

The maximum spacing of longitudinal GFRP reinforcement, s, is limited as specified by ACI CODE-440.11-22,[1] Sections 24.3.2a and 24.3.2b

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

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

where f_{fs} is stress at service loads, MPa.

The development length of the longitudinal GFRP reinforcement is governed by Code Section 25.4.2.1, as the greater of Eq. (4), (5), and (6) given herein

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

where f_{fr} *is* tensile stress in GFRP reinforcement required to develop the full nominal section capacity, MPa; c_b is lesser of: a) the distance from center of a bar to nearest concrete surface; and b) one-half the center-to-center spacing of bars being developed, or one-half the center-to-center spacing of the bars, mm; d_b *is* nominal bar diameter, mm; and ω is 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.

$$20d_b \tag{5}$$

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

The reinforcement area shall be provided as greater of area required by the ultimate factored moment demand and area necessary to ensure that the flexural strength exceeds the cracking strength, indicated in ACI CODE-440.11-22,[1] Sections 7.6.1.1 and 24.4.3.2, provided as Eq. (7) and (8)

$$A_{f_{min}-1} = \frac{2.1}{f_{fu}}A_g \tag{7}$$

$$A_{f_{min}-2} = \frac{20{,}000}{E_f} \tag{8}$$

where A_g is gross area of the cross section, mm^2.

Concrete cross-sectional dimensions shall be selected to avoid diagonal compression failure as in ACI CODE-440.11-22[1] section 22.5.1.2, provided as Eq. (9)

$$V_u \leq \Phi 0.2 f_c' bd \tag{9}$$

where V_u is factored shear force at a section, kN.

The nominal shear strength can be calculated as per ACI CODE-440.11-22,[1] Section 22.5.1.1, given as

$$V_n = V_c + V_f \tag{10}$$

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

The one-way shear strength provided by concrete can be calculated as the greater of two expressions from ACI CODE-440.11-22[1], Sections 22.5.5.1a and 22.5.5.1b, as given herein

$$V_c = 0.42\lambda_s k_{cr}\sqrt{f_c'}\,bd \tag{11}$$

$$V_c = 0.066\lambda_s\sqrt{f_c'}\,bd \tag{12}$$

where k_{cr} is ratio of the depth of elastic cracked section neutral axis to the effective depth, given by the code commentary Section R22.5.5.1, as shown herein

$$k_{cr,rect} = \sqrt{2\rho_f n_f + (\rho_f n_f)^2} - \rho_f n_f \tag{13}$$

where $\rho_f = A_f/b_d$ is the reinforcement ratio; A_f is the area of GFRP longitudinal reinforcement, mm^2; and $n_f = E_f/E_c$ is the modular ratio.

$$n_f = \frac{E_f}{E_c} = \text{Modular Ratio}$$

where E_c is modulus of elasticity of concrete (MPa), calculated as given by the Code Sections 19.2.2.1a and 19.2.2.1(b), given as Eq. (14) and (15).

$$E_c = w_c^{1.5}0.043\sqrt{f_c'} \tag{14}$$

$$E_c = 4700\sqrt{f_c'} \tag{15}$$

$\lambda_s = \sqrt{2/(1+0.004d)}$ is size effect factor, as given in ACI 440.11-22,[1] Section 22.5.5.1, Table 22.5.5.1.3, and should be less than or equal to 1.0.

Similarly, two-way shear strength is calculated as maximum strength calculated with Eq. (22.6.5.2a) and (22.6.5.2b), as given herein

$$v_c = 0.83\lambda_s k_{cr}\sqrt{f_c'} \tag{16}$$

$$v_c = 0.13\lambda_s\sqrt{f_c'} \tag{17}$$

where v_c is stress corresponding to nominal two-way shear strength of slab or footing, MPa.

CSA S806-12 code requirements—Chapter 8 of CSA S806-12[3] contains the provisions for the design of concrete members with FRP reinforcement. All the FRP-RC sections shall be designed so that the failure of the section is initiated by the crushing of concrete in the compression zone. However, if the factored resistance of a section is greater than 1.6 times the moment due to the factored loads, the concrete section can be designed so that failure is controlled by FRP rupture.

The Code Section 8.2.3 specifies that the minimum clear concrete cover in RC members shall be twice the diameter of a bar ($2d_b$) or 30 mm, whichever is greater. The ultimate strain in concrete at the extreme compression fiber shall be assumed to be equal to 0.0035 (that is, different from the ACI assumption of 0.003), and its tensile strength shall be neglected.

The Code Section 8.4.2 states that the minimum reinforcement of a flexural member shall be proportioned so that factored resisting moment (M_r) is at least 1.5 times greater than the cracking moment (that is, $M_r \geq 1.5M_{cr}$). Also, the minimum reinforcement area in slabs equal to $(400/E_f)A_g$ shall be provided in each of the two orthogonal directions. The reinforcement shall not be less than $0.0025A_g$ and shall be spaced no further than three times the slab thickness or 300 mm, whichever is less.

The provisions for one-way shear strength are given in Section 8.4.4, which states that the factored shear resistance of members with GFRP longitudinal reinforcement shall be determined as per Eq. (8) to (14) in CSA S806-12, provided as Eq. (18)

$$V_r = V_c + V_{sF} \tag{18}$$

where V_r is the factored shear resistance, kN; V_c is factored shear resistance provided by concrete, kN; and V_{sF} is factored shear resistance provided by FRP shear reinforcement, kN.

