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Finite Element Modeling of Insulated FRP-strengthened RC Beams Exposed to Fire

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ABSTRACT: This paper presents a finite element (FE) model for the thermo-mechanical analysis of insulated FRP-strengthened reinforced concrete (RC) beams exposed to fire. In the model, the effects of loading, thermal expansion of materials, and degradations in both the mechanical properties of materials and the bond behavior at FRP-to-concrete and steel-to-concrete interfaces due to elevated temperatures are all considered. The validity of the FE model is demonstrated through comparisons of FE predictions with results from existing standard fire tests on insulated FRP-strengthened RC beams.

1 INTRODUCTION

Despite its great success in the past two decades, the fiber reinforced polymer (FRP) strengthening technology suffers from one major limitation when indoor applications are considered. FRP composites show poor performance in fire as the polymer matrix typically has a low glass transition temperature, T_g. The polymer transforms into a soft and viscous material with severe stiffness and strength degradations when it is subjected to temperatures close to T_g. In addition, the polymer matrix may ignite under high heat fluxes, resulting in the generation of smoke and the spread of flames. Therefore, a layer of insulation material is often applied on the bonded FRP reinforcement to maintain the fire safety of the strengthened reinforced concrete (RC) member.

A direct approach for evaluating the fire endurance of an RC member strengthened with FRP is to conduct a standard fire test. Limited standard fire tests (Bisby et al. 2005a; Gao et al. 2010; Williams et al. 2008) have indicated qualitatively that FRP-strengthened RC members with appropriate design and insulation can achieve satisfactory fire performance. However, such standard fire tests are usually very expensive and time-consuming and therefore, their usefulness is limited in providing a comprehensive, quantitative understanding of the fire performance of insulated FRP-strengthened RC members covering wide ranges of various design parameters.

As an alternative to standard fire tests, numerical models for the fire resistance analysis of FRP-strengthened structural members have been developed. Bisby et al. (2005b) proposed a sectional model for the fire resistance analysis of FRP-confined RC columns. Hawileh et al. (2009) employed a nonlinear finite element (FE) model to study the heat transfer and deformation mechanisms in an insulated FRP-strengthened RC T-beam which was tested by Williams et al. (2008). In their work, both the external FRP and the internal reinforcing bars were assumed to be fully bonded with the concrete although bond failure between FRP and concrete is a common failure mode in FRP-strengthened RC beams. Indeed, the bond between FRP and concrete may degrade more rapidly than FRP itself under elevated temperatures.

The fire endurance analysis of insulated FRP-strengthened RC members is more challenging than that of un-protected FRP-strengthened RC members as for the latter the contribution of FRP can be simply ignored [e.g. Han et al. (2006)]. Additional aspects that need to be considered in the former include the temperature-dependent behavior of FRP and interactions among FRP, concrete and steel reinforcement at elevated temperatures. This paper presents a generic and advanced FE model based on ABAQUS to simulate the thermal and structural responses of insulated FRP-strengthened RC members exposed to fire.

2 THE FE MODEL

2.1 *Thermal and mechanical properties of steel, concrete and FRP at elevated temperatures*

The thermal conductivity, specific heat and thermal expansion of steel and concrete are defined following EN 1992-1-2 (2004). The thermal properties of carbon FRP sheets at elevated temperatures are determined according to Griffis et al. (1981) but the longitudinal thermal expansion coefficient of carbon FRP sheets is assumed to be zero based on ACI 440.2R-08 (2008).

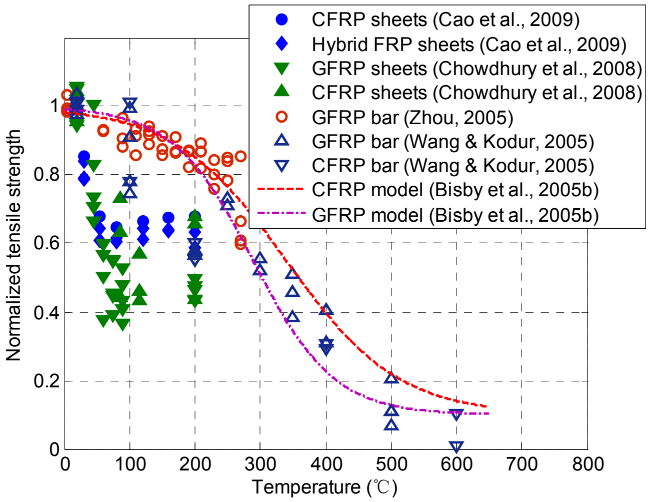
The uni-axial compressive stress-strain model for concrete at elevated temperatures given in EN 1992-1-2 (2004) is adopted; this model already includes the temperature and transient creep effects. The tension-softening behavior of concrete is represented by a tensile stress-crack displacement relationship based on the fracture energy and crack band concepts. Concrete is modeled using a damaged plasticity model, in which the yield surface developed by Lee and Fenves (1998) and the Drucker-Prager flow potential function are employed. The uni-axial tensile stress-strain relationship of reinforcing steel at elevated temperatures is also defined according to EN 1992-1-2 (2004).

