

Design of concrete beam reinforced with GFRP bars as per ACI codal provisions

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Abstract. This document provides design principles for concrete beams reinforced with glass fiber reinforced polymer (GFRP) bars per the ACI 440.1R-15 regulation. One of the main advantages of using glass fiber reinforced polymer rods instead of traditional steel reinforced rods is their lighter weight and higher corrosion resistance. However, the bending failure mode of FRP reinforced concrete (FRP-RC) beams is brittle rather than ductile because the elasticity of fiber reinforced polymer (FRP) bars is linear until failure and the elongation at break is small. For FRP-RC elements, concrete crushing compression failure, which gives various warnings before failure, is the preferred failure mode. In other words, unlike the usual design practice for reinforced concrete (steel-RC) beams, for FRP-RC beams, an over-reinforced structure is preferable to an under-reinforced structure. In addition, since the FRP RC member has low rigidity of the FRP rod, it bends more and cracks larger than the steel RC member. These factors limit the field of application of FRP. Here is a design example of a rectangular beam with tension reinforcement according to ACI regulations.

Keywords: Fibre-reinforced polymer (FRP), Glass fibre reinforced polymer (GFRP), anisotropic, compression-controlled, flexural behaviour

1 Introduction

Composites with fibers embedded in polymer resins, also known as fiber reinforced polymers (FRPs), are an alternative to steel rebar in concrete structures. Fiber-reinforced polymer reinforcements consist of continuous aramid fibers (AFRP), carbon fibers (CFRP),

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or glass fibers (GFRP) embedded in a resin matrix. The mechanical behavior of fiber reinforced polymer (FRP) reinforcements differs from that of conventional steel reinforcements [1]. Therefore, it is necessary to change the conventional design concept of concrete structures for FRP reinforcement. Fiber-reinforced polymer materials are anisotropic and feature high tensile strength only in the direction of the reinforcing fibers. This anisotropic behavior affects not only the adhesive performance, but also the shear strength and dowel behavior of FRP rods[2]. Also, the FRP material does not deform. Rather, it is flexible to failure. The design method should consider the lack of ductility of concrete members reinforced with FRP bars. The ACI 440R was first developed by him in 2001 as a guide for the design and construction of structural concrete with FRP bars. Similar design-related documents have been produced in other countries and regions such as Japan (Japan Society of Civil Engineers 1997b), Canada (CAN/CSA-S6-06, CAN/CSA-S806-12), and Europe (fib 2007, 2010). Increase [3]. We have sufficient analytical and experimental information on FRP reinforced concrete and extensive practical experience to put this knowledge into practice. The advantages of FRP are a) impermeability to chloride ions and chemical attack, b) higher tensile strength than steel, c) light weight - 1/4 to 5 times the weight of steel rebar. d) less concrete cover and e) less admixture. No corrosion required. f) In corrosive environments service life is significantly longer than steel. Compared with steel, FRP has the following advantages: a) FRP has linear elasticity to failure and steel yields. b) FRP is anisotropic while steel is isotropic. c) Due to the low modulus of FRP bars, the structure often compromises maintainability. controlled. d) FRP bars have a lower creep rupture threshold than steel. e) The coefficient of thermal expansion is different in the longitudinal and radial directions. f) Life in fire and high temperature applications is shorter than that of steel [4-5].

