Computational study on prestressed concrete members exposed to natural fires

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The present paper is aimed at investigating the structural behaviour in bending of prestressed concrete members exposed to natural fires, i.e. fires with a heating and a cooling phase. The fire scenarios considered are charac-terized by a heating phase that coincides with the ISO834 standard fire and a linear cooling branch. To accurately track the structural behaviour, the usual constitutive models for concrete, ordinary and prestressing steel at high temperature are adapted to account for the different behaviour of the materials upon unloading and cooling. Parametric analyses are carried out on typical prestressed sections (an I-girder and a double-tee), in order to highlight the detrimental effect of longer fire durations (up to 120 min) and lower cooling rates (3 °C/min) as well as the variability of the structural behaviour with the variation of the load level.

The results show that in members characterized by massive sections (I-girder in the present study) and exposed to natural fires, limiting the attention to the heating phase is not sufficient, as the maximum temperature in the prestressing steel may be reached long (even hours) after the onset of cooling (in accordance with tests reported in the literature), leading to delayed failure. Moreover, within the range of variation of the cooling rate (3–10 °C/min, ranging from slow to fast cooling) and load level (M/M_u = 0.15–0.30, ranging from low to high load ratio), the structural behaviour exhibits significant variations in the cooling phase of the fire, from an almost complete recovery of the initial configuration to runaway failure.

Keywords:
Prestressed concrete
Natural fires
Cooling
Unloading

1. Introduction

Prestressed concrete members are widely used in a variety of structural applications. In comparison to ordinary reinforced concrete members, they offer several advantages, such as higher speed of construction and larger span-to-depth ratios [1]. The use of prestressing allows improving the mechanical performance of reinforced concrete members, by limiting cracking phenomena, and thus increasing the stiffness. Optimization of structural behaviour, which is typical of the prefabrication industry where prestressing is widely used, makes it also possible to reduce the overall dimensions of the members: in fact, several types of prestressed concrete members available on the market are characterized by thin webs.

In comparison to reinforced concrete members, prestressed concrete members are more sensitive to fire [2]. In detail: (a) cold worked prestressing steel is more sensitive to high temperatures than ordinary hot rolled steel; (b) the reduced thickness that characterizes prestressed concrete members results in lower steel protection: this is particularly true in structures that were built before the publication of the current standards, where the typical prescriptions concerning concrete cover are not respected; and (c) the lack of connectivity between the structural elements (which is directly related to construction time savings) results in statically-determinate structures, where the redistribution of the internal forces is not possible, to the detriment of the structure's global stability. Finally, prestressed concrete members in demanding structures (such as industrial facilities) are subjected to a higher risk of fire exposure, because of the higher availability of combustible materials. As previously mentioned, this is particularly dangerous for several existing structures, which do not comply with current code provisions as regards concrete cover.

The first systematic researches on the fire resistance of prestressed concrete members were reported by Troxell [3], who compared fire resistance tests conducted in Europe and in the United States. A basic finding was that prestressed members could resist fires lasting up to 4 h, by providing a suitable concrete cover. Moreover, the beneficial effects of the continuity over intermediate supports and of axial restraints were pointed out. Finally, no major differences were found between pre- and post-tensioned members (with bonded tendons). Another systematic

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research on the behaviour of prestressed concrete members exposed to fire was carried out by Gustaferro [4], who studied the behaviour of prestressed concrete members exposed to standard fires. Once again, the key role played by cover and restraint conditions was pointed out. Moreover, for continuous (and thus redundant) structures, the importance of providing negative reinforcement over the supports to allow moment redistributions was highlighted. A summary of the fundamentals concerning precast prestressed members can be found in Ref. [5]: as regards simply-supported members without end rotational restraints, a simple and straightforward method to determine the structural end point (and thus the fire resistance) is devised, which consists in comparing the applied moment with the moment capacity in the most stressed section (midspan).

In recent years, post-tensioned flat slabs with unbonded tendons and prestressed hollow core slabs have been the most investigated prestressed members among those available on the market. The reason for this is to be found in the high sensitivity to fire of the former (especially if there is no ordinary reinforcement to provide additional flexural capacity), and in the shear-deficient behaviour of the latter (since no transverse reinforcement is present).

One of the most significant studies on bonded and unbonded post-tensioned concrete slabs was carried out by Ellobody and Bailey [6], who investigated the influence on the structural behaviour of several parameters, such as aggregate type, duct material and restraint conditions. The results showed that the presence of boundary restraints and the use of aggregates without silica are beneficial. As for the duct material, slabs with metallic ducts exhibited larger deflections than those with plastic ducts.

Regarding hollow-core slabs, they have been the object of several studies in recent years [7–9]. The results show that the thermal and structural response can be adequately simulated by means of numerical models (despite some peculiarities, such as the presence of voids with the ensuing problems related to heat transfer), and that factors such as load level, concrete cover and hole size considerably affect fire resistance. Moreover, the failure of hollow-core slabs is primarily governed by the fire scenario, or, in fact, by strength and stiffness loss in the strands, which results in a decrease of the load-bearing capacity and an increase of deflections. Finally, as previously mentioned, another critical issue in hollow-core slabs is the shear strength [10], in consideration of the fact that these elements do not have transverse reinforcement.

More recently, extensive fire resistance experiments on nine bonded prestressed continuous concrete beams were performed by Hou et al. [11], investigating the influence of several variables, including concrete cover, load level and prestressing level. The test results proved that the influence of load level and concrete cover is very significant. Moreover, as should be expected, continuous beams are characterized by a higher fire resistance than simply supported beams.

All the aforementioned research works are focused on the standard fire (that is usually adopted in laboratory testing), i.e. a post-flashover cellulosic fire without cooling phase. When focusing on buildings consisting of prestressed concrete members, however, natural fires are undoubtedly more significant, mainly because of the large room and variable amount of fire load that characterizes this type of structures [12]. Within this context, in the design phase it may be advantageous from the economic point of view (and also more realistic) to resort to natural fires, based on the actual geometry of the compartment and on the available fire load. As a matter of fact, the most recent version of EN 1991-1-2 [13] allows resorting to performance-based design through the use of natural fires, taking as many significant parameters as possible into consideration, so as to eventually yield a more accurate representation of the real behaviour. Within this framework, it is clear that the structural behaviour of prestressed concrete members exposed to natural fires is definitely of interest for the designer.

Research works dealing with natural fires have been limited so far. The experimental research carried out by Gales and Bisby, reported in Ref. [14], who investigated the response of continuous and restrained post-tensioned concrete slabs, with unbonded tendons, exposed to natural fires, should be mentioned. Despite the relatively simple structural system, the deflection seems to be governed by several parameters, and is not yet fully understood.

More recently, the critical factors influencing the residual behaviour of reinforced concrete beams were investigated by Kodur and Agrawal [15]: a finite element model was set up to account for different material properties of both concrete and reinforcing steel, during the fire exposure phase (heating and subsequent cooling) but also during the residual phase (post-cooling phase). It was clearly pointed out that in natural fires the maximum ambient temperature does not give a clear indication of the maximum temperature in the rebars. As a matter of fact, the maximum temperature in the steel depends on the duration of the heating phase and on the cooling rate.

To date, no simplified methods are available for assessing the fire resistance of reinforced concrete members exposed to natural fires. EN 1992-1-2 [16] indicates the possibility of using the 500 °C isotherm method, but only if the thermal profiles inside the member are similar to those caused by the standard fire: this is certainly not the case in the cooling phase, when the external layers undergo cooling while the internal layers are still hot. As a matter of fact, the numerical studies carried out so far on concrete members all took advantage of advanced calculation methods, by using commercial softwares and performing demanding 3D thermo-mechanical analyses [15,17]. Simple approaches are thus needed, in order for practitioners to take advantage of the possibilities offered by the use of natural fires, without the need of computationally-demanding analyses (even in the case of members with simple structural layout).

The objective of this paper is to investigate the structural behaviour of simply supported prestressed members exposed to natural fires, in order to highlight the possibility of delayed failure (i.e. in the cooling phase of the fire), and to clarify the role played by duration of the heating phase, load ratio and cooling rate. To this end, the typical sectional approach used for simply supported concrete members, both pre- and post-tensioned with bonded tendons, is used. The constitutive models of concrete, ordinary and prestressing steel are suitably modified to properly take into account the different irreversible phenomena that take place in a full heating-cooling cycle. The modelling approach is validated against an experimental test showing satisfying agreement. The role of the different parameters coming into play is then investigated by means of parametric analyses carried out on two typical prestressed concrete sections, namely an I-girder and a double-tee, in order to highlight the peculiar features that characterize the structural behaviour of the studied members exposed to natural fires.

