A PROPOSAL OF GUIDELINES FOR BENDING RESISTANCE OF FRP RC MEMBERS IN FIRE SITUATION

Emidio NIGRO
Associate Professor
DIST – Structural Engineering, University of Naples Federico II, Italy
emidio.nigro@unina.it

Giuseppe CEFARELLI
PhD
DIST – Structural Engineering, University of Naples Federico II, Italy
giuseppe.cefarelli@unina.it

Antonio BILOTTA
PhD
DIST – Structural Engineering, University of Naples Federico II, Italy
antonio.bilotta@unina.it

Gaetano MANFREDI
Full Professor
DIST – Structural Engineering, University of Naples Federico II, Italy
gamanfre@unina.it

Edoardo COSENZA
Full Professor
DIST – Structural Engineering, University of Naples Federico II, Italy
cosenza@unina.it

Abstract
Several building codes are available for the design of concrete structures reinforced with FRP; nevertheless knowledge on their structural behavior in case of fire is lack and no reliable design method is available.

The experimental data available in the literature about the behavior of fiber-reinforced composite materials as well as of concrete slabs reinforced with FRP bars or grids at high temperatures have been carefully examined, determining deterioration curves of bars mechanical properties as a function of the temperature.

On the other hand, a calculation incremental-iterative procedure, developed by the authors on the FE solution of the thermal problem and on the assessment of the moment-curvature law of the cross-section at high temperatures, allows the bearing capacity of the FRP reinforced concrete slabs under fire conditions to be assessed. The procedure has been validated on experimental results of fire tests on simply-supported concrete slabs reinforced with FRP bars carried out by the authors. The incremental-iterative procedure allowed the calibration of a simplified analytical method inspired to the well-known “isotherm 500°C method”, also suggested by the EN1992-1-2 for concrete members reinforced with steel bars. A proposal of simplified design method for bending resistance of FRP reinforced concrete members in fire situation is finally suggested.

Keywords: Fire, FRP, codes, flexure.

1. Introduction
Critical aspects for steel reinforced concrete (RC) members of some buildings and many bridges are durability and maintenance. Indeed, steel bars suffer the phenomenon of corrosion that can reduce structural capacity and capabilities of the structure. A significant contribution to overcome this
problem was provided by civil engineering industry with development of composite materials such as Fiber Reinforced Polymer. Due to high strength to weight ratio, these materials are often used for external strengthening: sheets and plates for External Bonded Reinforcement (EBR) technique and strips and bars for Near Surface Mounted (NSM) technique [1]. However, also FRP round bars for internal reinforcement are often preferred to traditional steel reinforcement [2] to avoid the spalling phenomenon due to corrosion [3] or to provide magnetic transparency (e.g. for structures that host advanced diagnostic equipment).

From a technical point of view, the vulnerability of organic polymers at high temperatures is probably the biggest drawback for the FRP bars. For this reason many examples of FRP-RC structures are available worldwide, but they are often structures for which fire is not a significant design condition. Despite fire is an event that can not be ignored for many civil structures as well as parking lots and industrial structures, many international codes provide suggestions for design of concrete structures reinforced with (FRP) bars in place of traditional steel reinforcement [2],[4],[5],[6] but few provisions and no calculation model are suggested that take account of fire conditions guidelines (for example [6] suggests not using FRP reinforcement in cases where fire resistance is an essential requirement). This lack in codes clearly reflects poor confidence in the use of FRP-RC members that can be improved only if a clear distinction between FRP and FRP-RC members is done. In fact, high temperatures result in a deterioration of the mechanical properties of FRP bar (both strength and Young’s modulus, [7]) as well as bond properties [8]. In this sense, the glass transition temperature, $T_g$, at which the resin of FRP shows a damage level quite high [9], represents a key parameter. Nevertheless, efforts to increase the value of $T_g$ (now between 80°C to 200°C) may also be useless for these applications if increases will not very relevant (more than 500°C) and thus compatible with temperatures reached during a fire. On the other hand, it is worth noting that deterioration of mechanical and bond properties depends on fibre type [7], surface treatment type [8] and, above all, the temperature attained in FRP bars, that can be strongly reduced when bars are embedded in concrete due to its low thermal conductivity. Therefore, performance of FRP-RC members in fire condition can be very better than FRP members even if no particular fire protection system is used (e.g. passive protection systems, consisting of normal coatings or special insulating materials).

