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Collapse of concrete columns during and after the cooling phase of a fire

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Abstract
A study has been performed on the collapse of reinforced concrete columns subjected to natural fire conditions during and after the cooling phase of the fire. The aim is, first, to highlight the phenomenon of collapse of concrete columns during and after the cooling phase of a fire and then, to analyze the influence of some determinant parameters. The main mechanisms that lead to this type of failure are found to be the delayed increase of the temperature in the central zones of the element and the additional loss of concrete strength during the cooling phase of the fire. A parametric analysis considering different fires and geometric properties of the column shows that critical conditions with respect to delayed failure arise for short-duration fires and for columns with low slenderness or massive sections.

Keywords
Fire resistance, reinforced concrete, concrete column, cooling phase, residual strength

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Introduction

In the prescriptive approach based on the standard ISO fire, the fire resistance of a structural element has to be ensured during the period of time that is required by law and no verification has to be made about the performance of the element thereafter. Because the temperature of the ISO fire is continuously increasing, the temperatures in the element are also continuously increasing and the load-bearing capacity of the structure is continuously decreasing. As a consequence, verification at the required fire duration guarantees stability at any previous instant in the fire.

If the behavior of a structure or structural element is assessed in a performance-based environment, a more realistic representation of the fire will be used that comprises not only a heating phase but also a cooling phase during which the temperature of the fire is decreasing back to ambient temperature. The influence of such realistic fire scenarios in the evaluation of the fire resistance is a key issue in the performance-based approach, as presented for example by Fike et al. [1] for concrete-filled hollow structural section columns. The required duration of stability may be longer than the duration of the heating phase; it may even be required that the structure survives the total duration of the fire until complete burnout. In that case, the load-bearing capacity of the structure continues decreasing after the maximum gas temperature is attained, finally reaching a minimum value and eventually recovering partially or completely when the temperatures in the structure are back to ambient.

The continuation of degradation of the load-bearing capacity after the maximum gas temperature is attained has two main causes.

Firstly, the temperatures in the structure may continue increasing while the gas temperature is decreasing. For an unprotected steel structure, this will be the case until the gas temperature has become lower than the steel temperature. For a thermally protected steel structure, the increase of steel temperature will continue for a longer time due to the inertia of the insulation. For a concrete structure, the zones of the members that are near the surface will exhibit a decrease soon after the gas temperature has become lower than the surface temperature but, as will be shown in the next section, the central zones may continue to have an increase in temperature for a significant duration, all the more if the section is massive.

The second reason is to be found in the material behavior. While steel recovers strength and stiffness as soon as its temperature decreases, completely or partially depending on the type of steel and the maximum temperature attained [2], concrete remains severely damaged after cooling. Concrete does not recover its strength, and also some indications exist that tend to prove that there is an additional loss of strength during cooling from maximum temperature to ambient [3].

For the designer, this implies that verification in the load domain at the time of maximum gas temperature does not guarantee against collapse at a later stage. Verification must necessarily be performed in the entire time domain by a step-by-step iterative method. Some authors have been interested in the residual load-bearing capacity of structural elements after exposure to fire, for example Hsu et al.

[4,5] for reinforced concrete beams. However, no study has been performed, to the authors' knowledge, on the risk of collapse during the cooling phase of a fire. The evolution of material properties in the cooling phase must be available to perform such an analysis.

For fire fighters, the possibility of collapse occurring after the time of maximum gas temperature, if confirmed, might be a real threat. In fact, fire temperatures can increase only if there is no fire fighting operation and collapse during this phase is only an issue if it leads to progressive collapse outside the fire compartment. On the contrary, the intervention of fire fighting forces will usually lead to a decrease of the fire temperatures and, if collapse occurs during this phase, it may be at a time when fire brigades are in or near the vicinity of the fire compartment and they thus may be endangered by the collapse. Collapse during the cooling phase of a fire occurred, for example, in a full-scale fire test conducted in 2008 by Wald [6] in the Czech Republic. The composite steel and concrete floor, working in tensile membrane action, collapsed shortly after the wood based natural fire entered into the cooling phase. One of the possible failure modes that is suspected is by failure of the concrete compression ring that was established in the slab.

A structural failure that would occur at a later stage, when the temperatures in the compartment are back to ambient, may be an even greater threat because it would occur at the time of first inspection, not only by the fire brigades but also possibly by other people. Such a tragic incident occurred in Switzerland in 2004 when seven members of a fire brigade were killed by the sudden collapse of the concrete structure in an underground car park in which they were present after having successfully fought the fire [7].

The two reasons that load-bearing capacity may continue decreasing after the peak gas temperature is reached explain why concrete structures are more prone to this phenomenon than steel structures. This article analyzes reinforced concrete columns subjected to natural fires in order to verify that the possibility of structural collapse during or after the cooling phase is real and, if so, what the parameters and conditions are that more likely lead to this undesirable behavior.

Analysis for reinforced concrete columns

The time–temperature fire curves in Figure 1 were taken from the parametric fire model of Annex A in Eurocode 1 [8]. The factor Γ that appears in Equation (A.2 a) of this article was given the value of 1.0, which makes the heating phase of the time–temperature curve of this natural fire model approximate the standard curve. Figure 1 shows the different fire curves that were used, differing from each other by the duration of the heating phase, from 15 to 240 min.

In the discussion of the results, the following terms will be used:

Phase 1 is the heating phase of the fire, when the gas temperature is increasing from 20°C to the maximum temperature. The duration of phase 1 is t_{peak};

