CURRENT FRP-REINFORCED CONCRETE DESIGN TRENDS IN ACI 440.1R

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1 INTRODUCTION AND OBJECTIVES

For the first time since its inception in 2001, the 2006 edition of ACI 440.1R [1] for the design of structural concrete members reinforced with FRP bars has been published as a "conventional" technology document instead of an "emerging" technology guideline. This category upgrading reflects the successful application of FRP reinforcing bars as concrete reinforcement in a wide variety of projects worldwide together with the rising consensus among researchers and design engineers that current FRP-reinforced concrete design theories and recommendations have also reached an acceptable level of maturity.

The upgrading of [1] comes accompanied by the introduction of design recommendations for shear design, indirect deflection control and flexural crack control of concrete beams and one-way slabs reinforced with FRP bars, together with the addition of punching shear design provisions for two-way slabs with FRP bars. The indirect deflection control and punching shear design provisions were previously unavailable in earlier ACI 440.1R versions.

The most important feature of the newly adopted or improved design procedures is in the fact that they are mechanistic and general in nature. This feature represents a significant drifting from precursor ACI 440.1R design procedures which were usually based on traditional steel-reinforced concrete design recommendations modified and empirically-tuned through modification factors. The main advantage of having incorporated mechanistic design procedures in ACI 440.1R-06 lies in the fact that most of these design procedures can be applied equally well to FRP- or steel-reinforced concrete structures. This design philosophy trend not only provides designers with more transparent design methodologies but also paves the way for a potential unification of FRP- and steel-reinforced concrete design procedures in the future.

This paper describes in detail the latest changes that were adopted in ACI 440.1R-06. The information is divided into two categories: refined design procedures and new design procedures. The paper also describes those design aspects that are currently undergoing discussion at task group level to attempt replacing existing first generation design procedures with more robust design rules for the next revision of ACI 440.1R. The paper also identifies areas where additional research is needed. The main motivation behind the latter is to plan ahead and facilitate the job of professional designers when faced with the task of designing concrete structures or members in which the use of FRP reinforcing bars is not currently addressed by ACI 440.1R-06.

2 IMPROVED DESIGN PROCEDURES IN ACI 440.1R-06

2.1 Flexural Cracking Control in Concrete Beams and One-way Slabs with FRP Bars

Prior to 2006, flexural cracks in ACI 440.1R were controlled directly based on a modified version of the traditional Gergely-Lutz [2] model. The original Gergely-Lutz model was modified through an empirical factor to account for the particular properties of FRP reinforcement.

The crack control design provisions in ACI 440.1R-06 are based on the mechanistic approach by Frosch [3]. The adoption of this crack control model in [1] was influenced to a large extent by ACI Committee 318's decision to replace the traditional Gergely-Lutz model in the ACI 318 code [4] with

Frosch's model because of unacceptable crack width predictions using the empirical Gergely-Lutz model for the case of concrete beams and slabs with large concrete covers.

In ACI 440.1R-06, the maximum flexural crack width at the tension face of a beam or one-way slab with FRP bars is calculated as

$$w = 2\frac{f_r}{E_r}\beta\, k_b\sqrt{d_c^2 + \frac{s}{2}} \tag{1}$$

where f_r is the reinforcing bar stress, calculated assuming elastic-cracked conditions, E_r is the modulus of elasticity of the reinforcement, β is the ratio of the distance from the neutral axis to the tension face of the member to the distance from the neutral axis to the centroid of the tensile reinforcement, d_c is the cover thickness from the tension face to the center of the closest reinforcing bar, s is the bar spacing (taken as the member width for a single bar case) and k_b is a coefficient that accounts for the bond characteristics of the reinforcement. For FRP bars with bond similar to that of ordinary steel reinforcing bars, k_b is set equal to one. For FRP reinforcing bars with better bond, k_b is less than unity whereas for FRP reinforcing bars with inferior bond properties, k_b is greater than one. If k_b is unknown, it shall be taken as 1.4 for non-smooth bars, as recommended in [5].