On this point, the literature provides a broad state of the art on the behavior in fire events of concrete structures reinforced or strengthened with FRP [10] and some results of flexure tests performed on FRP-RC members exposed to conventional fire conditions [11],[12],[13],[14],[15],[16]. In particular, [16] showed that higher fire resistance for FRP-RC slabs can be obtained by using larger concrete cover thickness and through the use of carbonate aggregate concrete. Moreover, they pointed out that it is necessary to consider the effects of two important factors on the fire endurance of RC slabs, namely the applied load and the reinforcement bond degradation. [15] pointed out the importance of distinguishing between temperature at which deterioration of the bond strength is attained and that at which decrease of the tensile strength in the bars is attained. Indeed, bond test results showed that bond strength between FRP and concrete decreases substantially when the glass transition temperature ($T_g$ ≥ 180°C) was attained. Moreover, the results of tensile tests at 400°C - 500°C showed a bar strength reduction ranging between 30% and 80%. Finally, a full-scale test, performed on a concrete slab reinforced with the same GFRP bars, highlighted that failure after 90 minutes of fire exposure was not attained due to the rupture of the bars. Indeed, the longitudinal bars were all lap spliced in the middle of the slab and failure was attained due to the loss of bonding between bars and concrete in the midspan of the slab. Hence a premature failure of the concrete member was manifested when FRP reached the glass transition temperature, $T_g$, due to the use of overlapping bars. Indeed, the bars’ temperature typically attains such high values that loss of adhesion at the concrete-FRP interface may rapidly occur and then cause the swift failure of the structure.

On the basis of these results, authors recently performed fire tests to evaluate resistance and
deformability of FRP-RC slabs in fire situations (see [17], [18] and [19]). Failure of slabs was attained also by the rupture of the fibers in the middle of the member because continuous reinforcement from side to side of the concrete element was used, contrary to what was observed in [15]. In addition, the results showed that the specimens failed due to the rupture of the fibers in the middle of the member only when large zones near the supports were not directly exposed to fire; this allowed suitable anchorage of bars at the ends to be ensured once the glass transition temperature is reached in the fire-exposed zone of the slab and resin softening reduces adhesion at the FRP-concrete interface (see [17] and [18]). Moreover, in [19] it was shown that a large length of this zone not directly exposed to fire required for anchoring straight bars can be reduced if the bars are bent at the end. Therefore if bar anchorage allows pull-out of bars to be avoided, slab failure is attained due to very high temperatures, that is, much higher than the glass transition temperature $T_g$, and the failure mechanism is shifted from the loss of bonding to the limit strength of the bars. Clearly, as stated above, bar temperatures largely depend on the concrete cover whereas fire endurance depends on fiber strength at high temperatures and hence may depend on fiber type. Although further theoretical and experimental studies are required, the use of FRP as internal reinforcement of concrete members seems possible also for constructions in which safety in case of fire is a key requirement.

Among the standards listed above, Canadian one [5] provides specific design requirements with regard to the effects of high temperatures on concrete slabs reinforced with FRP bars. A series of nomograms are provided to estimate the concrete cover necessary to maintain the temperature in the FRP bars within an acceptable limit defined as “critical temperature” (i.e. the temperature of FRP reinforcement at which is expected slab collapsing under service loads in fire conditions). However, the method can be applied if the critical temperature of the member is known.

In order to identify main parameters playing a role in the behavior of FRP-RC member and to suggest simplified methods of design and check as well as technical suggestions, useful for the development of guidelines on the subject, theoretical, numerical and experimental results of the recent studies carried out by the authors are summarized below. On the basis of these results, a proposal of simplified design method for bending resistance of FRP RC members in fire situation is finally suggested.