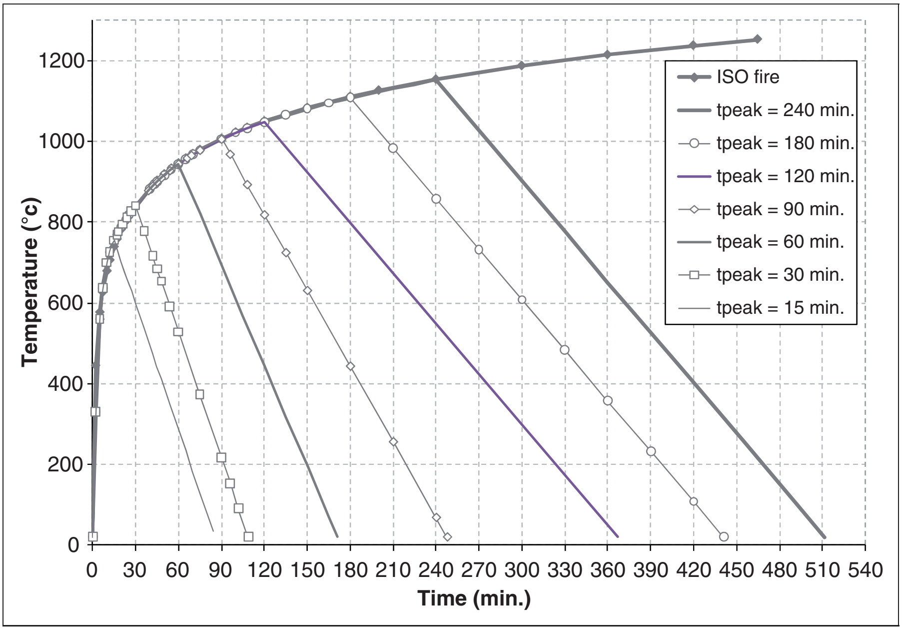

Figure 1. Different fire curves considered.

Phase 2 is the cooling phase of the fire, when the gas temperature is decreasing from the maximum temperature to 20°C. The end of phase 2 takes place at time, t_{20};

Phase 3 is the phase after the fire, when the gas temperature is back to 20°C.

The analyses were performed on reinforced columns exposed to the fire on three sides, with the fourth side having adiabatic conditions. This may be representative of a column along the side wall of a building, with the fourth side protected from the effects of the fire by the façade. The nonlinear finite element software SAFIR developed at the University of Liege [9] for the simulation of structures subjected to fire was used. The temperature distribution in the sections was determined by 2D nonlinear transient analyses, as shown in Figure 2. For the structural analysis, the columns were discretized longitudinally by means of Bernoulli beam type elements and the cross sections of the beam elements are divided into fibers that match the 2D elements of the thermal analysis.

Columns were chosen because the loss of concrete strength and stiffness is more detrimental in column like elements than in beam like elements, for which the behavior is more driven by the properties of the steel reinforcement.

The parameters that were considered in this study are:

1. The duration of the heating phase of the fire.
2. The effective length of the column.
3. The section of the column.
4. The duration of the cooling phase.

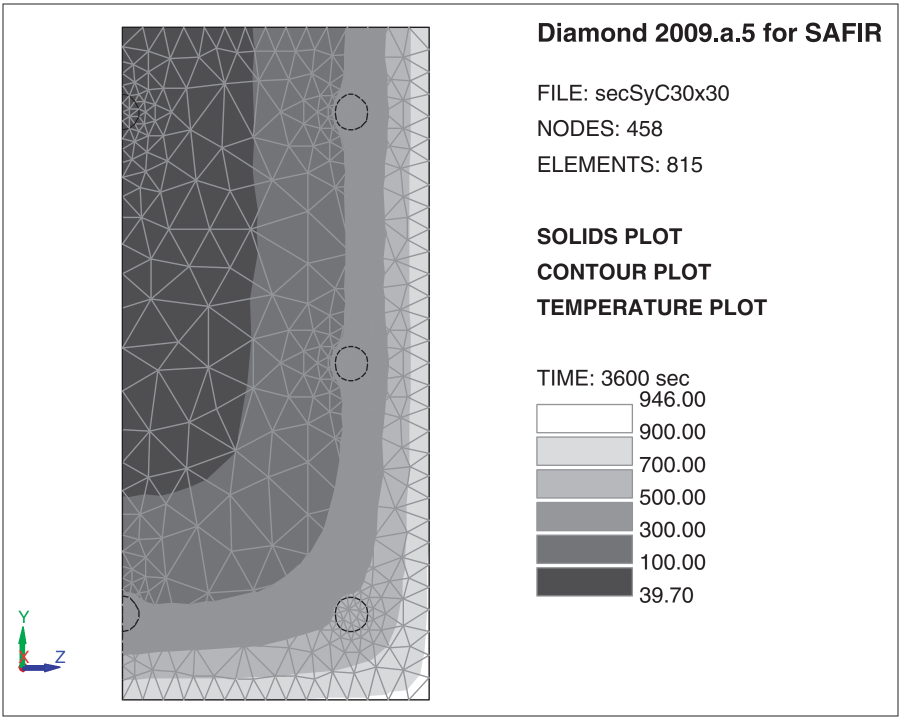

Figure 2. Isotherms after 60 min in a section heated on three sides (1/2 modeled).

Many other parameters might influence the possibility of a delayed collapse such as, for instance, the mechanical properties of the materials that make up the column. In this study, only those parameters are considered that can be evaluated, even if in a qualitative manner, by fire fighters arriving on site.

The section analyzed as the basic case has a 300×300 mm side section with 8 bars of 16 mm diameter and a concrete cover of 30 mm. The column is simply supported at both ends with a length of 4 m. A sinusoidal imperfection with maximum amplitude of $L/300$ has been introduced in the direction of the thermal gradient (leading to bending along one single axis). The results reported here are for the eccentricity toward the unexposed side of the section.

Simply supported concrete columns are commonly found in one floor industrial or commercial buildings. In moment resistant frames found in multiple-floor residential or office buildings, the columns are subjected to bending moments, typically with a bi-triangular distribution, while at the same time supported by a certain degree of rotational restraint. Such a situation cannot be evaluated by analyzing single columns because it is not possible to impose, for the degree of freedom linked to rotation at the ends of the element, the degree of rotational restraint and the amplitude of bending moment at the same time. Only one condition can be applied to any degree of freedom. The question of whether delayed failures are possible in moment resisting frames can only be addressed by the analysis of complete frames, which is beyond the scope of this study.

Material models

The reinforcing bars were represented in the model and the thermal properties of steel and concrete in the heating phase were taken from EN 1994-1-2 [10]. This means that thermal conductivity of concrete was taken with the upper limit in the sense of EN 1992-1-2 [11]. Siliceous concrete was chosen, with a density of $2400\,kg/m^3$ and a water content of $46\,kg/m^3$. The emissivity was taken as 0.7 and the coefficient of convection was $35\,W/m^2K$. Thermal properties of steel were considered as fully reversible during cooling.

