Flexural Capacity of Concrete Beam Reinforced with GFRP Bars

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Abstract. This paper proposes an analytical study to predict the flexural capacity of structure elements reinforced with glass fibre reinforced polymer (GFRP) bars. A nonlinear stress-strain curve of concrete is employed to consider the contribution of concrete under compression. The analytical calculation is validated against a published experiment, and the results are also compared to the ACI 440.1R-15 code. The results showed that the rupture modes predicted in the analytical study were comparable to those of the experiment, and the average ratio of flexural capacity calculated using Todeschini’s nonlinear curve to the experiment ranged from 0.78 to 0.86. Furthermore, flexural capacity obtained using nonlinear stress-strain curve of Todeschini for specimens with reinforcement ratios higher than the balanced reinforcement ratio performed better than that obtained using the ACI code.

1. Introduction
Nowadays, steel bars are the sole solution to reinforce concrete structures in Indonesia. Plain concrete supplies the compression force resistance, and the steel bars provide tension force that the concrete cannot provide. On the other hand, an aggressive environment would shorten the service life of the reinforced concrete (RC) structures since chloride ions and CO₂, resulted from the carbonation of concrete would corrode reinforcement bars. Corroded steel bars initiate cracking, spalling the concrete cover, and causes the loss of usability [1, 2]. The alternative remedy to address the corrosion process does not eliminate the problems of steel corrosion in the RC structures [3, 4]. On the other hand, maintenance cost carried out to recover the affected structures is doubled compared to the construction of a new structure with an alternative construction method [5, 6].

Demand for the use of fibre reinforced polymer (FRP) bars in place of steel bars ascended when the market needs for nonferrous reinforcing bars have been growing since the late 1980s. FRP bar is favourable because it has beneficial characteristics, such as high tensile strength, light-weight, and corrosion-resistant [7-9]. FRP products have been investigated widely for the application of strengthening RC beam [10-13] and RC beam-column joints [14], as well as the use of FRP bars as internal reinforcement [15]. An analytical study conducted on RC frame building showed that buildings reinforced with carbon FRP had a loading capacity 1.9 times higher compared to that reinforced with conventional steel bars [15].

The purpose of this research is to investigate an analytical procedure for calculating the flexural capacity of beams reinforced with glass FRP bars. The contribution of concrete force in compression
is calculated by using the Todeschini’s nonlinear curve [16]. The accuracy of results is confirmed to the experiments of published article [17] and also compared to the ACI 440.1R-15 [7].

2. Method of Analysis

2.1. Description of specimens
The specimens analysed in this study are taken from a published paper [17]. Two series of simply-supported beams, namely FB and HFB, were constructed and tested under monotonic loading. All specimens were identical and had a span of 3000 mm to maintain the ratio of shear span \((a)\) to effective depth \((d_f)\) of 3.4. Table 1 presents details of specimens analysed in this study.

| Specimen code | \(f'_c\) (MPa) | GFRP bar | Width (mm) | Depth (mm) | \(a/d_f\) |
|---------------|----------------|-----------|-------------|-------------|--------------|
| FB-2          | 34             | 2 D 13    |             |             |              |
| FB-3          | 34             | 3 D 13    |             |             |              |
| FB-4          | 34             | 4 D 13    |             |             |              |
| FB-6          | 34             | 6 D 13    |             |             |              |
| FB-8          | 34             | 8 D 13    |             |             |              |
| HFB-3         | 45             | 3 D 13    | 300         | 400         | 3.4          |
| HFB-4         | 45             | 4 D 13    |             |             |              |
| HFB-6         | 45             | 6 D 13    |             |             |              |
| HFB-8         | 45             | 8 D 13    |             |             |              |
| HFB-10        | 45             | 10 D 13   |             |             |              |

2.2. Material characteristics
Based on targeted concrete strength, the beams were separated into two groups, namely FB and HFB. The concrete strength of FB specimens was 34 MPa, while the concrete strength of HFB specimens was 45 MPa. All beams were reinforced with GFRP bars with a nominal diameter of 12.7 mm. The GFRP bars were manufactured from E-glass fibre type through a pultrusion process combined with thermosetting polyester resin. Outside of the bar surface was sand coated with helical glass fibre strands to enhance its bond characteristics.

2.3. Flexural strength calculation procedure
2.3.1. Method 1 – Todeschini’s nonlinear concrete curve. The procedure to calculate the flexural capacity of RC beams reinforced with GFRP bars proposed in this study is developed based on the strain compatibility and internal forces equilibrium. The variation of concrete and FRP bar stress and strain along the depth of beam cross-section is taken into consideration to attain this condition. The actual behaviour of stress-strain of the concrete in compression [16] is also adapted in this study.

