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A Performance Indicator for Structures under Natural Fire

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Highlights

- Structural failure may occur during or after the cooling phase of a fire.
- All typologies of structural components are concerned by delayed failures.
- An indicator is proposed to quantify a component vulnerability to delayed failure.
- It is used to compare the performance of different components in natural fire.

ABSTRACT

Fires in buildings are characterized by a heating phase followed by a cooling phase, yet the effects of the latter on structures are not well covered in the current approaches to structural fire engineering. Indeed the actual requirement of non-occurrence of structural failure at peak temperature does not guarantee against a delayed failure during or after the cooling phase of a fire, which puts at risk the fire brigades and people proceeding to a building inspection after a fire. Therefore there is an urgent need to better comprehend and characterize the materials and structures behavior under decreasing temperatures. Sensitivity to delayed failure of a structural component depends on its typology and constituting materials. In particular, two structural components with the same Fire Resistance rating (R) under standardized fire may exhibit very distinct behavior under natural fire, one of them being more prone to delayed failure than the other. With the aim of quantifying this effect, a new indicator is proposed that characterizes the performance of structures under natural fire conditions. The paper presents the methodology to derive this new indicator as well as results for different typologies of structural components. Parametric analyses highlight the prime influence of constitutive material and thermal inertia of the element on the post-peak behavior. Used in conjunction with the Fire Resistance rating, it is shown how the new indicator carries additional and significant information for classifying structural systems in terms of their fire performance and propensity to delayed failure.

Keywords: Natural Fire; Structures; Fire Resistance; Performance-Based Design; Temperature; Concrete; Steel; Timber; Cooling

1. Introduction

Fire Resistance rating (R) has been widely used as the reference indicator for assessing the performance of structures in fire. It is defined as the duration during which a structural component fulfils predefined criteria with respect to structural integrity, stability and temperature transmission [1] under monotonically increasing standardized fire conditions. It has been used for several decades in fire engineering [2-4] and is the reference indicator of fire performance in many codes [5-6]. Practically, it gives comparative information for components in which the fire-induced temperatures would be monotonically increasing. The Fire Resistance is therefore a convenient and efficient indicator to characterize a structural component by mean of a single scalar, the quantity of information provided being deemed as sufficient in a prescriptive environment.

In a performance-based environment, a more realistic representation of the fire may be used that comprises a heating phase followed by a cooling phase during which the temperature in the compartment is decreasing back to ambient temperature. The influence of such realistic fire scenarios on the behavior of structural components is a key issue in the performance-based approach, as shown for example for concrete-filled hollow structural section columns [7] or for single-plate shear connections in which the tensile forces created during the cooling phase can lead to failure [8]. 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, for instance in high-rise buildings due to requirements related to the egress time [9]. However, the fact that the structure exhibits stability at the time of maximum gas temperature does not guarantee against failure at a later stage. Typically, the load-bearing capacity of a 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. This delayed decrease in load-bearing capacity may be caused by the combination of various phenomena among which the delayed temperature increase in the sections due to thermal inertia and the non-recovery or additional loss of material mechanical properties during cooling. A previous research carried on the behavior of reinforced concrete columns under natural fires has indicated that there was a possibility of structural failure during and even after the cooling phase of a fire [10].