When concrete is heated, large amounts of water evaporate. The moisture that is released in the fire environment dissipates in the open air, being entrained outside the compartment through openings with the flow of combustion products. This water is thus not available and does not re-enter in the material during cooling and, furthermore, part of the water that is released during heating was chemically bound and the physico-chemical reactions that liberate the molecules of water are not reversible. It has thus been considered in the model that the energy consumed for the evaporation is not recovered during cooling. The specific mass of concrete, which decreases during heating because of the release of water, has been considered as constant during cooling, with a value that corresponds to the value at the maximum temperature. When the temperature increases in the concrete, the thermal conductivity has a tendency to decrease [10]. According to Schneider [12], this loss in conductivity is also associated with the loss of moisture during heating. It has thus been assumed that the decrease of thermal conductivity is not reversible and, during cooling, the thermal conductivity of concrete keeps the value corresponding to the maximum temperature.

The mechanical properties of the steel reinforcing bars have been considered as reversible, which means that stiffness and strength are recovered to full initial values during cooling. In the thermal elongation curve, the plateau corresponding to the phase change that occurs around 800°C at a level of 11×10^{-3} during heating occurs at slightly lower temperatures, around 700°C, at a level of 9×10^{-3} during cooling. When steel is back to ambient temperature, there is no residual thermal expansion. If yielding has occurred in the bars during the heating process, it is not recovered during heating; the plastic strain will not decrease during cooling.

For concrete, a residual thermal expansion or shrinkage has been considered when the concrete is back to ambient temperature. The value of the residual value is a function of the maximum temperature and is given in Table 1, taken from experimental tests made by Schneider in 1979 and mentioned in Schneider [12]. Negative values indicate residual shortening whereas positive values indicate residual expansion.

Compressive strength of concrete does not recover during cooling. According to EN 1994-1-2, an additional loss of 10% has been considered during cooling.

Table 1. Residual thermal expansion of concrete.

T_{max} (°C)	$\varepsilon_{residual}$ (20°C) (10^{-3})
20	0
300	−0.58
400	−0.29
600	1.71
800	3.29
≥900	5.00

This means that, for example, if the compressive strength has decreased from 1.00 to 0.50 at a given temperature, it will decrease to 0.45 when cooling back to ambient temperature. This assumption is of course the key for all the predictions presented in this article and thus affects the reliability of the conclusions. In a recent paper, Yi-Hai and Franssen [13] have shown, from the analysis of hundreds of experimental results reported in the literature, that the additional reduction during cooling may be even greater than the 10% reduction considered in Eurocode 4. In the stress–strain relationship of concrete, the strain corresponding to the peak stress was considered during cooling as fixed to the value that prevailed at the maximum temperature [14]. This hypothesis is also present in Fig. C.2 of Eurocode 4 [10].

Influence of the duration of the heating phase

Figure 3 summarizes the analyses performed to examine the influence of the duration of the fire.

Each curve in the Figure is related to one of the fires shown in Figure 1 and is the result of numerous simulations performed with different load levels. From the load-bearing capacity at time $t = 0$, here 2220 kN, the load has been progressively reduced and numerous simulations have been performed, each one for a different load, yielding a fire resistance time that increases as the load is decreased. In a real building, the columns are subjected to different loads, the level of which is not known to the fire fighters. What each curve shows is that, for the curve marked as '$t_{peak} = 15$ min' for example, if a column is subjected to a load of 1600 kN, it will fail after 60 min. Each of these curves shows whether there is a potential dangerous range of the load that could lead to collapse during or after the heating phase. If this is the case, the fact that a delayed collapse will occur or not in a building will depend on the actual load level on the columns.

The load that gives a fire resistance time equal to the duration of the heating phase t_{peak} is marked on the curve; for the fire with a heating phase of 15 min, this load is 1970 kN. If the load applied on the column is greater than 1970 kN, failure will occur during the heating phase of the fire. For loads less than 1970 kN, failure of the column occurs in the cooling phase of the fire, at least when the load is greater than a value noted as t_{20} on the curve; for the 15 min fire, this failure time of 86 min is obtained for a load of 1520 kN. For loads less than 1520 kN, collapse occurs in phase 3 of the fire, when gas temperature is back to ambient. For the same fire, collapse can occur as late as 270 min after the beginning of the fire, that is, more than 3 h after the end of the fire, for a load of 1390 kN. Any load less than 1390 kN will not lead to the collapse of the column, which is marked by the fact that the curve has a horizontal asymptote.

For the fire that has a heating phase of 240 min, any load greater than 229 kN will lead to collapse during the heating phase, that is, in less than 240 min. If the load happens to be between 170 and 229 kN, collapse will occur during the cooling phase. If the load is less than 170 kN, there will be no collapse of the column and indefinite stability is ensured; there is no possibility of collapse after the cooling phase for this fire duration.

It is thus possible, according to this model, to have in certain cases a structural collapse several hours after the end of the fire. According to Figure 3, this can occur only if the duration of the heating phase is not greater than 90 min. In fact, for longer fires, the horizontal asymptote starts for times shorter than t_{20} which means that, for these longer fires, if the load is so low that failure did not occur during the cooling phase, failure will not occur after the cooling phase. It can also be observed that the critical load range, that is, the range of loads leading to a failure in phase 3

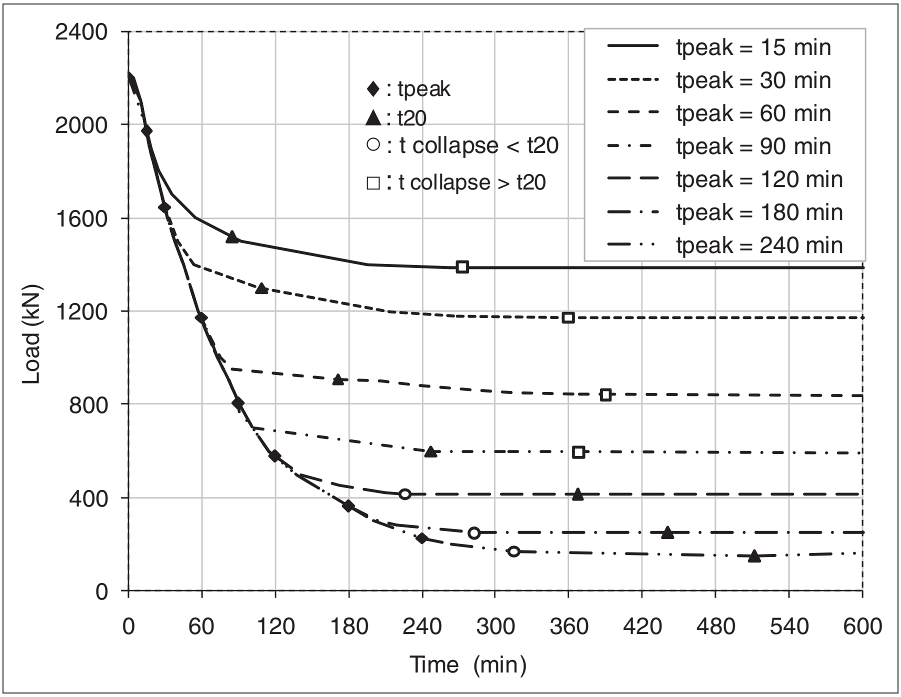

Figure 3. Influence of the duration of the heating phase (1).

of the fire, is broader for short fires and decreases as the duration of the fire increases.

