MODELING THE FIRE RESPONSE OF CONCRETE BEAMS REINFORCED WITH GFRP BARS

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ABSTRACT: In the last two decades, researchers and engineers studied the possibility of reinforcing slabs and beams with fiber-reinforced polymer (FRP) bars instead of the conventionally used steel reinforcement. Such FRP bars have high strength and corrosion resistance properties. However, the fire resistance of reinforced concrete (RC) members is still a concern when reinforced with such materials. The major objective of this paper is to develop a 3D finite element (FE) model and perform nonlinear thermal-stress analysis to predict the fire performance and time to failure of concrete beams reinforced with GFRP bars. The developed model includes temperature-dependent thermal and mechanical material properties. The predicted fire resistance of the beam specimen is compared with published experimental data. The numerically predicted and measured experimental results are in close agreement. The developed FE model would be used as a valid tool to study the effect of key parameters on the fire performance of RC beams reinforced with FRP bars.

1. Introduction

Over the last two decades, several research studies investigated the performance of concrete beams when reinforced with fiber-reinforced polymer (FRP) materials at room temperature (Alsayed, 1998; Robert and Benmokrane, 2010). The advantages of using such reinforcing bars over the conventional steel reinforcement are their high strength and corrosion resistance properties. However, FRP bars have a low glass transition temperature ($T_g$). Thus, they poorly perform when subjected to elevated temperatures due to their rapid loss of elastic modulus and tensile strength and are susceptible to combustion (Abbasi and Hogg, 2006; Bisby, 2003). Accordingly, the performance of reinforced concrete (RC) members when reinforced with FRP bars should be examined under fire loading.

Limited experimental and numerical studies had been conducted so far of the fire performance of concrete beams reinforced with FRP bars due to the limited and expensive fire experimental facilities and extensive amount of preparation. Thus, further experimental and numerical studies should be carried out to better understand the fire performance of concrete members when reinforced with FRP bars.

Upon the limited studies, Abbasi and Hogg (2006) investigated experimentally the fire performance of a RC beams reinforced in flexure with Glass-FRP (GFRP) bars, having a bottom concrete cover of 75 mm. The beams were designed according to Eurocode 2 (2004) and ACI-440 (2001). The beams were statically loaded up to the concrete cracking limit and then subjected to the standard ISO834 (1970) fire curve. The tested beam achieved a fire endurance of 128 minutes, which is higher than the design allowable limit of 90 minutes. The failure criterion was based on the BS 476: Part 20 standard (1987), which limits the mid-span deflection to $L/20$, where $L$ is the span length. It was concluded that a concrete cover thickness of 70 mm is adequate for concrete beams reinforced with GFRP bars is adequate to achieve a fire endurance limit of 90 minutes.
The aim of this paper is to develop a finite element (FE) model that can predict the behaviour of the tested beam by Abbasi and Hogg (2006) at all stages of fire loading. The FE model will be developed in ANSYS version 11.0 (2007) and a coupled thermal-stress analysis will be performed to capture the response of the tested specimen when subjected to the ISO 834 fire curve scenario.

2. Finite Element Model Development

The developed FE model has the same geometrical and material properties of the tested rectangular beam specimen by Abbasi and Hogg (2006). The total length, span length, width, and thickness of the tested specimen was 4400, 4250, 350, and 400 mm, respectively. The bottom concrete cover the FRP flexural reinforcement was 70 mm. The beam specimen was reinforced in flexure with seven #12 mm GFRP bars.

The discretized mesh of the developed FE quarter model is shown in Fig. 1. A quarter model is chosen due to the symmetrical properties of the tested specimen along the vertical and transverse axes. Symmetry is simulated by restraining the motion perpendicular to the axis of symmetry. In order to perform thermal-stress analysis, thermal and structural elements should be assigned to the concrete and GFRP materials. The used thermal elements for the concrete and GFRP reinforcement were SOLID70 and LINK33, respectively. In addition, structural LINK8, SOLID65, and SOLID45 elements were used for the GFRP bars, concrete, and steel supports, respectively (ANSYS, 2007). It should be noted that bond-slip between the GFRP bars and adjacent concrete surfaces did not occur during fire testing (Abbasi and Hogg, 2006). Thus, perfect bond is assumed between the GFRP bars and adjacent concrete surfaces. However, the simulation of bond-slip at elevated temperatures should be considered in future studies. In addition, failure is assumed in this study to occur when temperature in the GFRP bar reaches 462°C (Abbasi and Hogg, 2006). Full details of the technique used for simulating the fire performance of concrete members reinforced with GFRP bars can be found in a previous study conducted by the authors (Hawileh and Naser, 2012).

