DESIGN AND CONSTRUCTION OF CONCRETE
REINFORCED WITH FRP BARS: AN EMERGING TECHNOLOGY

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ABSTRACT

This paper presented at the 2001 ACI Dallas Convention reports on design guidelines and present experience in the use of FRP composites for the internal reinforcement of concrete structures. FRP bars have been and are being used to replace conventional steel rebars for a host of reasons, but perhaps the most relevant is that of prevention of reinforcement corrosion. The principles for design and construction have been recently established and proposed to industry by the American Concrete Institute (ACI). The fundamental principles at the basis of this document are rooted in the steel-reinforced concrete practice with modifications to account for the physico-mechanical characteristics of FRP. Some unresolved questions remain pertaining to specifications, test methods, detailing, validation and long-term durability (including fire resistance). Resolving these issues will increase the degree of confidence in the technology and allow for its more economical exploitation. This can only occur with diligent and targeted applications by engineers and contractors.

KEYWORDS

Construction, design, detailing, development length, fiber-reinforced polymers, flexure, reinforced concrete, reinforcement, shear, testing.

BRIEF BIO
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INTRODUCTION

In this paper, presented at the 2001 ACI Dallas Convention, reference is made to the key features of a recently published ACI document (ACI Committee 440, 2001), which provides recommendations for design and construction of FRP-reinforced concrete (RC) structures as an emerging technology. The objective is to provide the reader with a meaningful overview of the design protocol for FRP-reinforced concrete members. The ACI 440.1R-01 document only addresses non-prestressed FRP reinforcement and is not intended to be standard at this time, given the novelty of the proposed technology. It is however envisioned that this will occur in the near future as the construction industry becomes familiar with composites and the prescriptions of the guide are validated on a wide basis.

For this paper, only notations critical to the understanding of its contents are defined and the equations are expressed in US customary units. Following a presentation of the key concepts, the paper discusses topics for future implementation and sample applications.

PRINCIPLES

FRP materials are anisotropic and are characterized by high tensile strength with no yielding in the direction of the reinforcing fibers. This anisotropic behavior affects the shear strength and dowel action of FRP bars, as well as their bond to concrete performance. Proposed design procedures account for a lack of ductility in concrete reinforced with FRP bars. Both strength and working stress design approaches are acceptable according to the provisions of the ’95 edition of ACI 318 (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 most instances, serviceability criteria will control the design.
Design Values. The design tensile strength that should be used in all design equations is given as $f_{fu} = C_E f_{fu}^*$, 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 and exposure conditions; and $f_{fu}^*$ = manufacturer’s guaranteed tensile strength of an FRP bar defined as the mean tensile strength of a sample population minus three times the standard deviation ($f_{fu}^* = f_{u,ave} - 3\sigma$). The design rupture strain should be determined similarly (i.e., average minus three times the standard deviation), whereas the design modulus of elasticity is the same as the average value reported by the manufacturer. Design parameters in compression are not addressed by the guide since the use of FRP rebars in this instance is discouraged.

Flexure Behavior and Failure Modes. The non-ductile behavior of FRP reinforcement necessitates reconsideration of 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 failure mode is marginally more desirable for flexural members reinforced with FRP bars (Nanni, 1993) since the member does exhibit some pseudo-plastic behavior before failure. 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.
**Φ factor.** When concrete crushing controls, a strength-reduction factor of 0.70 is adopted. Furthermore, a Φ 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 linear transition between the two values of Φ, a section controlled by concrete crushing is defined as a section in which the reinforcement ratio, $\rho_f$, is greater than or equal to 1.4 times the balanced reinforcement ratio, $\rho_{fb}$, ($\rho_f \geq 1.4 \rho_{fb}$) and a section controlled by FRP rupture is defined as one in which $\rho_f < \rho_{fb}$.

**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 $M_n$ and $M_{cr}$ are the nominal and cracking moment, respectively). The minimum reinforcement area is obtained by multiplying the existing ACI 318-95 limiting value 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 318-95 with the addition of a corrective coefficient, $k_b$, for the bond quality. The $k_b$ coefficient 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 unity. 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.

Shear

Several issues need to be addressed when using FRP as shear reinforcement, namely: FRP has a relatively low modulus of elasticity; FRP has a high tensile strength and no yield point; tensile strength of the bent portion of an FRP bar is significantly lower than the straight portion; and FRP has low dowel resistance.

According to ACI 318-95, 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 the 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 ($\rho_s E_s$) necessary to develop the same flexural capacity. For practical design purposes, the value of $\rho_s$ can be taken as 50% of the maximum allowed by the code (i.e., $0.5 \rho_{s,max}$ 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. (1) (noting $V_{c,f}$ cannot be larger than $V_c$).