Factored shear resistance provided by concrete for members with effective depth greater than 300 mm, with no axial load may be calculated as per Section 8.4.4.5, provided as Eq. (19)

$$V_c = 0.05\lambda \emptyset_c k_m k_r (f_c')^{\frac{1}{3}} b_w d_v \tag{19}$$

where λ is the factor to account for concrete density; Φ_c is the strength reduction factor, taken equal to 0.65 as per Section 6.5.3.2; b_w is minimum effective web width, mm; d_v is effective shear depth, taken as the greater of $0.9d$ or $0.72h$, mm; and k_m is the coefficient accounting for the effect of moment at a section on shear strength, calculated as per Eq. (8) to (18) in the Code and provided in Eq. (20)

$$k_m = \sqrt{\frac{V_f d}{M_f}} \leq 1.0 \tag{20}$$

where V_f is the factored shear force, kN; d is distance from extreme compression fiber to the centroid of longitudinal bar, mm; M_f is factored moment, kN·m; and k_r is coefficient accounting for the effect of reinforcement rigidity on its shear strength, calculated as per Eq. (8) to (19) in CSA S806-12 and provided as Eq. (21)

$$k_r = 1 + (E_f \rho_{FW})^{1/3} \tag{21}$$

where ρ_{FW} is longitudinal FRP reinforcement ratio.

The concrete strength calculated in accordance with Section 8.4.4.5 in CSA S806-12[3] shall not be greater than Eq. (22) and less than Eq. (23) as stated in Section 8.4.4.5.

$$V_c \leq 0.22 \emptyset_c \sqrt{f_c'} b_w d_v \tag{22}$$

$$V_c \geq 0.11 \emptyset_c \sqrt{f_c' b_w d_v} \tag{23}$$

In determination of V_c, f_c' shall not be taken greater than 60 MPa.

Different from ACI CODE-440.11-22,[1] CSA S806-12[3] Section 8.4.4.6 states that sections within a distance of $2.5d$ from the face of the support where the support causes compression in the beam parallel to the direction of shear force at a section, V_c shall be calculated as the value determined according to Section 8.4.4.5 (Eq. (19)) multiplied by the factor k_a (that is, factor to account for the arching effect on shear strength) as per Section 8.4.4.6, provided in Eq. (24)

$$k_a = \frac{2.5}{\frac{M_f}{V_f d}} \geq 1.0 \tag{24}$$

The value of k_a shall not exceed 2.5.

CSA S806-12,[3] Section 8.4.4.7, addresses shear modification for members with size exceeding 300 mm and without minimum transverse shear reinforcement, the value of V_c calculated as per Section 8.4.4.5 (CSA S806-12[3]) shall be multiplied by the factor k_s (that is, factor to account for size effect) as given in Section 8.4.4.7 (CSA S806-12[3]) and provided in Eq. (25)

$$k_s = \frac{750}{450 + d} \leq 1.0 \tag{25}$$

Punching shear resistance can be calculated as per CSA S806-12,[3] Section 8.7.2, which states that factored shear due to punching shall not exceed the limits specified by Eq. (8-39), (8-40), and (8-41) of CSA S806-12,[3] provided as Eq. (26), (27), and (28)

$$v_r = \left(1 + \frac{2}{\beta_c}\right)\left[0.028\lambda \emptyset_c (E_f \rho_F f_c')^{\frac{1}{3}}\right] \tag{26}$$

where v_r is factored shear stress resistance, MPa; β_c is ratio of long side to short side of column; E_f is modulus of elasticity of FRP reinforcement, MPa; and ρ_f is reinforcement ratio.

$$v_r = \left[\left(\frac{\alpha_s d}{b_o}\right) + 0.19\right] 0.147\lambda \emptyset_c (E_f \rho_F f_c')^{\frac{1}{3}} \tag{27}$$

where $\alpha_s = 4$ for interior columns, 3 for edge columns, and 2 for corner columns.

$$v_r = 0.056\lambda \emptyset_c (E_f \rho_F f_c')^{\frac{1}{3}} \tag{28}$$

When calculating v_r using Eq. (26) to (28), the value of f_c' shall not be taken greater than 60 MPa. If the effective depth of the structural slab system exceeds 300 mm, the value of v_r obtained from Section 8.7.2[3] shall be multiplied by $(300/d)^{0.25}$ to include the effect of member size, as stated in CSA S806-12,[3] Section 8.7.4.

The development length of bars in tension shall be either determined directly from the tests or shall be taken as the greater of 300 mm he value obtained from Section 9.3,[3] as provided in Eq. (29)

$$l_d = 1.15 \frac{k_1 k_2 k_3 k_4 k_5}{d_{cs}} \frac{f_F}{\sqrt{f_c'}} A_b \tag{29}$$

where d_{cs} is the smaller of: a) the distance from closest concrete surface to the center of the bar being developed; and b) two-thirds of center-to-center spacing between bars being developed, mm; k_1 is bar location factor taken equal to 1.3 for horizontal reinforcement placed so that more than 300 mm of fresh concrete is cast in the member below the development length or splice and 1.0 for other cases; k_2 is concrete density factor is taken equal to 1.3, 1.2, and 1.0 for low-density, semi-low-density, and normalweight concrete; k_3 is bar size factor is taken equal to 0.8 for $A_b \leq 300$ mm^2 and 1.0 for $A_b \geq 300$ mm^2; k_4 is bar fiber factor is taken equal to 1.0 for GFRP and CFRP and 1.25 for AFRP; and k_5 is bar surface profile factor is taken equal to 1.0 for surface roughened or sand-coated surfaces, 1.05 for spiral pattern surfaces, 1.0 for braided surfaces, 1.05 for ribbed surfaces, and 1.80 for indented surfaces.

