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Guides and Specifications for the Use of Composites in Concrete and Masonry Construction in North America

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Abstract

This paper covers the latest developments in the preparation of design guidelines, construction specifications, and inspection and quality control recommendations related to the use of composites in construction. The forms of FRP composites addressed are limited to bars and laminates for reinforcement of concrete and masonry structures (new construction and repair/rehabilitation). FRP bars are being used as the internal reinforcement in concrete members when the conventional steel bars may be undesirable for a host of reasons (e.g., corrosion), and principles for design and construction have been established and proposed to industry by the American Concrete Institute (ACI). Conversely, strengthening of concrete members with externally bonded FRP composites in the form of laminates or near surface mounted (NSM) bars can now be considered an “acceptable practice.” Also in this case, the design and construction principles for use in practice are being finalized by ACI. The drivers for FRP strengthening technology are several, but perhaps the most relevant one is the ease of installation. On the wave of historical structures restoration projects conducted in Europe, there is an increasing interest in masonry-type applications even though no institution-sanctioned guidelines are available at present.

Introduction

In this paper, reference is made primarily to two technical documents produced by ACI Committee 440 under the new ACI series of emerging technology. The first one has been recently published (ACI Committee 440, 2001) and provides recommendations for design and construction of FRP reinforced concrete (RC) structures. The second one is under development (ACI Committee 440, 2001a) and provides guidance for the selection, design, and installation of FRP systems for externally strengthening concrete structures.

It should be noted that only notations critical to the understanding of the paper are defined herein and equations are expressed in US customary units.

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Design and Construction of Concrete Reinforced with FRP Bars

FRP materials are mostly anisotropic, do not exhibit yielding, and for design purpose, are considered elastic until failure. Design procedures should account for a lack of ductility in concrete reinforced with FRP bars. Both strength and working stress design approaches are considered by ACI. In particular, the guide makes reference to provisions as per ACI 318-95 Building Code Requirements for Structural Concrete and Commentary (ACI Committee 318, 1995). An FRP RC member is designed based on its required strength and then checked for serviceability and ultimate state criteria (e.g., crack width, deflection, fatigue and creep rupture endurance). In many instances, serviceability criteria may control the design. This ACI document does addresses prestressed concrete (PC) applications.

Design Values. The design tensile strength that should be used in all design equations is given in Eq. (1). The design rupture strain should be determined similarly, whereas the design modulus of elasticity is the same as the value reported by the manufacturer.

$$f_{fu} = C_E f^*_{fu} \qquad (1)$$

where:

f_{fu} = design tensile strength of FRP, considering reductions for service environment

C_E = environmental reduction factor, given in Table 1 for various fiber types (column Int.) and exposure conditions

f^*_{fu} = guaranteed tensile strength of an FRP bar defined as the mean tensile strength of a sample of test specimens minus three times the standard deviation ($f^*_{fu} = f_{u,ave} - 3\sigma$)

Table 1: C_E factor for various fiber systems and exposure conditions (ACI 440 2001 and 2001a).

Exposure condition	Carbon		Glass		Aramid	
	Int.[a]	Ext.[b]	Int.[a]	Ext.[b]	Int.[a]	Ext.[b]
Interior exposure	1.0	0.95	0.8	0.75	0.9	0.85
Exterior exposure	0.9	0.85	0.7	0.65	0.8	0.75
Aggressive environment	n/s	0.85	n/s	0.50	n/s	0.70

[a] = New construction/internal; [b] = Strengthening/external; n/s = Not specified

Design parameters in compression are not addressed since the use of FRP bars in this instance is discouraged.

Flexure

Behavior and Failure Modes. If FRP reinforcement ruptures, failure of the member is sudden and catastrophic. However, there would be some limited warning of impending failure in the form of extensive cracking and large deflection due to the significant elongation that FRP reinforcement experiences before rupture. The concrete crushing

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failure mode is marginally more desirable for flexural members reinforced with FRP bars (Nanni, 1993). In conclusion, both failure modes (i.e., FRP rupture and concrete crushing) are acceptable in governing the design of flexural members reinforced with FRP bars provided that strength and serviceability criteria are satisfied. To compensate for the lack of ductility, the member should possess a higher reserve of strength. The suggested margin of safety against failure is therefore higher than that used in traditional steel-RC design.