2 Glass fibred reinforced polymer (GFRP) bars

Under tensile loading, GRP rods do not exhibit plastic behavior (yielding) prior to fracture. The tensile behavior of FRP rods composed of a single fiber material is characterized by a linear elastic stress-strain relationship up to failure. Reinforced concrete sections are generally designed to ensure stress-controlled behavior caused by the yielding of steel prior to concrete fracture. Yield in steel provides ductility and warns of component failure. The non-ductile behavior of FK reinforcement requires re-evaluation of this approach. Failure of the FRP stiffener causes sudden and catastrophic failure of the component. However, because FRP stiffeners undergo large elastic strains before failure, there is limited warning of impending failure in the form of crack propagation or large deflections [6]. In either case, the members do not exhibit the ductility commonly found in tension-controlled concrete beams reinforced with steel rebar, and the rebar undergoes plastic deformation prior to concrete fracture. Compression-controlled behavior is slightly desirable for FRP rod-enhanced deflection. Since the concrete fractures before the tensile failure of the FRP rebar, the bending elements exhibit a certain inelastic behavior before failure. Therefore, both compression and tension control sections are acceptable in FRP bar reinforced flexure designs as long as strength and serviceability criteria are met. Components require higher strength reserves to compensate for the lack of ductility. Therefore, the recommended margin of safety against failure is higher than that of conventional reinforced concrete structures [7].

3 Design Concepts (As per ACI 318 and ACI 440)

- a) Determine the service loads:

Calculate factored moment $M_u = Wl^2/8$

W =weight of the beam

Factored load $W_u = 1.2(\text{Self weight of beam} + \text{Dead load}) + 1.6 (\text{Live load})$

Factored moment $M_u = Wl^2/8$

b) The bending capacity of FRP reinforced deflection depends on whether it is controlled by concrete crushing or by FRP failure [8]. By comparing the FRP reinforcement rate and the balance reinforcement rate where concrete crushing and FRP failure occur at the same time, the control limit state can be known. Since FRP does not yield, the design tensile strength is used to calculate the FRP reinforcement balance. The FRP step-up ratio can be calculated by equation (1).

$$\rho_f = \frac{A_f}{bd} \tag{1}$$

The balanced FRP reinforcement ratio can be computed from equation (2)

$$\rho_{fb} = 0.85\beta_1 \frac{f'_c}{f_{fu}} \frac{E_f \epsilon_{cu}}{E_f \epsilon_{cu} + f_{fu}} \tag{2}$$

When the boost ratio is less than the balance ratio ($\rho_f < \rho_{fb}$), the FRP failure limit state controls. Otherwise, ($\rho_f > \rho_{fb}$) is the concrete failure limit condition.

c) Balance of Fiber Reinforced Polymer Reinforcement means the reinforcement in deflection such that the strength design reaches the maximum design strain limit strain 0.003 assumed for the fiber reinforced polymer (FRP) tensile reinforcement at the same time as the concrete under compression. is the amount and distribution of . d) The bending ability of FRP reinforced deflection depends on whether it is controlled by concrete crushing or by FRP failure. By comparing the FRP reinforcement rate and the balance reinforcement rate where concrete crushing and FRP failure occur at the same time, it is possible to know the control limit state. Since FRP does not yield, the design tensile strength is used to calculate the FRP reinforcement balance [9].

e) If the cross section is stress controlled ($\rho_f \leq \rho_{fb}$), a minimum level of reinforcement should be provided to prevent failure in concrete cracks (M_{cr} is the crack moment). The minimum reinforcement provisions of ACI 318 are based on this concept, and with modifications he also applies to FRP reinforcement components [10-11].