2. Analysis procedure

The failure of simply supported prestressed members in fire can be identified by taking into consideration the most stressed section, i.e. midspan (Fig. 1). This criterion is typically used for checking the safety under standard fires [4,5]. The same basic concept is applied in the following to the case of natural fires to identify the structural end point, i.e. the situation when the moment capacity is overcome by the applied moment. For simply supported members under distributed loads (or similar) and uniformly exposed to fire, failure is still governed by the most stressed section.

To this end, a sequentially-coupled thermo-mechanical analysis is carried out on the most stressed section. The first step is to perform a 2D thermal analysis of the section. In the numerical applications presented in the following, the thermal analyses were performed by means of the commercial software ABAQUS 6.16 [18]. Clearly, the thermal field could be worked out equally well by means of other softwares, or taking advantage of other approaches, such as the approximated analytical expressions by Wickström [19].

The output of the thermal analysis, consisting of the temperature values, is used as input for the mechanical analysis, in order to work out

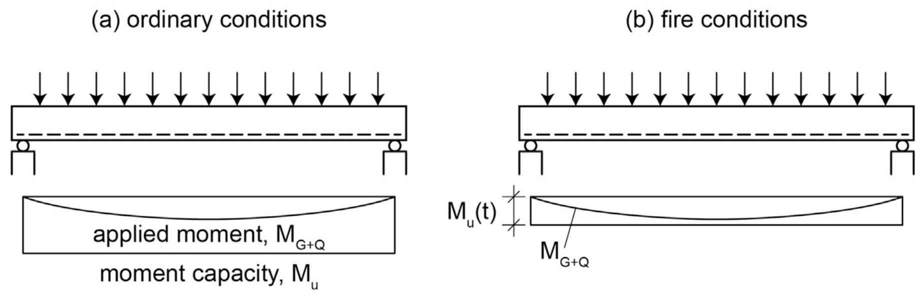

Fig. 1. Moment capacity and applied moment in simply supported members: (a) at ambient temperature; and (b) in fire conditions.

the evolution of the mechanical properties of the materials as a function of temperature. The mechanical analysis is based on the following basic assumptions:

- the Bernoulli-Navier hypothesis, namely that plane sections remain plane;
- perfect bond between the steel bars (ordinary and prestressed) and the surrounding concrete.

While the first assumption is widely accepted for performing the structural analysis of members such as beams and columns, the second assumption is reasonable in zones where large bending moments are expected and the shear forces are low. Both assumptions are reasonable when failure is governed by bending at midspan, this being typically the case for slender elements such as the ones considered herein.

The objective of the mechanical sectional analysis is to obtain the deformations (axial strain and curvature) of the section as a function of time; the failure of the most stressed section is then indicated by a divergence of the deformation values. The sectional analysis requires an iterative procedure, where the final results for a given fire duration are obtained once the equilibrium equations are satisfied, within the prescribed tolerance on the total axial force N (in pure bending $N = 0$) acting on the section ($\Delta N < 10^{-5}$). The iterative procedure was implemented in a purposely developed code, and enabled the deformation history to be traced over the whole fire duration. Sectional failure is thus attained if equilibrium is not possible, with the aforementioned divergence of the deformations (structural end point).

It is worth pointing out that this sectional approach is applicable to both pre-tensioned members and post-tensioned members with bonded tendons, because of the bond between the strands (in the former case) or the tendons (in the latter case, achieved by means of grouting the ducts) and the surrounding concrete. In members post-tensioned with unbonded tendons, where the compliance between the tendons and the concrete takes place only at the extremities, the structural analysis requires the deformations along the whole member to be considered, and thus requires a complete structural analysis of the whole member.

3. Mechanical properties

3.1. Concrete

3.1.1. Thermal strain

The irreversibility of thermal dilation is one of the peculiar features of a full heating-cooling cycle. As thermal dilation is, at least in part, due to irreversible phenomena (micro- and macro-cracking) that take place upon heating because of the thermal incompatibility between aggregates and cement paste, a certain amount of residual deformation is to be expected after cooling [20]. As reported in Ref. [21], a significant increase of the thermal expansion can be observed between 500 and 600 °C. This increase is probably due to macro-cracking caused by thermal stresses between aggregates and cement paste. Those macro-cracks were observed on the specimens after the tests. Clearly, at the macroscopic level it is difficult to separate the thermal expansion of the concrete from the deformation due to the opening of the cracks. Whatever the real cause, a certain amount of residual elongation ensuing from heating will remain also after the cooling phase. The main factors governing the amount of residual thermal deformation are the maximum temperature experienced, and the aggregate type used in the concrete mix.

The rather limited number of experimental results [22] on residual thermal deformation suggests that concrete thermal deformation can be considered as fully recoverable if the maximum temperature experienced is lower than 100 °C. On the other hand, if the maximum temperature is in the range between 200 and 400 °C, little residual shrinkage is to be expected. For higher maximum temperature values, residual elongation is to be expected. As for the influence of the aggregate, concretes with siliceous aggregates exhibit larger residual deformations than limestone or expanded clay concretes. Results from the experiments performed by Franssen [21] suggest that the maximum temperature governs the amount of residual elongation, and that for maximum temperatures lower than 500 °C, no significant residual elongation is to be expected. For maximum temperatures beyond 500 °C, residual deformation varies between 55 and 65% of the elongation corresponding to the maximum temperature. Bearing in mind the main the aforementioned evidences, the following assumptions were adopted in the present work:

- the residual deformation for maximum temperatures below 500 °C, be it contraction or elongation, is negligible;
- above 500 °C, the residual thermal deformation is equal to 50% of the maximum thermal dilation experienced.

Clearly, these assumptions lead to a discontinuity of the residual thermal deformation at 500 °C: whereas this may lead to strange numerical result in principle, it was not the case in all the analyses presented herein, also because the strain compatibility along the section tends to accommodate the strain discontinuities at different points.

The adopted thermal strain evolution during heating and cooling is plotted as a function of temperature in Fig. 2: the evolution of the thermal dilation with temperature was computed according to the provisions of [16] for concrete with siliceous aggregate.

3.1.2. Constitutive model

Accounting for the cooling phase in a constitutive model implies several peculiar issues to be addressed. If only the heating phase is considered, assuming monotonic loading at each point generally yields good results, though some unloading, due to thermal gradients, takes place in the core of the section. However, this unloading during heating takes place following the initial loading branch. On the contrary, if the temperature-time curve includes a cooling phase, the constitutive laws should be suitably modified: EN 1992-1-2 [16] clearly points out that the constitutive law for concrete in compression should be suitably modified in the case of natural fires; in more detail, the possibility of unloading should be explicitly included, as well as the additional strength decay in the cooling phase, and the constitutive law modified accordingly.

In the following, the loading branch coincides with the stress-strain

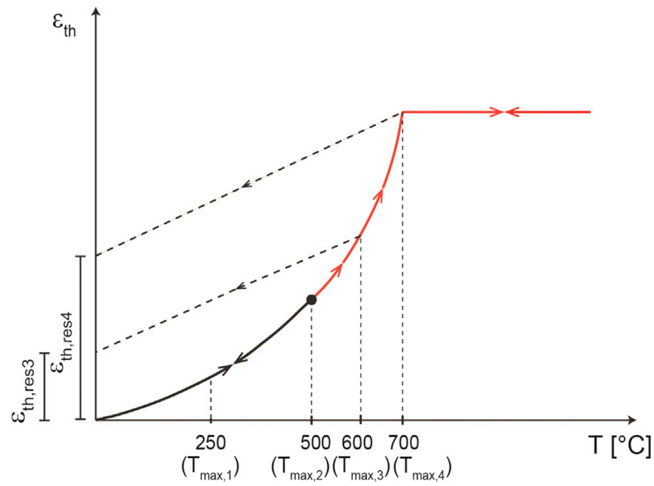

Fig. 2. Free thermal strain of concrete at elevated temperatures and during subsequent cooling.

law provided in Ref. [16], whereas unloading is assumed to take place along a linear branch, with the slope defined by the residual elastic modulus. This assumption complies with the fact that the transient and creep strains (irreversible strain components, with the transient strain being the largest component of the load-induced thermal strain) that are responsible for the increased deformability of concrete upon first heating, are not recovered during unloading, and are not present at all when the residual behaviour of concrete is considered [20,23]. As a consequence, the residual elastic modulus at any temperature is larger than the elastic modulus in hot conditions, the difference being the irreversible components of strain:

$$E_{res} = k \cdot E_{hot} \tag{1}$$

where E_{hot} is the initial tangent modulus of elasticity in hot conditions, and k is a constant value (with $k > 1$, as commonly found in laboratory tests [24,25]). As can be seen in Fig. 3a, once the stress corresponding to the peak strain is reached, only plastic loading (with increase in strain increment) or unloading along a linear branch (if strain increment is negative) can occur. It is worth noting that the stress can go to zero with a strain value (i.e. cracking strain) greater than zero. Below this strain value, as is usually done in the numerical analysis of concrete members, the contribution of the tensile stresses is considered negligible.