Figure 1 shows a comparison of the theoretical moment-curvature behavior of beam cross-sections designed for the same factored strength, $\Phi M_{n,}$ following the design approach of ACI 318 and ACI 440 (including the recommended strength reduction factors). Three cases are presented in addition to the steel reinforced cross section: two sections reinforced with GFRP bars and one reinforced with CFRP bars. For the section experiencing GFRP bar rupture, the concrete dimensions are larger than for the other beams in order to attain the same design capacity.

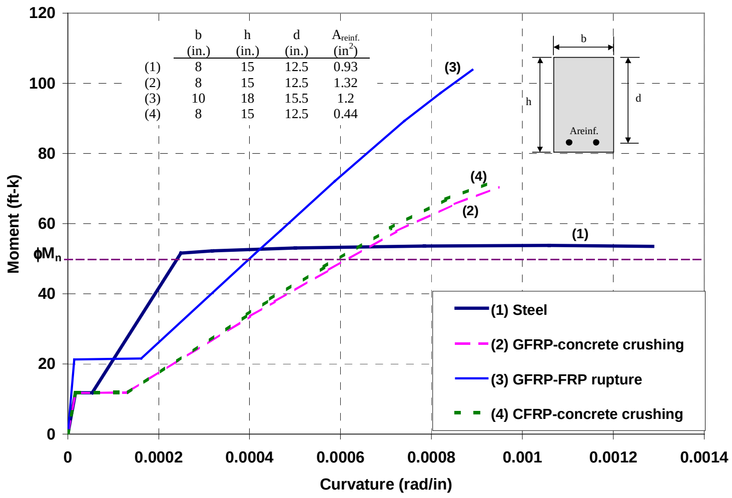

Figure 1. Moment-curvature relationships for RC sections using steel and FRP bars (ACI 440 2001).

<u>Φ factor</u>. Because FRP members do not exhibit ductile behavior, a conservative strength-reduction factor is adopted and set equal to 0.70 rather than 0.90 as per steel RC. Furthermore, because a member that experiences an FRP rupture exhibits less plasticity than one that experiences concrete crushing, a strength-reduction factor of 0.50 is recommended for FRP rupture-controlled failure. While a concrete crushing failure mode can be predicted based on calculations, the member as constructed may not fail accordingly. For example, if the concrete strength is higher than specified, the member can fail due to FRP rupture. For this reason and in order to establish a transition between the two values of Φ, a section controlled by concrete crushing is defined as a section in which the reinforcement ratio is larger or equal to 1.4 times the balanced reinforcement ratio ($\rho_f \geq 1.4\, \rho_{fb,}$) and a section controlled by FRP rupture is defined as one in which $\rho_f < \rho_{fb}$.

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Minimum reinforcement. If a member is designed to fail by FRP rupture, $\rho_f < \rho_{fb}$, a minimum amount of reinforcement, $A_{f,min}$, should be provided to prevent failure upon concrete cracking (that is, $\Phi M_n \geq M_{cr}$ where M_{cr} is the cracking moment). The minimum reinforcement area for FRP reinforced members is obtained by multiplying the existing ACI 318 limiting equation for steel by 1.8 (i.e., 1.8 = 0.90/0.50 which is the Φ ratio).

Crack Width. For FRP reinforced members, the crack width, w, can be calculated from the expression shown in ACI 318 with the addition of a corrective coefficient, k_b, for the bond quality. The k_b term is a coefficient which accounts for the degree of bond between the FRP bar and the surrounding concrete. For FRP bars having bond behavior similar to steel bars, k_b is assumed equal to one. Using the test results from Gao et al. (1998), the calculated values of k_b for three types of GFRP bars were found to be 0.71, 1.00, and 1.83. When k_b is not known, a value of 1.2 is suggested for deformed FRP bars.

Creep rupture and fatigue. Values for safe sustained and fatigue stress levels are given in Table 2. These values are based on experimental results with an imposed safety factor of 1/0.60.

Table 2. Creep rupture and fatigue stress limits in FRP reinforcement (ACI 440 2001 and 2001a).

Fiber type	Glass FRP	Aramid FRP	Carbon FRP
Creep rupture stress limit, $F_{f,s}$	$0.20\,f_{fu}$	$0.30\,f_{fu}$	$0.55\,f_{fu}$

Shear

Issues to be addressed when using FRP as shear reinforcement include: relatively low elastic modulus; high tensile strength; no yield point; tensile strength of the bent portion significantly lower than the straight one; and low dowel resistance.