DESIGN EXAMPLE

Design of GFRP-RC foundation as per ACI CODE-440.11-22

The bottom of the square footing is located 0.91 m below the basement slab (that is, original footing given in *ACI Reinforced Concrete Design Handbook*[4]). Therefore, it is considered a shallow foundation.[1] The square footing is redesigned with applicable Code provisions for one- and two-way slabs as stated in ACI CODE-440.11-22[1] Section 13.3. The minimum base area of the shallow foundation was selected to satisfy the code requirements in Section 13.3.1.1. It requires that the minimum base area of the foundation shall be proportioned not to exceed the permissible bearing pressure when subjected to forces and moments applied to the foundation. It was observed that with applicable load combinations and allowable soil capacity provided in the *ACI Reinforced Concrete Design Handbook*,[4] the minimum required base area of footing was 12.2 m^2. Therefore, it was decided to use a 3.6 x 3.6 m foundation that slightly exceeds the required dimensions. The dimensions of the footing and critical section for one- and two-way shear verification are shown in Fig. 2, where b_1 and b_2 are the length and width of footing ($b_1 = b_2$ for this case of square footing), and b_1' and b_2' are the critical perimeter dimensions for two-way shear ($b_1' = b_2'$ for this case of square column). Also shown are the critical sections for one-way shear (that is, at a distance d from the column face) and two-way shear (that is, at a distance $d/2$ from the column face), and c_1 and c_2 are column dimensions (that is, 610 x 610 mm, as provided in *ACI Reinforced Concrete Design Handbook*).[4]

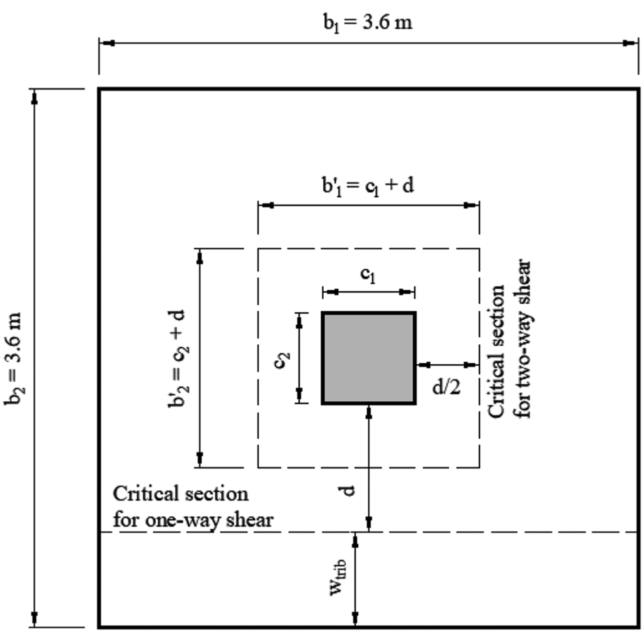

Fig. 2—Square footing, column dimensions, and critical sections for one-way and two-way shear.

The column does not impart a moment to the footing so that the soil pressure under the footing is uniform. ACI 440.11-22[1] Section 13.2.6.2 states that for one-way shallow foundations and two-way isolated footings, it is permissible to neglect the size effect factor specified in Sections 22.5 and 22.6 for one-way and two-way shear provisions, respectively. Consequently, the size effect factor was neglected in both calculations, and it was assumed that shear strength is only provided by concrete cross section.

The tributary area contributing to one-way shear and two-way shear were equal to 2.47 and 10.7 m^2, respectively. The k_{cr} value was first calculated using a reinforcement ratio (ρ_f) of 0.004 and a modular ratio (n_f) 1.8, resulting equal to 0.11. (Note: $\rho_f = 0.004$ was adopted to meet both strength and serviceability requirements.) However, Code Section R22.5.5.1 requires a lower bound of 0.16 on the value of k_{cr} (that is, $k_{cr} = 0.16$) in Eq. (22.5.5.1b); hence, this value was used to calculate shear strength.

Ignoring the size effect factor and using normalweight concrete, the GFRP-RC footing required a larger thickness for one-way shear than its steel-RC counterpart subjected to the same loads (that is, to 0.94 m, versus 0.91 m). Using $h = 0.94$ m, the one-way shear strength of GFRP-RC footing calculated as per ACI CODE-440.11-22[1] Sections 22.5.5.1a and 22.5.5.1b was equal to 815 kN, which exceeds the demand of 786 kN.