Natural fire scenarios are thus associated with a threat that was entirely disregarded as long as standardized fires were considered: the possibility of a structural failure arising after the time of maximum temperature in a compartment, referred to as delayed failure hereafter. A delayed failure has been observed, for example, in a full-scale fire test on a composite steel and concrete floor conducted in 2008 in the Czech Republic [11]. In 2004 in Switzerland, a delayed failure of an underground car park killed seven members of a fire brigade, who were present in the car park after having successfully fought the fire when the concrete structure suddenly collapsed [12]. Yet research works on structural behavior after the time of maximum temperature are very scarce and focus mainly on residual load-bearing capacity [13-16]. It is the opinion of the authors that more attention should be brought to the structural behavior during and right after the cooling phase of the fire, because the vulnerability of a structure is important in these phases during which the elevated temperature has not dissipated yet. In this purpose the definition of a suitable measure of performance for structures under natural fire is needed to allow for comparative analyses. Clearly, the Fire Resistance indicator is not suitable to characterize a structure sensitivity to delayed failure, as it is based on monotonically increasing temperatures. Therefore, a new indicator is proposed in this paper to complement the information carried by the Fire Resistance with information about the behavior under natural fire conditions. This indicator allows comparing and classifying structural systems in terms of their sensitivity to delayed failure.

The next Section presents the theoretical basis supporting the indicator definition. The definition allows associating an unequivocal value of the indicator for any given structural component under a given load. The method to derive the indicator is described in Section 3.

In Section 4, the indicator is numerically assessed for a series of applications with concrete, steel and timber materials. In a previous research [10], it was shown that for concrete columns the situations of delayed failure were more likely to arise for short-duration fires and columns with low slenderness or massive sections. It is shown here how these results can be interpreted in terms of the new indicator. The study is extended to other typologies of structural elements for comparing these typologies in terms of performance under natural fire. The results are discussed in Section 5.

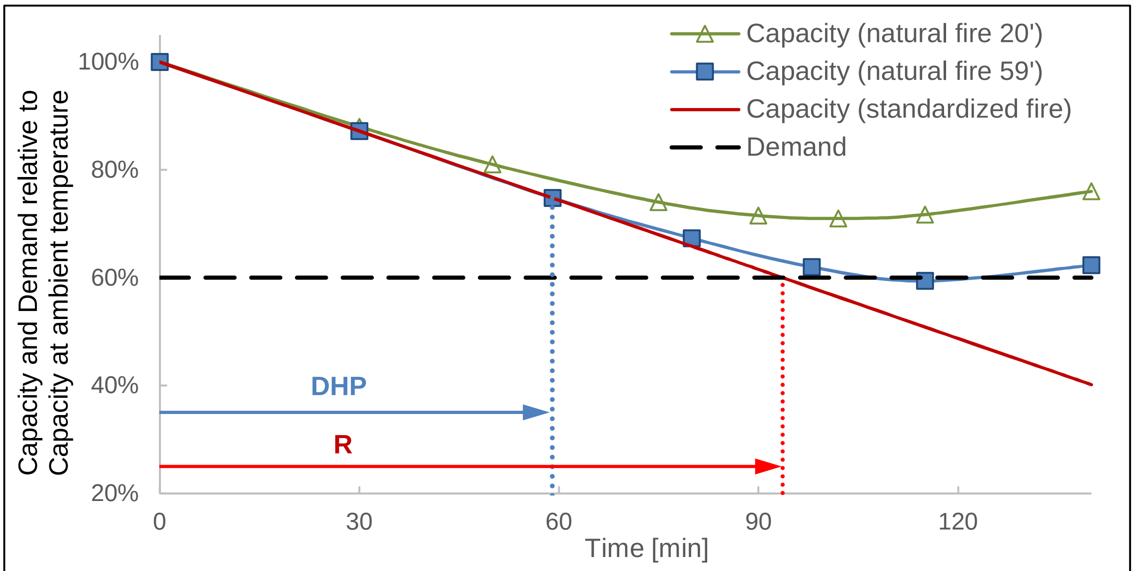
2. Theoretical Definition of the Indicator

2.1 Capacity Evolution under Standardized and Natural Fire

Let us consider a structural component subjected to a certain load (demand) which is considered constant during the fire. During the fire, the temperature increase across the section of the component leads to a decrease of the mechanical properties of the constituting materials, resulting in a decrease of the load-bearing capacity of the component.