The reference crack width limits for Eq. (1) are, respectively, 0.5 mm (0.020 in.) and 0.7 mm (0.028 in.), for exterior and interior exposure conditions. These limits were adopted from [6]. They are more relaxed than those associated with conventional reinforced concrete design due to the superior corrosion resistance of FRP. The main advantage of Eq. (1) is that the procedure is mechanistic and it is thus applicable to both steel- and FRP-reinforced concrete beams and one-way slabs.

2.2 Shear Design of Concrete Beams and One-way Slabs with FRP Bars

As in ACI 318, the nominal shear strength of a reinforced concrete cross section, V_n, in ACI 440.1R is defined as the sum of the shear resistance provided by the concrete, V_c, and the steel shear reinforcement, V_s. Prior to 2006, the concrete shear contribution in FRP-reinforced beams was calculated based on an empirical equation in which the FRP axial stiffness effect on the concrete shear strength contribution was weighted against that of steel reinforcement. In an attempt to rely on a more transparent design procedure, ACI 440.1R-06 adopted the beam shear model proposed in [7] to evaluate the concrete shear strength of beams reinforced with FRP bars. The concrete shear capacity V_c of flexural members using FRP bars as main reinforcement can be evaluated according to Eq. 2.

$$V_c = \frac{2}{5}\sqrt{f_c'}\,b_w c \tag{2}$$

where b_w is the width of the web and c is the cracked transformed section neutral axis depth. For singly reinforced rectangular beams, c may be computed as kd, equal to

$$k = \sqrt{2\rho_f n_f + (\rho_f n_f)^2} - \rho_f n_f \tag{3}$$

where $\rho_f = \dfrac{A_f}{b_w d}$. Equation (2) accounts for the axial stiffness of the FRP reinforcement through the neutral axis depth. This equation provides a reasonable factor of safety for FRP reinforced concrete specimens across the range of reinforcement ratios and concrete strengths tested to date [7]. Equation (2) may be rewritten as Eq. (4), which is simply the ACI 318 beam shear equation for steel reinforcement, V_c, modified by the factor 12/5 k which accounts for the axial stiffness of FRP.

$$V_c = \left(\frac{12}{5}k\right)\frac{1}{6}\sqrt{f_c'}\,b_w d \tag{4}$$

2.3 Anchorage and Development Length in Straight FRP Bars

The development length equation for FRP reinforcing bars in ACI 440.1R-06 is based on the work of Wambeke and Shield [8] who, based on concepts introduced by Orangun et al [9] and GFRP-reinforced beam bond test results reported in the literature, developed the following equation to relate the average bond stress, u, normalized by the square root of the concrete compressive strength, to the normalized cover to the center of the bar, C/d_b, and the normalized splice length, d_b/l_e.

$$\frac{u}{0.083\sqrt{f_c'}} = 4 + 0.3\frac{C}{d_b} + 100\frac{d_b}{l_e} \tag{5}$$

where C is the lesser of the cover to the center of the bar or one-half of the center-on-center spacing of the bars being developed and l_e is the embedded length. Any benefits from confinement on bond strength in Eq. (5) are neglected due to the low relative rib area of most common GFRP reinforcing bars.

Solving Eq. (5) for the developable bar stress for a given cover and embedment length, results in

$$f_{fe} = \frac{0.083\sqrt{f_c'}}{\alpha}\left(13.6\frac{l_e}{d_b} + \frac{C}{d_b}\frac{l_e}{d_b} + 340\right) \le f_{fu} \tag{6}$$

in which C/d_b should not be taken larger than 3.5 and α is a top bar effect factor equal to 1.5 for reinforcing bars with more than 300 mm of concrete cast below. Equation 6 can be applied to AFRP and CFRP bars. The rationale behind Eq. (6) is that the bar stress value varies linearly from zero to f_{fe} along the first $20d_b$ of the bar embedment. ACI 440.1R-06 recommends embedment lengths of at least $20d_b$. Equation (6) is not valid for embedment lengths greater than $100d_b$.