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

(1)

The ACI 318-95 method used to calculate the shear contribution of steel stirrups, $V_s$, is applicable when using FRP as shear reinforcement with the provision that its stress level, $f_f$,..
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 (i.e., $f_{f_v} < f_{fb} = \text{strength of the bent}$). The stress level in the FRP shear reinforcement at ultimate for use in design is given by $f_{f_v} = 0.002E_f \leq f_{fb}$ . An expression for $f_{fb}$ is given in ACI 440.1R-01.

**Development Length**

The development length of FRP reinforcement can be expressed as shown in Eq. (2) as a function of the bar diameter, $d_b$, and the design strength. This should be a conservative estimate of the development length of FRP bars controlled by pullout failure rather than concrete splitting.

$$\ell_{df} = \frac{d_b f_{fu}}{2700}$$  \hspace{1cm} (2)

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.

**UNRESOLVED ISSUES**

As pointed out in ACI 440.1R-01, future research is needed to provide information in areas that are a real or perceived disadvantage, are still unclear, or are in need of additional evidence to validate performance. A list of detailed research topics is shown below.

**Materials:** behavior of FRP-reinforced members under elevated temperatures; minimum concrete cover requirement for fire resistance; fire rating; effect of transverse expansion of
FRP bars on cracking and spalling of concrete cover; creep rupture behavior and endurance limits of FRP bars; end treatment requirements of saw-cut FRP bars; and strength and stiffness degradation of FRP bars in harsh environment.

Among the major limitations of the present document is the lack of recognition of the importance of the polymeric resin. In particular, the role of the resin is paramount in determining the durability properties of the FRP systems including fire resistance. As the ACI document attempts to differentiate performance based on fiber type, the same needs to be accomplished in terms of resin type.

**Flexure/axial force:** behavior of FRP-RC compression members; behavior of flexural members with tension and compression FRP reinforcement; design and analysis of non-rectangular sections; maximum crack width and deflection prediction and control; minimum depth for deflection control; and long-term deflection behavior.

**Shear:** concrete contribution to shear resistance; failure modes and reinforcement limits; and use of FRP bars for punching shear reinforcement in two-way systems.

One of the drawbacks in the present formulation of shear capacity appears to be the limitation on the permissible design stress level in the FRP shear reinforcement at ultimate given by

$$f_{fc} = 0.002E_f \leq f_{fb}.$$  

The 0.002 strain limit was imposed to control crack width ensuring a better aggregate interlock. It was not based on any experimental evidence, but made sense as the value corresponding to Grade 60 steel yielding point. The shear capacity of an RC beam reinforced with FRP is penalized by cracking control twice, and its design results in an uneconomical use of the reinforcement. Experimental evidence shows the attainment of
higher strain values (Wang 1998, Zhao and Maruyama 1995, Okamoto et al. 1994). Given the high strain to failure of FRP, the engineer could consider using higher design values such as, for example, 0.00275 (corresponding to the yield strain of Grade 80) as allowed by ACI 318 for welded fabric. In no case, however, should the effective strain in FRP shear reinforcement exceed 0.004. This upper limit value of 0.004 has been selected for consistency with the new guide for externally applied FRP reinforcement being developed by ACI Committee, which states: “In no case, however, should the effective strain in FRP laminates exceed 0.004.” when referring to FRP laminates used as shear reinforcement. As a confirmation of the validity of this limit, the diagram given in Fig.1 shows experimental results from the cited references versus theoretical shear capacity values using both the present strain limit for stirrups (i.e., 0.002 – diamond symbols in the figure) and the proposed upper-limit value of 0.004 (i.e., square symbols in the figure). The predicted values are all on the safe side. Theoretical values are not multiplied by the $\Phi$ factor.

**Detailing**: standardized classification of surface deformation patterns; effect of surface characteristics of FRP bars on bond behavior; lap splices requirements; and minimum FRP reinforcement for temperature and shrinkage cracking control.

**Structural systems/elements**: behavior of concrete slabs on ground reinforced with FRP bars.

**Test methods**: bond characteristics and related bond-dependent coefficients; creep rupture and endurance limits; fatigue characteristics; coefficient of thermal expansion; durability characterization with focus on alkaline environment and determination of related environmental reduction factors; strength of the bent portion; shear strength; and compressive strength.
One of the test methods under development allows obtaining the strength capacity of 90-degree bents for FRP bars used as stirrups in shear reinforcement or used as anchors. The photographs given in Figures 2 and 3 show a specimen under test and the fractured bent. This method is intended for use in laboratory tests in which the principal variables are the size or type of the FRP bar and radius of the bent. It consists of a tension test conducted using a unique fixture which has three components, upper and lower steel parts and interchangeable aluminum corner inserts, machined to fit a specific bar diameter and bent radius. Instrumentation may be used depending on the parameters being monitored. If elastic modulus and strain distribution are required, strain gages can be mounted directly on the FRP bent. This test method has several advantages, which include ease and reliability. For example, in the case of one of the glass FRP bar systems used for validation, the coefficient of variation for 12 consecutive tests remained below 5.5%.