Using $h = 0.94$ m, the two-way shear strength was calculated as per Section 22.6, resulting equal to 2684 kN, which was less than demand of 3590 kN. Hence, the concrete cross section thickness was increased to 1.12 m to satisfy two-way shear requirements. As shown in Table 3, the two-way shear strength at a thickness equal to 1.12 m is 3488 kN, which is greater than the demand of 3413 kN. It should be noted that the two-way shear strength for the steel-RC is 5902 kN at a thickness equal to 0.91 m, as also shown in Table 3. This may be because shear strength in steel-RC cross section depends on effective cross section where a section between two cracks is considered. Hence, the entire section

Table 3—Design of steel-RC and GFRP-RC footing as per ACI 318-19[2] and ACI CODE-440.11-22[1]

		Steel-RC ACI 318-19						GFRP-RC ACI CODE-440.11-22					
		Demand			Capacity			Demand			Capacity		
Quantity		h, m	Moment, kN·m	Shear, kN	$\frac{A_{s\_req}}{A_{s\_pro}}$	Moment, kN·m	Shear, kN	h, m	Moment, kN·m	Shear, kN	$\frac{A_{f\_req}}{A_{f\_pro}}$	Moment, kN·m	Shear, kN
One-way shear		0.91	—	850	—	—	925	1.12	—	578	—	—	986
Two-way shear			—	3651		—	5902		—	3413		—	3488
Flexural strength	(ASTM D7957)		1356	—	0.85	2045	—		1356	—	0.83	4706	—
	(ASTM D8505)										0.84	4717	

contributes to the shear strength. However, in the case of GFRP-RC, only uncracked concrete above the neutral axis is considered effective in resisting the applied forces.

The critical section for the maximum moment was assumed at the face of the column as shown in Fig. 3. The tributary area contributing to the moment was equal to 5.4 m^2 and the ultimate moment calculated was equal to 1356 kN-m. The reinforcement area required to meet strength requirements was equal to 0.015 m^2. However, to meet serviceability requirements stated in ACI CODE-440.11-22,[1] Sections 24.3.2(a), 24.3.2(b), and 24.3.2.2, and temperature and shrinkage requirements stated in Section 7.6.1.1, the provided reinforcement area was increased to 0.018 m^2. In this footing design, M29 bars were placed at 127 mm center to center. The flexural capacity of GFRP-RC footing designed as per ACI CODE-440.11-22[1] was equal to 4706 kN·m. The reinforcement area for steel-RC footing was equal to 0.007 m^2, and its moment capacity was 2045 kN·m (refer to Table 3). A sketch of dimensions and reinforcement details of GFRP-RC footing designed as per ACI CODE-440.11-22[1] are provided in Fig. 4.

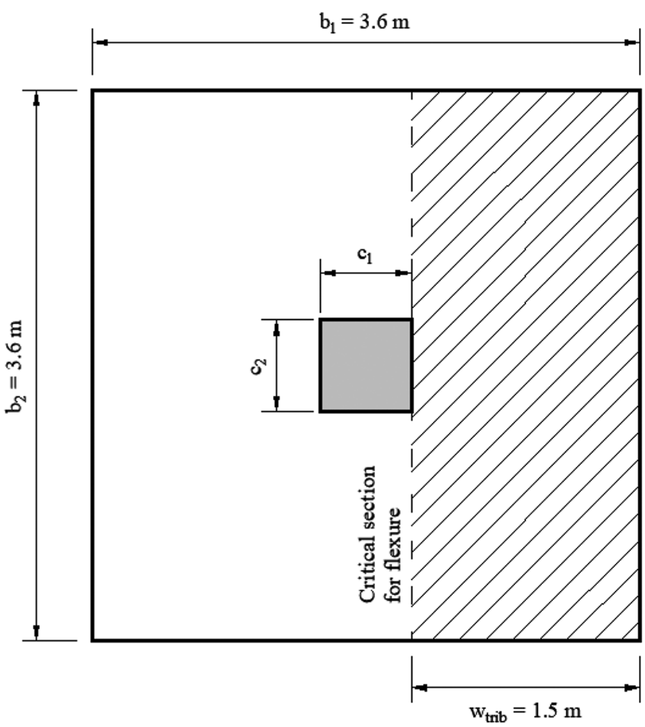

Fig. 3—Critical section for moment.

In the summer of 2023, ASTM published new specification ASTM D8505/D8505M, which defines the physio-mechanical properties of a new generation of GFRP bars.[7] These bars have higher elastic modulus and strength compared to ones specified in ASTM D7957/D7957M.[5,7] While ACI CODE-440.11-22 does not cover these bars, the footing was redesigned as per ASTM D8505/D8505M to investigate their influence on the design. The properties of new-generation bars are provided in Table 1.

The use of high-elastic-modulus and high-strength bars in the design of GFRP-RC footing resulted in the reduction of required reinforcement ratio. The shear strength equations in ACI CODE-440.11-22 depend on the axial stiffness of GFRP reinforcement, which is incorporated by factor k_{cr}, with lower bound of 0.16 on its value. Even though using new-generation bars resulted in reduction of required reinforcement ratio, the lower bound on the value of k_{cr} controlled the shear design. Therefore, the shear strength of the footing remained the same.