Little information is available on the mechanical behavior of FRP plates/sheets for structural strengthening applications at elevated temperatures in the published literature. Bisby et al. (2005b) collected existing test data and proposed a sigmoid function model for the strength and stiffness degradations of FRP composites at elevated temperatures. It should be noted that most of the test data collected by Bisby et al. (2005b) were from the tensile tests of FRP bars or tendons. Therefore, Bisby et al.'s (2005b) model may not be suitable for FRP sheets commonly used to strengthen RC beams as such FRP sheets are usually formed via a wet lay-up process and possess a much lower T_g than pre-cured FRP products.

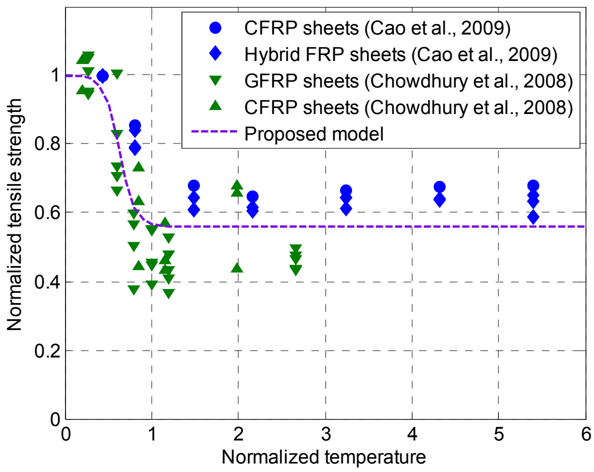
Figure 1a presents the available tensile strength test data for FRP sheets at elevated temperatures (Cao et al. 2009; Chowdhury et al. 2008). The test data from Wang and Kodur (2005) and Zhou (2005) for FRP bars and the models proposed by Bisby et al. (2005b) are also shown in the figure for comparison. In the figure, the tensile strengths of FRP bars/sheets at different temperatures are normalized by that obtained at room temperature. It is clearly seen that the tensile strength degradation of FRP sheets is more severe than that of FRP bars. As the performance of FRP sheets/bars depends predominately on the glass transition temperature, T_g, of the polymer matrix, the value of T_g needs to be taken as a key parameter in any mathematical relationship representing the degradation of tensile strength with elevated temperature. A sigmoid function as given below and used by Bisby et al. (2005a) seems appropriate for the tensile strength degradation of both FRP bars and FRP sheets:

$$\frac{f_T}{f_0} = \left(\frac{1-a_1}{2}\right)\times\tanh\left(-a_2\times\left(\frac{T}{T_g}-a_3\right)\right)+\left(\frac{1+a_1}{2}\right) \qquad (1)$$

where f_0 and f_T are the tensile strengths at ambient temperature and at an elevated temperature T, respectively; and $a_1 = 0.556$, $a_2 = 5.624$ and $a_3 = 0.630$ are empirical factors. Figure 1b shows this proposed relationship where the tensile strength is normalized by the tensile strength at ambient temperature and the temperature by the T_g value of the polymer matrix. Due to a lack of test data, it is assumed that relationship described by Eq. 1 can be also used to evaluate the degradation of elastic modulus of FRP sheets at elevated temperatures.



(a) Normalized strength vs. temperature



(b) Normalized strength vs normalized temperature

Figure 1. Tensile strength degradations of FRP sheets at elevated temperatures.

2.2 Bond models for interfaces

For insulated FRP-strengthened RC members exposed to fire, the bond behavior of FRP-to-concrete and reinforcing steel-to-concrete interfaces becomes important. Bond deteriorations at both interfaces with elevated temperature may significantly affect the cracking behavior of concrete and the occurrence of debonding failure in the insulated system. However, such bond deteriorations have received little attention in existing work. In the present FE model, the bond-slip responses at FRP-to-concrete and steel-to-concrete interfaces at elevated temperatures are explicitly represented through the use of nonlinear spring elements. A perfectly rigid contact condition is assumed for the normal direction between reinforcing steel bars and concrete. In the tangential direction, the CEB-FIP (1993) bond-slip model is adopted as the ambient temperature model for steel-to-concrete interfaces; appropriate bond strength and interfacial energy reductions are incorporated into this model to reflect bond deteriorations at elevated temperatures (Dai et al. 2010).

