Flexural Capacity Predictions and Comparisons of GFRP Reinforced Beams

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Abstract. The use of concrete material in construction always requires steel bars as reinforcement, particularly to resist the tensile stresses occurred in concrete. This is because concrete has very low tensile strength. However, the use of reinforcing steel creates another issue, namely its durability against corrosion. This paper discusses the use of GRP reinforcement as a replacement to steel reinforcement particularly when it is used in the aggressive environmental zones. GFRP has the advantage of being a corrosion resistance, it is not like the steel. Different failure mechanisms are also found between steel and GFRP. The steel yields first prior to failure, but not with the GFRP since they remains almost elastic until failure. This is due to the difference between the properties of the steel and GFRP materials, thus this issue leads to the difference in both flexural design procedures. This paper studies the comparison of the theoretical flexural strength design of various codes worldwide as well as several methods for predicting the flexural capacity of beams proposed by various researchers. Various percentages of flexural reinforcement possibly used in beams are discussed and compared. Some of the codes used for comparisons include the FIB Bulletin 40, ECP 208-2005, ACI 440.1R-15, and CSA 806-12.

1. Introduction

The nominal bending moment capacity of FRP-reinforced concrete members is also determined similarly as that for the steel-reinforced concrete members [1]. Most of the FRP-reinforced flexural members are designed in over-reinforced condition. However, both under- and over-reinforced conditions are permitted as long as the deflections and crack widths (serviceability limits states) satisfy the requirements. FRP bars do not yield and will behave linearly elastic until failure and in the design, the yield strength of steel bars is replaced by the ultimate strength of FRP bars [2].

Concrete sections reinforced with steel bars are normally designed to be in tensile-controlled condition. This ensures that the steel bars will yield first prior to concrete crushing. The early yielding of steel bars stated a ductile behavior of the beam. This provides a good warning prior to its failure. The FRP bars do not indicate a clear yielding mechanism. The design methods used for determining the bending moment capacity of reinforced concrete beams with steel bars must be modified to accommodate the different mechanism of FRP bars to define for the FRP-bar linear stress-strain mechanism [3]. The behavior of concrete members reinforced by FRP bars are determined using linearly-elastic condition and remain unyielding up to failure. This mechanism is different with the traditional reinforced concrete with steel bars where the failure mechanism is always controlled by the crushing of compressive concrete before or after the steel bar yielding. For FRP-reinforced concrete members, they might display either the rupture of the FRP bars or the crushing of compressive concrete as a failure mode. Since FRP bars are not possible to attain a ductile failure manner, most of researchers and design codes proposed that the concrete crushing better to occur first rather than the FRP bar failure. This is due to concrete crushing is slightly less brittle than FRP bar failure [4-8].

2. Parametric Study

The bending moment capacities are investigated using four beam specimens which were designed with adequate GFRP longitudinal reinforcing bars with ratios of 0.5% (2#13); 0.8% (2#16); 1.1% (4#13); and 1.7% (4#16). The GFRP stirrups for all beams were set to be #6-100 mm. The dimensions of all the beams are 200-mm width, 3000-mm long, and 300-mm deep. The details of the typical beams are shown in Figure 1 and the beams cross sectional data is listed in Table 1. The following are the given mechanical properties of the beams: f_c' = 25 MPa; f_{fu}^* = 708 MPa (#13); f_{fu}^* = 683 MPa (#16); E_f = 43.9 GPa (#13); E_f = 46.7 GPa (#16).