By keeping track of the strain corresponding to the peak stress, loading and unloading can occur as many times as required, with the cracking strain shifting and the stress-strain curve changing shape.

As for thermal damage, during the heating phase the stress-strain curves are adapted to allow for decreasing compressive strength and increasing deformability. During the cooling phase, since thermal damage is considered as a function of maximum temperature only, the stress-strain curves are not updated. This last assumption implies that the extra-damage taking place during the cooling phase is neglected. It is worth noting, however, that the difference between "hot" compressive strength and residual compressive strength is usually rather limited (10% according to the provisions contained in EN 1994-1-2 [26]).

3.2. *Steel*

3.2.1. *Thermal strain*

The thermal strain for both prestressing and reinforcing steel is computed according to the provisions of [16]. No residual thermal strain has been assumed for steel, which is considered a material that recovers its properties upon cooling. This assumption is in agreement with the rather limited number of experimental results available to date [27,28].

3.2.2. *Constitutive model*

For both prestressing and reinforcing steel, elastic-perfectly plastic stress-strain laws with linear unloading branch were assumed (Fig. 3b). As a consequence, there is no difference between the proportional limit and the yield strength. Since for significant fire durations and/or at impending collapse steel is likely to be yielded, this assumption plays a minor role on the results. The main difference between ordinary and prestressing steel is the different thermal decay of the mechanical properties (that is higher in the latter). Note, however, that ordinary steel is destined to play a minor role with regards to the possibility of failure, as in prestressed concrete members the bending capacity is mostly provided by the prestressing steel.

The evolution of the stress-strain law at any given point is governed by the maximum temperature experienced and loading history. The accumulation of residual plastic deformation changes the position, but not the shape of the stress-strain diagram. Note that the effects of creep (and therefore the relaxation of prestressing steel) are implicitly taken into account by reducing the modulus of elasticity with increasing temperature.

4. Validation of the modelling approach

The validation of the numerical model is performed against the results presented in Ref. [29]. A set of three reinforced concrete beams were subjected to fire curves with heating and cooling phase, and then

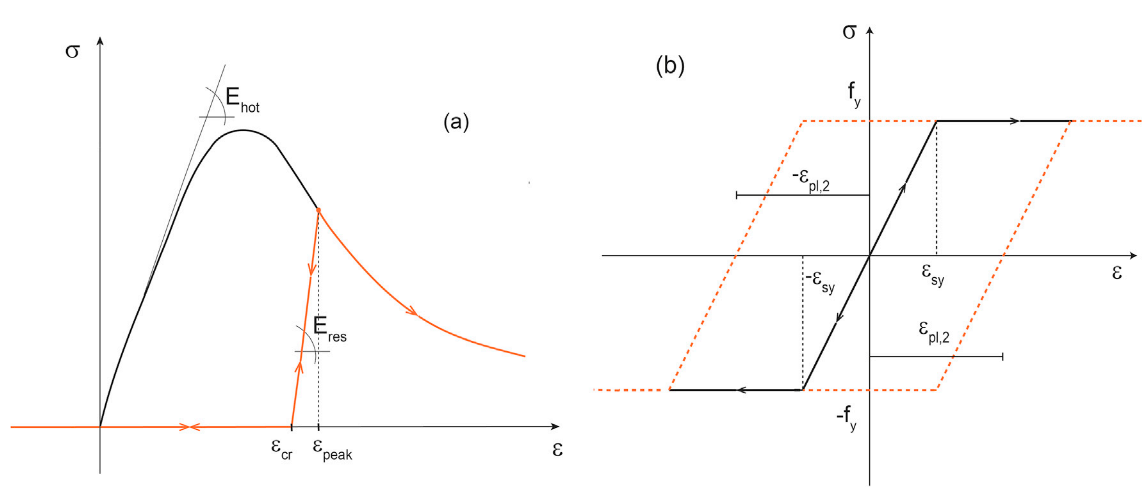

Fig. 3. (a) Stress-strain model for concrete in compression at a given temperature, in loading and unloading; (b) stress-strain model for steel at a given temperature, in loading and unloading conditions.

tested for measuring their residual bearing capacity. Two beams (B1 and B3) were tested with axial restraints at the extremities, while B2 was simply supported.

Beam B1 was made from normal-strength concrete (NSC, $f_c = 56$ MPa); the other two, were of high-strength concrete (HSC, $f_c = 100$ MPa). The reinforcement consisted of ∅19 mm bars as bottom reinforcement, two ∅13 mm bars as top reinforcement, while the shear reinforcement consisted of ∅6 mm stirrups at a spacing of 150 mm (Fig. 4). The steel of the flexural reinforcement had specified yield strength of 420 MPa.

During the fire tests, beams B1 and B2 were tested under a so called "short" design fire (SF), with the ambient temperature quickly peaking 1100 °C, a rest period at the maximum temperature until 60 min, followed by a descending branch with a cooling rate of approximately 5 °C/min. Beam B3 was tested under a long and severe design fire (LF), with peak temperature of 1100 °C until 120 min, followed by a cooling branch with a temperature decrease of 9 °C/min.

For the purpose of validation, B2 was chosen, because its structural layout (simply supported) is the same as that of the prestressed members studied herein. In the case of beam B2, the sectional approach described in the previous section is expected to correctly capture the structural behaviour.

The lower curve of thermal conductivity suggested in Ref. [16] was used for the thermal analysis; moreover, the thermal behaviour was considered fully reversible. The comparison between numerical and experimental results is shown in Fig. 5a. On the whole, the agreement is good, both in the heating and in the cooling phase. There is a slight overestimation of the maximum temperature that could be due the heat dispersion along the length of the member, which is present in the tests but cannot be captured by means of 2D analyses.

Decay of the concrete strength, irrecoverable in the cooling phase, and thermal strain were assumed as per [16] for calcareous aggregate. The thermal strain was considered irreversible upon cooling if the maximum concrete temperature exceeds 500 °C; the value of the residual thermal strain was taken equal to 50% of the value at the maximum temperature. The decay of the yield strength and elastic modulus of steel was assumed as per [16] and the behaviour was considered irreversible, while the thermal strain (also taken from Ref. [16]) was considered fully recoverable. The comparison between numerical and experimental results is shown in Fig. 5b. The agreement is satisfying, as regards the value of the maximum deflection and its time of occurrence: it is worth noting that no information were provided as regards the evolution of the thermal and mechanical properties with temperature. On the whole, the numerical approach appears to overestimate the deflections throughout the heating phase: this fact could be explained by considering that in the numerical analyses the contribution of concrete in tension was neglected. Moreover, the decay of the mechanical properties prescribed in Ref. [16] does not account for the effects of a pre-load (that was present in the tests), which leads to a less pronounced decay of the mechanical properties.

5. Parametric analyses

Parametric analyses were performed on the sections of two typical prestressed concrete members, namely an I-girder and a double tee (Fig. 5). The two sections, representative of simply supported members commonly found in industrial buildings [1], are interesting for a variety of reasons: the I-girder has a thin web with a massive bottom flange (where most of the prestressing steel is located), while the double tee has very thin webs and flange, and consequently a low cover to the prestressing steel located in the webs.

The aforementioned characteristics are instrumental for highlighting the role played by the concrete cover in delaying the attainment of the maximum temperature in the prestressing steel. Moreover, more than 95% of the bending capacity in ordinary conditions (determined by means of sectional analysis) of the double-tee section is provided by the prestressing steel, while in the I-girder section the prestressing steel provides 65% of the bending capacity, the remainder being provided by the ordinary reinforcing steel, that is always present to ensure additional resources in the event of cracking.

As it was done in previous studies [15,17,30], the temperature-time curve was derived from the parametric fire model devised in Ref. [13], by setting the factor Γ (that accounts for parameters such as ventilation and thermal inertia of the enclosure) equal to 1. In this way, the heating phase coincides with the ISO-834 fire curve (Fig. 7 [31]). As for the cooling phase, a linear decrease of temperature with time was adopted [13,32].

The thermal properties were assumed following the provisions of EN 1992-1-2 [16] that allows to choose the value of conductivity between the upper and lower limit; therefore, the mean value of conductivity was used; moreover, the specific heat for dry concrete was assumed. As it was done in the validation (obtaining a good agreement with the test results), the thermal properties were considered reversible in cooling and are thus a function of temperature only. Finally, the presence of ordinary and prestressing steel was neglected in the thermal analysis.