3. Material Properties at Elevated Temperatures

Thermal and mechanical temperature dependent properties are required to perform transient thermal stress analysis. The mechanical properties of the GFRP bars degrade dramatically at elevated temperature (Abbasi and Hogg, 2006; Bisby, 2003). The required thermal material properties to conduct transient thermal analysis are the thermal conductivity \( k_0 \), specific heat \( C_0 \), and density \( \rho \). The thermal and mechanical properties (elastic modulus \( E \), Poisson’s ratio \( \mu \), and coefficient of thermal expansion in the longitudinal direction \( \alpha_L \) and transverse direction \( \alpha_T \)) of the different materials at room temperature are listed in Table 1.
### Table 1 - Thermal and mechanical material properties at room temperature

<table>
<thead>
<tr>
<th>Material</th>
<th>$E$ (GPa)</th>
<th>$\mu$</th>
<th>$k_0$ (W/mm K)</th>
<th>$C_o$ (J/kg K)</th>
<th>$\alpha_L$ (1/K)</th>
<th>$\alpha_T$ (1/K)</th>
<th>$\rho$ (kg/mm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>30.5</td>
<td>0.20</td>
<td>2.7×10⁻³</td>
<td>722.8</td>
<td>6.08×10⁻⁷</td>
<td>-</td>
<td>2.32×10⁻⁷</td>
</tr>
<tr>
<td>GFRP</td>
<td>40.8</td>
<td>0.28</td>
<td>4.0×10⁻⁵</td>
<td>1310</td>
<td>6.58×10⁻⁶</td>
<td>33.7×10⁻⁵</td>
<td>1.60×10⁻⁶</td>
</tr>
</tbody>
</table>

The thermal conductivity, specific heat, density, elastic modulus, tensile strength, compressive strength, and stress-strain curves of the concrete material as a function of increasing temperature were taken from the provided guidelines by Eurocode 2 (2004) and factors given by Zhou and Vecchio (1987).

It should be noted that due to the lack of data of temperature dependent thermal material properties for the GFRP bars, the thermal properties of the GFRP bars at room temperature are used in the transient thermal analysis of this study. The temperature thermal and mechanical dependent material properties of the concrete material are reported elsewhere (Hawileh and Naser, 2011). In addition, the GFRP bars are assumed to behave elastically up to failure. The reduction in the elastic modulus ($K_E$) and tensile strength ($K_\sigma$) as a function of increasing temperature are based on the reduction factors suggested by Abbasi and Hogg (2006), and presented in Eqs. 1 and 2.

\[
K_E = 1 - 0.0017\Delta T \quad (1)
\]
\[
K_\sigma = 1 - 0.0025\Delta T \quad (2)
\]

where, $\Delta T = T - 20 ^\circ C$

### 4. Loading and Boundary Conditions

The first stage of conducting a coupled thermal-stress analysis is to perform transient thermal analysis, in which the ISO834 (1970) fire curve is applied as nodal temperatures at the beam’s soffit and two vertical sides. The obtained temperature distribution from the transient thermal analysis is applied to the structural model as nodal temperatures at different specified time steps. The simply supported beam is also loaded with a sustained gravity load of 40kN that was uniformly distributed along the beam specimen. The second stage consisted of performing stress analysis to obtain the deformation and stresses of the beam specimen at all stages of fire loading. It should be noted that the nodal temperatures of the ISO834 fire curve shown in Fig. 2 are applied in terms of several load steps. Each load step is composed of several smaller sub-steps that are solved using the Newton-Raphson technique (ANSYS, 2007).

![Fig. 2 - ISO834 fire curve (ISO834, 1970)](image-url)
5. Results and Discussions

Figure 3 shows the temperature distribution throughout the beam after 30 and 120 minutes of fire exposure. In addition, Fig. 4 shows the vertical displacement of the beam after 120 minutes of fire exposure.

(a) After 30 minutes of fire exposure

(b) After 120 minutes of fire exposure

FIG. 3 – Temperature Distribution after 30 and 120 minutes of fire exposure

Fig. 4 – Vertical displacement (in mm) after 120 minutes of fire exposure
In order to validate the accuracy of the developed model, the predicted and experimentally obtained time to failure (fire resistance) of the beam specimen is compared. The fire resistance of RC beams is defined when one of the following thermal and strength criteria is reached:

1. The temperature of the GFRP bars exceeds the critical temperature of 400°C.
2. The temperature of the top beam surface exceeds 140°C.
3. The vertical displacement of the beam specimen exceeds $L/20$, where $L$ is the beam’s span length (mm).
4. The rate of deflection exceeds $L^2/900d$ (mm/min), where $d$ is the effective slab’s depth (mm).

Table 2 compares the predicted and experimentally obtained fire resistance of the beam specimen.

<table>
<thead>
<tr>
<th>Slab</th>
<th>Fire Resistance (mins.)</th>
<th>Controlling failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experimental</td>
<td>119</td>
<td>Temperature limit of 400°C</td>
</tr>
<tr>
<td>FE</td>
<td>116</td>
<td>Temperature limit of 400°C</td>
</tr>
<tr>
<td>FE/Exp.</td>
<td>0.975</td>
<td>-</td>
</tr>
</tbody>
</table>

It is clearly indicated from Table 2 that the experimental and numerically predicted fire resistance of the beam specimen are 119 and 116 minutes, respectively and failure is controlled by the temperature limiting criterion of 400°C. Thus, the difference between the predicted and experimental fire resistance is 2.5%. It should be also noted that both the FE model and fire test achieved a fire resistance of more than 90 minutes, and thus passes the required rating set by building regulations for fire safety. Thus, it can be concluded that the developed model can reasonably predict the fire resistance of concrete beams reinforced with GFRP bars.

6. Conclusion

It could be concluded from the presented results of this numerical study that the developed FE model can accurately predict the fire resistance of concrete beams reinforced with GFRP bars. The validated model should be used in a future study to validate the fire response of other tested beam specimens reinforced with FRP bars. In addition, the developed numerical model could be used in design oriented parametric studies to examine the influence of various parameters on the fire performance of concrete beams reinforced with GFRP bars.

7. References