The analytical procedure is begun by assuming the location of the neutral axis \((c)\) of the RC beam as seen in Figure 1. The section and material properties for both concrete and the GFRP bars are the first input, and then both strains at the concrete \((\varepsilon_c)\) and GFRP bars \((\varepsilon_f)\) as seen in Figure 2 are calculated based on the assumed neutral axis location. Later, the compressive force at concrete \((C_c)\) and the tensile force at GFRP \((T_f)\) can be determined based on the respective strain of the concrete and the GFRP bars. The assumed neutral axis applies if it satisfies the equilibrium of the internal forces in the concrete section. Compressive force at concrete can be calculated using equation (1).
Figure 1. Proposed procedure to determine flexural capacity of RC with FRP bars

\[ C_c = k_1 \times f'_c \times c \times b_w \]  
(1)

Where \( C_c \) is the concrete force, \( k_1 \) is the ratio of average stress to the maximum stress, \( f'_c \) is the concrete strength, \( c \) is the location of the neutral axis, and \( b_w \) is the width of the beam. Finally, the flexural capacity (\( M_n \)) of the concrete beam is computed by summing the product of the internal forces to the neutral axis location as seen in equation (2).

\[ M_n = (C_c \times z_1) + (T_f \times z_2) \]  
(2)

Where \( T_f \) is the tensile force at GFRP, \( z_1 \) is the distance from the concrete force to the neutral axis, and \( z_2 \) is the distance from the tensile force of FRP to the location of the neutral axis.

2.3.2. Method 2 - equivalent rectangular concrete stress-block. Figure 3 shows the strain and stress distribution as well as the internal forces in the FRP concrete section as suggested by ACI 440.1R-15
Depth of the concrete stress-block ($a_f$) is computed according to Equation 7.2.2b or 7.2.2f of ACI 440.1R-15 [7]. The depth of the rectangular stress-block depends on the value of the ratio of FRP bars used ($\rho_f$) compared to its balanced ratio ($\rho_{fb}$), and the width of the rectangular concrete block is the product of the constant $\alpha_1$ and the concrete strength ($f'_c$).

For $\rho_f \geq \rho_{fb}$

$$a_f = \frac{A_f \cdot f_f}{0.85 \cdot f'_c \cdot b_w}$$

Where $a_f$ is depth equivalent stress block, $A_f$ is the area of FRP bars, $f_f$ is the stress in FRP bars under tension, $f'_c$ is the concrete strength, and $b_w$ is the width of the beam.

Figure 2. Stress-stress distribution in RC section according to Todeschini’s nonlinear concrete curve: (a) concrete section, (b) strain distribution, (c) stress in concrete and FRP bars

Figure 3. Stress-stress distribution in RC section according to ACI 440.1R-15 [7]: (a) concrete section, (b) strain distribution, (c) stress in concrete and FRP bars
And for $\rho_f < \rho_b$

$$a_f = \beta_1 \left( \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fu}} \right) d_f$$

(4)

Where $a_f$ is depth equivalent stress block, $\beta_1$ is a factor that depends on the concrete strength, $\varepsilon_{cu}$ is the ultimate strain in the concrete, and $\varepsilon_{fu}$ is the rupture strain of the FRP reinforcement. The neutral axis location of the concrete section ($c$) can be calculated with equation (5).

$$c = \frac{a_f}{\beta_1}$$

(5)

The moment nominal capacity ($M_n$) can be calculated with equation (6).

$$M_n = A_f \cdot f_f \left( d_f - \frac{a_f}{2} \right)$$

(6)

Where $A_f$ is the area of FRP bars, $f_f$ is the stress in FRP bars under tension, and $d_f$ is the effective depth of the beam, measured from the compression concrete fibre to the centroid of the FRP bars. Tensile stress in the FRP can be computed with the equation 7.2.2d of ACI 440.1R-15 [7].

$$f_f = \left[ \frac{(E_f \varepsilon_{cu})^2}{4} + 0.85 \cdot \beta_1 \cdot f_c \cdot \frac{\rho_f}{E_f} \cdot \varepsilon_{cu} - 0.5 E_f \cdot \varepsilon_{cu} \right] \leq f_{fu}$$

(7)

Where $\rho_f$ is the ratio of FRP bars to the concrete section area, $E_f$ is the modulus of elasticity of FRP bars, and $f_c$ is the concrete strength.

3. Results and Discussion

3.1. Rupture mode

Table 2 compares the rupture modes in the experiment [17] and the rupture modes that resulted in the analytical study. Failure of beams in the experiment occurred in two modes. Specimens FB-2 to FB-4 and HFB-3 as well as HFB-4 failed due to the rupture of GFRP bars. The rest of the beams experienced failure in the form of concrete crushing, where cracks were observed in the constant moment region. Rupture modes in this analytical study are defined by comparing the ratio of FRP bars to the balanced ratio. The FRP rupture governs the failure of the concrete section when the reinforcement bar ratio is less than the balanced ratio. Table 2 shows that the rupture modes in the analytical study are comparable to those of the experiment except for the HFB-6 beam, where the specimen failed by bond failure.

