High-temperature behaviour of concrete slabs reinforced with FRP bars

Emidio Nigro , Gaetano Manfredi , Edoardo Cosenza & Giuseppe Cefarelli
Department of Structural Engineering, University of Naples "Federico II", Naples, Italy

ABSTRACT: The paper deals with the structural behaviour of concrete slabs reinforced with FRP bars or grids in the case of high temperature. The experimental data available in the literature about the behaviour of fiber-reinforced composite materials as well as of concrete slabs reinforced with FRP bars or grids at high temperature have been carefully examined, determining deterioration curves of bars mechanical properties as a function of the temperature. An accurate calculation procedure has been implemented and validated in order to assess the deterioration curves of the FRP reinforced concrete slabs bearing capacity depending on the main geometrical and mechanical parameters. Some simplified design methods have been also suggested.

1 INTRODUCTION

The durability and the actual life expectancy of concrete slabs reinforced with steel bars, above all if exposed to severe environmental conditions, represent nowadays key problems.

An interesting application, which has already been used in some countries in the bridge slab field, is represented by the use of FRP bars or grids instead of the traditional steel reinforcements. There are several advantages of the technological solution based on FRP bars or grids embedded in concrete, replacing steel bars: the excellent resistance to corrosion as well as to environmental agents, like chemical attacks and freeze-thaw cycles, the standardization and the simplicity of the building process, the facility of moving within the yard and the low transportation costs due to material lightness, the low times of placing.

Nevertheless, an important aspect of the concrete slabs reinforced with FRP behaviour is given by the effects of the exposure to fire at slab's top and bottom, possible both for bridge decks outside and for residential or industrial building decks. In those cases, the mechanical properties' deterioration of the fiber-reinforced composite materials and of the bond at high temperatures determines a reduction of the resistance and stiffness of the elements working in flexure. Obviously, the high temperature effects result critical for RC elements strengthened with externally bonded FRP reinforcements (Nigro et Al. 2006), but they results significant also for concrete elements reinforced with internal FRP bars (Bisby, Green & Kodur, 2005).

The aim of this paper is to examine the fire resistance of unprotected concrete slabs reinforced with FRP bars or grids, to individuate the main parameters that characterize the behaviour and to suggest simplified design/check methods.

2 THERMO-MECHANICAL CHARACTERIZATION OF THE MATERIALS

Studying the behaviour at the elevated temperatures of the structural members reinforced with FRP bars or grids it is necessary, for each material that composes the member, to define the thermal and mechanical properties in function of the temperature.

A brief synthesis of the FRP reinforced concrete elements behaviour background is contained in a recent paper of Bisby, Green & Kodur (2005). Several research activities are reported in the literature studying the high-temperature mechanical properties of FRP materials and the behaviour of FRP reinforced concrete elements in flexure exposed to standard fire (Blontrock et Al., 1999; Bisby, 2003; Bisby et Al., 2005; Kodur et Al., 2005; Bisby et Al., 2007).

The examination of the experimental data emphasizes the different bars thermo-mechanical behaviour also inside the same typology. Degradation of the thermo-mechanical properties at high temperatures is typically governed by fiber or polymer matrix types, fiber amount and external surface of the rods: Figure 1a shows the strength values f_{fu} of various CFRP bars in function of the temperature (the values are normalized to the strength at T=20°C). Therefore, any bar typology has specific mechanical properties at elevated temperatures.