2. Suggestions for guidelines

2.1 Fire resistance check

Resistance check in fire situations is expressed by the following well-known relationship [20]:

$$E_{fi,d,t} \leq R_{fi,d,t}$$  \hspace{1cm} (1)

in which $E_{fi,d,t}$ and $R_{fi,d,t}$ are the design effect and capacity in fire situation at time $t$ of fire exposure, respectively. The effect is calculated by combining the mechanical actions in accordance with Eurocode 0 [21] for accidental design situations. The capacity is calculated with partial safety factors $\gamma_M=1.0$. The effect of high temperatures on strength and stiffness of concrete can be assessed according to Eurocode 2 [20]. As concern FRP, properties at high temperature should be provided by the manufacturer.

2.2 Thermal analysis

Development and distribution of the temperature within structural elements exposed to fire (namely the thermal field) are evaluated by separating the thermal and mechanical response models according to European design code [22] for steel RC structures. Thermal field in FRP-RC slab should be assessed by solving heat propagation problem due to radiation and convection from the combustion gases to the exterior surface of the member; the presence of protective materials must to be considered. Concrete thermal properties should be assessed
according to Eurocode 2 [22] whereas the presence of FRP bars can be neglected in the thermal model due to their modest size [18].

From a design point of view, the designer can be facilitated through the use of simplified methods for the assessment of temperatures without having to carry out a detailed numerical analysis. An example of a simplified procedure is suggested in the Canadian Guidelines [5] (see also [25]). Inspired by the method proposed in Canadian standards [5], Nigro et al. ([7] and [18]) proposed simplified methods for the assessment of the temperature of bars embedded in slabs exposed to fire from beneath. The time-temperature curves are similar to those of the Canadian Guidelines, but are obtained by referring to the standard temperature-time curve of ISO834. Moreover, concrete thermal properties suggested by European codes were assumed because they provide safe predictions, as also shown in the previous section. In particular, the upper limit curve for concrete thermal conductivity [22] was assumed in order to maximize the temperature values attained in the FRP bars during fire exposure. The presence of the bar was neglected in the simplified method because it refers to the average temperature in the bar that is only slightly affected by the lower FRP conductivity than concrete, as stated above. For further details refer to [7], [18] and [26]. The FRP bar temperature curves $T(t,c)$ have the following expressions (1):

$$
t \leq 30 \text{ min} : \quad T(t,c) = A_1(c) \times t + 20 \quad \text{[°C]}
$$

$$
t \geq 30 \text{ min} : \quad T(t,c) = A_2(c) + A_3(c) \times t^{A_4(c)} \quad \text{[°C]}
$$

The temperatures are dependent on fire exposure time $t$ and concrete cover $c$, whereas the total height $h$ of the slab does not appear to be influential upon heat diffusion between the surface exposed to fire and the bars. The coefficients $A_i(c)$ that take into account parameter $c$ are reported in Table 1. The relationships (2) provide the curves represented in Figure 1.

**Table 1 - Coefficients Ai(c)**

<table>
<thead>
<tr>
<th>$c$ [mm]</th>
<th>$A_1$</th>
<th>$A_2$</th>
<th>$A_3$</th>
<th>$A_4$</th>
</tr>
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<tr>
<td>20</td>
<td>11.538</td>
<td>-4586.1</td>
<td>4221.2</td>
<td>0.0470</td>
</tr>
<tr>
<td>30</td>
<td>8.032</td>
<td>-2326.8</td>
<td>1935.7</td>
<td>0.0854</td>
</tr>
<tr>
<td>40</td>
<td>5.685</td>
<td>-892.3</td>
<td>592.2</td>
<td>0.1774</td>
</tr>
<tr>
<td>50</td>
<td>3.997</td>
<td>509.4</td>
<td>271.7</td>
<td>0.2561</td>
</tr>
<tr>
<td>60</td>
<td>2.792</td>
<td>-312.0</td>
<td>130.8</td>
<td>0.3400</td>
</tr>
</tbody>
</table>

**Figure 1 - Temperature vs. time as a function of concrete cover (simplified method)**

### 2.3 Mechanical analysis for bending moment resistance

The strains, and hence stresses, in bars of slabs can be estimated according to an incremental-iterative procedure [7], based on the following hypotheses:
- planarity of cross-section up to failure, so that strain profile is linear also during the fire exposure;
- perfect bond between concrete and FRP bars;
- concrete constitutive law according to suggestion provided by Eurocode 2 at high temperatures;
- no concrete tensile strength;
- FRP constitutive law linear elastic up to failure, also at high temperatures;
- no FRP compressive strength;