For FRP sheets-to-concrete interfaces, Gao et al. (2010) conducted double-lap shear tests to investigate bond deteriorations at elevated temperatures. The carbon fiber sheets (0.167 mm thick) used had a tensile strength of 2900 MPa and an elastic modulus of 230 GPa. The bond length and width of the FRP sheets were 120 mm and 100 mm, respectively. The compressive strength of concrete was 34.5 MPa. As shown in Figure 2, the failure load of the FRP-to-concrete interface at 40°C is slightly higher than that at 4°C but then decreases as the temperature further increases. For the current FE model, the bond-slip model proposed by Lu et al. (2005) for FRP-to-concrete interfaces at room temperature was modified to describe the temperature-dependent bond behavior described above. The interfacial fracture energy (the area underneath the bond-slip curve) of the FRP-to-concrete interface at an elevated temperature was derived from the temperature-dependent interfacial failure load (i.e. the pull-off load of the FRP sheet as shown in Figure 2), based on the fact that the failure load is proportional to the square root of the interfacial fracture energy. As a result, a temperature-dependent local bond stress-slip model for FRP sheets-to-concrete interfaces at elevated temperatures was established for use in the FE model (Figure 3) (Dai et al. 2010).

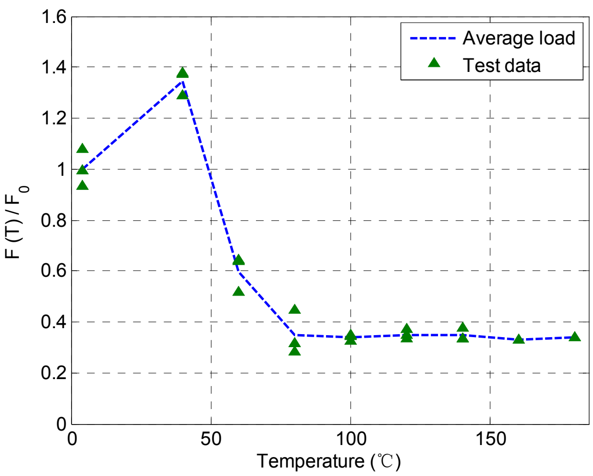


Figure 2. Failure load of FRP-to-concrete bonded joints versus temperature.

3 VERIFICATION OF THE FE MODEL

The FE model described above was implemented using ABAQUS and deployed to analyze full-scale fire tests conducted on three RC beams strengthened with CFRP sheets, which were protected with different insulation materials (Gao et al. 2010). Due to space limitation, only results for one RC beam insulated with a 40 mm calcium silicate board are presented herein. In the test, the temperatures at the FRP sheet-to-concrete interfaces at the mid-span and quarter-span cross-sections were measured by Type-K thermocouples. Other details of the tests, such as the arrangements of steel reinforcement and thermocouples, can be found in Gao et al. (2010).

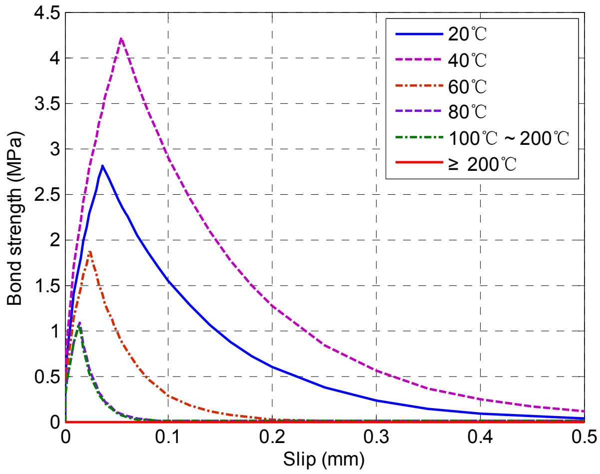
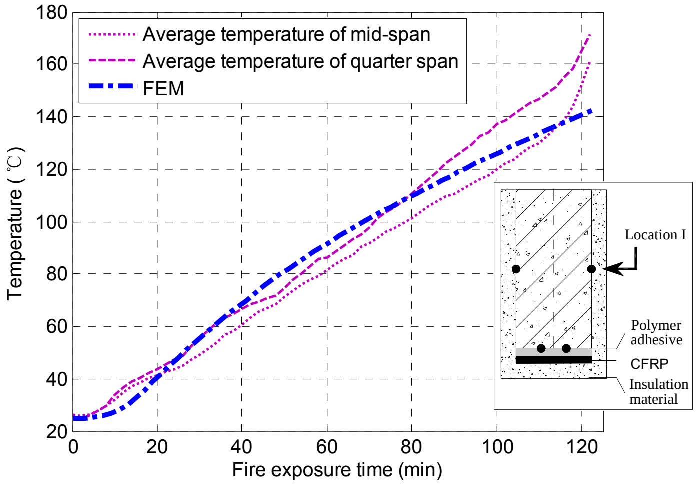


Figure 3. Proposed bond-slip curves of FRP-to-concrete interfaces at elevated temperatures.