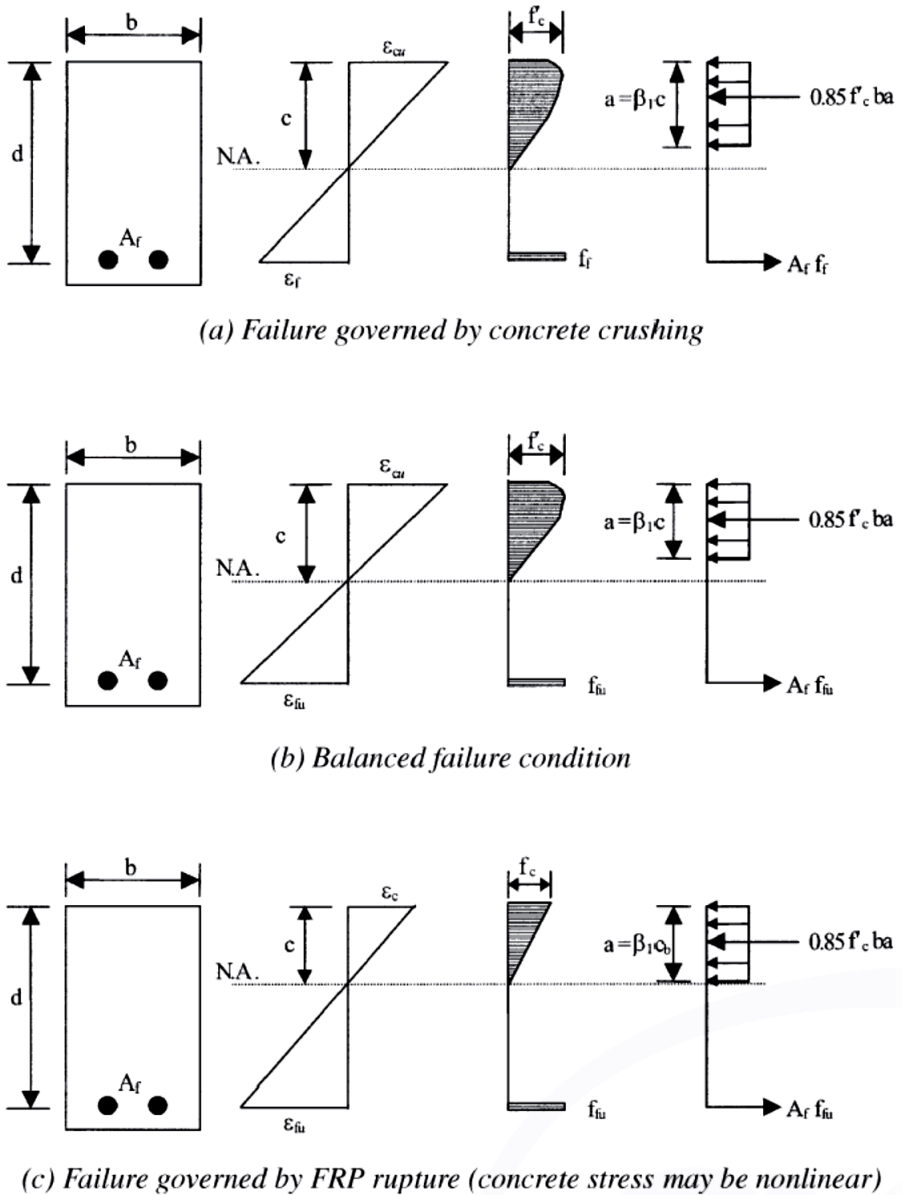


Figure 1. Strain and stress distribution at ultimate conditions

4 Design procedural steps

Design a rectangular beam of width $b = 300$ mm to have adequate flexural strength. The beam must resist service load moments $M_D = 76$ kN-m and $M_L = 47$ kN-m. Assume interior exposure conditions.

Compressive strength of concrete $f_c' = 28$ MPa

Tensile strength of FRP bar $f_{fu}^* = 550$ MPa

Modulus of elasticity of FRP $E_f = 45,000$ MPa

Environmental reduction factor $C_E = 0.8$

[440.1R, Table 6.2 Environmental reduction factor for various fibres and exposure conditions] Fibre type: Glass and Exposure condition: Concrete not exposed to earth and weather.