For a standardized fire, the temperature is continuously increasing in the compartment, so that the temperatures in the element are also continuously increasing and, assuming that all materials properties degrade under increasing temperatures, the load-bearing capacity is continuously decreasing. Failure occurs at the time when the capacity meets the demand, this time being defined as the Fire Resistance of the component. This is illustrated by the red curve in Figure 1 where, for the sake of simplicity, capacity is assumed to decrease linearly over time, which is usually not the case in practice.

For a natural fire, the temperature in the compartment or, more generally speaking, the thermal solicitation to the elements, is first increasing until a maximum and then decreasing back to ambient conditions. In that case, the load-bearing capacity of the component is first decreasing until reaching a minimum and then it may remain constant or recover, partially or completely, after the temperature has come back to ambient. Importantly, the time of the maximum fire temperature and the time of the minimum load-bearing capacity are generally not simultaneous, the latter arising later than the former. Contrary to standardized fire in which failure will always happen, failure under a natural fire situation depends on the severity of the natural fire. Figure 1 shows the evolution of capacity of a structural component for two different natural fires, the heating phase of which follows the standardized ISO fire for a duration of 20 minutes and 59 minutes respectively; failure arises for the longer natural fire only.



Fire Resistance (R): minimum exposure duration to a *standardized fire* that leads to failure *at that time*.
Duration of the Heating Phase (DHP): minimum exposure duration to a *natural fire heating phase* that affects the component to such an extent that it will fail *later*.

Figure 1. Structural failure occurs if the capacity meets the demand. Under natural fire, this may occur during or after the cooling phase.

The following observations can be drawn, illustrated by this hypothetical example:

- Given a structural component submitted to a certain load, the component will fail under some natural fires while it will not under others, depending on the severity of the fire. The severity is a measure of natural fires to be established; it will be discussed later.
- Consequently, for any structural component submitted to a certain load, a "critical natural fire" can be defined as the natural fire with minimum severity that will lead to the failure of the component.
- This "critical natural fire" may have a duration of the heating phase shorter than the Fire Resistance of the component; the fact that a structural component is R90 does not give sufficient information to conclude whether the component will fail if it is subjected to a natural fire with a heating phase of, say, 80 minutes.
- The failure that occurs under a natural fire may arise long after the Fire Resistance time of the component, as seen for instance in Figure 1 by comparing the time when the capacity meets the demand for the standardized fire and for the 59 minutes natural fire.

Note that the load in a structural component does not necessarily remain constant during a fire. The elevation of temperature in the materials produces thermal elongations together with a reduction of strength and stiffness, which may cause restraint forces when considering the interaction with the surrounding structure. As a result, the demand that is plotted in Figure 1 could be a curve (in all generality) instead of a horizontal line. The demand was assumed constant in this hypothetical example to simplify the discussion. Assuming otherwise would not change the discussion of this section; in particular, it remains true that the capacity can meet the demand during the cooling phase of a fire which heating phase was shorter than the Fire Resistance of the component.

Hence, the Fire Resistance does not give enough information to characterize the performance of structures under natural fire and a new indicator is needed to complement it. This indicator must be related to a certain measure of severity of the natural fire.

The following section discusses the characterization of natural fires by a severity measure. Then the new indicator based on this severity measure is introduced in Section 2.3.

2.2 Measure of Severity of a Natural Fire

Natural fires in a compartment can be characterized simply by a temperature-time relationship. Due to the variability in the parameters that affect the natural fire, an infinity of time-temperature relationships can be obtained in theory. For condensing the information contained in the full temperature-time curve into a characteristic measure of severity, several indicators can be considered, for instance the maximum temperature of the fire, the duration of the heating phase, the total duration of the fire until the temperature comes back to ambient, or certain measures of energy released by the fire.