The impact of using new-generation bars, however, was evident in flexure design of the footing. Even though minimum reinforcement was still controlled by serviceability requirements, the GFRP bars were comparatively less stressed, which allowed an increase in the required center-to-center spacing. Hence, the footing designed with new-generation bars required 20 M29 GFRP bars compared to 28 M29 when using the old-generation bars specified in ASTM D7957.[5]

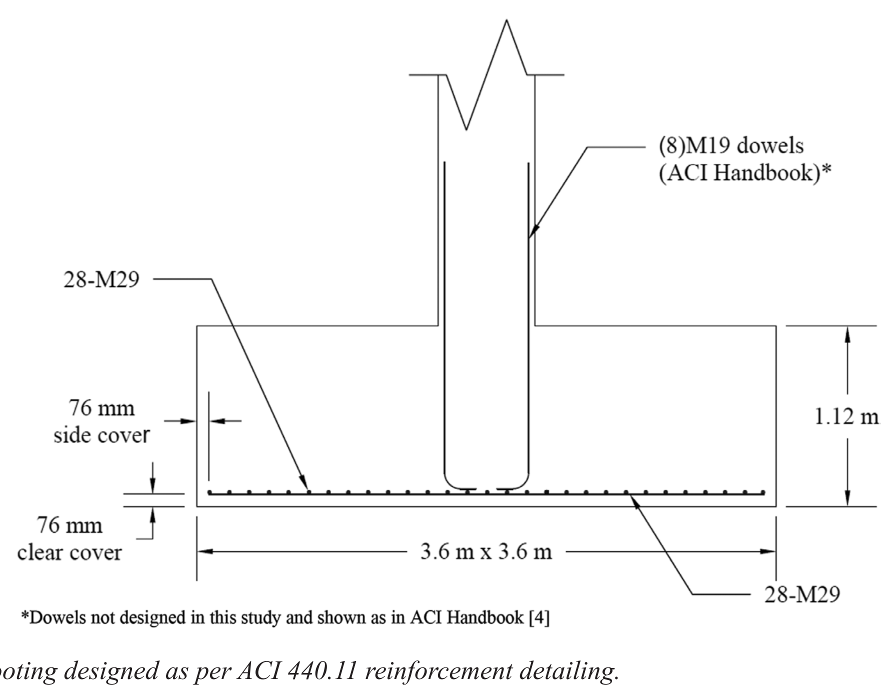

Fig. 4—GFRP-RC footing designed as per ACI 440.11 reinforcement detailing.

Table 4—Design of GFRP-RC footing as per ACI CODE-440.11-22[1] and CSA S806-12[3]

<table>
<tr><th rowspan="3" colspan="2">Quantity</th><th colspan="6">GFRP-RC ACI CODE-440.11-22</th><th colspan="6">GFRP-RC-CSA S806-12</th></tr>
<tr><th colspan="3">Demand</th><th colspan="3">Capacity</th><th colspan="3">Demand</th><th colspan="3">Capacity</th></tr>
<tr><th>h, m</th><th>Moment, kN·m</th><th>Shear, kN</th><th>$\frac{A_{f\_req}}{A_{f\_pro}}$</th><th>Moment, kN·m</th><th>Shear, kN</th><th>h, m</th><th>Moment, kN·m</th><th>Shear, kN</th><th>$\frac{A_{f\_req}}{A_{f\_pro}}$</th><th>Moment, kN·m</th><th>Shear, kN</th></tr>
<tr><td colspan="2">One-way shear</td><td rowspan="4">1.12</td><td>—</td><td>578</td><td rowspan="2">—</td><td>—</td><td>986</td><td rowspan="4">1.02</td><td>—</td><td>670</td><td rowspan="2">—</td><td>—</td><td>1055</td></tr>
<tr><td colspan="2">Two-way shear</td><td>—</td><td>3413</td><td>—</td><td>3488</td><td>—</td><td>3488</td><td>—</td><td>3522</td></tr>
<tr><td rowspan="2">Flexural strength</td><td>(ASTM D7957)</td><td rowspan="2">1356</td><td rowspan="2">—</td><td>0.83</td><td>4706</td><td rowspan="2">—</td><td rowspan="2">1356</td><td rowspan="2">—</td><td>0.16</td><td>8682</td><td rowspan="2">—</td></tr>
<tr><td>(ASTM D8505)</td><td>0.83</td><td>4717</td><td>0.17</td><td>7093</td></tr>
</table>

The GFRP-RC shallow foundation required a larger reinforcement area than steel-RC and higher values of thickness. The extra materials and excavation costs may impose limitations on its application.

Design of GFRP-RC footing as per CSA S806-12

In this section, the footing example taken from *ACI Reinforced Concrete Design Handbook*[4] was redesigned as per the guidelines of CSA S806-12.[3] GFRP reinforcement properties, admissible soil pressure, and concrete strength are the same as provided in Table 1.

The minimum base area of the footing remains the same as used previously (that is, 3.6 x 3.6 m). The initial concrete cross-section thickness adopted in the design as per CSA S806-12[3] was equal to the thickness of steel-RC footing (that is, 0.91 m), which later was increased to value shown in Table 4.

The one-way and two-way shear strength of the GFRP-RC footing was calculated as per CSA S806-12[3] Sections 8.4.4.5 and 8.7.2, respectively, using a concrete density factor (λ) equal to 1.0 corresponding to normalweight concrete. The coefficients k_m and k_r were calculated as per Section 8.4.4.5 equal to 0.70, and 8.37, respectively. The effective shear depth (d_v) was taken as the greater of the value 0.9d (where d is effective of cross section) and 0.72h, which was equal to 0.8 m. The size effect factor (k_s) for one-way shear was calculated as per Section 8.4.4.7, equal to 0.55 and arch effect equal to 1.1. The strength reduction factor used for shear design was equal to 0.65 as per CSA S806-12,[3] Section 6.5.3.2 (different from ACI CODE-440.11-22[1] where it is equal to 0.75). Using a footing thickness of 0.91 m, the one-way shear strength was calculated as per Section 8.4.4.5, Eq. (8-19), equal to 1072 kN, which was greater than the demand of 800 kN.