According to ACI 318, the nominal shear strength of a steel RC cross section, V_n, is the sum of the shear resistance provided by concrete, V_c, and the steel shear reinforcement, V_s. Similarly, the concrete shear capacity $V_{c,f}$ of flexural members using FRP as main reinforcement can be derived from V_c multiplied by the ratio between the axial stiffness of the FRP reinforcement ($\rho_f E_f$) and that of steel reinforcement ($\rho_s E_s$). For practical design purposes, the value of ρ_s can be taken as $0.5\rho_{s,max}$ or $0.375\rho_b$. Considering a typical steel yield strength of 420 MPa (60 ksi) for flexural reinforcement, the equation for $V_{c,f}$ is that shown in Eq. (2) (noting $V_{c,f}$ cannot be larger than V_c).

$$V_{c,f} = \frac{\rho_f E_f}{90\beta_1 f_c'} V_c \tag{2}$$

The ACI 318 method used to calculate the shear contribution of steel stirrups, V_s, is applicable when using FRP as shear reinforcement, with the provision that the stress level in the FRP shear reinforcement, f_{fv}, should be limited to control shear crack widths, maintain shear integrity of the concrete, and avoid failure at the bent portion of the FRP stirrup, f_{fb}. Eq. (3) gives the stress level in the FRP shear reinforcement at ultimate for use in design. An expression for f_{fb} is given in ACI 440.1R-01.

$$f_{fv} = 0.002E_f \leq f_{fb} \quad (3)$$

Development Length

The development length of FRP reinforcement can be expressed as shown in Eq. (4). This should be a conservative estimate of the development length of FRP bars controlled by pullout failure rather than concrete splitting.

$$\ell_{bf} = \frac{d_b\, f_{fu}}{2700} \quad (4)$$

Manufacturers can furnish alternative values of the required development length based on substantiated tests conducted in accordance with available testing procedures. Reinforcement should be deformed or surface-treated to enhance bond characteristics with concrete.

Design and Construction of FRP Systems For Strengthening

Information on material properties, design, installation, quality control, and maintenance of FRP systems used as external reinforcement is presented in this ACI document (ACI Committee 440, 2001a). This information can be used to select an FRP system for increasing the strength and stiffness of RC beams or the ductility of wrapped columns. This document does not address masonry walls.

It is recommended that the increase in load-carrying capacity of a RC or PC member strengthened with an FRP system be limited. The philosophy is that a loss of FRP reinforcement should leave a member with sufficient capacity to resist at least 1.2 times the design dead load and 0.85 times the design live load. Design recommendations are based on limit-states-design principles. This approach sets acceptable levels of safety against the occurrence of both serviceability and ultimate limit states (i.e., deflections, cracking, failure, stress rupture, fatigue). In determining the ultimate strength of a member, all possible failure modes and resulting strains and stresses in each material should be assessed. For evaluating the serviceability of an element, engineering principles, such as modular ratios and transformed sections, can be used.

FRP-strengthening systems should be designed in accordance with ACI 318 strength and serviceability requirements, using the outlined load and strength-reduction factors. Additional reduction factors applied to the contribution of the FRP reinforcement are recommended to reflect the limited body of knowledge. The environmental-reduction factor C_E to determine the FRP design strength and strain was given in Table 1 for different fiber types (see column Ext.). Similarity with values adopted for internal FRP reinforcement should be noted.

Flexure

Failure Modes. Guidance is given on the calculation of the flexural strengthening effect of adding longitudinal FRP reinforcement to the tension face of a rectangular RC member (concepts could be extended to cover T-sections and I-sections as well as PC). The nominal flexural capacity can be computed as per ACI 318. An additional reduction factor, $\psi_f = 0.85$, is applied to the flexural-strength contribution of the FRP reinforcement to account for uncertain reliability of the FRP reinforcement.