Under this hypotheses, the total strain of the generic layer of the cross section, $\varepsilon_{tot}$, is equal to (see [22], [27], [28]):

$$\varepsilon_{tot} = \varepsilon_T(T) + \varepsilon_\sigma(\sigma, T)$$

(3)

where:
- $\varepsilon_{tot}$ is the strain in FRP bar at time of fire exposure $t$.
- $\varepsilon_T(T)$ is the thermal strain of the generic layer (it corresponds to the thermal elongation $L/L$ depending on layer temperature $T$);
- $\varepsilon_\sigma(\sigma, T)$ is the mechanical strain of the generic layer (stress-related through the $(\sigma-\varepsilon; T)$ law).

Since:

$$\varepsilon_{tot} = \varepsilon_{med} + \chi \cdot y$$

(4)

with $\varepsilon_{med}$ and $\chi$ average strain and cross-section curvature respectively, the mechanical strain can be obtained as (see Figure 2):

$$\varepsilon_\sigma = \varepsilon_{tot} - \varepsilon_T = (\varepsilon_{med} + \chi \cdot y) - \varepsilon_T$$

(5)

During fire exposure, the reduction of adhesion in the exposed area of the member may have a limited influence on the load bearing capacity when FRP reinforcing bars are well anchored: this can be ensured if a sufficient continuity of FRP bars is provided and high temperatures are not reached at the end of FRP (see [19]) in case of a single bay or when the fire is confined in a single bay of a multiple-bay concrete slab structure and thus bars are well anchored in adjacent bays of the structure [29].

![Discretized cross-section](image)

**Figure 2 - Total, thermal and mechanical strains curves and stresses curve**

Nevertheless, the reduction of adhesion has potentially serious implications on the flexural deformability of a concrete slab, since unbonded reinforcement could lead to the development of fewer, larger flexural cracks. Moreover, such cracks could result in localized heating of the FRP bars, which could lead to premature tensile rupture of the bars. Finally note that at high temperatures the decay of adhesion for FRP bars is more significant than experienced by the bars of steel and it is also not well known, as it depends on the characteristics of different types of bars. However, this effect is very difficult to model and it is not taken into account in the procedure.
2.4 Iterative procedure

The proposed method is valid only for the assessment of the bending moment resistance of concrete slabs subjected to fire from the side of the fibers in tension. It fits within the simplified methods used in the analysis under fire conditions conducted for single elements. The method is in compliance to the well-known “isotherm 500°C method” suggested by EN1992-1-2 [22] for concrete members reinforced with steel bars.

Bending moment resistance depends on the type of failure: attainment of maximum tensile strain in FRP (Zone 1 of Figure 4) or maximum compressive strain in concrete, \( \varepsilon_{cu}^* \) (Zone 2 of Figure 4). In fire situation maximum compressive strain in concrete, \( \varepsilon_{cu}^* = 0.01 \) can be assumed [7]. In particular, in Zone 1 (Figure 4) rupture is achieved for attainment of limit strain in FRP, whereas strain in concrete is lower than limit strain, \( \varepsilon_{cu}^* \). Hence, for this failure mode, strain profile is characterized by a fixed point, \( \varepsilon_{tot,f} \), at centroid of FRP reinforcement. The strain \( \varepsilon_{tot,f} \) can be calculated as:

\[
\varepsilon_{tot,f} = \varepsilon_{fu,T} + \varepsilon_{T,f}
\]

where \( \varepsilon_{fu,T} \) and \( \varepsilon_{T,f} \) are reinforcement ultimate strain and thermal strain respectively, both assessed with reference to the bar temperature.

If the mechanical behavior of FRP bars at high temperatures is assumed linear elastic up to failure, the ultimate strain, \( \varepsilon_{fu,T} \), can be calculated through the following relationship:

\[
\varepsilon_{fu,T} = \frac{f_{fu,T}}{E_{f}} = \frac{\rho_f(T) \cdot f_{fu}}{\rho_E(T) \cdot E_f} \cdot \varepsilon_{fu,T=20^\circ C}
\]

being \( \rho_f(T) \) and \( \rho_E(T) \) two factors that take into account the decrease of FRP strength and Young’s modulus due to temperature \( T \).