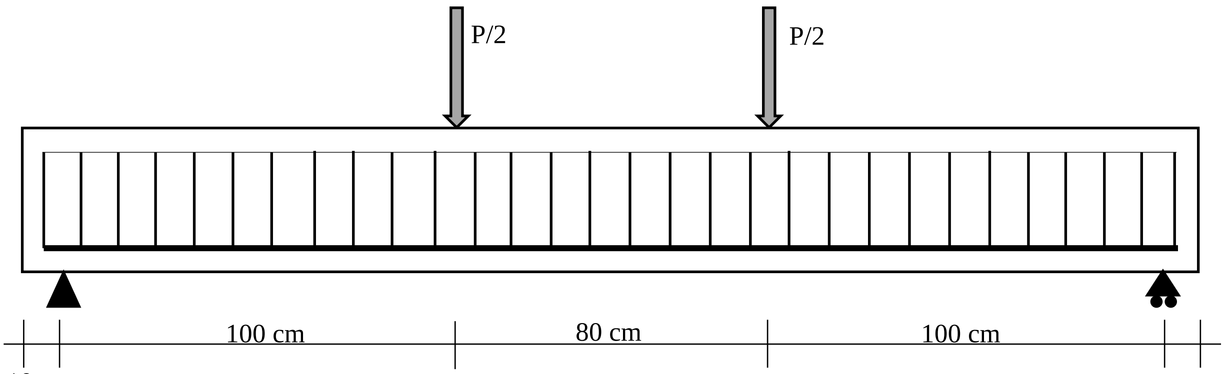
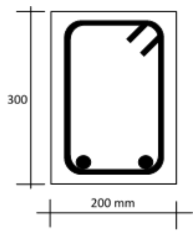
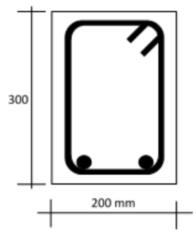
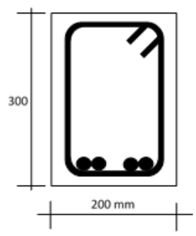
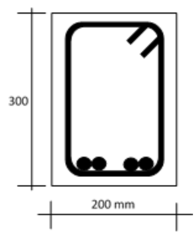


Figure 1 Typical details and test setup of the beams

Table 1 Cross-sectional details of the beams

Notation	2L13	2L16	4L13	4L16
Cross section				
Dimension	200x300x3000	200x300x3000	200x300x3000	200x300x3000
Reinforcement	2#13	2#16	4#13	4#16
Ratio	0.5%	0.8%	1.1%	1.7%
Stirrups	#6@100	#6@100	#6@100	#6@100

3. Results and Discussion

The FIB Bulletin 40 [9], which is identical to the EC 2 [10] provision, states that the ultimate bending moment (M_u) can be determined by Equation 1.

$$M_u = \eta \, f_{cd} \, b \, d^2 (\lambda \, \xi) \left(1 - \frac{\lambda \, \xi}{2}\right) \quad (1)$$

where:

$f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_c}$
α_{cc} = recommended value in EC2 is 1
$\left.\begin{aligned}\lambda = 0.8\\ \eta = 1.0\end{aligned}\right\}$ for $f_{ck} \leq 50$ MPa
$\left.\begin{aligned}\lambda &= 0.8 - \left(\frac{f_{ck} - 50}{400}\right)\\ \eta &= 1.0 - \left(\frac{f_{ck} - 50}{200}\right)\end{aligned}\right\}$ for $50 < f_{ck} \leq 90$ MPa
$\xi = \frac{X}{d} = \frac{\varepsilon_{cu}}{\varepsilon_f + \varepsilon_{cu}}$
$\varepsilon_f = \frac{-\varepsilon_{cu} + \sqrt{\varepsilon_{cu}^2 + \frac{4\,\eta\,\alpha_{cc}\,f_{ck}\,\lambda\,\varepsilon_{cu}}{\gamma_c\,\rho_f\,E_f}}}{2}$

The ECP 208-2005 [11, 12] provision states that the ultimate limit state bending moment of the cross section area can be calculated using Equation 2.

$$M_u = \left(\frac{A_f f_{fe}^*}{\gamma_f}\right)\left(d - \frac{a}{2}\right) \quad (2)$$

The depth of the equivalent stress block can be computed by Equation 3:

$$a = \frac{\frac{A_f f_{fe}^*}{\gamma_f}}{\left(\frac{0.67\,f_{cu}}{\gamma_c}\right)b} \quad (3)$$

The design stress in the FRP bars, f^*_{fe} can be computed using Equation 4:

$$\frac{f_{fe}^*}{\gamma_f} = \left(\frac{0.8\,d - a}{a}\right) E_f\ \varepsilon_{cu} \quad (4)$$

By substituting a in Equation 3 and Equation 4 for f^*_{fe}:

$$\frac{f_{fe}^*}{\gamma_f} = \left[\sqrt{\frac{(E_f \varepsilon_{cu})^2}{4} + \frac{0.536\,f_{cu}}{\mu_f\,\gamma_c} E_f\ \varepsilon_{cu}} - 0.5\,E_f\ \varepsilon_{cu}\right] \leq \frac{f_{fu}^*}{\gamma_f} \quad (5)$$

The ACI 440.1R-15 [13] provision states that the nominal flexural strength of an FRP-reinforced concrete member can be determined based on the controlling strength limit state (concrete crushing or FRP rupture), the strain compatibility, and the internal force equilibrium.