Figure 4 shows another presentation of the same results. On the horizontal axis is the duration of the heating phase of the fire whereas the applied load is on the vertical axis. Each curve of Figure 3 is on a vertical line in Figure 4 but, in Figure 4, only the characteristic points of the curves of Figure 3 have been marked. The plan is divided into four regions. Above the upper line is the region corresponding to failures in phase 1, the heating phase (e.g., above 1970 kN for a fire duration of 15 min and above 229 kN for a fire duration of 240 min). Under this line is the region of failures during phase 2, the cooling phase. Below the lowest line is the region that does not lead to collapse at all and the intermediate region. Existing only for fire durations shorter than 90 min, the intermediate region is dangerous since collapse can occur in phase 3, when the compartment is back to ambient temperature conditions. The fact that the band corresponding to the dangerous situation is so narrow explains why the ratio of structural collapse of buildings after the fire to that of the total number of building collapses is low. This is where the possible large consequence of the event, rather than the probability of occurrence, leads to a significant risk.

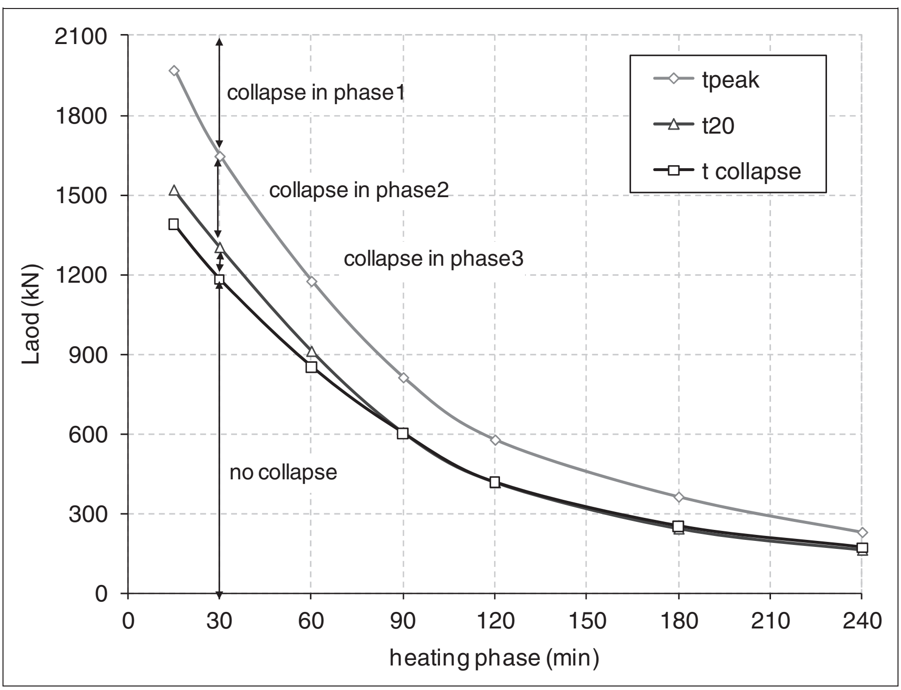

Figure 4. Influence of the duration of the heating phase (2).

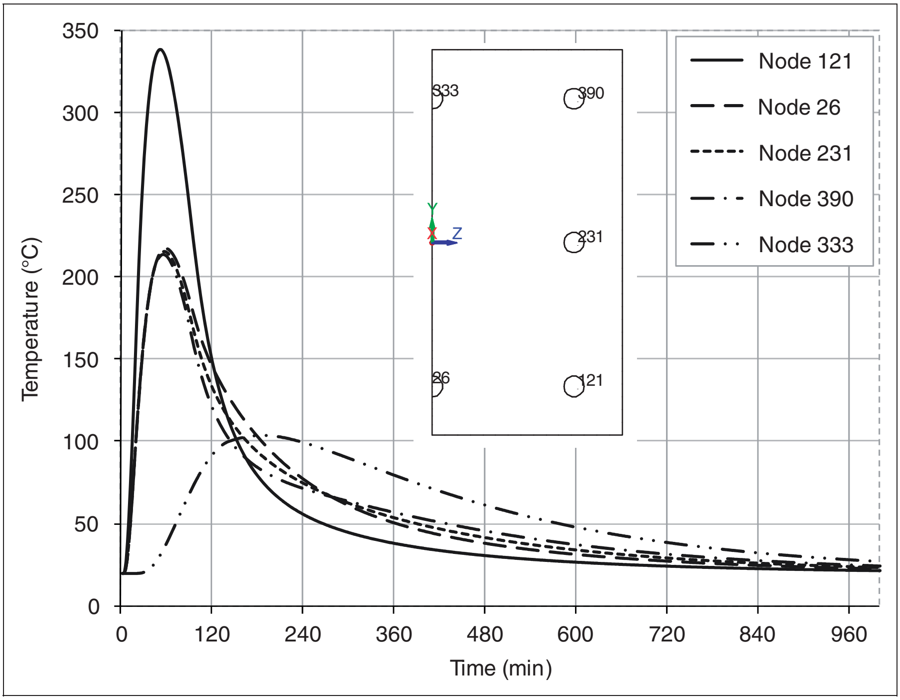

Figure 5. Evolution of the temperature in the bars for a 15 min fire.

Figure 5 shows the evolution of the temperature in the steel bars for a fire with a heating phase of 15 min, returning to 20°C after 86 min. Five curves only need to be plotted for the 8 bars of the section because of the symmetry in the temperature distribution in the section (Figure 2). Node 121 belongs to the bar in the heated corners; node 333 is for the bar in the center of the adiabatic boundary and the remaining three curves are for the other bars. It can be observed that, in all bars, the temperature continues increasing for at least 30 min after the time of maximum gas temperature and, for the bar that is away from the heated surfaces, the temperature keeps on increasing significantly for more than 60 min after the end of the fire. Similar behavior could be plotted for different points in the concrete material. This delayed temperature increase in the internal parts of the section, plus the additional 10% decrease in compressive strength when concrete cools down to ambient, explain the delayed failures.