“Average” behavioural deterioration curves \( \rho_f(T) \) and \( \rho_E(T) \) for different kinds of FRP bars (CFRP, GFRP, AFRP) were obtained by the authors ([7], [26]) based on available experimental data.

\[
\begin{align*}
\rho_f(T) &= \frac{f_{fu}(T)}{f_{fu}} = \frac{0.05}{0.05+8.0 \times 10^{-11} \cdot T^{0.35}} \quad \text{for GFRP bars} \\
\rho_f(T) &= \frac{f_{fu}(T)}{f_{fu}} = \frac{0.06}{0.06+2.0 \times 10^{-10} \cdot T^{0.35}} \quad \text{for CFRP bars} \\
\rho_E(T) &= \frac{E_f(T)}{E_f} = \frac{0.28}{2.4+9.0 \times 10^{-12} \cdot T^{4.4}} \quad \text{for GFRP bars} \\
\rho_E(T) &= \frac{E_f(T)}{E_f} = \frac{2.4}{2.4+9.0 \times 10^{-12} \cdot T^{4.4}} \quad \text{for CFRP bars}
\end{align*}
\]

The formulations of \( \rho_f(T) \) and \( \rho_E(T) \) for GFRP bars are reported in Figure 3. Moreover, such curves are compared with similar curves provided by [10]; the curves are in good agreement. As linear strain profile is assumed, the maximum compressive strain in concrete can be calculated as:

\[
\varepsilon_c = (\varepsilon_{fu,T} + \varepsilon_{T,f}) \cdot \frac{y_c}{d-y_c} \leq \varepsilon_{cu}^*
\]

Similarly in Zone 2 (Figure 4) rupture is achieved for attainment of limit strain in concrete whereas the FRP strain is lower than limit strain and the following relationships can be written:

\[
\varepsilon_c = \varepsilon_{cu}^*
\]

\[
\varepsilon_{tot,f} = \varepsilon_{cu}^* \cdot \frac{d-y_c}{y_c} \leq (\varepsilon_{fu,T} + \varepsilon_{T,f})
\]
For both types of failure (Zone 1 and Zone 2) the distance of the neutral axis from the concrete edge in compression, $y_c$, can be calculated through the following equilibrium:

$$N_c - N_f = 0$$  \hspace{1cm} (13)

being $N_c$ and $N_f$ compressive and tensile resultants.

![Figure 3 - Average behavioural deterioration relationships: $\rho f(T)$ and $\rho E(T)$](image)

**Figure 3** - Average behavioural deterioration relationships: $\rho f(T)$ and $\rho E(T)$

The bending resistance, $M_{Rd,f,t}$, at time $t$ of fire exposure can be calculated through rotational equilibrium. Similarly to the case of normal temperature, bending moment-curvature law ($M$-$\chi$; $N_{ext}$) of the critical cross-section for the imposed value of the axial force $N_{ext}$ and the current temperature field within the section [27] can be assessed. The numerical procedure to assess the moment-curvature law of the cross-section is iterative: more details can be found in [7] and [26]. The maximum of the moment-curvature diagram is $M_{Rd,f,t}$, namely the bending moment resistance of the cross section corresponding to the assigned external axial force, $N_{ext}$, and the exposure time, $t$. The failure of the FRP reinforced cross-section is assumed to occur when the ultimate strain is attained in either FRP bars or concrete. The bending moment at failure and the maximum moment could not be equal if at least one of the constitutive laws of the materials is characterized by a softening branch.

![Figure 4 – Strains and stresses at Ultimate Limit State (ULS)](image)

**Figure 4** – Strains and stresses at Ultimate Limit State (ULS)
2.5 Simplified method

The incremental-iterative procedure above can be easily implemented in a calculation program; by contrast, it is not immediate for a manual computation. Moreover, during the fire exposure, the axial load, $N_{ext}$, may change its value due to the restraint action. By adopting a member analysis according to [22], a value of $N_{ext}$ equals to zero can be assumed for a slab element. Therefore, a simplified method able to assess the bending moment resistance of unprotected FRP-RC slabs exposed to fire on the side of fibers under tension, $M_{Rd,fi,t}$, can be used.