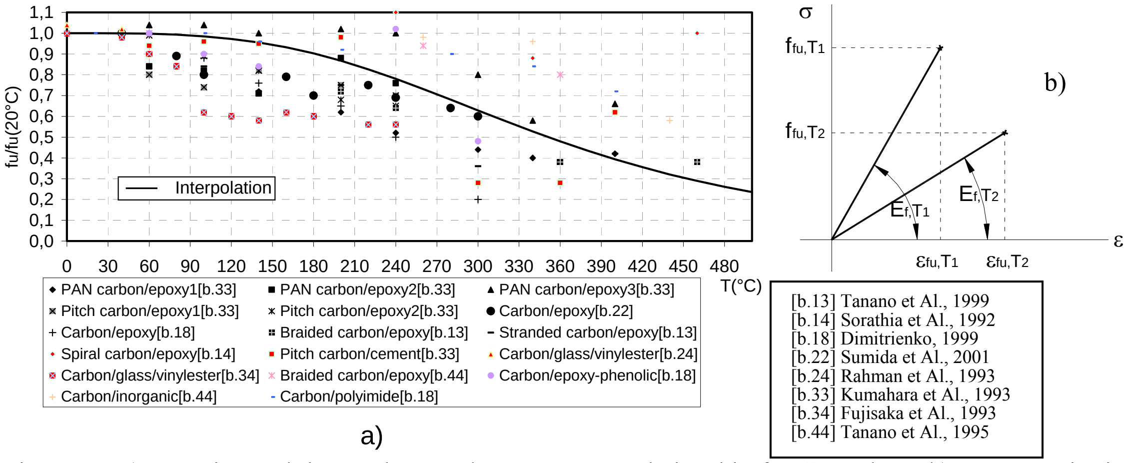


Figure 1 – a) Experimental data and strength-temperature relationship for CFRP bars; b) FRP constitutive laws for two difference temperatures

However, in order to achieve synthetic pointing on the FRP bars strength and tangent elastic modulus in function of the temperature, as well as in Bisby et Al. (2005), "average" behavioural deterioration curves for the different kind of composite materials that are utilized for the bars (CFRP, GFRP, AFRP) have been obtained (see Nigro et Al., 2007). As an example, the following *average strength-temperature curve* has been fitted for CFRP bars (see also Figure 1a):

$$\rho_f(T)=\frac{f_{fu}(T)}{f_{fu}}=\frac{A}{B+C\cdot T^D}=\frac{0{,}06}{0{,}06+1{,}99E^{-10}\cdot T^{3{,}33}} \tag{1}$$

Similar expressions have been utilized to assess the variation $\rho_E(T)$ of the *tangent elastic modulus*, leading to the definition of the stress-strain relationship of the FRP bars in function of the temperature *T*. This law is assumed elastic-brittle, as well as in normal conditions (Figure 1b). Further data concerning GFRP and AFRP bars are also reported in Nigro et Al. (2007).

With reference to the concrete, EN1992-1-2 (2004) suggests formulations, both concerning the thermal properties (thermal conductivity, specific heat and density) that concerning the mechanical properties (stress-strain relationship of the material at the elevated temperatures).

3 ASPECT OF THE FRP REINFORCED CONCRETE SLABS BEHAVIOUR

For evaluating the thermal field induced into the sections by the fire or other heat sources it is accepted the hypothesis of decoupling the thermal behaviour of the materials from the mechanical behaviour, as well as suggested by EN1992-1-2. This hypothesis is at the basis of the Fourier equation for the study of the heat propagation within solid bodies. Due to the variability of the material thermal properties with the temperature (thermal conductivity, specific heat), a numerical solution of the heat transfer problem has to be performed; in the paper the finite element technique is used by means of the well-known FIRES-T3 computer program (Iding et Al. 1997). The heat transfer on the boundary may be assumed to occur through a combination of radiation and convection, linearly or non-linearly modelled. The thermal flux is defined as well as suggested by EN1991-1-2 (2004).

According to usual FEM approach, the slab or beam cross-section is divided into a sufficient number of layers (1-D thermal analysis) and in each one the temperature is assumed as uniform and equal to that of its centroid. Each element is characterised by specific thermal properties, different from that of other elements and variable depending on its temperature.

For the considered time of exposure to prescribed environment conditions (fire or others), the thermal analysis gives the thermal field in the cross-section. A specific stress-strain law (σ-ε;T_i), which is a function of the local temperature and takes into account the variation of the material mechanical properties with the temperature (see. sec. 2), is considered for each element of the cross-section discretization.

Fire safety verification of the structural elements or of the complete structure can generally be assessed according to the Eurocodes and the recent international codes, in strength domain or in temperature domain or in time domain. Verification in the strength domain requires that the following design equation be satisfied:

$$E_{fi,d,t} \leq R_{fi,d,t} \tag{2}$$

in which $E_{fi,d,t}$ and $R_{fi,d,t}$ represent respectively the design values of the actions' effects (generally the "*quasi-permanent*" load combination is assumed) and the ultimate resistant capacity in fire conditions for a *t* duration of fire exposure (assessed, as well as is usually in the fire safety verifications, taking the partial safety factor for each material equal to 1.0). Verification in the temperature domain is proposed by the Canadian Guidelines CAN/CSA-S806-02 (2002) for reinforced slabs with FRP bars: in this code they fix as a resistance limit in fire conditions the time which takes for temperature inside the FRP bar to reach T_{cr} value *(critical temperature),* defined as the temperature to which FRP bars loses 50% of its resistance. The same guide contains several graphics which provides temperature values in the bars of slabs exposed to fire ASTM E119 as a function of fire time exposure, concrete cover, slab's thick and kind of concrete in question.