Although each of these measures presents its advantages, it was decided here to select the duration of the heating phase of the natural fire as the severity measure. This choice is supported, first, by the will to work with a measure in time unit, for consistency with the Fire Resistance indicator which also represents a duration. Therefore, the two indicators can be easily compared. Secondly, the duration of the heating phase of a natural fire has a direct practical significance and can be easily comprehended by the different stakeholders involved in fire safety, including the fire fighters who can relate it with the time of their intervention. Also, this is a quantity that, to some degree, can be estimated on site during a real fire.

In order to compare different natural fires based on a single scalar measure, a set of standard natural fire curves must be defined, with the only parameter being the duration of the heating phase. The parametric fire model of EN 1991-1-2 [5] was adopted here to define the set of standard natural fires. In this model, the value of the factor Γ was taken as 1.0 here, which makes the heating phase of the time–temperature curve of this natural fire model approximate the standard ISO curve. The only varying parameter is the time t_{max} which corresponds to the duration of the heating phase. Consequently, the selected set of standard natural fires is a generalization of the standard ISO fire that includes cooling down phases; the ISO fire being the particular case where t_{max} is infinite.

The parametric fire model has been derived to represent a post flashover and single zone situation. This assumption is usually considered valid under certain conditions, notably regarding the size of the compartment in which the fire develops.

For large, open compartments, in which spatial variance is known to be significant, different types of fire models have been proposed to capture more accurately the fire development. For instance, travelling fires models can capture the movement of the fire in large compartments and therefore account for the fact that all the fuel is not burning simultaneously. However these types of fire models are beyond the scope of the present research.

2.3 Duration of the Heating Phase (DHP) Indicator

For a structural component, there exists one "critical natural fire", based on a given severity measure, for which the component will fail. According to the choice made in Section 2.2, this fire is defined as the Eurocode parametric fire (with $\Gamma = 1$) of shortest duration of the heating phase t_{max} that will result in a failure of the component. This duration is referred to as Duration of the Heating Phase, DHP. The unicity of the DHP is associated to the choice of the (Γ 1) Eurocode parametric fire model, which leads to a well-defined temperature evolution in heating and cooling phases for a given t_{max}. Therefore, similarly to the Fire Resistance, the DHP is an unequivocal characterization of a certain performance of a given structural component loaded to a certain level. However, it is important to notice that, would a different natural fire model be chosen, or a different value of parameter Γ, the DHP would be modified, just as the Fire Resistance is dependent on the adopted prescriptive fire. Nevertheless, the DHP allows comparing with each other the performance of different structural systems under natural fire.

By definition, the DHP represents the minimum exposure time to standard ISO fire (followed by cooling phase in accordance with the Eurocode parametric fire model) that will eventually result in the failure of the structural component (either be it in the heating phase, in the cooling phase or after termination of the fire). It is always smaller than or equal to the Fire Resistance. It is important to notice that the DHP does not give any indication about the time of collapse. Generally, collapse can occur several minutes or hours after the time corresponding to the DHP, and it may even occur after the end of the fire, when the temperature in the compartment is back to ambient.

The DHP of the component discussed in Section 2.1 is shown in Figure 1. It is equal to 59 minutes. This means that the component loaded at 60% of its initial capacity will fail if it is subjected to the standard ISO fire during 59 minutes or more, followed by a cooling phase in accordance with the Eurocode parametric fire model. Failure does not arise after 59 minutes but after 110 minutes. On the contrary, the considered structural component is able to survive indefinitely to an exposure to the standard ISO fire shorter than 59 minutes, followed by a cooling phase in accordance with the Eurocode parametric fire model. For instance, Figure 1 shows that the load bearing capacity during and after exposure to a natural fire with 20 minutes of heating phase remains constantly higher than the demand on the component.

For this structural component, the Fire Resistance is equal to 93 minutes. This Fire Resistance is higher than the DHP (59 min), but lower than the time of delayed failure corresponding to the critical natural fire (110 min).