The evolution of the temperature in the same locations has been plotted in Figure 6 for the fire with a heating phase of 180 min, returning to ambient temperature after 441 min. For this longer duration fire, all temperature curves have almost reached their maximum value when the gas temperature is back to 20°C, at 441 min. This can explain why, in longer fires, there is no failure after the cooling phase of the fire because, if the temperature in the steel bars does not increase

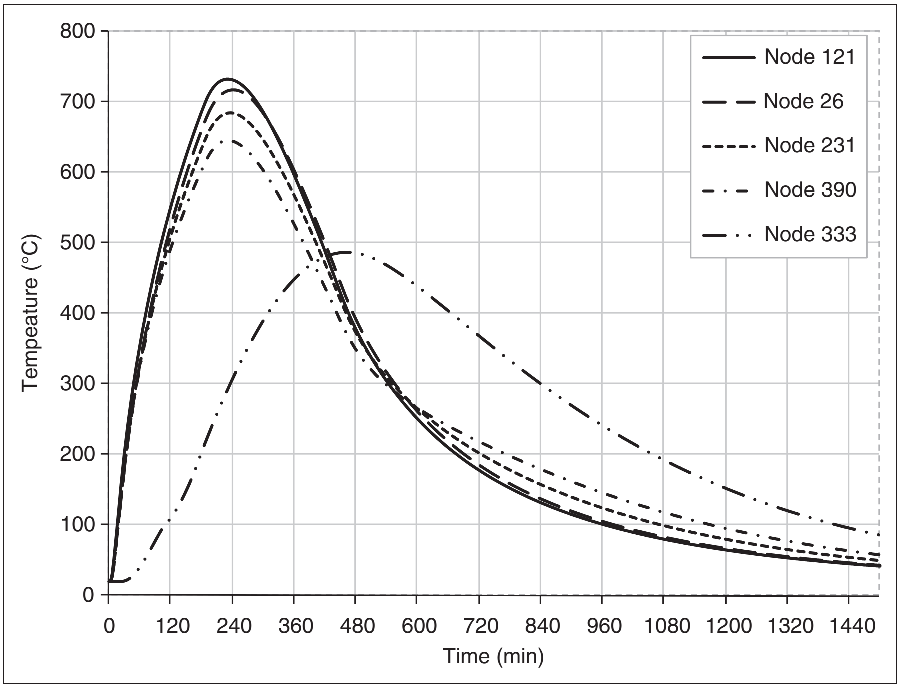

Figure 6. Evolution of the temperature in the bars for a 180 min fire.

further after the end of the fire, the steel properties have started to recover while, in the concrete, most of the decrease of strength and stiffness has already developed.

Influence of the effective length of the column

The analyses have been repeated for different lengths of the column and the results are summarized in Figure 7 for a heating phase that lasts for 60 min. Failures in phase 3 of the fire are observed only in columns with a length of 5 m or less. For longer columns, failure can be observed in phase 2, the cooling phase, but the range of loads leading to collapse in phase 2 is reduced as the length of the column increases. For a 9 m long column, nearly all failures are in the heating phase; if such a long column can survive to the heating phase of the fire, it is very likely to survive indefinitely.

For a longer fire with a heating phase of 120 min, no collapse during phase 3 was observed, even for short columns.

Figure 8 shows the evolution of the lateral displacement in a 4 m long column subjected to a fire with a heating phase of 60 min. Three different loads have been applied, one that leads to failure exactly after 60 min when the gas temperature is at the peak (curve noted 't_{peak}'), one that leads to failure at the end of the cooling phase when the gas temperature is back to 20°C (curve noted 't_{20}') and one that

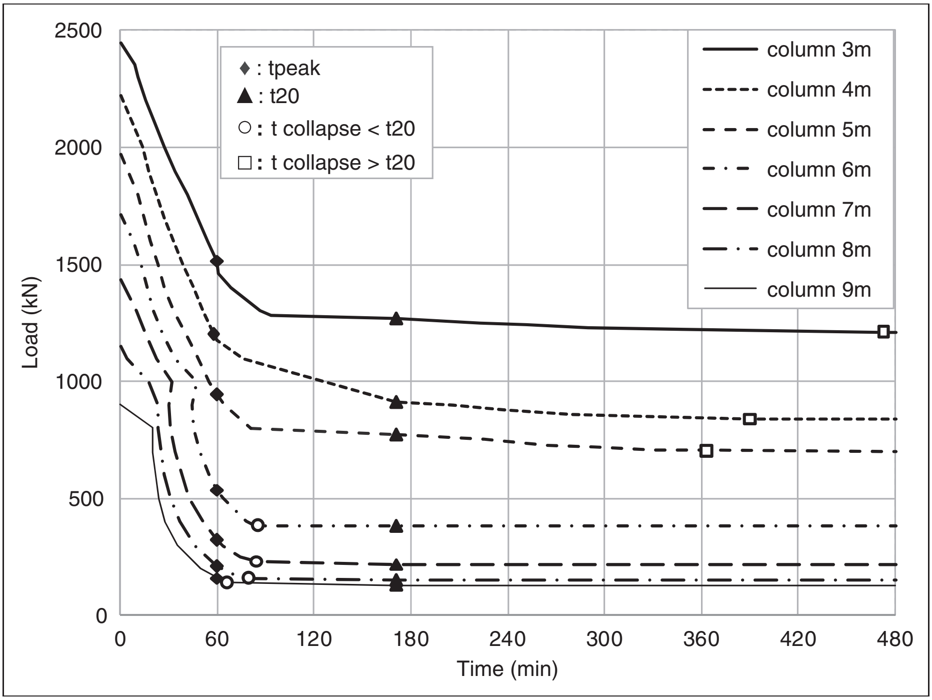

Figure 7. Influence of the length of the column.

leads to failure after more than 6 h (curve noted 't_{collapse}'). It can be observed that the displacements are toward the fire in the first minutes of the fire, driven by thermal gradients. After a while, heating of the material in the regions of the section near the fire leads to a decrease of stiffness in these regions and the effective neutral axis (with respect to stiffness) in the section moves away from the fire.