3.1 *Incremental-iterative procedure*

The assessment of the structural member flexural resistance at elevated temperature is performed, as for normal temperature, determining the bending moment-curvature law (M-χ;N_{ext}) of the critical cross-section for the imposed value of the axial force N_{ext} and the current temperature field within the section. The numerical procedure to assess the moment-curvature law of the cross-section is iterative and develops through the following steps.

The external axial force N_{ext} (N_{ext} = 0 in pure bending) and the distribution of the temperatures $T_i(t)$ within the section, related to the assigned exposure time t, are known and fixed for each (M-χ;N_{ext};t) diagram.

For an assigned curvature χ_j, a tentative value for the average strain ε_{med} of the cross-section is initially assumed and the corresponding distributions of strain ε_i and stress $\sigma_i = \sigma(\varepsilon_i)$ within the section are determined on the basis of the temperature-dependent stress-strain laws (see par. 2). The internal axial force N_{int} is then evaluated starting from the stress distribution:

$$N_{\text{int}} = \sum_{1}^{n_c} A_{c,i} \cdot \sigma_{c,i} + \sum_{1}^{n_f} A_{f,i} \cdot \sigma_{f,i} \quad . \tag{3}$$

Iterations varying the average strain ε_{med} of the section need up to satisfying the longitudinal equilibrium equation within a suitable error:

$$\left| N_{\text{int}} - N_{est} \right| \leq \delta \quad \rightarrow \quad \varepsilon_{med} \quad . \tag{4}$$

If the ultimate limit strains of the materials, dependent on the local temperature, are not exceeded,

$$\varepsilon_{c,i} \leq \varepsilon_{c,u}(T_i) \ , \ \ \varepsilon_{f,i} \leq \varepsilon_{f,u}(T_i) \tag{5}$$

then the bending moment M_j corresponding to the assigned curvature χ_j may be determined:

$$M_j = \sum_{1}^{n_c} A_{c,i} \cdot \sigma_{c,i} \cdot \left(y_{c,i} - y_G \right) + \sum_{1}^{n_f} A_{f,i} \cdot \sigma_{f,i} \cdot \left(y_{f,i} - y_G \right) \tag{6}$$

where $A_{c,i}$, $\sigma_{c,i}$, $\left(y_{c,i} - y_G \right)$ are the area, stress and distance between layer centroid and geometric cross-section baricentre for the concrete layer *i*, respectively. For the FRP bars the analogous terms are $A_{f,i}$, $\sigma_{f,i}$ and $\left(y_{f,i} - y_G \right)$.

The previous steps are repeated for a sufficient number of curvature χ_j in order to determine the overall moment-curvature diagram. The maximum of the moment-curvature diagram is the required ultimate bending moment $M_{u,t}$ of the cross section for the prescribed external axial force N_{ext} and the assigned fire exposure time *t*.

The failure of the FRP reinforced cross-section is assumed to occur when the ultimate strain is achieved at least in one material. The temperature-dependent ultimate strains of concrete is assumed according to EN1992-1-2 (2004), whereas the ultimate strain of the FRP is assumed in function of the reduction of the strength and tangent modulus (see par. 2).

The validation of the proposed incremental-iterative procedure, which has been performed with reference to both experimental and theoretical results available in the scientific literature, is omitted therein for sake of brevity (see Nigro et Al., 2007).

3.2 *Simplified method*

The proposed simplified method consists of calculating directly the ultimate bending moment $M_{u,t}$ of the cross-section through a procedure inspired to the well-known "isotherm 500°C method", also suggested by the EN1992-1-2 for concrete members reinforced with steel bars.

This method consists of calculating the ultimate bending moment of the section in flexure with the known distribution of temperature, by assuming a reduced section constituted of concrete with a lower temperature to 500°C, for which are assumed the main mechanical features at normal conditions, and from the FRP bars for which they consider the reduced strength according to reached temperature of the bars in the section.