The Fire Resistance and the DHP are standardized indicators and as such they are useful to compare and classify different structural systems. At same Fire Resistance, the lower the DHP of a structural system, the higher its sensitivity to delayed failure under natural fire. In terms of performance-based design, the Fire Resistance indicator is interpreted as an information about the time of resistance during the heating phase of a fire, although it is obviously not a direct measure of this time since the real fire conditions will differ from the prescriptive fire conditions. The DHP is interpreted as an information about the occurrence of delayed failure as a function of the instant when the fire started to decrease, whether by self-extinction or due to the action of the fire fighters.

3. Method to derive the DHP indicator

To obtain the DHP of a structural component under a given load ratio is a more complex operation than for the Fire Resistance, for two reasons.

First, searching for the DHP of a component is searching for a design fire curve (the "critical natural fire"). The process is thus iterative, consisting of several analyses under different applied parametric fires for the search of the minimum value of parameter t_{max} that leads to structural failure. Because of the iterative procedure, experimental testing is not practically applicable; analytical or numerical models must be used.

Secondly, except for the simplest elements, the analysis of a structural component under natural fire necessarily requires a verification in the entire time domain by a step-by-step iterative method, since verification in the load domain at the time of maximum gas temperature does not guarantee against failure at a later stage. Therefore, the analysis is usually performed by means of advanced numerical methods such as the non-linear Finite Element Method (FEM).

Additional information can be found in the DHP curve of a component, i.e. the relationship between the load ratio and the DHP of this component, as will be shown in Figure 4.

4. Applications

4.1 Description of the Numerical Analyses

In this section, numerical analyses of structural components under fire conditions are conducted to assess the indicators R and DHP of these components under different load ratios. The non-linear finite element software SAFIR [17] has been used for thermal and mechanical analyses. The natural fires used in the analyses are the parametric fires from Eurocode with $\Gamma = 1$, as explained in Section 2.2.

The thermal properties of steel and concrete in the heating phase were taken from EN 1994-1-2 [18]. Where concrete is used, siliceous concrete was chosen, with a density of 2400 kg/m³ and a water content of 48 kg/m³. The emissivity was taken as 0.7 and the coefficient of convection between concrete and the air was 35 W/m²K.

The mechanical behavior of structural steel follows the model of EN 1993-1-2 [19]. A loss of residual yield strength of 0.3 MPa/°C is assumed for steel once heated beyond 600°C. Below this temperature of 600°C, the steel strength is considered as reversible, which means that the strength is recovered to full initial value during cooling if the temperature in steel has not exceeded 600°C [20].

For concrete, a residual thermal expansion or shrinkage has been considered when the concrete is back to ambient temperature, the value of which is taken as a function of the maximum temperature according to experimental tests published in the literature [21]. The Explicit Transient Creep Eurocode model has been adopted to take into account the transient creep strain irreversibility during cooling [22-23].

As prescribed in EN 1994-1-2, an additional loss of 10% of the concrete compressive strength with respect to the value at maximum reached temperature has been considered during cooling. The additional reduction during cooling is supported by many experimental studies [24]. In fact, experimental results suggest that the additional reduction during cooling may be even greater than the 10% reduction considered in Eurocode. Recent research on high strength concrete has also indicated that the compressive strength for these concretes tends to reduce even after cooling, by an additional 10-20% reduction after a few days, and partially recovers after a longer time period (1-2 years) [25]. These findings have implications on the vulnerability to cooling phases and residual load bearing capacity of concrete members. In particular, if a greater reduction of compressive strength than what is actually prescribed in the code is considered during and after cooling, this could reduce the DHP for some of the concrete members. In the present research, it was chosen to adopt the assumption of Eurocode for the strength reduction during cooling because it is a standardized and widely accepted model. Therefore, it is unquestionable that the conclusions drawn from the analyses are valid with a standard model and do not depend on particular assumptions on the material model that would suit the needs for this particular research. Future research shall investigate further the evolution of material properties during cooling and determine whether a revision of the codes is required.

