NUMERICAL SIMULATION OF THE MECHANICAL BEHAVIOUR OF FRP RC TENSIONED ELEMENTS

IRENE VILANOVA MARCO  
MSc  
University of Girona  
EPS Campus Montilivi  
irene.vilanova@udg.edu*

ALBERT TURON TRAVESA  
PhD  
University of Girona  
EPS Campus Montilivi  
albert.turon@udg.edu

LLUÍS TORRES LLINAS  
PhD  
University of Girona  
EPS Campus Montilivi  
lluis.torres@udg.edu

MARTA BAENA MUÑOZ  
PhD  
University of Girona  
EPS Campus Montilivi  
marta.baena@udg.edu

Abstract

The use of fibre reinforced polymer (FRP) bars in reinforced concrete (RC) structures has emerged as an alternative to traditional RC due to the corrosion of steel in aggressive environments and the electromagnetic transparency of FRP. Because of their different mechanical properties, the cracking and deformability behaviour of FRP RC elements is quite unlike traditional steel reinforced concrete. In recent decades a number of authors and design codes have proposed several approaches to predict the behaviour of FRP RC elements. However, using finite element method (FEM) for the advanced modelling of FRP RC structures has not been investigated in depth. In this paper, a numerical method to model FRP RC tensioned elements using Abaqus FEM code is presented. The interaction between reinforcement and concrete is modelled using cohesive elements powered with a bi-linear bond-slip law obtained directly from experimental tests. Concrete mechanical behaviour is modelled using a damaged plasticity approach with Drucker-Prager criterion to define the failure surface. Numerical results are compared with experimental data and Eurocode 2 predictions.

Keywords: Concrete, FRP, tensile members, cohesive models, bond

1. Introduction

Conventional steel reinforced concrete structures can be corroded in aggressive environments, such as saline, which reduces their service life and increases maintenance and replacement costs [1, 2]. Fibre reinforced polymers (FRP) as reinforcement for concrete structures have emerged as an alternative to steel due their good behaviour in corrosive environments and the electromagnetic transparency.
The mechanical behaviour of FRP RC structures differs from traditional steel RC structures. FRP bars have a linear behaviour up to failure and a lower elastic modulus leading to larger deformations and crack widths. Therefore, the design of FRP RC structures may often be governed by serviceability limit states. The different analytical methods proposed to predict the behaviour of reinforced concrete structures under tensile loads take into account the cracking behaviour, and the tension stiffening effect, and allow calculations of deformations, crack spacing and cracks widths [1-6]. Most of these methods have been developed based on many simplifications. More recently, methods to model the mechanical behaviour of reinforced concrete structures under tensile load using advanced numerical tools such as the finite element method (FEM) have also been presented [7-9]. These models rely on fewer hypotheses and simplifications than the design codes, but have a much higher computational cost. Further research in this area is needed to reduce that cost and improve the constitutive laws used to model the material behaviour.

In this work, a numerical method to study the behaviour of FRP RC tensioned members is presented. The bond between concrete and rebar is modelled using in-house cohesive elements. These cohesive elements are fed with the material parameters obtained directly from experimental tests and reported in a previous work by Baena et al. [10]. The constitutive behaviour of the concrete is modelled using the concrete damaged plasticity (CDP) model available in ABAQUS [11]. Results obtained using this method are compared with data from a previous campaign reported in Baena et al. [6], and with crack spacing predictions from Eurocode 2 [4].

2. Description of the model

Three different constitutive laws are used in this work to model FRP RC members. It is assumed that the mechanical behaviour of the FRP is linear elastic until failure. Therefore, the FRP rebar is modelled using a linear elastic constitutive equation. For the concrete, the DCP model of ABAQUS/Explicit is used [11]. Finally, the interaction between the FRP rebar and concrete is modelled using cohesive elements. A brief description of the bond interface and the concrete damaged plasticity model is introduced in this section.

2.1 Bond interface model

Several analytical laws have been proposed to reproduce the bond behaviour between concrete and reinforcing bars under tensile loads [12-15]. In this work, the bi-linear cohesive law described in Figure 1 is used to describe the interaction between concrete and FRP rebar. The law was described by Turon et al. [16] and adapted by Gonzalez et al. [17] to be used in ABAQUS/Explicit code. Although the constitutive model was developed for the delamination of composite structures, it can also be used to model bonds between reinforcing bars and concrete, assuming that the interaction between concrete and FRP rebar can be collapsed to zero-thickness surface. This approach has been previously adopted by other authors such as Serpieri and Alfano [18, 19], who enhanced the cohesive formulation by introducing friction and dilatancy to model the interaction between concrete and reinforcement. In this work a simpler constitutive model is used because it is assumed that all the mechanisms taking place are contained in the fracture toughness, $G_c$, used in the constitutive equation.
The first part of the constitutive model is linear elastic up to the maximum bond stress $\tau_0$, involving an initial displacement $\Delta_0$. At this point the interface starts to become damaged. Once the maximum bond stress is reached and the crack begins to propagate, the stress is reduced up to the maximum displacement $\Delta_f$. $G_c$ is the fracture toughness needed to propagate the crack.