The two-way shear strength was calculated as per Section 8.7, Eq. (8-39), (8-40), and (8-41) (reproduced herein as Eq. (26), (27), and (28)). Given an interior square column (610 x 610 mm), the factor β_c was taken equal to 1.0 and a_s was taken equal to 4.0. The size effect factor for two-way shear (k_s) was calculated as per Section 8.7.4 equal to 0.78.

Using a footing thickness of 0.91 m, the two-way shear strength was calculated as per Section 8.7 equal to 3226 kN which was less than demand of 3612 kN. Hence, the thickness was increased to 1.02 m to satisfy two-way shear requirement resulting in a strength of 3522 kN which is greater than the demand of 3488 kN (refer to Table 4).The required thickness value (that is, 1.02 m) for two-way shear is 0.1 m (9%) less than that required for GFRP-RC footing designed as per ACI 440.11-22[1] (that is, 1.12 m).

When the footing thickness was increased to meet two-way shear requirements, the one-way shear capacity decreased to 1055 from 1072 kN due to size effect.

The critical section for a maximum moment is at the face of the column as shown in Fig. 3. The tributary area contributing to the moment was equal to 5.4 m^2 and the ultimate moment was equal to 1356 kN·m. The flexural reinforcement area used was greater of the value required to resist the ultimate moment and minimum reinforcement stipulated in CSA S806-12[3] Sections 8.4.2.1 and 8.4.2.3. It should be noted that the reinforcement area required for the ultimate moment was equal to 0.004 m^2. However, it was increased to 0.03 m^2 (6.5 times more than needed for moment) to meet the minimum reinforcement requirements, which required M29 bars placed at 76 mm center-to-center. The moment capacity of the footing becomes 8682 kN·m, which by far exceeds demand (refer to Table 4). A sketch with dimensions and reinforcement details of GFRP-RC footing designed as per CSA S806-12[3] is given in Fig. 5.

The ratio of reinforcement area required for ultimate moment to that of provided reinforcement area highlights the conservatism in code provisions for minimum reinforcement requirement. The minimum reinforcement requirements for slabs in CSA S806-12,[3] Section 8.4.2.3, that are also applicable to foundations may result in very large quantities of FRP flexural reinforcement. If the intention of this provision is to control shrinkage and temperature cracking, this reinforcement may not be effective in shallow foundations because bars are placed only at the footing bottom. Also, temperature variations and drying shrinkage may not be critical concerns in elements surrounded by soil.

Similar to ACI CODE-440.11, the footing was redesigned as per provisions of CSA S806-12 with new-generation bars as per specifications of ASTM D8505.[7] In CSA S806-12, both one-way and two-way shear provisions depend on elastic modulus and reinforcement ratio. However, the impact of using high-elastic-modulus bars was undermined by reduction in the required reinforcement ratio. Therefore, no positive impact was visible on the shear strength of the footing. On the other hand, the reinforcement area required for flexure design decreased when using high-elastic-modulus bars. For example, when the footing was designed with

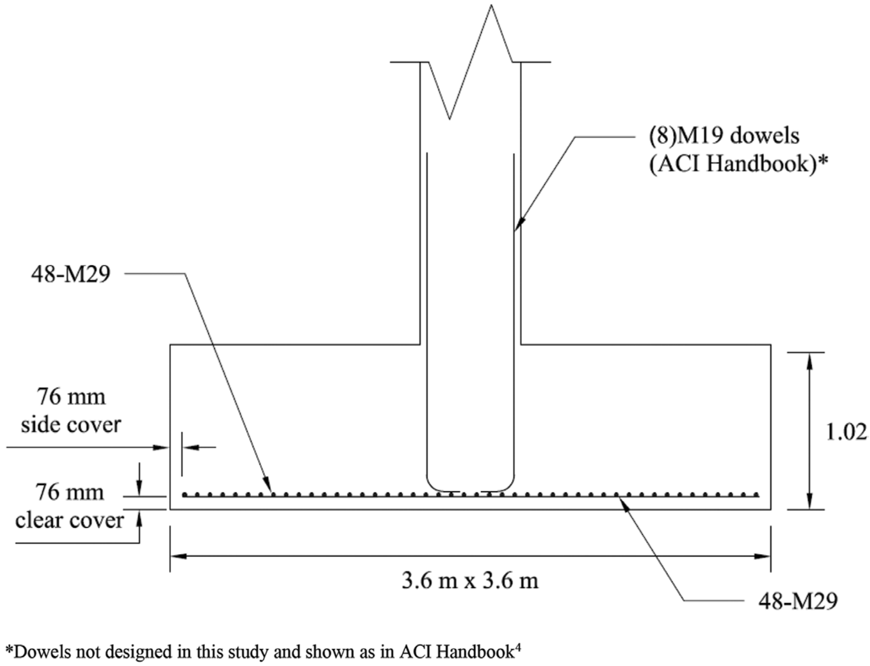

Fig. 5—GFRP-RC footing designed as per CSA S806-12 reinforcement detailing.

new-generation bars, the required number of bars decreased to 35 M29 bars against 48 M29 when using old-generation bars specified in ASTM D7957.