For both sections, the following reference mechanical properties at ambient temperature were assumed:

- concrete compressive strength: $f_c = 40$ MPa;
- modulus of elasticity and yield strength of mild steel: $E_s = 210$ GPa, $f_{sy} = 500$ MPa;
- modulus of elasticity and yield strength of prestressing steel: $E_p = 195$ GPa, $f_{py} = 1860$ MPa (note that since the constitutive law of prestressing steel is assumed to be elastic-perfectly plastic, Fig. 2b, there is no difference between yield and ultimate strength);
- prestressing level at the onset of fire: $\sigma_0 = 1000$ MPa.

The value of the prestressing level at the onset of fire was determined by considering the typical value of prestressing before long term losses (1200–1300 MPa), and accounting for 15–20% of decrease due to creep and shrinkage of concrete, and relaxation of the prestressing steel [33,34]. As for the evolution of the mechanical properties with temperature,

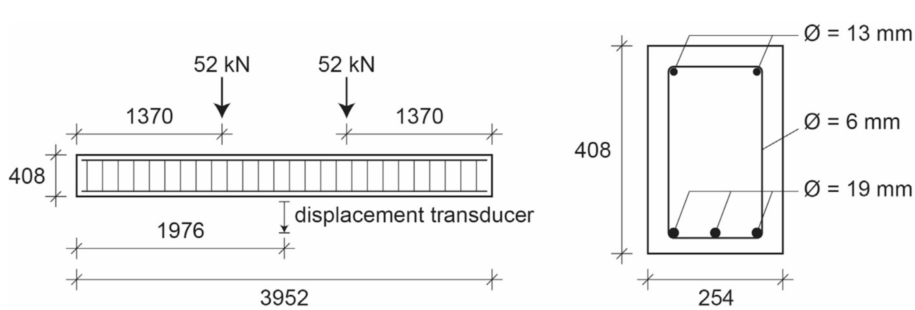

Fig. 4. Geometry and transverse section of the beam used for the validation [28].

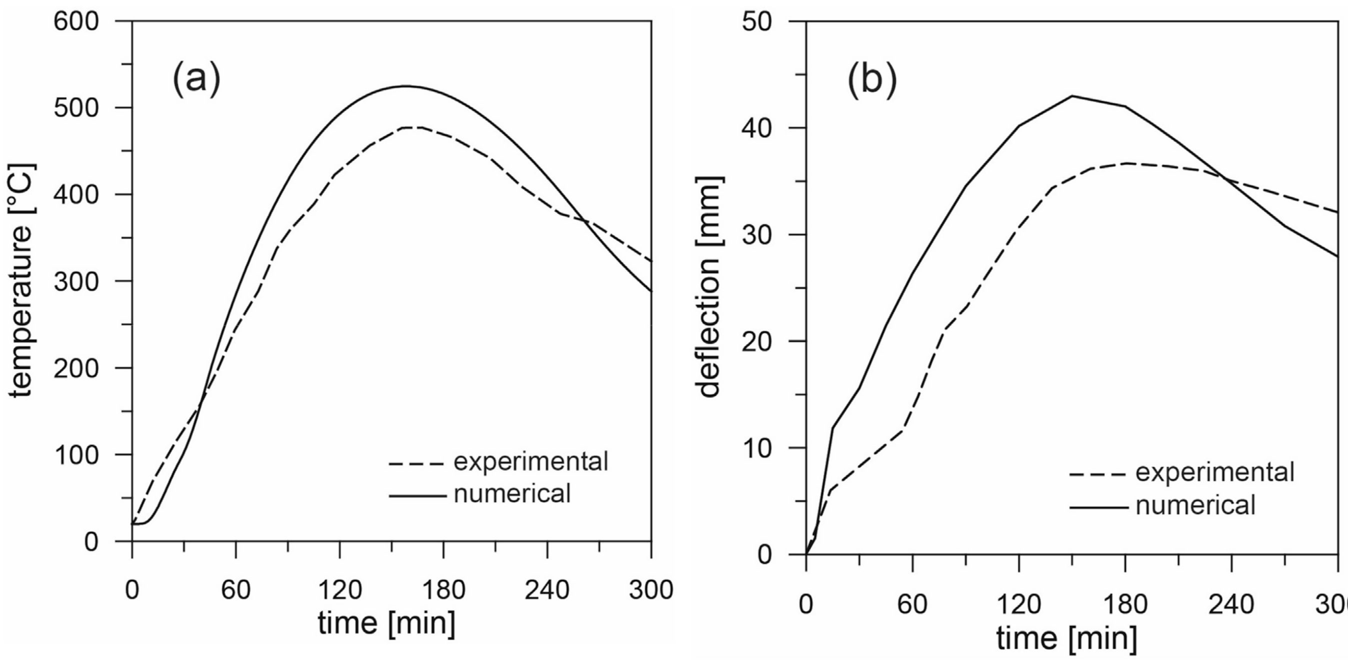

Fig. 5. Comparison between experimental results and numerical simulations: (a) temperature in the rebars; (b) deflection at midspan.

the values suggested in EN 1992-1-2 [16] for siliceous concrete, ordinary and prestressing steel were assumed. Note that siliceous concrete was chosen, because of its greater sensitivity to high temperatures in comparison to concretes containing other types of aggregate (e.g. calcareous or basalt). In prestressed concrete members, however, failure is expected to be governed by prestressing steel, and the thermal decay of concrete plays a minor role.

Several parameters were varied, in order to investigate their influence on the behaviour of the selected prestressed concrete beam sections, and thus to get sound conclusions about their influence on the failure.

A first set of preliminary analyses was devoted to investigate the role of the residual thermal deformation and level of prestressing, in order to check the validity of the assumptions made for the values of these two parameters. Moreover, the role of the irreversible strain components is studied through comparison of the proposed model with a recently formulated explicit model [35].

Fire severity, determined by the duration of the heating phase and the cooling rate, is certainly among the most critical parameters governing the response. If the failure of the section, whether in the heating or in the cooling phase of a fire, is to be studied, then the maximum temperature in the section, and most importantly in the prestressing steel, can provide useful information. The duration of the heating phase was varied from 30 to 120 min for the I-girder and from 15 to 60 min for the double tee (with the cooling rate fixed at 5 °C/min). and different cooling rates were considered (3, 5 and 10 °C/min), in accordance with [13] (where cooling rates ranging from 250 to 625 °C/h are devised), for a fixed heating phase duration of 60 min for the I-girder and 30 min for the double-tee.

Load level is a parameter of utmost importance. It can be conveniently expressed as the ratio of the externally applied bending moment to the ultimate moment capacity at ambient temperature. Three values were considered in this work: $M/M_u = 0$, 0.15 and 0.30, where M is the bending moment acting at midspan at the onset of fire and M_u is the bearing capacity in bending at midspan assuming the characteristic values of the materials properties (partial safety factors $\gamma_c = \gamma_s = \gamma_p = 1$). While the largest value can be considered realistic for members with a significant share of variable loads ($Q_{k1}/G_k = 1.5$, $\psi_{1,1} = 0.2$ [16]), the zero value is not possible in reality, as at the very least the self-weight will act as a load; it is interesting, however, in order to investigate the effect of prestressing alone, to better highlight the contribution of prestressing to the in-time evolution of the curvature of the section.

5.1. *Role of the thermal field*

Given that almost the full moment capacity of the double-tee section is provided by the prestressing steel, it is more than clear that a sizable loss of stiffness and strength has to be prevented in order to avoid failure. However, since the concrete cover is rather thin (average of 3 cm, Fig. 6a) very high temperatures are likely to be attained in most cases. In Fig. 8a, the average temperature in the strands is plotted as a function of the duration of the heating phase, for the reference cooling rate of 5 °C/min. It is worth noting that the temperature increase continues long after the onset of the cooling phase: as a matter of fact, the maximum temperature in the strands is reached at approximately 90 and 100 min for heating phase durations of 15 and 30 min respectively, and at 130 min for heating phase duration of 60 min. As previously mentioned, this can be attributed to the low conductivity and high thermal capacity of concrete, which delay the heat transfer towards the inner layers of the section. Note that failing to account for the cooling phase leads to a significant underestimation of the maximum temperature (from −40 to −75%). In Fig. 8b, the average temperature in the strands is plotted as a function of the cooling rate, for the reference duration of the heating phase of 30 min. The maximum temperature in the prestressing steel is significantly lower for a cooling rate of 10 °C/min than for 3 °C/min (−25%), something that reduces the likelihood of delayed failure in the cooling phase. Similar considerations hold for the I girder (Fig. 9a and b): in this case the prestressing steel has better protection than in the double-tee section, since all prestressing strands are located in the bottom flange (Fig. 6b). As a consequence, higher durations of the heating phase are needed to attain the same temperature levels as in the double-tee section.