The displacement of the neutral axis depends only on the temperature distribution in the section, which is the same for all column lengths. The lateral displacement of the column induced by the thermal gradient is proportional to the second-order power of the length of the column and, in a short column, this displacement is rather limited. The second-order bending moments are, in each section, proportional to the eccentricity of the load, which is the difference between the displacement due to the thermal gradient (in one direction) and the displacement of the neutral axis (that develops in the other direction). In the cooling phase of the fire, as long as the temperature increases in the column (Figure 5), the decrease of bending stiffness is overwhelming compared to the relatively modest influence of the decrease in thermal bowing that occurs simultaneously. In a short column, the eccentricity changes sign when the neutral axis moves and so do the displacements generated by the second-order moments.

If the evolution of the lateral displacement is plotted for a longer column (Figure 9), it is observed that the effects of the thermal gradient (proportional

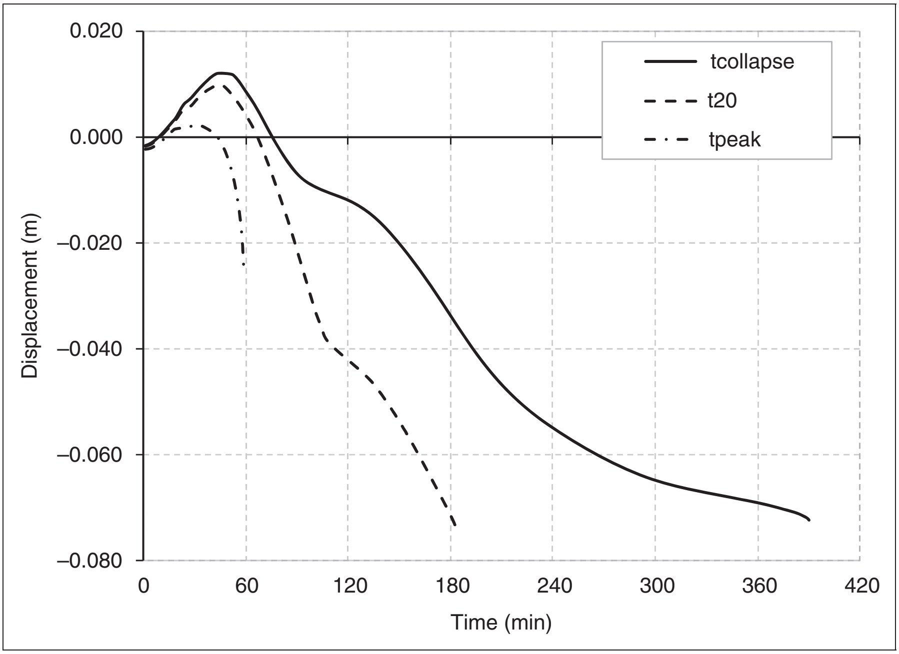

Figure 8. Lateral displacement at mid height for a 4 m column.

to L^2) are so important that the variation of position of the neutral axis is not sufficient to counteract the thermal curvature; the displacements remain in the direction of the fire. In that case, as soon as the temperature decreases in the regions of the section near the fire, the severity of the thermal gradient decreases and so does the lateral displacement. The column is then more likely to survive, even if the strength and stiffness continue decreasing a little bit more in phase 2 and 3. This explains why longer columns that survive to the heating phase are in a better position to survive also the cooling phase and, even more certainly, phase 3 of the fire.

Influence of the section of the column

The analyses have been repeated for a 4 m column with a section of 600 × 600 mm side containing 12 bars of 20 mm diameter with a concrete cover of 40 mm. The results are presented in Figures 10 and 11. Whereas failure in phase 3 of the fire was observed only for relatively short fires when the 300 × 300 mm side section was considered (Figure 3), delayed failures are observed here even for very long fires with a heating phase as long as 4 h; in Figure 11, to be compared to Figure 4, the band of dangerous collapses in phase 3 extends towards fire durations of 240 min, but for only 90 min with the smaller section. The range in loads leading to these failures after the fire is rather limited. For example, for the 120-min fire, failures after cooling can occur only if the load is between 5670 and 5270 kN.

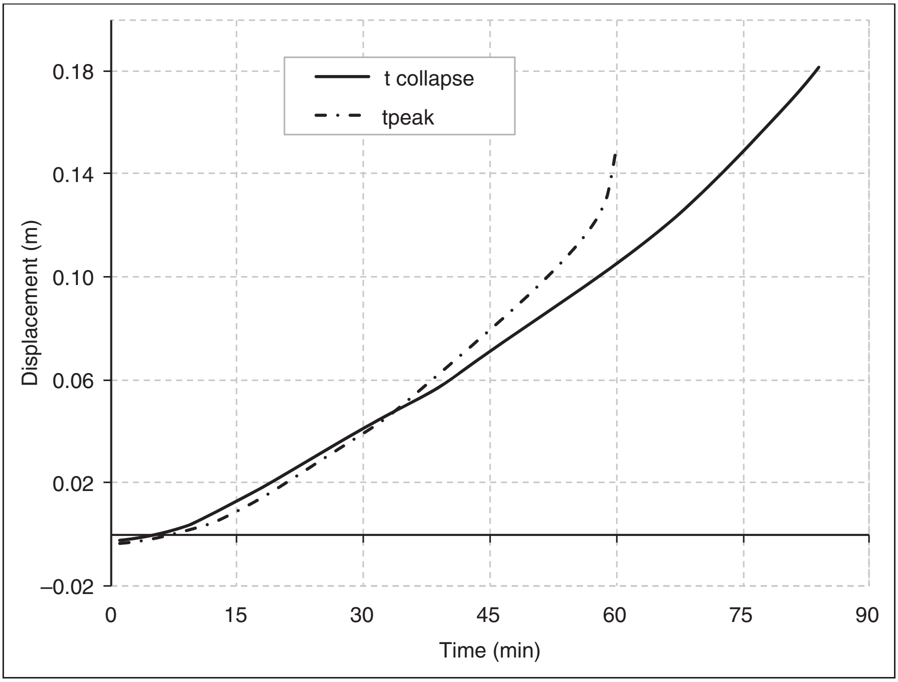

Figure 9. Lateral displacement at mid height for a 6 m column.

The dangerous range is reduced as the duration of the fire increases. More critical is the fact that, if the load happens to be in the dangerous range, delayed failures can occur as late as 15 h after the end of the fire.