DETAILING OF GFRP REINFORCEMENT

The minimum length required for the anchorage of GFRP reinforcement was calculated as per ACI CODE-440.11-22,[1] Section 25.4.2.1 for M29 bars. The bar location modification factor (y) was taken equal to 1.0 for tension reinforcement placed at 76 mm from the base of the footing. The factor, c_b/d_b, was equal to 2.18. The development length calculated as per Section 25.4.2.1, Eq. (25.4.2.1a) (Eq. (4)), was equal to 1.38 m, which was greater than those calculated with Eq. (25.4.2.1b) (Eq. (5)) and (25.4.2.1c) (Eq. (6)). Therefore, the value (that is, 1.38 m) obtained from Eq. (25.4.2.1a) was adopted in the footing design as per ACI CODE-440.11-22[1] and must be provided in the footing to develop full capacity of the section at the point of maximum moment.

Similarly, the development length was calculated as per CSA S806-12,[3] Eq. (9.1). The modification factor for bar location, k_1, was taken equal to 1.0; concrete density factor, k_2, equal to 1.0; bar size factor, k_3, 1.0; bar fiber factor, k_4, 1.0; and surface profile factor, k_5, was taken equal to 1.0 for sand-coated bars. The development length calculated was equal to 1.23 m. The value obtained from equation 9.1 (that is, 1.23 m) was greater than the minimum required 0.30 m as per Section 9.3.1. Hence, 1.23 m was adopted for footing design as per CSA S806-12[3] and must be provided in the footing to develop full capacity of the section at the point of maximum moment.

ACI CODE-440.11-22[1] and CSA S806-12[3] incorporate stresses in the bar (f_{fr}) in development length equations. Because footings designed as per CSA S806-12[3] required a larger reinforcement area to satisfy minimum reinforcement requirements, the bars were less stressed, consequently requiring less development length than in the case of ACI CODE-440.11-22.[1] The development length required for GFRP-RC as per CSA S806-12[3] is 71% and ACI CODE-440.11-22[1] is 92% more than that required for steel-RC, which required 0.72 m.

The use of new-generation bars resulted in the reduction of the required reinforcement ratio. Therefore, reinforcing bars were placed at bigger spacing compared to old-generation low-elastic-modulus bars. These bars were more stressed compared to closely spaced bars, thereby, required longer development length values. The required development length increased to 2.16 and 1.64 m, respectively, for ACI 440.11-22 and CSA S806-12, respectively.

OBSERVATIONS

Tureyen and Frosch[8] proposed a physical model for calculating concrete contribution to the shear strength of GFRP-RC beams. The model considered cracked section, rather than a section between two cracks, as in the case of ACI 318-19 shear equations.[2] This model was later adopted by ACI CODE-440.11-22[1] with modifications proposed by Nanni et al.[9] for calculating one-way shear, as provided in Eq. (11) and (12) of this manuscript. The modifications proposed by Nanni et al.[9] intended to avoid penalizing lightly reinforced sections. The one-way shear equation in ACI CODE-440.11-22[1] rendered a test-to-predicted ratio equal to 2.59 for 20 GFRP-RC beams, highlighting the conservatism involved in the equations.[10]

Ospina[11] suggested an equation for two-way shear prediction of GFRP-RC slabs, equal to twice the value of one-way shear proposed by Tureyen and Frosch.[8] Realizing the fact that the suggested equation will penalize lightly reinforced slabs, Nanni et al.[9] proposed modifications to the equation proposed by Ospina.[11] Both equations proposed by Ospina[11] and Nanni et al.[9] became part of ACI CODE-440.11-22[1] code, given as Eq. (16) and (17) in this manuscript. The analysis of two-way shear equation in ACI CODE-440.11-22

Table 5—Design of GFRP-RC footing as per ACI CODE-440.11-22[1] at different soil bearing capacities

<table>
<tr><th rowspan="3">Quantity</th><th colspan="7">Soil bearing capacity 268 kN/m²</th><th colspan="7">Soil bearing capacity 536 kN/m²</th></tr>
<tr><th colspan="5">Dimensions, m</th><th colspan="2">Capacity</th><th colspan="5">Dimensions, m</th><th colspan="2">Capacity</th></tr>
<tr><th>h</th><th>b_1</th><th>b_2</th><th>Development length, m</th><th>$\frac{A_{f\_req}}{A_{f\_pro}}$</th><th>Moment, kN·m</th><th>Shear, kN</th><th>h</th><th>b_1</th><th>b_2</th><th>Development length, m</th><th>$\frac{A_{f\_req}}{A_{f\_pro}}$</th><th>Moment, kN·m</th><th>Shear, kN</th></tr>
<tr><td>One-way shear</td><td rowspan="2">1.12</td><td rowspan="2">3.6</td><td rowspan="2">3.6</td><td rowspan="3">1.38</td><td rowspan="2">—</td><td>—</td><td>986</td><td>0.97</td><td rowspan="2">2.5</td><td rowspan="2">2.5</td><td rowspan="3">1.34</td><td rowspan="2">—</td><td>—</td><td>585</td></tr>
<tr><td>Two-way shear</td><td>—</td><td>3488</td><td></td><td>—</td><td>2813</td></tr>
<tr><td>Flexural strength</td><td>—</td><td>—</td><td>—</td><td>0.83</td><td>4706</td><td>—</td><td>—</td><td>—</td><td>—</td><td>0.92</td><td>2330</td><td>—</td></tr>
</table>

rendered a test-to-predicted ratio equal to 1.8 against a database of 51 elevated GFRP-RC slabs.[12-20] Conservatism will further increase when this equation is applied to the foundations. Using shear equations developed for elevated GFRP-RC slabs to shallow foundations leads to implementation challenges for comparatively new technology in the construction industry. As observed in the current study, ACI CODE-440.11-22,[1] shear provisions required cross sections that are 100 and 210 mm bigger than those required by CSA S806-12 and ACI 318-19,[2] respectively. Also, it required reinforcement area that is 170% bigger than that of a cross section with steel-RC (0.019 m^2 versus 0.007 m^2). The bigger reinforcement areas in ACI CODE-440.11-22[1] intend to meet detailing requirements (that is, crack width and stress at service loads), which may not be critical concerns in the footings.