5.2. *Role of residual thermal deformation and level of prestressing*

Fig. 10 shows typical curves of the mechanical response of the two sections, for a load ratio $M/M_u = 0.15$ and a cooling rate of 10 °C/min, in order to highlight the role played by the residual thermal deformation and the level of prestressing. From Fig. 10a (I-girder, duration of the heating phase = 60 min) it can be noted that the adopted value of the residual thermal deformation $\varepsilon_{th,res}$ plays a marginal role on the

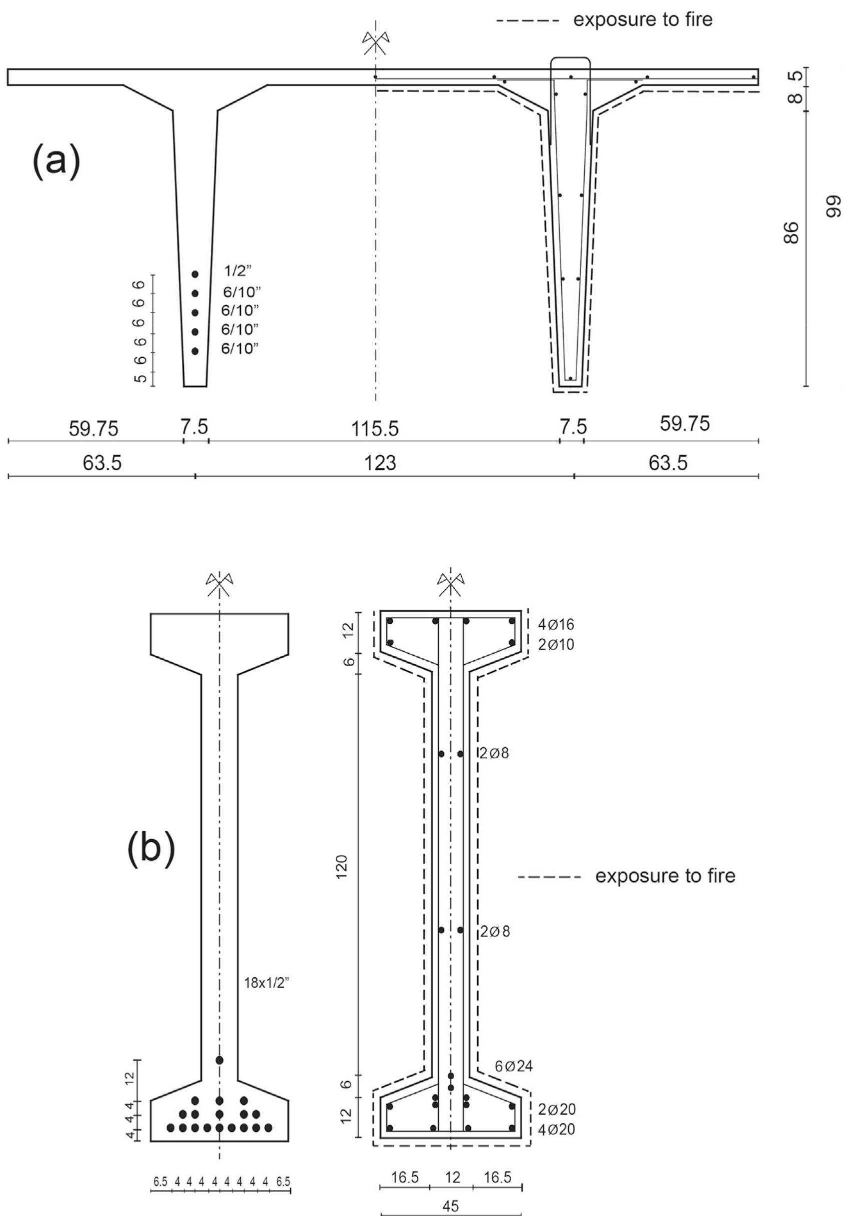

Fig. 6. Prestressed sections considered in the numerical analyses (prestressing steel on the left, ordinary steel at the right): (a) double-tee section; and (b) I-girder section.

maximum curvature, within a range of variation between 50 and 65% of the thermal deformation corresponding to the maximum temperature: this finding might explain why in some recent works [15,35], where the residual behaviour was investigated, more simplified assumptions concerning the thermal and mechanical properties were adopted (no unloading considered). The role of the residual thermal deformation increases when the strands are in the cooling phase: from Fig. 10a it can be seen that the difference between the values of curvature for the two values of residual thermal deformation considered is increasing with time, reaching eventually 20%. As for the role of the prestressing level at the onset of fire σ_0, Fig. 10b (double-tee, duration of the heating phase = 30 min) shows that within the typical range of 900–1100 MPa it is definitely of minor importance. It is worth noting that the initial decrease of curvature (corresponding to hogging deflections) is due to the prevailing effect of prestressing, which is enhanced by the increased deformability of concrete at the beginning of the heating phase, over the thermal dilations and strand relaxation (that would lead to sagging deflections).

5.3. Role of different strain components – implicit vs explicit model

As previously mentioned, a key aspect concerning the behaviour of concrete in full heating-cooling cycles is definitely the contribution of the different strain components, most of which are irreversible.

Within the available constitutive models for concrete, the term accounting for transient creep is considered either implicitly or explicitly [23,36]. The widely used Eurocode model only considers transient strain implicitly, by summing this strain component to the instantaneous stress-related strain, to give the so-called mechanical strain ε_m. The total strain is then obtained as the sum of mechanical and thermal strain:

$$\varepsilon_{tot} = \varepsilon_m + \varepsilon_{th} \tag{2}$$

The stress can then be evaluated on the basis of the mechanical strain only. The same approach is adopted in this study, as regards the behaviour of concrete in the loading phase. Implicit models, such as the Eurocode model, are widely used because of their simplicity.

In explicit models, the mechanical strain is expressed as the sum of

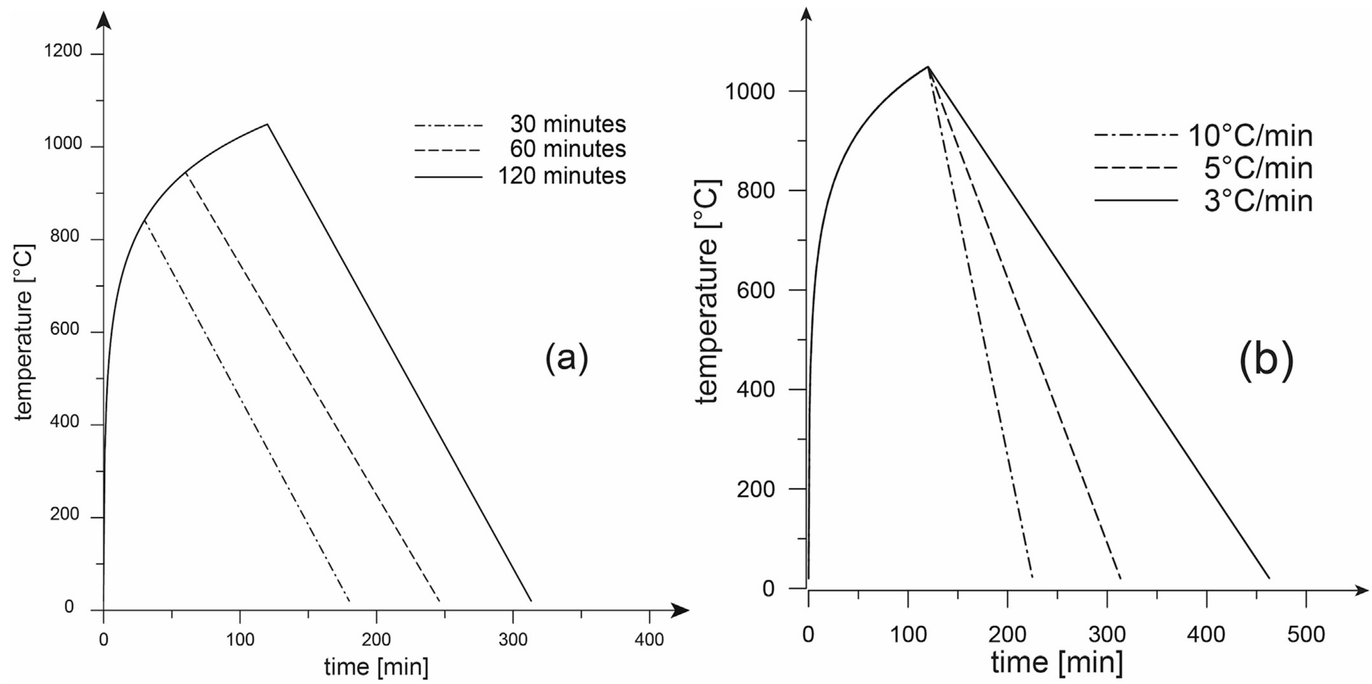

Fig. 7. Fire scenario: (a) variation of the duration of heating phase, at constant cooling rate (5 °C/min); and (b) variation of the cooling rate, at constant duration of the heating phase (120 min).