In this way the calculation of the ultimate bending moment resistance can be done in a similar way to the usual calculation at normal conditions, determining the depth of the neutral axis y_c from the balance to the translation and by calculating the value of the ultimate bending moment in fire situation from balance to rotation. For rectangular section with single FRP reinforcement level, the two resolving equations become:

$$\psi \cdot b \cdot y_c \cdot f_{ck} - \rho_f(T) \cdot f_{fu} \cdot A_f = 0 \tag{7}$$

$$M_{Rd,fi,t} = \rho_f(T) \cdot f_{fu} \cdot A_f \cdot (d - \lambda \cdot y_c) \quad , \tag{8}$$

being $\rho_f(T)$ the bar's strength reduction coefficient depending on the temperature in the bar (see equations (1)); the bar temperature *T* can be evaluated from interpolated curves given in eq. (9) and figure 2, without any thermal analysis. In equations (7) and (8), the non-dimensional coefficients ψ and λ represent the resultant of the compression stresses and its distance from the extreme compression fiber, respectively, divided by $b \cdot y_c \cdot f_{ck}$ and by y_c, respectively.

4 APPLICATIONS TO FRP REINFORCED SLABS EXPOSED TO FIRE

The incremental-iterative procedure described in section 3.1, based on the FE solution of the thermal problem and on the assessment of the moment-curvature law of the cross-section in presence of elevated temperatures, is applied to FRP reinforced concrete slabs' check in fire conditions, pointing out the influence some parameters have on both thermal and mechanical behaviour of slabs exposed to ISO834 fire at the bottom, like the concrete cover *c*, the kinds of FRP bars (CFRP , GFRP , AFRP), the height of slab *h* and the mechanical percentage of reinforcing bars. In the thermal analysis the "superior limit" curve for concrete thermal conductivity is assumed (EN1992-1-2), in order to maximize the reached temperature values in FRP bars during the thermal transitory.

For example, for evaluating the influence of the concrete cover, a concrete slab with width of 1.0 m, thickness of 0.25 m and reinforced with 195.6 mm^2 of CFRP bars is considered in the analyses. The concrete is of class C30/37 (f_{ck} = 30 N/mm^2), the CFRP bars have strength f_{fu} = 2068 N/mm^2 and elastic modulus E_f = 124 GPa.

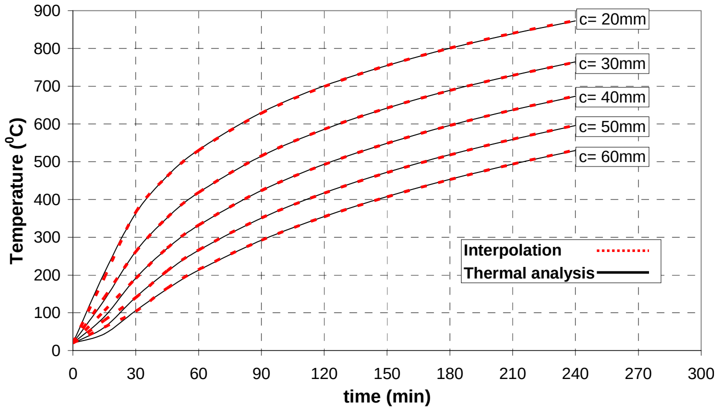
Thermal analysis of reinforced slabs, exposed to ISO834 fire, gives the temperature in the FRP bars during the fire exposure. The curves $T(t,c)$ in the bars can be seen in Figure 2 as a function of fire exposure time *t* and for different values of the cover *c*. The slab's total height *h* instead does not appear as an influent parameter on heat diffusion between the invested surface by fire and the bars' position which basically depends on cover *c*.

The time-temperature curves are similar to the ones of the Canadian Guidelines, but they're obtained by referring to the standard fire curve ISO834 and to the concrete's thermal properties suggested by Eurocodes. They can be interpolated through the following expressions (see Figure 2):

$$t \leq 30 \quad \text{min}: \qquad T(t,c) = A_1(c) \cdot t + 20$$
$$t \geq 30 \quad \text{min}: \qquad T(t,c) = A_2(c) + A_3(c) \cdot t^{A_4(c)} \tag{9}$$

As the coefficients $A_i(c)$ are function of the cover c, like it's shown in Figure 2, the interpolated curves are basically coincident with the ones deriving from the reference thermal analysis.

c [mm]	A_1	A_2	A_3	A_4
20	11,538	-4586,1	4221,2	0,0470
30	8,032	-2326,8	1935,7	0,0854
40	5,685	-892,3	592,2	0,1774
50	3,997	-509,4	271,7	0,2561
60	2,792	-312,0	130,8	0,3400



a) b)

Figure 2 – a) Interpolated curves coefficients (9); b) Bar-temperature Vs Exposure-time curves in function of concrete cover.