Note that this approach neglect the possible membrane effect as well as the effects of bending moment redistribution in continuous multi-span structures, which typically allow more resistant mechanisms to be developed and the fire resistance to be increased.

As the temperatures in the concrete in compression are very low due to its high thermal conductivity the concrete constitutive law at 20°C (see Figure 5) can be used. Moreover the following non-dimensional coefficients $\psi$ and $\lambda$, representing the compressive stress resultant in concrete and its distance from the top edge, divided to $b \cdot y_c \cdot f_{ck}$ and $y_c$ respectively, can be defined:

$$\psi = \frac{y_c}{0} \int \sigma(y)dy \quad \frac{y_c}{y_c \cdot f_{ck}}$$

$$\lambda = \frac{y_c}{0} \int \sigma(y) \cdot (y_c - y)dy \quad \frac{y_c}{y_c \cdot \int \sigma(y)dy}$$

For rupture in Zone 1 (FRP failure) translation equilibrium can be written:

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

valid for $\varepsilon_c = (\varepsilon_{fu,T} + \varepsilon_{T,f}) \cdot \frac{y_c}{d - y_c} \leq \varepsilon_{cu}$

whereas rotation equilibrium can be written:

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

For rupture in Zone 2 (concrete failure) translation equilibrium can be written:

$$\overline{\psi} \cdot b \cdot y_c \cdot f_{ck} - \rho_E (T) \cdot E_f \cdot A_f \cdot \left( \varepsilon_{cu} \cdot \frac{d - y_c}{y_c} - \varepsilon_{T,f} \right) = 0$$

valid for $\varepsilon_{tot,f} = \varepsilon_{cu} \cdot \frac{d - y_c}{y_c} \leq (\varepsilon_{fu,T} + \varepsilon_{T,f})$

whereas rotation equilibrium can be written:

$$M_{Rd,fi,t} = \rho_E (T) \cdot E_f \cdot A_f \cdot \left( \varepsilon_{cu} \cdot \frac{d - y_c}{y_c} - \varepsilon_{T,f} \right) \cdot (d - \overline{\lambda} \cdot y_c)$$

Dimensional parameters $\psi \in \lambda$ clearly depends on the maximum concrete strain $\varepsilon_{cu}$. In both cases (FRP or concrete rupture) the following parameters can be assumed in eqns. (16) to (21) (see [7]):

$$\varepsilon_{cu} = 0.01 \quad \psi (e) = \overline{\psi} = 0.75 \quad \lambda (e) = \overline{\lambda} = 0.5$$

(22)
2.6 Anchoring length

In order to attain a full flexural strength of the FRP-RC member an adequate anchorage of bars is necessary, as well as suggested in [6]. In this regard, in [30] full scale test results, extensively presented in [17], [18] and [19] were used to investigate the bond behavior of FRP bars embedded in concrete at high temperature and assess a procedure to predict bond stress, slip, and load transfer at elevated temperature, based on both the results of numerical thermal analysis and the predictions of a bond theoretical model adjusted for fire situations. The design procedure outlined for calculating the minimal required anchoring length proves a valuable approach for the practicing engineer and stands together with the experimental and numerical results. Design nomograms were also shown as examples of application of the procedure, even if the results clearly depend on the properties of the FRP bars (i.e. fiber type, Young’s modulus, tensile strength, diameter, surface treatment) and the strength of the concrete. However, they can be extended to different types of FRP bars as long as the manufacturer provides the adhesion properties of the bars, at least under ambient temperature conditions.

The constitutive shear stress-slip law for FRP bars to concrete interface identified by [31] can be used. The law is characterized by:

- an ascending branch \( \tau_b(s) = \tau_m \left( \frac{s}{s_m} \right) ^\alpha \) for slip \( s \leq s_m \),

and

- a softening branch \( \tau_b(s) = \tau_m \left( 1 + p - p \frac{s}{s_m} \right) \) for slip \( s > s_m \).

Therefore, the “double branch” constitutive law is completely defined when the values of the maximum shear stress, \( \tau_m \), and the corresponding slip, \( s_m \), as well as coefficients \( \alpha \) and \( p \), are predetermined. These parameters were able to be determined by using an identification procedure, minimizing the scatter between theoretical predictions and experimental results of pull-out tests.