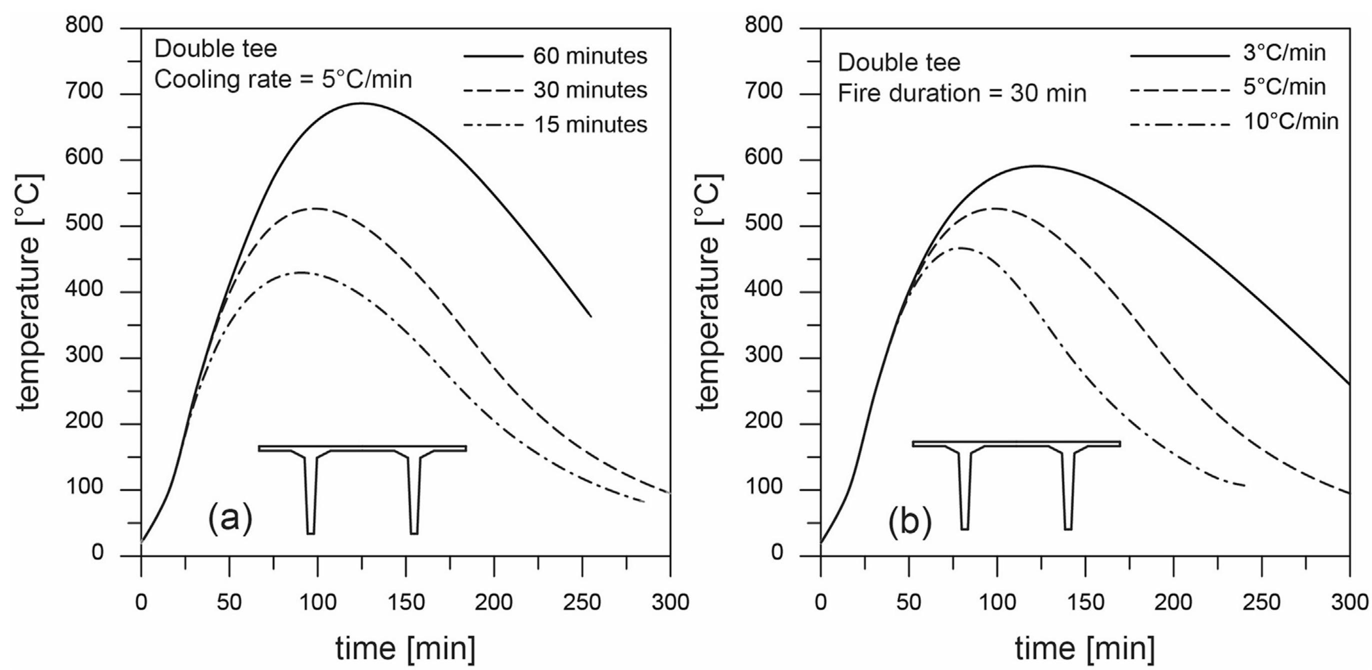

Fig. 8. Double-tee section: (a) average temperature in the strands as a function of the duration of the heating phase (cooling rate = 5 °C/min); (b) average temperature in the strands as a function of the cooling rate (fire duration = 30 min).

instantaneous stress-related strain and transient creep strain; therefore, the term for transient creep is explicitly introduced:

$$\varepsilon_{tot} = \varepsilon_{\sigma} + \varepsilon_{tr} + \varepsilon_{th} \tag{3}$$

Among the several explicit models available, the most recent is the one developed by Gernay and Franssen [37], and validated against the results from tests on axially unrestrained concrete cylinders subjected to different stress-temperature regimes. The advantage of this model is the fact that it accounts for all the possible load-temperature combinations, hence also for the cooling phase of a fire. The formulation of the model is rather simple, as it has the same general form as the widely used EC2 model. These are the reasons why this model was chosen to validate the implicit model used in this study.

A limited number of analyses was performed (I girder exposed to fire with a heating phase duration of 60 min, cooling rate of 10 °C/min and load level $M/M_u = 0.15$), and the time evolution of the sectional curvature obtained with the two models was compared (Fig. 11a). The agreement between the two models is definitely good: slight differences

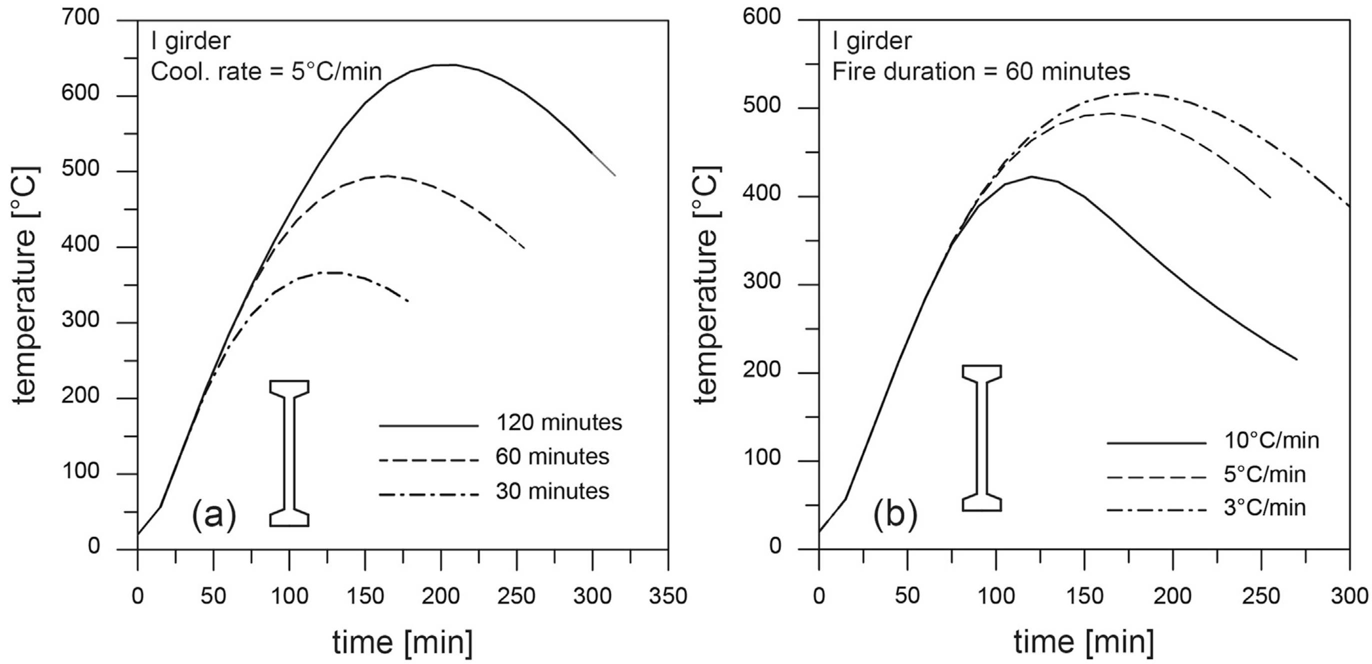

Fig. 9. I-girder section: (a) average temperature in the strands as a function of fire duration (cooling rate = 5 °C/min); (b) average temperature in the strands as a function of the cooling rate (fire duration = 60 min).