$$M_n = A_f f_f \left(d - \frac{a}{2}\right) \quad (6)$$
$$a = \frac{A_f f_f}{0.85 f'_c\, b}$$

$$f_f = E_f \varepsilon_{cu} \frac{\beta_1 d - a}{a}$$

The CSA S806-12 [14] provision calculates the ultimate moment resistance using Equation 7 as follows:

$$M_r = T\left(d - \frac{\beta_1 c}{2}\right) = A_f f_f \left(d - \frac{\beta_1 c}{2}\right) \tag{7}$$
$$f_f = 0.5\, E_f\, \varepsilon_{cu} \left[\left(1 + \frac{4\alpha_1 \beta_1 \phi_c f'_c}{\rho_f \phi_f E_f \varepsilon_{cu}}\right)^{1/2} - 1\right]$$

$$\alpha_1 = 0.85 - 0.0015\, f'_c \geq 0.67$$
$$\beta_1 = 0.97 - 0.0025\, f'_c \geq 0.67$$

$$\phi_c = 0.6 \text{ (concrete cast in situ)}$$
$$\phi_f = 0.75 \text{ (GFRP)}$$

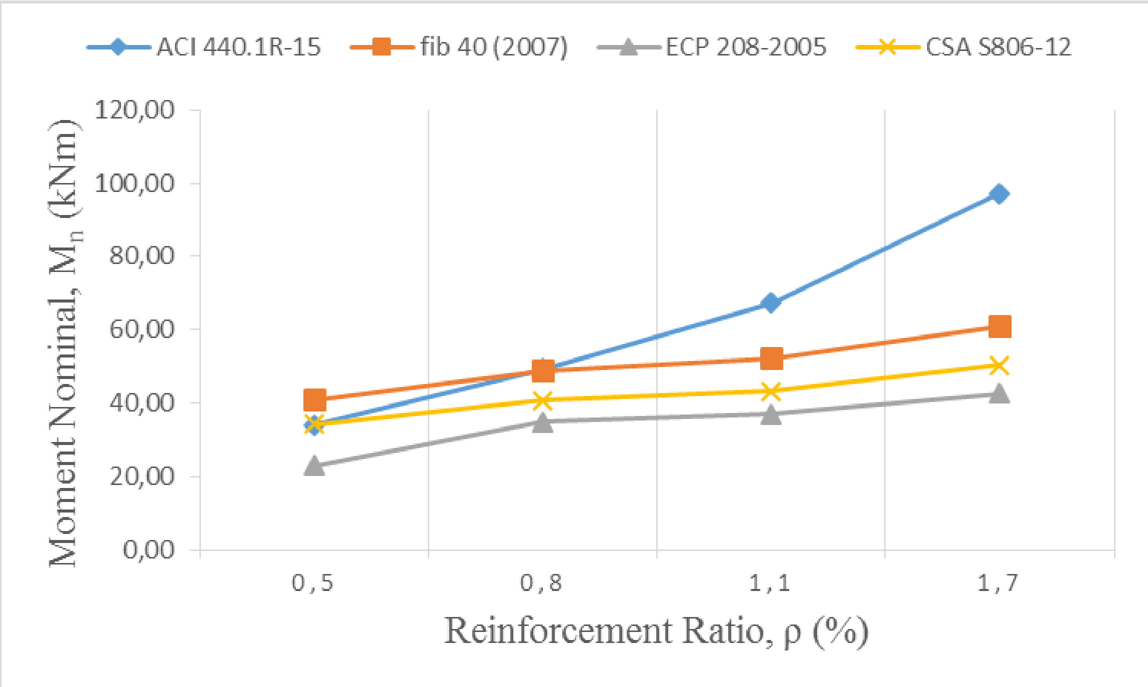


Figure 2 Nominal bending moment capacities of FRP-reinforced concrete beams

4. Conclusions

From the comparisons above, it can be concluded that bending moment capacities of the beams (refer to the ACI 440.1R-15) increase with the increase of reinforcement ratio. The ACI 440.1R-15 overestimates the bending moment capacities of the GFRP-reinforced concrete beams opposed to the other codes. On the other hand, the ECP 208-2005 underestimates the bending moment capacities of the reinforced concrete beams reinforcing with GFRP.

FIB Bulletin 40 and CSA 806-12 indicate that the increase of GFRP-reinforcement ratio will increase the bending moment capacities of the beams which are lower than the ACI 440.1R-15 and higher than the ECP 208-2005.

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