The fracture toughness and the maximum bond stress are taken from pull-out test results [10]. The authors tested 88 specimens, changing the type of reinforcement bar (carbon, glass and steel). The maximum bond stress is defined as the maximum point of the bond stress-slip curve and the fracture toughness as the area below the curve (Figure 2).

2.2 Concrete model

The CDP model from the ABAQUS library [11] is used in the finite element code to define concrete behaviour. The model considers the total strain as the addition of elastic ($\varepsilon^e$) and plastic ($\varepsilon^p$) strain, $\varepsilon = \varepsilon^e + \varepsilon^p$.

The stress-strain relation associated with damage corresponds to:

$$
\sigma = \left(1 - d\right)D_0^e : \varepsilon
$$

where $d$ is the damage variable ($d=0$ no damage, $d=1$ fully damaged) and $D_0^e$ is the non-damaged elastic modulus. Damage is associated with the damaged mechanisms of concrete – cracking in tension and crushing in compression – that involve a decrease of the elastic modulus.
Damage is governed by hardening variables $\sim_{pl}$ and the effective stress $d = d(-, \sim_{pl})$. The hardening variables at compression ($\sim_{c}$) and tension ($\sim_{t}$) are considered independent.

Microcracks and crushing are proportional to the hardening variables. These variables control the yield surface and the degradation of the elastic modulus. However, they are also related to the energy dissipation that generates microcracks. The yield surface $F(-, \sim_{pl})$, which determines the failure state, is the effective tensile zone, based on the formulation proposed by Lubiner et al. [20] and by Lee and Fenves [21] (Figure 3). A flow rule defines how the yield surface develops under plastic strain.

![Figure 3. Yield surface of DCP model [11].](image)

3. Validation of the presented method

3.1 Description of the model

To validate the model, results from an experimental campaign conducted by the authors and reported in a previous work (Baena et al. [6]) have been used. Four GFRP RC elements axially loaded in tension were tested. Each test specimen was a concrete prism 1300 mm long; the nominal diameter ($d_n$) and the cross-section size are summarized in Table 1.

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>Cross-section size (mm)</th>
<th>Nominal diameter $d_n$ (mm)</th>
<th>Reinforcement ratio $\rho$ (%)</th>
<th>Concrete compressive strength $f_{cm}$ (MPa)</th>
<th>Concrete tensile strength $f_{ct}$ (MPa)</th>
<th>Concrete elasticity modulus $E_c$ (MPa)</th>
<th>Bar elasticity modulus $E_{FRP}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>13_170</td>
<td>170</td>
<td>13.7</td>
<td>0.51</td>
<td>48.1</td>
<td>1.75</td>
<td>27315</td>
<td>37418</td>
</tr>
<tr>
<td>16_170</td>
<td>170</td>
<td>16.9</td>
<td>0.71</td>
<td>46.6</td>
<td>2.58</td>
<td>34514</td>
<td>42835</td>
</tr>
<tr>
<td>19_170</td>
<td>170</td>
<td>19.1</td>
<td>1.00</td>
<td>56.2</td>
<td>2.10</td>
<td>33275</td>
<td>40595</td>
</tr>
<tr>
<td>16_110</td>
<td>110</td>
<td>15.9</td>
<td>1.69</td>
<td>56.2</td>
<td>2.34</td>
<td>33275</td>
<td>38758</td>
</tr>
</tbody>
</table>

The experimental results of the compressive and tensile strength of concrete ($f_{cm}$ and $f_{ct}$ respectively), and the moduli of elasticity ($E_c$ for concrete and $E_{FRP}$ for reinforced bar) are also given in Table 1.
The mechanical properties needed for the bond-slip law, fracture toughness and bond strength are given in Table 2. The bond-slip parameters depend on the bar diameter. Further details of the bond behavior can be found in [6].

Table 2. Mechanical properties of the bond-slip law.