3.2. Flexural capacity

The comparison of the flexural capacity of beams gained in the experiment, analytical calculation using Todeschini’s concrete curve, and ACI code is presented in Table 3. Overall, the flexural capacity predicted using Todeschini’s concrete curve underestimates the flexural capacity gained in the experiment. Figure 4 shows that the average ratio of flexural capacity calculated using Todeschini’s curve to those of the experiments for the FB series specimen is 0.78 with the standard deviation of 6%. On the other hand, as seen in Figure 5, the average ratio of the flexural capacity of Todeschini’s curve to those of experiments for the HFB series beam test is 0.86 with the standard deviation of 7%.
Table 2. Comparison of rupture mode between the experiment [17] and this analytical study

| Code of Specimens | $A_f$ (mm$^2$) | $\rho_f$ | $\rho_{balance}$ | Rupture mode |
|-------------------|--------------|---------|-----------------|--------------|
| FB-2              | 265.46       | 0.0025  | 0.0051          | rupture of FRP |
| FB-3              | 398.20       | 0.0037  | 0.0051          | rupture of FRP |
| FB-4              | 530.93       | 0.0050  | 0.0051          | rupture of FRP |
| FB-6              | 796.39       | 0.0075  | 0.0051          | concrete crushing |
| FB-8              | 1061.86      | 0.0100  | 0.0051          | concrete crushing |
| HFB-3             | 398.20       | 0.0037  | 0.0061          | rupture of FRP |
| HFB-4             | 530.93       | 0.0050  | 0.0061          | rupture of FRP |
| HFB-6             | 796.39       | 0.0075  | 0.0061          | bond failure |
| HFB-8             | 1061.86      | 0.0100  | 0.0061          | concrete crushing |
| HFB-10            | 1327.32      | 0.0125  | 0.0061          | concrete crushing |

Furthermore, for the specimens with reinforcement ratios higher than the balanced reinforcement ratio (FB-6, FB-8, HFB-8, HFB-10), flexural capacity calculated per Todeschini’s curve resulted better compared to those of ACI code. In this case, compression concrete reaches the ultimate strain ($\varepsilon_{cu} = 0.003$) while the strain in the FRP bars is lower than its ultimate strain. The stress-strain concrete curve is fully formed, and causing the contribution of concrete force to the flexural capacity of the beam section is maximum.

Table 3. Comparison of flexural capacity between the experiment [17], analytical calculation in this study and ACI 440.1R-15

| Code of Specimens | Flexural capacity (kN-m) | Ratio |
|-------------------|--------------------------|-------|
|                   | $^{[17]}$ Experiment | Todeschini’s curve | ACI 440.1R-15 | Todeschini’s curve to Experiment | ACI 440.1R-15 to Experiment |
| FB-2              | 68.94                    | 52.41  | 60.23 | 0.76 | 0.87 |
| FB-3              | 111.18                   | 77.81  | 90.35 | 0.70 | 0.81 |
| FB-4              | 125.88                   | 102.64 | 120.46 | 0.82 | 0.96 |
| FB-6              | 171.54                   | 148.14 | 146.41 | 0.86 | 0.85 |
| FB-8              | 222.60                   | 166.67 | 164.71 | 0.75 | 0.74 |
| HFB-3             | 93.24                    | 78.27  | 91.02 | 0.84 | 0.98 |
| HFB-4             | 119.04                   | 103.56 | 121.36 | 0.87 | 1.02 |
| HFB-6             | 200.46                   | 153.12 | 163.50 | 0.76 | 0.82 |
| HFB-8             | 218.04                   | 193.18 | 184.50 | 0.89 | 0.85 |
| HFB-10            | 219.36                   | 211.71 | 202.18 | 0.97 | 0.92 |

Mean 0.78 0.85
STD 0.06 0.08

Furthermore, for the specimens with reinforcement ratios higher than the balanced reinforcement ratio (FB-6, FB-8, HFB-8, HFB-10), flexural capacity calculated per Todeschini’s curve resulted better compared to those of ACI code. In this case, compression concrete reaches the ultimate strain ($\varepsilon_{cu} = 0.003$) while the strain in the FRP bars is lower than its ultimate strain. The stress-strain concrete curve is fully formed, and causing the contribution of concrete force to the flexural capacity of the beam section is maximum.
Figure 4. Comparison of flexural capacity ratios of Todeschini’s curve to the experiment and the ACI code to the experiment for FB specimen series

Figure 5. Comparison of flexural capacity ratios of Todeschini’s curve to the experiment and the ACI code to the experiment for HFB specimen series
4. Conclusion
In this study, an analytical procedure to predict the flexural capacity of beam reinforced with GFRP bars was conducted. A nonlinear stress-strain relationship was used to examine the contribution of concrete under uniaxial compression. The flexural capacity obtained in the analytical study was compared to the results of an experiment published in the literature and to the results calculated using the ACI 440.1R-15 code. The results demonstrated that the rupture modes predicted by the analytical study were comparable to the rupture modes observed in the experiment. Furthermore, flexural capacity gained with Todeschini’s nonlinear curve for the specimens with reinforcement ratios higher than the balanced reinforcement ratio resulted better compared to that of ACI code.

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