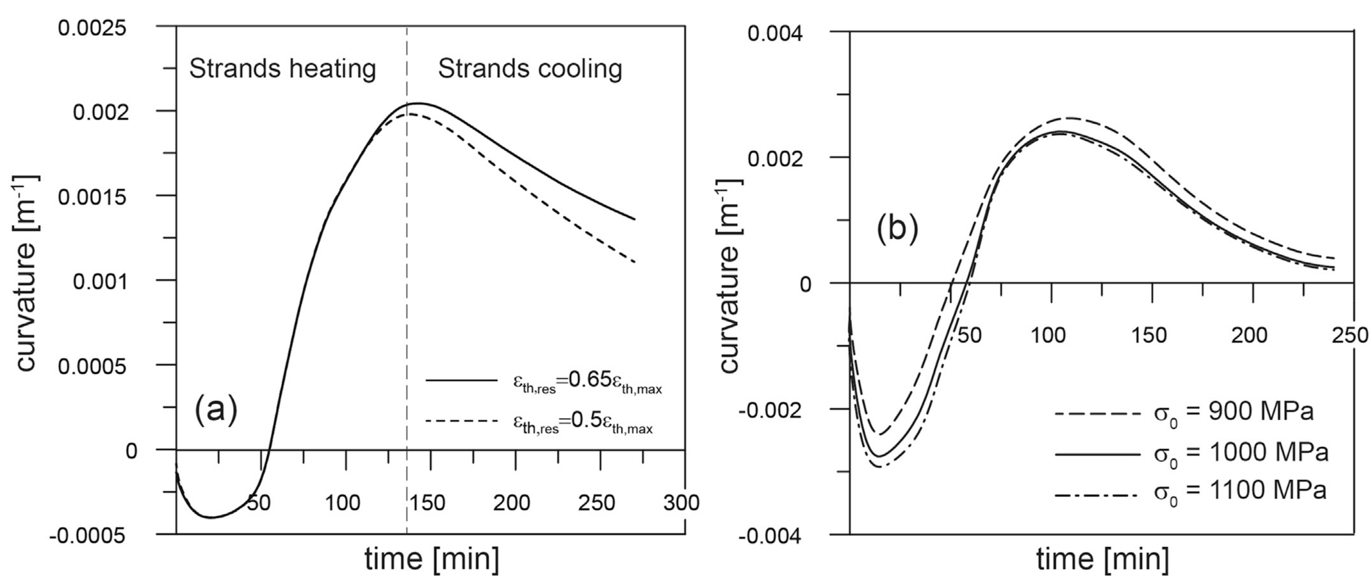

Fig. 10. (a) Influence of the residual thermal strain on the structural behaviour; (b) influence of the prestressing level on the structural behaviour.

are visible only in the cooling phase, where unloading is likely to occur. As for the role played by the elastic modulus of concrete (taken as k·E_{hot}, where the reference value for k was chosen as 1.25), it proved to be marginal (Fig. 11b).

5.4. Role of the duration of the heating phase

Fig. 9 shows the response of the two sections as a function of the duration of the heating phase, for a load level $M/M_u = 0.15$ and a cooling rate = 5 °C/min.

The double-tee section is certainly more critical (Fig. 12a): due to the extremely high temperatures attained in the strands (see also Fig. 8a), it is only able to survive fires characterized by shorter durations of the heating phase (15 and 30 min), while it fails at the early stage of the cooling phase for a duration of the heating phase of 60 min.

The I-girder section shows a more complex behaviour (Fig. 12b). In the heating phase it does not exhibit failure for 30 and 60 min, while it fails for duration of the heating phase of 120 min. This was expected, since from Fig. 7b it is evident that the average temperature of the strands starts decreasing long (almost 2 h) after the onset of cooling. Even though the load level is rather low, upward bending only takes place in the very first phases of the fire, when the prestressing level and stiffness of the strands are to a great extent maintained. Later on, as the temperature in the prestressing steel further increases, the response is governed by downward bending. Note that for the fire duration of 60 min, the I-girder fails in the cooling phase, due to the delayed heating of the prestressing steel.

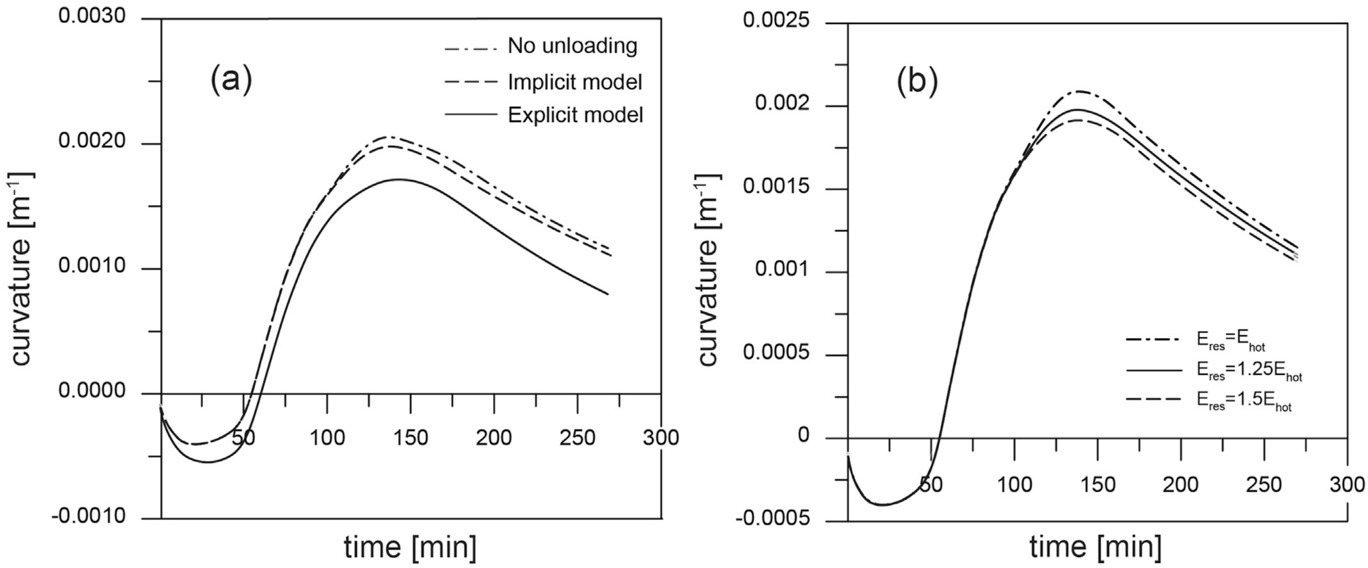

Fig. 11. (a) Comparison of the proposed stress-strain model (implicit model, $E_{res} = 1.25{\cdot}E_{hot}$) with the model by Gernay and Franssen (explicit model [34]) and with the EC2 constitutive model (without unloading); (b) influence of residual elastic modulus adopted for the unloading phase.

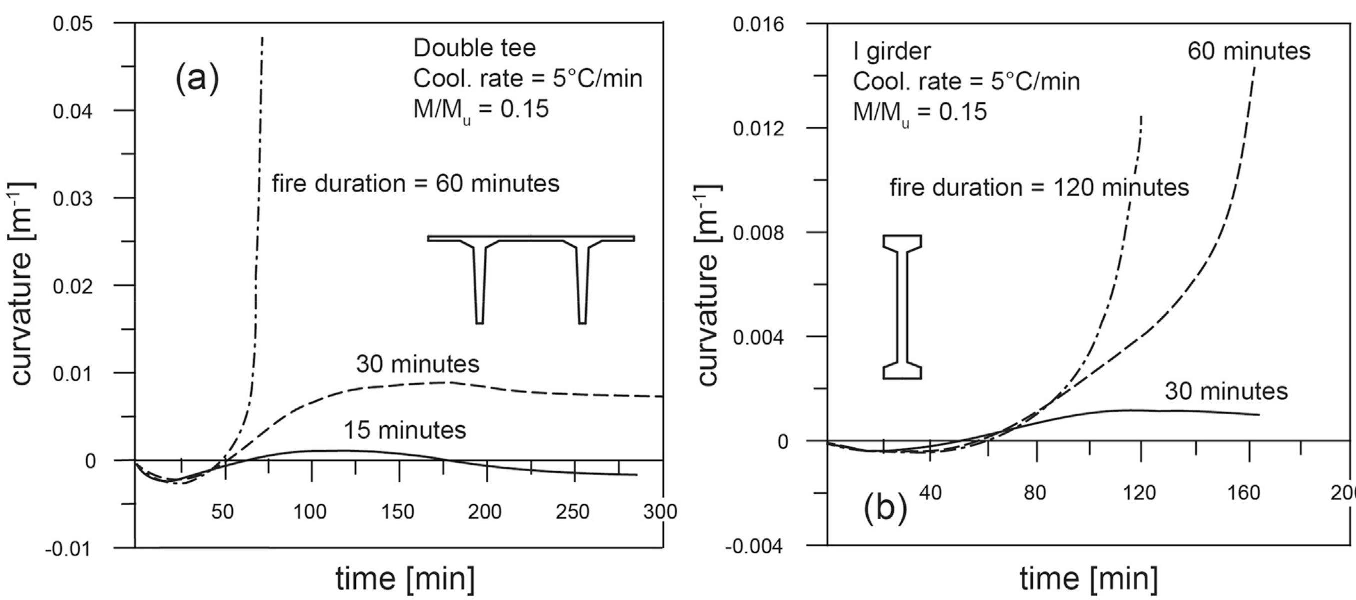

Fig. 12. Influence of fire duration ($M/M_u = 0.15$, 5 °C/min): (a) double tee; and (b) I girder.

5.5. Influence of the cooling rate

Fig. 13 shows the response of the two sections as a function of the cooling rate, for a load level $M/M_u = 0.15$ and fire durations = 30/60 min (double tee/I girder). Cooling rates are set to 3, 5, 10 °C/min, to simulate different cooling regimes. Only in the case of the I girder, the additional value of 50 °C/min is also considered.