As FRP materials are linearly elastic until failure, the stress level in the FRP reinforcement is always proportional to strain. The effective strain level in the FRP reinforcement at the ultimate-limit state can be found from Eq. (5).

$$\varepsilon_{fe} = \varepsilon_{cu}\left(\frac{h-c}{c}\right) - \varepsilon_{bi} \leq \kappa_m \, \varepsilon_{fu} \tag{5}$$

with:

$$\kappa_m = \begin{cases} 1 - \dfrac{nE_f t_f}{2,400,000} & \text{for} \quad nE_f t_f \leq 1,200,000 \\ \dfrac{600,000}{nE_f t_f} & \text{for} \quad nE_f t_f > 1,200,000 \end{cases}$$

where: n is the number of plies; E_f is the elastic modulus of each ply; and t_f is the ply thickness. The term κ_m is a factor no greater than 0.90 that is meant to limit the strain in the FRP reinforcement to prevent debonding or delamination. This term recognizes that laminates with greater stiffness are more prone to delamination. The κ_m term is only based on a general recognized trend and on the experience of engineers practicing the design of bonded FRP systems.

Φ factor. The strength-reduction factor depends on the strain in the steel at ultimate, ε_s. Φ is set equal to 0.90 for ductile sections ($\varepsilon_s \geq 0.005$) and 0.70 for brittle sections where the steel does not yield. A linear transition for the reduction factor between these two extremes is then established.

Stress limits. To avoid plastic deformations, the existing steel reinforcement should be prevented from yielding at service load levels. The stress in the steel at service should be limited to 80% of the yield stress. Similarly, to avoid failure of an FRP-reinforced member due to creep-rupture of the FRP, stress limits for these conditions should be imposed on the FRP reinforcement. Limits on sustained and fatigue stresses are those listed in Table 2 and are identical to those for internal FRP reinforcement.

Shear

The nominal shear capacity of an FRP-strengthened concrete member can be determined by adding the contribution of the FRP reinforcing to the contributions from the reinforcing steel and the concrete. An additional reduction factor, ψ_f, is applied to the contribution of the FRP system. The additional reduction factor, ψ_f, should be selected based on the known characteristics of the application but should not exceed 0.85 for two- and three-sided wrapping schemes and 0.95 for completely wrapped elements.

The shear strength provided to a member by the FRP system should be based on the fiber orientation and an assumed crack pattern (Khalifa et al., 1998). It can be determined by calculating the force resulting from the tensile stress in the FRP along the assumed crack with an expression similar to that of steel reinforcement. To compute the tensile stress in the FRP shear reinforcement at ultimate, f_{fe}, it is necessary to calculate the effective strain in the FRP, ε_{fe}.

FRP systems that do not enclose the entire section (two- and three-sided wraps) have been observed to delaminate from the concrete before the loss of aggregate interlock of the section. For this reason, bond stresses should be analyzed to determine the usefulness of these systems and the effective strain level that can be achieved. The effective strain is calculated using a bond-reduction coefficient, κ_v, applicable to shear (see Eq. (6)).

$$\varepsilon_{fe} = \kappa_v \varepsilon_{fu} \leq 0.004 \text{ (for U-wraps or bonding to two sides)} \tag{6}$$

The bond-reduction coefficient is a function of the concrete strength, the type of wrapping scheme used, and the stiffness of the laminate (Khalifa et al. 1998). The methodology for determining κ_v has been validated for members in regions of high shear and low moment, such as simply supported beams. The methodology has not been confirmed for shear strengthening in areas subjected to simultaneous high shear and moment loads, such as continuous beams. In such situations, conservative values for κ_v are recommended.

Compression Members

The axial compressive strength of a non-slender member confined with an FRP jacket is calculated using the conventional expressions of ACI 318 substituting for f'_c the factored confined concrete strength $\psi_f f'_{cc}$. The additional reduction factor is set to ψ_f = 0.95.

Vertical displacement, section dilation, cracking, and strain limitations in the FRP jacket can also limit the amount of additional compression strength that can be achieved with an FRP jacket. The axial demand on an FRP-strengthened concrete member should be computed with the load factors required by ACI 318 and the axial compression strength should be calculated using the strength-reduction factors, ϕ, for spiral and tied elements required by ACI 318.

If the member is subjected to combined compression and shear, the effective strain in the FRP jacket should be limited based on the criteria given by $\varepsilon_{fe} = 0.004 \leq 0.75\varepsilon_{fu}$.