4.2 Reinforced Concrete Columns

The first application deals with reinforced concrete (RC) columns exposed to fire on three sides, with the fourth side having adiabatic conditions. The column is simply supported at both ends. A sinusoidal imperfection with amplitude of L/300 has been introduced. This imperfection is introduced in the unfavorable direction, which is the direction of the thermal gradient for high slenderness and the direction opposite to the thermal gradient for low slenderness, because for columns with low slenderness the effects of the variation of the neutral axis position overcome the displacements due to thermal gradients. The concrete compressive strength is equal to 25 MPa and the steel reinforcement yield strength is 400 MPa. The column analyzed as the basic case is a 4 meter long column with a square cross section of 300 mm side, 8 bars of 16 mm diameter and a concrete cover of 30 mm. Two other configurations are then studied, varying by the effective length and section of the column, see Table 1. The Fire Resistance and DHP of the columns are established using the software SAFIR.

Case no.	Length L_0 [m]	Section [m]	Steel bars no. [-]	Steel bars diameter [mm]
1	4.0	0.30 x 0.30	8	16
2	6.0	0.30 x 0.30	8	16
3	6.0	0.45 x 0.45	12	16

Table 1. Geometry of the 3 studied reinforced concrete columns.

The results are presented in Table 2. For the different columns, the indicators R and DHP are given for applied load ratios of 60% to 30%. The load ratio is defined as the ratio between the vertical load applied in the fire situation at the top of the column and the ultimate strength at time t = 0 ($R_{d,fi}$), determined using SAFIR. When subjected to a 50% applied load ratio, the Case 1 column is characterized by a couple (R; DHP) of (70; 34). This means that the column has a fire resistance of 70 minutes, but it will fail if it is subjected to a natural fire with heating phase longer than 34 minutes.

The relationship between R and DHP is plotted in Figure 2 for the three columns. It shows that the DHP is always lower than the fire resistance, which reveals the possibility of delayed failure for the columns. Also, the DHP appears to be approximately a linear function of R. Finally, considering a required fire resistance for the three columns, the tendency to exhibit failure during the cooling phase will be more pronounced for case 3 column than for the others and for case 1 column than for case 2. This confirms that delayed failures are more prone to arise in columns with low slenderness and massive sections, as mentioned in [10].

Time in min	Case 1		Case 2		Case 3	
Load ratio	R	DHP	R	DHP	R	DHP
60%	52	17	23	7	80	29
50%	70	34	29	10	109	53
40%	92	56	36	19	154	79
30%	121	84	49	31	214	119

Table 2. Indicators R and DHP for the 3 RC columns under different load ratios.

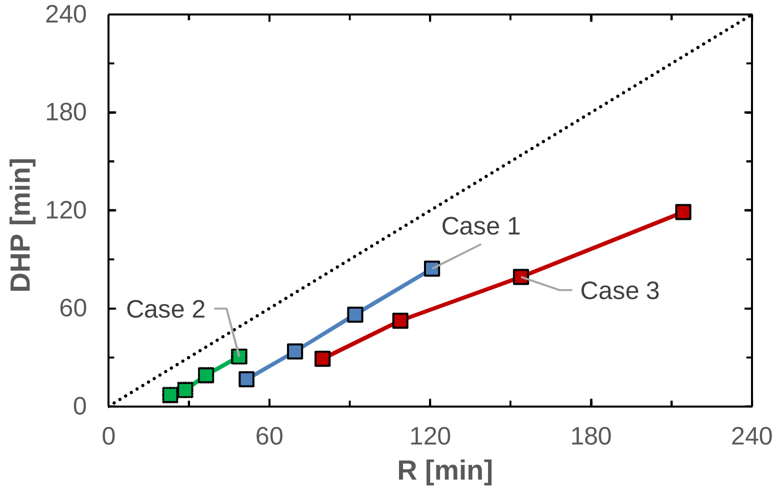


Figure 2. R-DHP indicators for the RC columns. A structural component with a response in the R-DHP space that is distant to the bisector has a higher propensity to delayed failure.