3 NEW DESIGN PROCEDURES IN ACI 440.1R-06

3.1 Indirect Deflection Control of Concrete Beams and One-way Slabs with FRP Bars

This design provision came about due to lack of guidance in earlier versions of the ACI 440.1R guide regarding preliminary dimensioning of concrete beams and one-way slabs with FRP bars. Unlike some traditional structural concrete design codes, such as ACI 318, where direct deflection calculations can be waived if minimum member depth requirements are met, the indirect deflection control provisions in ACI 440.1R-06 are meant only for member pre-dimensioning purposes. This is the result of deflections in FRP-reinforced concrete members being more sensitive to the variables affecting deflection compared to steel-reinforced members because of the brittle-elastic nature, variable stiffness and specific bond characteristics of FRP reinforcing bars.

The indirect deflection control design provisions in ACI 440.1R-06 are based on the work of Ospina et al [10] who defined a minimum depth, h, for concrete beams and one-way slabs as defined by a maximum span to depth ratio, L/h, evaluated as

$$\frac{L}{h} \le \frac{48\eta}{5K_1}\left(\frac{1-k}{\varepsilon_f}\right)\left(\frac{\Delta}{L}\right)_{max} \tag{7}$$

In Eq. (7), $\eta = d/h$; $(\Delta/\ell)_{max}$ is the limiting service load deflection-span ratio; k is the ratio of the compressive concrete zone to the effective flexural depth, evaluated assuming cracked-elastic conditions; K_1 is a boundary condition factor, taken as 1.0, 0.8, 0.6, and 2.4 for uniformly loaded simply-supported, one-end continuous, both-ends continuous, and cantilevered spans, respectively; and ε_f is the service reinforcement strain at midspan or at the support for cantilevered members.

The maximum span-to-depth ratio limitation per Eq. (7) corresponds to a limiting curvature associated with a given reinforcement strain and a target deflection-span ratio Δ/L. This indirect deflection control procedure can be applied to any type of reinforcement provided that the reinforcement stress-strain response follows Hooke's law across the reinforcement stress range along which deflections are controlled.

Ospina and Gross [11] modified Eq. (7) by the ratio I_e/I_g to account for concrete's tension stiffening effect in the minimum member depth predictions. They used the I_e and I_g values defined by ACI

440.1R-06. This led to the development of Table 1 which was adopted by ACI 440.1R-06 for preliminary dimensioning of FRP-reinforced concrete beams and one-way slabs. Tabulated values are based on an assumed service deflection limit of *L/*240 under total service load, and reinforcement ratios of $3.0\rho_{fb}$ and $2.0\rho_{fb}$ for beams and slabs, respectively.

Table 1 Recommended Minimum Thickness of Non-prestressed Beams or One-way Slabs

	Minimum Thickness, h			
	Simply-supported	One end continuous	Both ends continuous	Cantilever
Solid one-way slabs	ℓ/13	ℓ/17	ℓ/22	ℓ/5.5
Beams	ℓ/10	ℓ/12	ℓ/16	ℓ/4

It is worth noting that the minimum member thickness requirements of Table 1 do not guarantee that all deflection considerations will be satisfied in a project.

3.2 Punching Shear of Concrete Two-way Slabs with FRP Bars

In earlier versions of ACI 440.1R, no guidance was given on the punching shear design of two-way slabs with FRP bars. This design provision was introduced after a series of test results [12, 13, 14] of isolated column-supported FRP-reinforced two-way slabs subjected to uniform gravity loads demonstrated that the axial stiffness of the FRP reinforcement, as well as the concrete strength, f'_c, significantly affect the transverse shear response of two-way slabs with FRP bars. Test results also indicate that an increase in the top FRP mat stiffness increases punching shear capacity and decreases the ultimate slab deflection and that punching shear failure in slabs reinforced with FRP bars occurs suddenly and is brittle in nature. Although not addressed directly by ACI 440.1R-06, the shear response of two-way slabs with FRP grids at punching does not exhibit a sharp load drop as observed in slabs reinforced with either steel or FRP bars. Instead, they continue to absorb energy in a relatively stable fashion following initial failure as evidenced in [14] and [15].