EXAMPLE OF APPLICATION: GFRP-REINFORCED CONCRETE BRIDGE DECKS

The original Sierrita de la Cruz Creek Bridge in Potter County, Texas, was replaced because it had become structurally deficient and functionally obsolete (Bradberry, 2001). The new bridge is 168.6 m (553 ft) long with a superstructure consisting of seven spans using 24.1-m (79-ft) prestressed concrete (PC), Texas Type “C” beams. The superstructure is divided into three units namely: a two-span unit on the north end and a three-span unit in the middle with epoxy-coated steel-RC decks and a two-span unit at the south end, each with a top mat GFRP bar and bottom mat epoxy-coated steel-RC deck. The transverse slab reinforcement is the primary load-carrying reinforcement of the slab and there is no provision for the development of tension ties associated with slab arching action.
Design forces were determined by a one-way analysis of the slab as well as from the empirical formula (AASHTO, 1996). The one-way analysis was performed assuming a 300-mm (1-ft) longitudinal strip of the transverse slab, continuous over knife-edged supports representing the six PC beams. Because the bottom mat of steel reinforcement was held to no more than the 150-mm (6-in) standard spacing and because panels were used for most bays, positive moment regions were adequately reinforced. Thus, only negative moments were considered in determining the required GFRP bar size and maximum spacing. Only the slab overhangs had the potential to reach full flexural capacity. Crack width, rather than strength and allowable stress limits, was the controlling design factor and determined bar size and spacing for the GFRP bars in the deck. The choice of the value of maximum acceptable crack width was in this case 0.5 mm (0.02 in) (CSA, 1996). The calculated maximum stress for this crack width was less than 15% of the guaranteed ultimate strength of the bar. For the deck slab design, #6 GFRP bars spaced at 140 mm (5.5 in) were required, versus the #5 epoxy-coated steel bars at 150 mm (6 in), which is the standard size and spacing. To summarize, GFRP bars spaced 7% closer than the standard epoxy-coated steel reinforcement, each with 42% more cross-sectional area than the standard steel reinforcement, was required in the top mat for the GFRP-reinforced deck as compared with a fully epoxy-coated steel-reinforced deck. Figures 4 and 5 show the GFRP reinforcement placement and the final product.

Like the Sierrita de la Cruz Creek bridge project, the 53rd Avenue bridge project in Bettendorf, Iowa, used a top GFRP reinforcing mat (negative moment only) of its deck. Design moment and shear values were based on HS-20 truck loading following the approach prescribed in AASHTO. In the direction transverse to traffic, in order to meet strength and serviceability (i.e., crack width) requirements, the design called for #6 GFRP bars spaced at
114 mm (4.5 in.) on center. The calculated stress in the GFRP bars at service level was 0.16 $f_{fu}$. Reinforcement size and arrangement for the negative moment region of the overhang (pedestrian sidewalk) was similar to that of the deck. A clear concrete cover of 50 mm (2.0 in.) was used throughout, and a minimum 914 mm (36 in.) lap splice length was also recommended. For temperature and shrinkage reinforcement (i.e., longitudinal top bars), #5 GFRP bars spaced at 229 mm (9 in.) as for the Sierrita de la Cruz Creek Bridge were recommend. Shear strength checks were based on two-way action for the bridge deck and one-way action for the overhang. Figures 6 and 7 show construction phases.

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 reinforcement for concrete structures should allow the construction industry to take full advantage of this emerging technology. Applications for concrete construction using internal FRP reinforcement are rapidly developing. The example mentioned in this paper is that of bridge decks.

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<table>
<thead>
<tr>
<th>Exposure condition</th>
<th>Carbon</th>
<th>Glass</th>
<th>Aramid</th>
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<td>Interior exposure</td>
<td>1.0</td>
<td>0.8</td>
<td>0.9</td>
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<tr>
<td>Exterior exposure</td>
<td>0.9</td>
<td>0.7</td>
<td>0.8</td>
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Table 2: Creep rupture and fatigue stress limits in FRP reinforcement

<table>
<thead>
<tr>
<th>Fiber type</th>
<th>Glass</th>
<th>Aramid</th>
<th>Carbon</th>
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<td>Creep rupture stress limit, $F_{fs}$</td>
<td>$0.20f_{fu}$</td>
<td>$0.30f_{fu}$</td>
<td>$0.55f_{fu}$</td>
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</table>
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