The design tensile strength $f_{tu} = C_E * f_{tu}^* = (0.8)(550) = 440 \text{ MPa}$ [440.1R, Eq. (6.2a)]

Step1:

Reinforcement ratio is assumed (for design of a reinforced concrete member of unknown dimensions)

So, assume $\rho_f = 1.5\rho_{fb}$

ρ_f = fibre-reinforced polymer reinforcement ratio

ρ_{fb} = fibre-reinforced polymer reinforcement ratio producing balanced strain conditions

$$\rho_{fb} = 0.85\beta_1 \frac{f'_c}{f_{fu}} \frac{E_f \epsilon_{cu}}{E_f \epsilon_{cu} + f_{fu}} \quad [440.1R, \text{Eq. (7.2.1b)}]$$

ϵ_{cu} = ultimate strain in concrete

$E_f * \epsilon_{cu} = (45,000) (0.003) = 135 \text{ MPa}$

$\rho_{fb} = 0.85 * 0.85 * (28/440) * ((135)/(135+440)) = 0.01079$

$\rho_f = 1.5\rho_{fb} = 1.5(0.01079) = 0.01619$

Because $\rho_f \geq 1.4\rho_{fb}$, the section is compression-controlled means the concrete crushing limit state controls. Compression-controlled behavior is marginally more desirable for flexural members reinforced with FRP bars. By experiencing concrete crushing prior to tensile rupture of the FRP reinforcement, a flexural member Nanni 1993b does exhibit some inelastic behavior before failure.

The strength design philosophy states that the design flexural strength at a section of a member should exceed the factored moment.

$$\phi M_n \geq M_u$$

Design flexural strength refers to the nominal flexural strength of the member multiplied by a strength reduction factor ϕ . *Strength reduction factor for flexure*—Because FRP members do not exhibit ductile behavior, a conservative strength reduction factor should be adopted to provide a higher reserve of strength in the member. Based on ACI 318, the ϕ factor for design of a compression-controlled section is 0.65.

$\phi = 0.65$

Step2:

Compute bd^2 required.

First, determine the required design moment strength:

$\phi M_{n,reqd} = M_u = 1.2M_D + 1.6M_L = 1.2(76) + 1.6(47) = 166.4 \text{ kN-m}$ [318-11, Eq. (9-2)]

$M_{n,reqd}$ = Required nominal moment capacity (N-mm)

M_u = factored moment at section (N-mm)

Calculate the stress in the tensile reinforcement (f_f) at ultimate conditions for the assumed value of ρ_f

$$f_f = \sqrt{\frac{(E_f \epsilon_{cu})^2}{4} + \frac{0.85\beta_1 f'_c}{\rho_f} E_f \epsilon_{cu}} - 0.5E_f \epsilon_{cu} \leq f_{fu} \quad [440.1R, \text{Eq. (7.2.2d)}]$$

$$f_f = \sqrt{\frac{(135)^2}{4} + \frac{0.85(0.85)(28)}{(0.01619)} (135)} - 0.5(135) \leq 440$$

$f_f = 349 \text{ MPa}$

f_f = stress in FRP reinforcement in tension (MPa)

Use the moment capacity equation to determine required dimensions for the cross section

$$M_u = \phi M_n = \phi \rho_f f_f \left(1 - 0.59 \frac{\rho_f f_f}{f'_c} \right) b d^2 \quad [440.1R, \text{Eq. (7.2.2e)}]$$

$$(166.4)(10^6) = (0.65)(0.01619)(349) \left(1 - 0.59 \frac{(0.01619)(349)}{(28)} \right) b d^2$$

$$b d^2 = 51.43 \times 10^6 \text{ mm}^3$$

Step3:

Size of the beam:

$$(b d^2)_{\text{Provided}} \geq (b d^2)_{\text{required}}$$

$$\text{Recall } b = 300 \text{ mm, so } d = \sqrt{\frac{51.43 \times 10^6}{300}} = 414 \text{ mm}$$

Step4:

Now, determine the required reinforcement, select bars, and determine depth.

$$A_{f, \text{reqd}} = \rho_f b d = (0.01619)(300)(414) = 2011 \text{ mm}^2$$

A_f = Area of fibre-reinforced polymer (FRP) reinforcement (mm²)

Select four No.25 bars ($A_f = 2040 \text{ mm}^2$)

Examining alternative designs using other bar sizes may require changing the assumed value of f_{fu}^* to a value appropriate for the selected bar size.