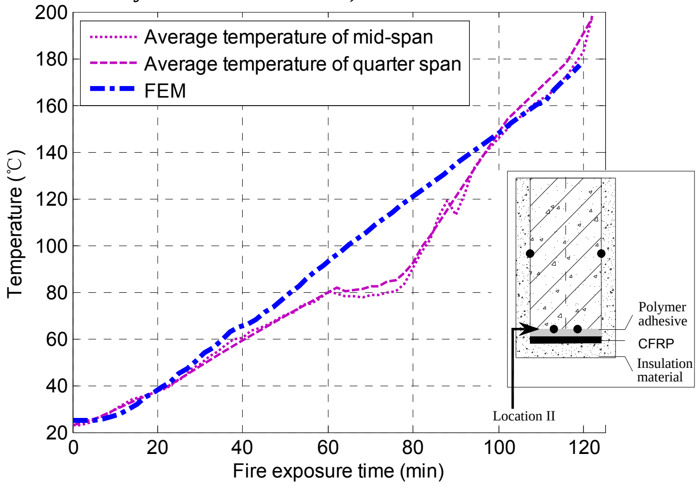
Figure 4 shows experimental and predicted temperatures at various sectional depths as a function of fire exposure time. At the mid-height of the beam, a close agreement between experimental and predicted temperatures is seen. At the soffit of the beam, the predicted temperatures at the FRP-to-concrete interface differ significantly from the experimental values at fire exposure times of 60 and 80 minutes; otherwise, there is reasonable agreement between the two sets of results. In the test, the temperature experienced a plateau 60 minutes after the fire test commenced and a more rapid increase between 80 minutes and 100 minutes. However, the FE modeling could not capture these variations. This plateau was possibly due to the migration of moisture from the insulation material to the surface of the FRP sheet due to the better moisture-resistance of the latter. In Figure 5, the test and predicted mid-span deflections of the insulated FRP-strengthened RC beam are shown as a function of the fire exposure time. It is seen that the predicted results agree very closely with the test results. It can thus be concluded that the FE model is capable of providing reasonably accurate predictions for both the temperature field and the structural response of an insulated FRP-strengthened RC beam.

4 CONCLUDING REMARKS

In this paper, an advanced FE model has been presented for the evaluation of fire performance of insulated FRP-strengthened RC beams. In the model, the bond−slip responses of FRP-to-concrete and steel-to-concrete interfaces at elevated temperatures are explicitly represented. The accuracy of the FE model has been verified through comparisons between predicted results and results from a standard fire test on an insulated FRP-strengthened RC beam conducted by Gao et al. (2010). FE predictions for both the temperature field and the structural response are in reasonably close agreement with the test data. The model is being exploited in parametric studies for the development of fire endurance design guidelines for insulated FRP-strengthened RC beams.



(a) Location 1 (at the mid-height of the beam and between the insulation layer and the concrete)



(b) Location II (at the soffit of the beam and between the FRP and the concrete)

Figure 4. Temperatures versus fire exposure time

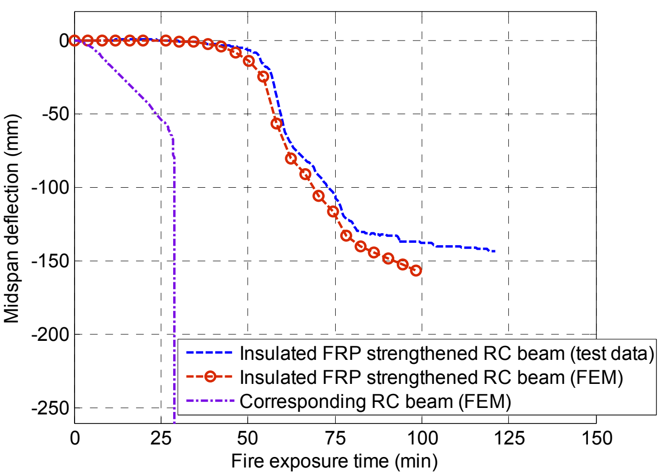


Figure 5. Mid-span deflections versus fire exposure time

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