Based on the work of Ospina [16], who extended the mechanistic Tureyen and Frosch beam shear model to two-way slab shear design, ACI 440.1R-06 introduced the following equation to calculate the concentric punching shear capacity of FRP-reinforced two-way concrete slabs either supported by interior columns or subjected to concentrated loads.

$$V_c = \frac{4}{5}\sqrt{f_c'}\,b_o c \tag{8}$$

where b_o is the perimeter of critical section for slabs and footings, and c is the cracked transformed section neutral axis depth, equal to kd, with k calculated as in Eq. (3) except that ρ_f now refers to the slab reinforcement ratio. For slabs with orthogonal FRP reinforcing mats, the slab reinforcement ratio is calculated as the average of the reinforcement ratios in each direction. In the evaluation of Eq. (8), b_o should be computed at 0.5d away from the column or loading patch and the shape of the critical surface should be the same as that of the column. Intuitively, Eq. (8) can be rewritten as

$$V_c = \left(\frac{12}{5}k\right)\frac{1}{3}\sqrt{f_c'}\,b_o d \tag{9}$$

which is simply the basic ACI 318 concentric punching shear equation for steel-reinforced slabs modified by the factor 12/5 k which accounts for the effect of the axial stiffness of the FRP reinforcement on the slab shear strength. The precursor of Eq. (9) is reported in [16] as

$$V_c = Nk\sqrt{f_c'}\,b_o d \tag{10}$$

with N = 5/6. Equation (10) provides very conservative punching shear capacity estimates for FRP-reinforced two-way slabs across the range of reinforcement ratios and concrete strengths tested to date. As pointed out by Ospina [16], much more accurate predictions can be obtained by setting N = 5/4. Nevertheless, ACI 440.1R-06 adopted N = 4/5 because it leads to a two-way shear strength that is twice the beam shear strength and also provides an extra degree of conservatism in design because of the limited number of punching shear test results on slabs with FRP bars to date. It is expected for this code provision to be refined as more test results become available.

4 ONGOING ACI 440 H DISCUSSION

Two main topics are being currently discussed among ACI 440 H task groups on deflections and cracking for potential implementation in the ACI 440.1R guideline. These refer to direct deflection control and indirect flexural cracking control.

4.1 Direct Deflection Control in FRP-reinforced Concrete Members

Deflections in ACI 440.1R-06 are calculated based on an effective moment of inertia, I_e , calculated per Eq. (11). This equation is largely influenced by the well known Branson's equation.

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \beta_d I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \le I_g \tag{11}$$

The term β_d is an empirically-tuned factor, defined by ACI 440.1R-06 as "a reduction coefficient related to the reduced tension stiffening exhibited by FRP-reinforced members". Equation (11) is identical to the I_e equation given in previous versions of the ACI 440.1R guideline except for the β_d term which was redefined based on a statistical evaluation of FRP-reinforced concrete beam test data reported by Gross at the 2004 San Francisco meeting of ACI Subcommittee 440H. The term β_d is now defined as

$$\beta_d = \frac{1}{5}\left(\frac{\rho_f}{\rho_{bf}}\right) \le 1 \tag{12}$$

where ρ_{bf} is the balanced reinforcement ratio, defined as

$$\rho_{fb} = 0.85\beta_1 \frac{f_c'}{f_{fu}} \frac{E_f \varepsilon_{cu}}{E_f \varepsilon_{cu} + f_{fu}} \tag{13}$$