For the reference load level, both sections fail during the cooling phase for the lowest cooling rate (3 °C/min), while they are able to withstand the full heating-cooling cycle for higher cooling rates. In both cases, a variation of the cooling rate from 5 to 10 °C/min brings in a sizable reduction of the residual displacements. For the I girder, the deflections caused by the largest value of cooling rate (50 °C/min) are almost fully recovered in the cooling phase. In all cases, the structural behaviour (and the deformation reversal) is governed by the temperature decrease of the strands, that is indicated with a full dot in Fig. 13a,b.

Clearly, fast cooling rates are beneficial, even more for shorter fire durations, as they prevent prestressing steel from reaching critically high temperatures, on the condition that they don't induce spalling, because of the steep thermal gradients.

5.6. Influence of load level

The load level plays a very significant role in determining sectional response to fire (Fig. 14). The bending moment due to external loads tends to induce downward bending (positive curvature), while prestressing alone tends to cause the opposite response of upward bending (negative curvature, Fig. 14b, $M/M_u = 0$). In the double-tee section (Fig. 14a, duration of the heating phase = 15 min, cooling rate = 5 °C/min), the load level $M/M_u = 0.15$ brings in a residual negative curvature (upward deflection) that is of the same order of magnitude of that at the beginning of the fire, while the load level $M/M_u = 0.30$, though not causing collapse, causes a residual downward deflection, that is several times larger than the initial one. In the case of the I girder (Fig. 14a,

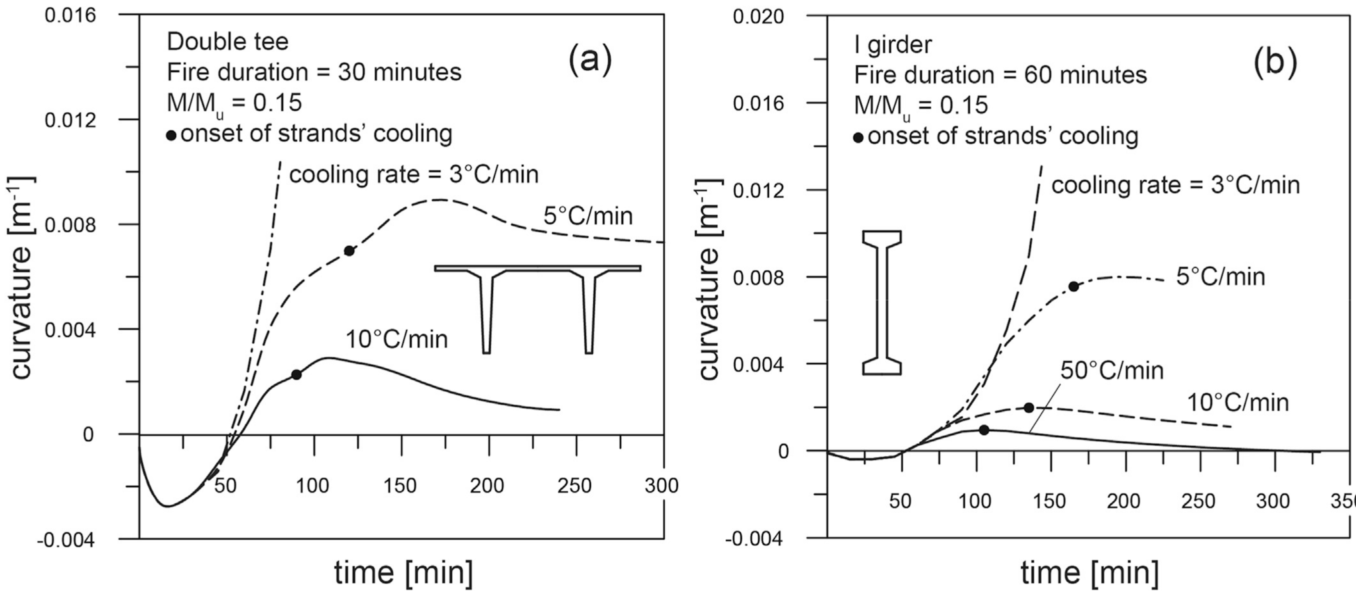

Fig. 13. Influence of the cooling rate: (a) double tee (fire duration = 30 min; $M/M_u = 0.15$); and (b) I girder (fire duration = 60 min; $M/M_u = 0.15$).

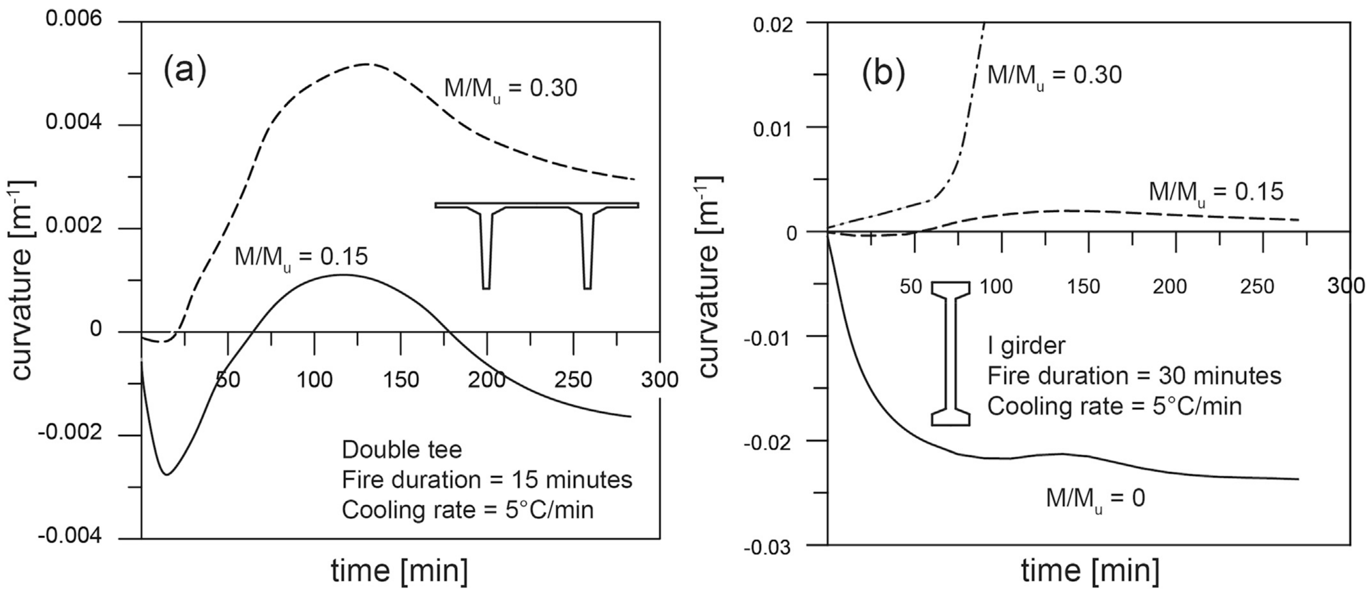

Fig. 14. Influence of load level: (a) double tee (fire duration = 15 min; cooling rate = 5 °C/min); and (b) I girder (fire duration = 30 min; cooling rate = 5 °C/min).

duration of the heating phase = 30 min, cooling rate = 5 °C/min), varying the load level from $M/M_u = 0$ to 0.30 significantly changes the shape of the curves: as a matter of fact, without applied loads upward deflections prevail, while for the highest load level there is failure in the cooling phase (approximately 60 min after the onset of cooling).

6. Conclusions

The numerical investigations carried out allow drawing some general conclusions on the structural behaviour of prestressed concrete members exposed to natural fires.

The influence of section geometry is determined mainly by the concrete cover and available "thermal mass". As a result, for the same load level, the double-tee section exhibits the worst performance in terms of fire resistance. For both sections, however, the results clearly indicate that heating of the prestressing steel continues long after the onset of the cooling phase, especially for low fire durations. As a consequence, depending on the values of fire duration, cooling rate and load level, failure can take place also in the cooling phase.

Cooling rate and load level both prove to be important factors. With long durations of the cooling phase, higher temperatures can be reached in the prestressing steel, especially in more massive sections. In the two sections examined, doubling the cooling rate from 5 to 10 °C/min brings in a sizable reduction of the residual displacements. As for the load level, depending on whether the effects of prestressing or those of the external applied loads prevail, the residual displacements can be either upward deflections or downward deflections.

Generally speaking, irreversible effects, such as residual thermal deformation and accumulation of plastic deformations, which in principle may play an important role in the cooling phase, are definitely of minor importance as regards failure. On the contrary, their importance increases in members surviving the whole fire, where the assessment of the residual displacements can give indications on the level of thermal damage. This topic, which is of crucial importance from the practical point of view, needs to be investigated in more detail.

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