Two reasons can explain the higher sensitivity of wider sections to delayed failure. The first is linked to the fact that, if the length of the column remains constant, wider sections lead to lower slenderness, which has been identified as more critical in the previous section of this article. The second reason may be traced to the fact that, in larger sections, the temperatures in central zones of the section keep on increasing long after the gas temperature is back to ambient, which is not the case for smaller sections (Figure 6).

Delayed failures are thus strongly linked to the presence of a certain zone of the section in which the temperature continues to increase after the peak of the gas temperature. This may explain why, when a 4 m long column with a section 300×300 mm was analyzed as heated on four sides, no delayed failure could be observed.

Influence of the duration of the cooling phase

It has been shown that the fires with the shortest heating phase are the most dangerous for the risk of collapse after the cooling phase. For a given

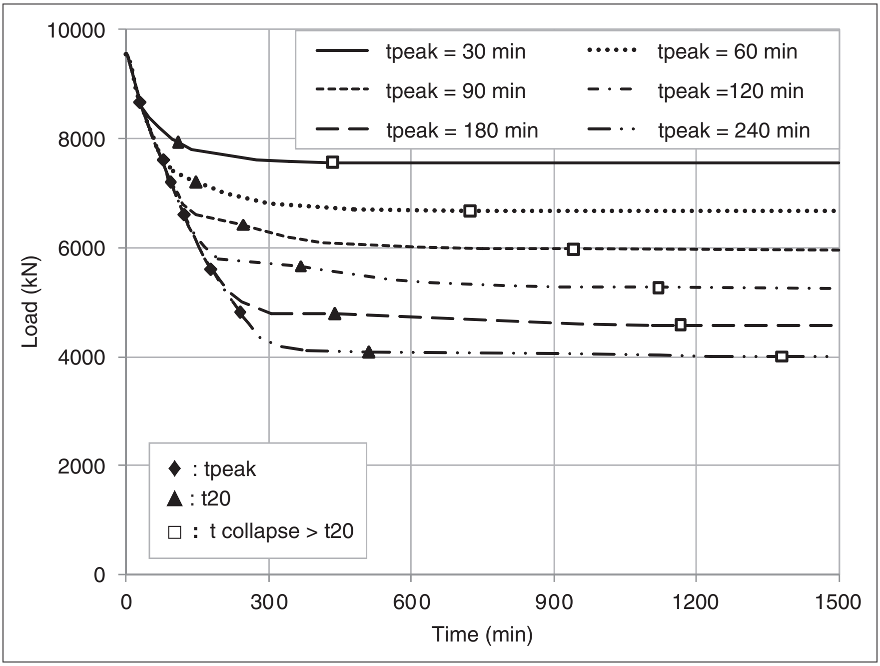

Figure 10. Evolution of the fire resistance for a 600×600 mm^2 side section.

duration of the heating phase, the influence of the duration of the cooling phase has been examined. This could have practical implications for the fire fighting strategy; is it more important for the firefighters to stop the fire as quickly as possible or to have it decreasing progressively, from the point of view of the risk of collapse after the cooling phase? Different fires with a heating phase of 45 min and cooling phases ranging from 0 to 195 min were considered (Figure 12).

Figure 13 shows that, when the fire can be extinguished rapidly, the structure can survive with higher loads than when the fire is allowed to cool down slowly. If the load is 1400 kN, for example, stopping the fire in less than 15 min allows the column to survive indefinitely. Extinguishing the fire within 45 to 75 min leads to a failure after extinguishment, some 150 or 60 min after the end of the fire, respectively. Any slower cooling will lead to a collapse during cooling, approximately 30 min after the beginning of the cooling phase.

It is thus probably a good idea to decrease the temperatures in the compartment as fast as possible in order to increase the probability of survival for the structure. It has nevertheless to be noted that, when the fire has been extinguished, there is dangerous period of about 6 to 7 h during which a delayed collapse can occur, depending on the load level. This period is more or less the same for all cooling rates.

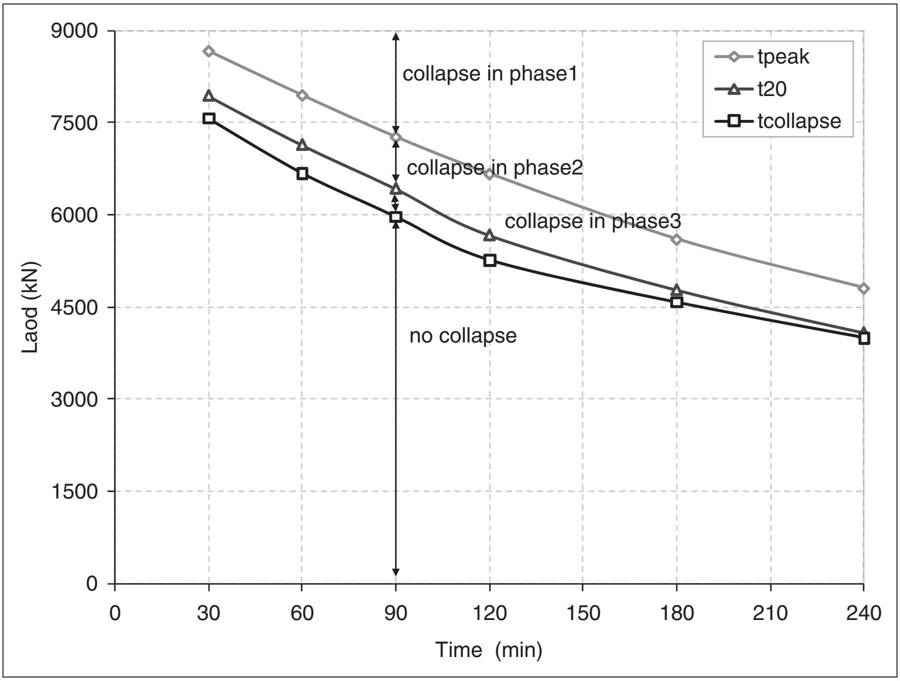

Figure 11. Failure in different phases for a 600x600 mm^2 side section.

Conclusions

The numerical analyses undertaken here for concentrically loaded simply supported concrete columns heated on three sides show that a failure during the cooling phase of a fire is a possible event and, more dangerous, that a failure of the structure is still possible when the fire has been completely extinguished, in some cases several hours after conditions in the fire compartment have become tenable again and a first inspection of the building might be under way.