Equation (13) was empirically calibrated for rectangular FRP-reinforced sections and is not directly applicable for FRP-reinforced T-beams. Its adequacy has been questioned by Bischoff [17]. First of all, the β_d dependency on ρ_{bf} is conceptually incorrect because this would imply that different deflections can be predicted for members reinforced with FRP bars that have similar stiffness but different ultimate tensile strength, f_{fu}, as would be the case of GFRP and AFRP. Since deflection calculation is a problem associated with the serviceability limit state, the procedure should not be linked to a ultimate state limit parameter such as f_{fu} . ACI Subcommittee 440H task group on deflections is currently addressing this observation for the next revision of the ACI 440.1R document. A second observation refers to the β_d definition in ACI 440.1R-06. FRP-reinforced concrete beams and one-way slabs do not have reduced tension stiffening because of the FRP reinforcement properties but because the tension stiffening component in the original Branson's equation becomes so high for FRP-reinforced concrete beams and one-way slabs that it has to be reduced to realistic levels through β_d [17].

The original Branson's equation was empirically derived for steel-reinforced beams with I_g / I_{cr} less than 4. Branson's equation tends to overpredict the stiffness of members with low FRP reinforcement ratios, i.e. members with $I_g / I_{cr} > 5$, and therefore to underestimate their deflections. Thus, the β_d term should be defined simply as a factor that reduces the tension stiffening component in Branson's equation for deflection calculations in members with FRP to more realistic levels. Since Branson's equation works well for beams with $2 < I_g/I_{cr} < 3$, limiting the reduced stiffness ratio I_g / I_{cr} to a value not greater than 4.0 should provide a reasonable amount of tension stiffening for FRP RC members and correct for the shortcomings of the current ACI 440.1R approach.

Instead of redefining β_d to reduce the I_g contribution to the weighted average calculation of the I_e term, Bischoff [18,19] proposes an alternative expression, based on a rational concept of providing a weighted average of flexibility rather than stiffness, to evaluate the effective moment of inertia of both steel and FRP-reinforced concrete beams and one-way slabs.

$$I_e = \frac{I_{cr}}{1-\eta\left(\frac{M_{cr}}{M_a}\right)^2} \leq I_g \qquad \text{with} \qquad \eta = 1-\frac{I_{cr}}{I_g} \tag{14}$$

Equation (14) is valid only if the applied moment is equal to or greater than the cracking moment. If M_a is significantly lower than M_{cr}, then the deflection calculation should be based on I_g. If M_a is slightly less than M_{cr}, the designer is advised to assume a cracked section since factors such as shrinkage and temperature may cause the section to crack under these conditions. One of the advantages of Eq. (14) is the deletion of the empirically-tuned β_d term. The ACI 440 H task group on deflections is currently evaluating the validity of Eq. (14) in light of reported test results.

4.2 Indirect Flexural Cracking Control in Concrete Beams and One-way Slabs with FRP Bars

Rather than controlling cracks directly, Ospina and Bakis [20] propose an indirect flexural crack control procedure in terms of maximum bar spacing. The procedure follows the format of the ACI 318 flexural crack control model which replaced in 1999 the traditional z-factor approach (based on Gergely-Lutz model) for steel-reinforced concrete beam and one-way slab design with an indirect procedure that calculates a maximum bar spacing, also eliminating exposure condition dependence. The proposed procedure does not represent a significant departure from the current ACI 440.1R-06 crack control model. Instead, it is just a re-arrangement of the existing ACI 440.1R-06 crack control rules. Re-arranging Frosch's crack width equation and solve for the maximum bar spacing results in

$$s = 2\sqrt{\left(\frac{w\,E_r}{2f_r\beta\,k_b}\right)^2 - d_c^2} \tag{15}$$

Ospina and Bakis [20] propose

$$s = 1.2\frac{E_r w}{f_r k_b} - 2.5\,c_c \leq 0.95\frac{E_r w}{f_r k_b} \tag{16}$$

which is just a discontinuous representation of Eq. (15) following the ACI 318 crack control equation format. Equation (16) controls flexural cracks indirectly because the bar spacing requirement is linked to a limiting crack width value. It can be applied regardless of whether the reinforcement is steel or FRP, and explicitly accounts for the influence of the limiting crack width on the prescriptive bar spacing. Equation (16) is meant to be used in conjunction with crack width limits specified by codes.