<table>
<thead>
<tr>
<th>SPECIMENT</th>
<th>Bond strength τ_{max} (MPa)</th>
<th>Fracture toughness G_F (mJ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>13_170</td>
<td>17.06</td>
<td>237.26</td>
</tr>
<tr>
<td>16_170</td>
<td>17.33</td>
<td>279.91</td>
</tr>
<tr>
<td>19_170</td>
<td>14.57</td>
<td>225.54</td>
</tr>
<tr>
<td>16_110</td>
<td>17.33</td>
<td>279.91</td>
</tr>
</tbody>
</table>

ABAQUS/Explicit [11] is used to model the RC members. An Abaqus C3D8R element is employed to model the concrete and the rebar material. On average, 61,000 elements per specimen are used to model the ties. Each element consists of eight nodes and has an approximate size of 5 mm. Cohesive elements placed between the concrete and the reinforcing bar are used to simulate the bond behaviour. Taking advantage of the symmetry, only an eighth is analysed (Figure 4). A displacement controlled analysis is performed, increasing the displacement applied at the free end of the rebar.

Figure 4. Meshed eighth part of the concrete specimen.

3.2 Results

In this section the results of the method are presented, compared with the experimental data and discussed. The comparison between the experimental and the FEM prediction is shown in Figure 5.

The numerical results have a linear behaviour until the cracking load \( P_{cr} \) is reached. As seen in Figure 5, the FEM model captures the cracking process when the load increases. Once all the cracks are formed (stabilized cracking), the trend of the modelling results in a line parallel to, or slowly approaching, the bare bar response. The sudden changes in the FEM curve are attributed to the nature of the analysis, an explicit scheme, and the sudden energy release when a new crack appears.

The FEM simulation can also provide the crack pattern obtained during the analysis. The final crack patterns obtained for each specimen are shown in Figure 6. Crack spacing, \( S_m \), can be predicted using equation 2 from Eurocode 2:
where $k_1$ is 0.8 for ribbed and 1.6 for smooth steel bars, $k_2$ is 0.5 for flexural and 1.0 for tensile loading, and $\rho_{\text{eff}}$ is the ratio of internal reinforced to the effective area of the concrete in tension.

The crack spacing obtained from the FEM simulations, together with the experimental results and Eurocode 2 provisions (Eq. 2), are given in Table 3.

For lower reinforcement ratios, $\rho<1$, the FEM predictions underestimate crack spacing, while for $\rho<1$ the crack spacing is overestimated in the FEM simulations. Although the random effects associated with the cracking process always have to be taken into consideration when analysing crack patterns, the FEM predictions of mean crack spacing are more accurate than predictions made using Eurocode 2 equations, at least for the specimens with higher reinforcement ratios.

### Table 3. Comparison between numerical and experimental results for crack spacing.

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>Experimental $S_{\text{m,exp}}$ (mm)</th>
<th>Eurocode 2 $S_{\text{m,EC2}}$ (mm)</th>
<th>$S_{\text{m,EC2}}/S_{\text{m,exp}}$</th>
<th>FEM model $S_{\text{m,FEM}}$ (mm)</th>
<th>$S_{\text{m,FEM}}/S_{\text{m,exp}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>13_170 ($\rho=0.51%$)</td>
<td>264.96</td>
<td>586.00</td>
<td>2.2</td>
<td>188.30</td>
<td>0.7</td>
</tr>
<tr>
<td>16_170 ($\rho=0.71%$)</td>
<td>227.75</td>
<td>486.00</td>
<td>2.1</td>
<td>145.00</td>
<td>0.6</td>
</tr>
<tr>
<td>19_170 ($\rho=1.00%$)</td>
<td>113.27</td>
<td>434.50</td>
<td>3.8</td>
<td>116.00</td>
<td>1.0</td>
</tr>
<tr>
<td>16_110 ($\rho=1.69%$)</td>
<td>123.34</td>
<td>241.26</td>
<td>1.9</td>
<td>138.50</td>
<td>1.1</td>
</tr>
</tbody>
</table>
4. Conclusions
A numerical method to predict the response of GFRP RC members under tensile load is presented. Cohesive elements are used to model the interaction between concrete and reinforcement and a concrete damaged plasticity model is used to define the concrete behaviour. The method is validated by comparing its predictions with experimental data. It reproduces satisfactorily the cracking process and the experimental results in terms of load-deformation. The method also provides satisfactory results for crack spacing when compared with those obtained using Eurocode 2 equations.

5. Acknowledgements
The authors acknowledge the support provided by the Spanish Government (Ministerio de Educación y Ciencia), Project BIA2010-20234-C03-02.

6. References