No. 25 bar diameter: nominal diameter or diameter of reinforcing bar $d_b = 25.4 \text{ mm}$

[440.6-08, Table 7.1]

For interior exposure, clear cover is 38 mm

[440.5-08, Table 3.1]

Assuming No. 13 stirrups: diameter of the stirrup is 12.7 mm

$$h = (414) + (38) + (12.7) + \frac{(25.4)}{2} = 477 \text{ mm}$$

h = Overall height of flexural member, in. (mm)

Round up to be conservative. So, select a 300 x 500 mm beam.

Step 5:

Determine capacity of cross section:

$$d = 500 - \left[(38) + (12.7) + \frac{(25.4)}{2} \right] = 437 \text{ mm}$$

$$\rho_f = \frac{A_f}{b d} = \frac{2040}{(300)(437)} = 0.01556 \quad [440.1R, \text{Eq. (7.2.1a)}]$$

$$\frac{\rho_f}{\rho_{fb}} = \frac{0.01556}{0.01079} = 1.44$$

$$\text{Because } \rho_f \geq 1.4 \rho_{fb}, \phi = 0.65 \quad [440.1R, \text{Eq. (7.2.3)}]$$

$$f_f = \sqrt{\frac{(E_f \epsilon_{cu})^2}{4} + \frac{0.85 \beta_1 f'_c}{\rho_f} E_f \epsilon_{cu} - 0.5 E_f \epsilon_{cu}} \leq f_{fu} \quad [440.1R, \text{Eq. (7.2.2d)}]$$

$$f_f = \sqrt{\frac{(135)^2}{4} + \frac{0.85(0.85)(28)}{(0.01556)}(135) - 0.5(135)} \leq 440$$

$$f_f = 357 \text{ MPa}$$

$$M_n = \rho_f f_f \left(1 - 0.59 \frac{\rho_f f_f}{f'_c} \right) b d^2 \quad [440.1R, \text{Eq. (7.2.2e)}]$$

$$= (0.01556)(357) \left(1 - 0.59 \frac{(0.01556)(357)}{28} \right) (300)(437)^2 = 281.0 \times 10^6 \text{ N-mm} = 281.0 \text{ kN-m}$$

$$\phi M_n = (0.65)(281.0) = 182.3 \text{ kN-m} > M_u = 166.4 \text{ kN-m} \quad \text{OK}$$

Many designs for FRP-reinforced concrete are governed by serviceability requirements related to crack control, deflections, and creep rupture, rather than by flexural strength requirements.

5 Conclusions

The following conclusions were drawn from analytical studies and design of rectangular beams reinforced with glass fiber reinforced polymer (GFRP) rods according to the Codal provisions of ACI 440.1R-15.

1. Glass fiber reinforced polymer (GFRP) bars can replace steel rebar in concrete structures
2. Glass fiber reinforced polymer (GFRP) rods are anisotropic and feature high tensile strength only in the direction of the reinforcing fibers. This anisotropic behavior affects not only the bonding performance, but also the shear strength and dowel behavior of FRP rods. Additionally, GFRP materials do not yield. Rather, it is flexible to failure. The design method should take into account the lack of ductility of concrete members reinforced with GFRP bars.
3. When subjected to tensile loads, GFRP rods do not exhibit plastic behavior (yielding) before failure. The tensile behavior of GFRP rods made from a class of fibrous materials is characterized by a linear elastic stress-strain relationship up to failure.
4. Compression controlled behavior becomes slightly more desirable for GFRP rod enhanced deflection. Since the concrete fractures before the tensile failure of the FRP rebar, the bending elements exhibit a certain inelastic behavior before failure.
5. Components require a higher strength reserve to compensate for the lack of ductility. Therefore, the recommended margin of safety against failure is higher than that of conventional reinforced concrete structures.

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