5 RESEARCH NEEDS

Just as ACI 440.1R-06 entered a stage where first-generation design procedures are being replaced by mechanistic, more general design models, there are still plenty of areas where significant research work is required.

5.1 Flexural Design

The tension-controlled and compression-controlled terminology adopted by ACI 440.1R-06 is not fully consistent with the flexural design definitions given in ACI 318-05. Despite the substantial differences between steel and FRP reinforcement, the flexural design rules of ACI 440.1R-06 are still referenced to whether the balanced reinforcement ratio is exceeded or not. This parameter was abolished in the ACI 318 code. Unless there is evidence demonstrating it should be retained, ACI 440

H is encouraged to examine alternative flexural design approaches that do not use the balanced reinforcement ratio as a reference parameter.

Flexural design provisions for FRP-reinforced two-way slabs are virtually non-existing. There is absolutely no guidance, and even less experimental evidence, indicating how to design and detail an FRP-reinforced two-way slab. There is no information on distribution rules to allocate flexural moments in column and middle strips and also to deal with torsional effects.

5.2 Shear Design

As far as beam shear design is concerned, there is a need to evaluate through experimental testing the validity of the Tureyen-Frosch model for FRP-reinforced concrete T-beams.

There is also very limited guidance available on punching shear design of one-way slabs and bridge decks. This is surprising because bridge decks constitute perhaps one of the most popular FRP applications. The punching model proposed by El-Gamal, El-Salakawy and Benmokrane [21] is robust; in fact, to the author's best knowledge, it is the only punching model in the literature to date that accounts for the effect of edge restraint conditions on the punching capacity of slabs with FRP bars. However, the procedure is empiric in nature which may conflict with ACI 440's decision to adopt the mechanistic Tureyen-Frosch model for shear design.

Further research, in addition to the tests reported by Zaghloul and Razaqpur [22], is needed to examine the effect of moment transfer on the punching shear capacity of FRP-reinforced two-way slabs at interior columns. It is also required to evaluate the current punching shear model applicability for two-way slabs at interior columns in presence of unbalanced moments. The shear response of slabs at edge and corner column locations also deserves examination.

5.3 Column Longitudinal Reinforcement

In several instances, FRP reinforcement has become potentially usable for columns in industrial facilities for reasons of magnetic transparency. There is really no reason to prevent the use of longitudinal FRP reinforcement in columns if the members are well confined.

5.4 Serviceability

There is limited experimental evidence examining long-term deflections in FRP-reinforced concrete beams and one- and two-way slabs. The minimum thickness requirements of Table 1 need be validated through experimental work.

5.5 Development Length and End Anchors

There is also research need for anchor terminations that can replace bent-up ends and long lapped bars for large diameter reinforcement.

5.6 Thin FRP Grid Reinforcement

There is limited experimental evidence and supporting theory on design of concrete members with thin FRP grid reinforcement. These include structural elements where the grid reinforcement is used either as main or as secondary reinforcement, including bridge decks, T-beams, double-tee beams, shotcrete applications, and architectural panels.

6 CONCLUSION

This paper reported the latest changes introduced in ACI 440.1R-06 as part of a conceptual plan to replace first-generation empirically-tuned design models with mechanistic, general design procedures. The paper also described those design aspects that are currently undergoing discussion at task group level for potential inclusion in the next version of the ACI 440.1R guideline and identifies areas where substantial research work is required.

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