At load levels near ultimate, damage to the concrete in the form of significant cracking in the radial direction occurs. The FRP jacket contains the damage and maintains the structural integrity of the column. At service load levels, this type of damage should be avoided. In this way, the FRP jacket will only act during overloads that are temporary in nature. To ensure that radial cracking will not occur under service loads, the stress in the concrete is limited to $0.65f'_c$. In addition, the stress in the steel should remain below $0.60f_y$ to avoid plastic deformation under sustained or cyclic loads. By maintaining the specified stress in the concrete at service, the stress in the FRP jacket will be negligible.

Near-Surface Mounted (NSM) FRP Reinforcement

Although not directly addressed in ACI 440 2001a, the use of NSM FRP bars is a promising technology for increasing flexural and shear strength of deficient RC and PC members (De Lorenzis and Nanni, 2001). The advantages of NSM FRP bars compared with externally bonded FRP laminates are the possibility of anchoring the reinforcement into adjacent RC members, and minimal surface preparation work and installation time. The method used in applying the bars is as follows. A groove is cut in the desired direction into the concrete surface, the groove is then filled halfway with adhesive paste, and the FRP bar is placed in the groove and lightly pressed. This forces the paste to flow around the bar and fill completely between the bar and the sides of the groove. Finally, the groove is filled with more paste and the surface is leveled. As this technology emerges, the structural behavior of RC and PC elements strengthened with NSM FRP bars needs to be fully characterized.

Unreinforced Masonry (URM) Strengthening

At present, ACI provisions do not cover the use of composites for the strengthening of masonry structures even though some recommendations are available from other organizations (ICBO, 1997). Also a subcommittee of the Masonry Standards Joint Committee (i.e., Strengthening, Repair, and Rehabilitation), which is a three-society coordinated effort, is considering the use of composites as a suitable technology for masonry repair/upgrade.

FRP materials in the form of laminates and bars have been used for the strengthening of URM elements similarly to concrete members. In the case of strengthening with NSM FRP bars, there is a preference in placing them into grooves cut into the bed joints (i.e., structural repointing). To be completely successful, masonry retrofit work should be carried out with the least possible irrevocable alteration to the building appearance, as many URM buildings are part of the cultural heritage.

For masonry walls strengthened with FRP laminates, research results have shown that debonding of the FRP laminate from the masonry substrate is the controlling mechanism of failure. This has been evident in masonry walls strengthened to resist either in-plane or out-of-plane loads. Therefore, there is a need to determine the effective strain of the laminate as a function of the amount of strengthening. For clay units, debonding may have a direct relationship with the porosity of the masonry itself. For this reason, walls built with different and representative types of masonry units should be investigated. To date, there is a tendency to use types and quantities of FRP composites similar to those used for the strengthening of RC elements.

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Based on the premise of debonding as a controlling mode of failure, anchorage systems to avoid this failure mode need be developed. The use of steel angles may be effective when the wall is subjected to in-plane loads. However, when subjected to out-of-plane loads, the wall may be locally fractured in the anchorage regions due to the restraint of the wall movement (Tumialan et al., 2000). FRP bars have been successfully used for anchoring laminates in RC joists strengthened in shear. The anchoring technique consists of placing the fiber sheet under the bar embedded in a slot saw-cut in the base material.

Structural repointing is an alternative to strengthen masonry walls where aesthetics is important. Specifically for the case of walls subjected to in-plane loads, the effective strain developed in the bars needs to be estimated for different strengthening schemes. It has been observed that the failure in these walls is the result of the loss of bond between the adhesive and the masonry units (Tinazzi et al., 2000). The strengthening of only one side of a wall represents a frequently-encountered field situation where there is the presence of a veneer wall. Thus, the crack growth may be larger on the un-strengthened face of the wall. The crack between masonry units and the mortar in the un-strengthened face has been observed to travel through the wall thickness until debonding of the adhesive paste from the masonry units occurs. At this point the wall fails because the tensile stresses are no longer transferred to the FRP bars.

The interaction of strengthened walls with the surrounding structural elements (i.e. beams and columns) is of paramount importance since the effectiveness of the strengthening may be dangerously overestimated due to premature failures (e.g. crushing of masonry units at the boundary regions) in the masonry itself (Tumialan et al., 2000) or because the effects of the strengthening may produce undesirable changes in stiffness.

Conclusions

Even with some unresolved issues that should become a priority for future research, it can be concluded that the availability of design and construction guides developed by ACI for the use of FRP internal and external reinforcement for new and existing structures should allow the construction industry in North America to take full advantage of this emerging technology.

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