The main mechanisms for these delayed failures are to be found in the fact that temperatures in the central zones of the element can keep on increasing even after the gas temperature is back to ambient and also in the fact that concrete loses additional strength during cooling compared to the situation at maximum temperature. It has been shown that the most critical situations with respect to delayed failure arise for short fires and for columns with low slenderness (short length and/or massive section).

Rapid cooling of the gas temperature increases the probability for the column to survive the fire indefinitely. Yet, after the fire has been put out, there is a period of several hours during which the column is still under danger of collapse if the load is sufficiently high, whatever the speed with which the fire has been extinguished.

As generic constitutive models have been used here for steel and for concrete, it does not really make sense to compare the results obtained from the simulation

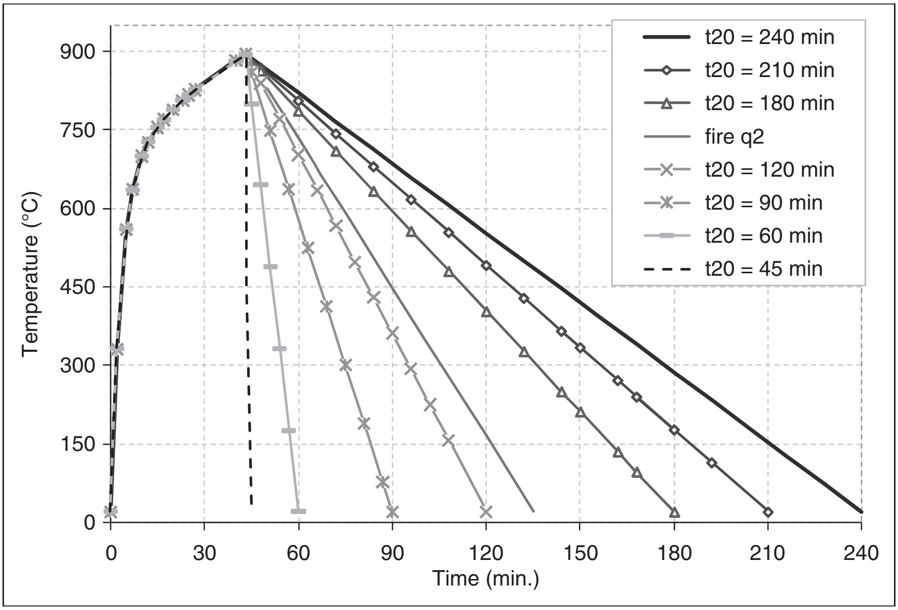

Figure 12. Fires with different cooling phases.

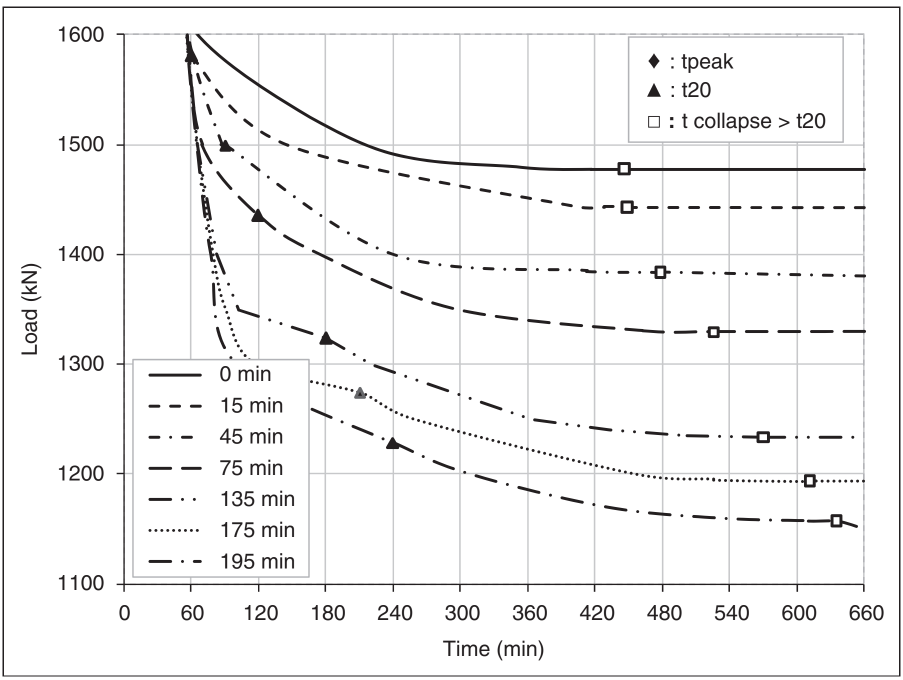

Figure 13. Evolution of the resistance for different durations of the cooling phase.

with the results of experimental tests (provided that such results are available). A generic model is typically used for a structural analysis; that means to examine the behavior of structural elements, on the basis of a material model that is widely accepted and considered in the scientific community as reproducing, on the average, the type of material behavior observed for most concrete mixes. By using the same material model as used in other structural analyses, any dissatisfaction with the results cannot be blamed on the choice of material model.

Still, a very strong assumption of the concrete constitutive model of the Eurocodes used in this analysis is that the effects of transient creep can be incorporated implicitly in the mechanical strain term. Although this assumption has proven to yield quite accurate results in structures heated by a standard time–temperature curve, its validity for representing the behavior of concrete during a cooling phase may be questioned because transient creep is, in reality, not reversible. Recent numerical simulations have been made by the authors on the same columns as the one studied here, but now with an explicit transient creep constitutive model [15,16]; it appears that the occurrence of collapse during or after the cooling phase is predicted even more often with an explicit creep model than with the implicit model of Eurocode 2 that has been used for the results presented in this article.

The question whether delayed collapse is also possible in other types of structures such as, for example, in moment resisting frames, has not been addressed and could be the subject of other studies in the future.

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Nomenclature

Γ	=	time factor used with the parametric time–temperature curves of Eurocode
t_{peak}	=	time corresponding to the end of the heating phase - duration of the heating phase of the fire
t_{20}	=	time corresponding to the end of the cooling phase – duration of the fire (heating and cooling)
L	=	length of the column
T_{max}	=	maximum temperature
$\varepsilon_{residual}$	=	residual thermal expansion of concrete at 20°C as a function of the maximum reached temperature
t	=	time
$t_{collapse}$	=	fire resistance time for the lowest load that induces the collapse of the column for the considered fire

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