In order to take into account the effect of high temperature on the bond between FRP and concrete the expression for the normalized bond strength provided [8], taking into account the effect of the temperature, \( T \), and the bar properties (i.e. normalized residual bond strength, \( \tau_r \), degree of cross linking, \( C_r \), and glass transition temperature of the polymer at the bar surface, \( T_g \)), can be used:
\[
\tau^*(T) = 0.5 \cdot (1 - \tau^*_r) \cdot \tanh \left( -\frac{0.02}{C_r} \left[ T - \left( T_g + \frac{k_i}{0.02} C_r \right) \right] \right) + 0.5 \cdot (1 + \tau^*_r)
\]  

(25)

The normalized residual bond strength, \(\tau^*_r\), can be completely neglected for a temperature higher than 250°C, even if a little residual strength (i.e. \(\tau^*_r = 0.10\)) is provided by the original model by [8]. This assumption was made because the model is calibrated on experimental results of tests performed in the range 0-250°C. Hence for higher temperatures the model’s reliability has not yet been validated.

Therefore the differential equation that governs the bond problem of FRP bars can be expressed as:

\[
\frac{d^2 s(z)}{dz^2} - \frac{4}{E(T(z))} \tau^*(s(z), T(z)) = 0
\]

(26)

where \(E\) and \(\phi\) are respectively the Young modulus and diameter of FRP bars, and \(\tau^*\) is a function of the bar temperature via Eqn. (25). The dependence of Young’s modulus on temperature \(E(T)\) can be taken into account with the relationship (9). Note that creep of the polymer resin at elevated temperatures could play an important role for the bond of FRP bars under sustained load. This phenomenon is only partially taken into account by adopting the relationship (9) calibrated on experimental tests probably performed in a short time.

In order to solve Eq. (26) and then to determine the development length of FRP bars, \(l_{d,fi,t}\), in fire conditions at time, \(t\), an iterative finite differences procedure can be used and the theoretical minimum anchoring length (namely development length), \(L_b\), in zones which are not directly exposed to fire but which are necessary to transfer the forces between FRP bars and concrete, can be determined [30].

Values of parameters \(\tau_m, s_m, a,\) and \(p\) defining the bond stress-slip law according to the procedure in [31] as well as the values of strength and Young modulus according to [24] should be determined. The glass transition temperature of the polymer at the bar surface, \(T_g\), obtained for example through a DMA test, is also required. Strength and Young modulus laws against temperature, as well as the normalized residual bond strength, \(\tau^*_r\), and degree of cross linking, \(C_r\), can be defined by using data from the literature. Of course, more reliable results can be obtained if these values are experimentally identified, instead of using data from the literature.

3. Conclusions

A significant contribution to overcome problems of durability and maintenance of the structures was provided by civil engineering industry with development of composite materials such as Fiber Reinforced Polymer (FRP). The susceptibility of polymers at high temperatures is probably the biggest drawback for the FRP bars. For this reason many FRP-RC structures are often realized only when fire is not a significant design condition, also because few specific design requirements with regard to the effects of high temperatures on concrete slabs reinforced with FRP bars are provided by international codes.

In the framework of fire safety engineering the authors recently carried out an extensive research on fire resistance of concrete slabs reinforced with FRP bars or grids in the absence of specific protective materials (e.g. passive protection systems such as normal plaster or special insulation materials). Research key aspects were recalled with reference to both experimental work carried out on nine full-scale structural members and theoretical analysis conducted for the development of interpretative mechanical models and simplified methods for the evaluation of the bearing capacity in fire situations. Even if some simplifications were
assumed respect to more advanced check procedures developing in fire safety engineering, experimental and theoretical results comply to the fire analysis conducted for single elements, currently available in the major design codes.

Based on these results, technical suggestions for the drafting of guidelines regarding the design of concrete members reinforced with FRP bars subject to fire have been provided. Further research developments will be finally devoted to (a) evaluation of effects of structural continuity, such as membrane effect and possible bending moment redistribution, and (b) shear strength of concrete slabs reinforced with FRP bars.

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5. References