There can be a double use of the curves $T(t,c)$. Once the bar's *critical temperature* has been defined, as for example utilising the material strength reduction curves $\rho_f(T)$ defined in section 2, diagrams of Figure 2 or relations (9) can get an estimation of necessary time for the bar to reach critical temperature. Moreover, they're useful for the application of the simplified method described in section 3.2, allowing the estimation of the reduction coefficient $\rho_f(T)$ of the material strength to use in the relations' application (7) and (8). In both cases we can notice the advantage of avoiding the thermal field's calculation in the slab.

As a further application, the ***ultimate bending moment resistance of slabs reinforced with FRP*** and exposed to fire at the bottom is evaluated both with the incremental-iterative procedure, then according to the simplified analytic method described in par. 3.2. In particular, they're represented in Figure 3, as a function of exposure time, in terms of curves of the adimensional parameter

$$\rho_M(t) = \frac{M_u(t)}{M_u(t_0)} = \frac{M_{Rd,fi,t}}{M_u(t_0)} \tag{10}$$

defined as the ratio between the ultimate bending moment $M_u(t) = M_{Rd,fi,t}$ at fire exposure time t and the ultimate bending moment $M_u(t_0)$ at the beginning of the thermal transitory (evaluated according to unitary partial safety factors). Each curve refers to a fixed value of concrete cover.

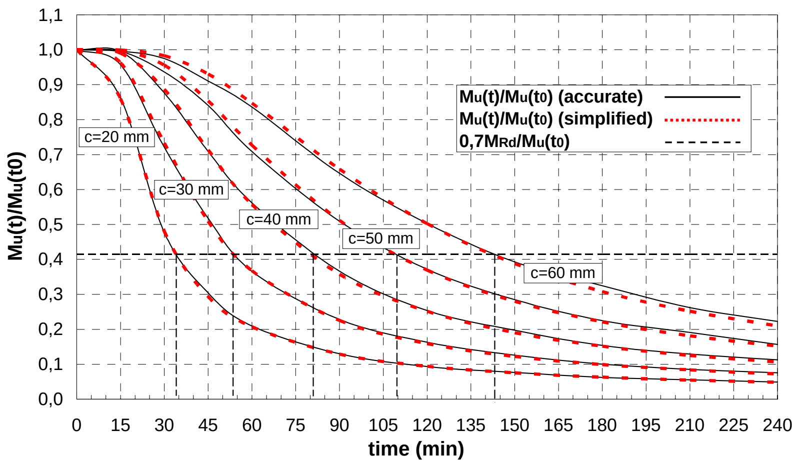


Figure 3 – Resistant bending moment reduction curves (slabs reinforced with CFRP bars)

First it is possible to notice the almost perfect coincidence between the simplified method and the accurate one. Therefore, the simplified method, sets completely free the estimation of the slabs bending moment resistance in fire conditions, thanks to the calculation of thermal field in the slab using relationships (9), in all cases where the compress zone of the section presents lower temperatures to 500°C.

In Figure 3 it is also represented the horizontal straight line $M_{Sd,fi}/M_u(t_0)$, which corresponds to the required flexural resistance during fire exposition, being $M_{Sd,fi} = \eta_{fi} \cdot M_{Rd}$ the design bending moment in the fire situation (the load level for fire design η_{fi} is assumed as 0.7) and posing $M_{Sd} \equiv M_{Rd}$, as M_{Rd} is the flexural resistance for normal temperature design (calculated using the relevant partial safety factors). The straight line's intersection with the adimensional resistance curves, puts in evidence the big influence of bars cover c on maximum time exposure to fire, which goes from about 35 min to 145 min with c variable from 20 to 60 mm.

5 CONCLUSIONS

The analysis of bibliography experimental results about the behaviour of composite materials and concrete slabs reinforced with FRP bars in case of high temperatures, has stressed both the necessity of a specific thermo-mechanical characterization of materials, even considering the differences we can find between the diverse commercial FRP kinds, and it has also allowed to check the reliability of an accurate calculation procedure oriented towards concrete reinforced members in flexure with FRP bars in fire situation. The thermo-mechanical analysis led by referring to concrete slabs reinforced with FRP bars exposed to fire at bottom and by using resistance reduction curves of characteristic bars of homogeneous classes of materials, allowed to deduce diagrams which provide the temperature in the bars according to variation of time exposure to ISO834 fire and of the cover, useful for the slab's check in the temperature domain (critical temperature method). The comparison with the accurate reference procedure gave us the chance to prove the applicability to reinforced slabs with FRP bars of a simplified analytical method inspired to the well-known isotherm 500°C method.

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