The development length equation in ACI CODE-440.11-22 results in very large values (that is, 92% more than steel-RC), and this, coupled with the challenge of adding a hook at the end of long longitudinal bars, makes design impractical and costly. In the current design example, the required dimensions are large enough to compensate the required development length. However, when the soil stiffness increases or the loads are smaller, the required footing dimensions decrease; thereby, it will be difficult to meet the required development length within the available dimensions. To illustrate this effect, the soil bearing capacity was made twice the value originally given in the *ACI Reinforced Concrete Design Handbook*,[4] (that is, from 268 to 536 kN/m^2). Consequently, the required footing dimensions decreased to 2.5 from 3.6 m as in the case of original footing, as shown in Table 5. Though the footing dimensions decreased but the stress in the bars did not change significantly as minimum reinforcement area required by ACI CODE-440.11-22[1] controls in both cases. Therefore, the required development length was equal to 1.32 m, slightly less than required originally. Adjusting a development length equal to 1.32 m within available dimensions will be difficult. The required development length and available dimensions in two cases discussed previously are provided in Table 5.

The current development length equation is based on the test data obtained more than two decades ago, with bars used in those tests that are no longer used in construction projects.[21] Therefore, it is necessary to reassess and update the development length equation based on recent literature which incorporates improvements in the material and surface properties,[22,23] thereby developing a more representative equation for calculating development length for GFRP-RC members.

CONCLUSIONS AND RECOMMENDATIONS

In this study, an example of square footing subject to axial load only was taken from *ACI Reinforced Concrete Design Handbook*[4] and redesigned with glass fiber-reinforced polymer (GFRP) reinforcement compliant with ASTM D7957 as per ACI CODE-440.11-22[1] and CSA S806-12[3] to show the implications of code provisions. The concrete strength f_c' was assumed to be 28 MPa, bond coefficient k_b = 1.20, and concrete cover was 76 mm in the design of GFRP-reinforced concrete (RC) for both codes.

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

- GFRP-RC footing designed as per ACI CODE-440.11-22 required more concrete cross-section thickness to satisfy shear requirements than steel-RC designed as per ACI 318-19. The thicker cross section may lead to implementation challenges, particularly on sites with water-table issues. Similarly, ACI CODE-440.11-22 required a higher longitudinal reinforcement area to satisfy detailing provisions.
- The GFRP-RC footing designed as per CSA S806-12 required a concrete cross-section thickness slightly more than that of steel-RC, but less than as per ACI 440.11-22. However, the longitudinal reinforcement area was much higher than in the other two cases.
- It was observed that ACI CODE-440.11-22 shear equations disregard arching effect in thicker members for one-way shear and adopts an empirical coefficient in two-way shear that seems conservative. Hence, the required thickness of a shallow foundation is bigger than that designed as per CSA S806-12.
- The equations for computing development length in GFRP are more demanding than in the case of steel (that is, 92% more than that of steel-RC). This is challenging when dealing with footings of relatively small dimensions.
- The use of new-generation high-elastic-modulus, high-strength bars did not affect the shear strength. However, a positive impact was noticed on the flexural capacity of GFRP-RC footings.

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NOTATION

A_{fv} = area of shear reinforcement, mm^2
b = width of cross section, mm
c_b = lesser of: a) distance from center of bar to nearest concrete surface; or 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
d_v = effective shear depth, taken as greater of 0.9d or 0.72h, 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, MPa
f_{fr} = tensile stress in GFRP reinforcement required to develop full nominal section capacity, MPa
f_{fs} = stress at service loads, MPa
k_b = bond-dependent coefficient
k_{cr} = ratio of depth of elastic cracked section neutral axis to effective depth
k_m = coefficient considering effect of moment at section on shear strength
k_r = coefficient considering effect of reinforcement rigidity on its shear strength
M_f = factored moment, kN·m
M_u = ultimate factored moment at section, kN·m
n_f = modular ratio
P_u = ultimate factored load, kN
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, kN
V_f = nominal shear strength provided by GFRP shear reinforcement, kN
V_n = nominal shear strength, kN
V_r = factored shear resistance, kN
V_{sF} = factored shear resistance provided by FRP shear reinforcement, kN
V_u = factored shear force at section, kN
v_c = stress corresponding to two-way shear strength of slab or footing, MPa
v_r = factored shear stress resistance, MPa
w_c = density, unit weight of normal weight concrete, kg/m^3
β_c = ratio of long side to short side of column
ε_f = strain in GFRP flexural reinforcement
Φ = strength reduction factor
λ = factor to account for concrete density
λ_s = size effect factor
ρ_{FW} = longitudinal FRP reinforcement ratio
ω = bar location modification factor

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