4.3 Reinforced Concrete Beam

The next study case deals with a reinforced concrete beam exposed to fire on three sides. The beam has a rectangular cross section of 500 mm height by 220 mm width reinforced by 3 lower bars of 20 mm diameter and 3 upper bars of 10 mm diameter, with a concrete cover of 40 mm. Figure 3 shows the discretization of the section and temperature distribution after 60 minutes. The beam is simply supported at both ends with a length of 6 m. The applied load is uniformly distributed on the beam and maintained constant during the fire. The mechanical strength of concrete and steel reinforcement are 30 MPa and 500 MPa, respectively.

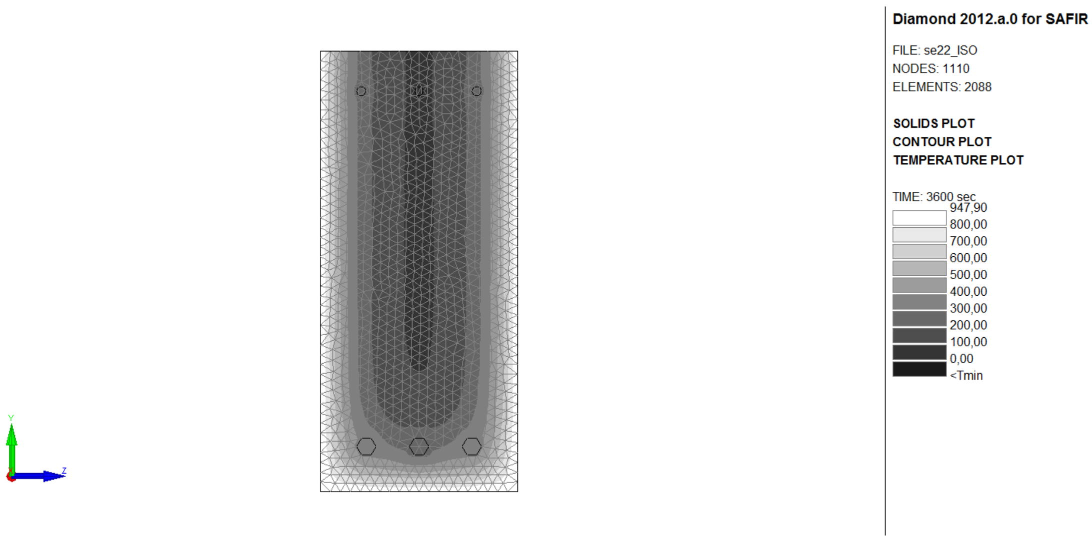


Figure 3. Temperature distribution in the RC beam section after 60 minutes of ISO fire.

Figure 4 shows the results for the reinforced concrete beam in a different presentation than that of Figure 2. On the vertical axis is the ratio between the applied load and the initial ultimate strength. The minimum applied load ratio leading to failure has been established under four natural fires of respectively 60, 90, 120 and 180 minutes of heating phase duration. The load ratios corresponding to these DHP are found as 70%, 41%, 16% and 10% respectively, meaning that the RC beam loaded at 41% of its ultimate capacity at ambient temperature will fail under a natural fire which heating phase lasts longer than 90 minutes. Figure 4 also shows the fire resistance time R and the latest time of failure, corresponding to the failure time of the RC beam when subjected to the DHP natural fires.

The difference between R and DHP increases from 34 minutes for a load ratio of 70% to 60 minutes for a load ratio of 10%. For instance, the RC beam loaded at 41% of its ultimate load capacity at ambient temperature is characterized by R equal to 128 minutes and DHP equal to 90 minutes. This significant difference denotes a sensitivity to delayed failure for this type of RC beams mainly due to the delayed temperature increase in the steel reinforcement.

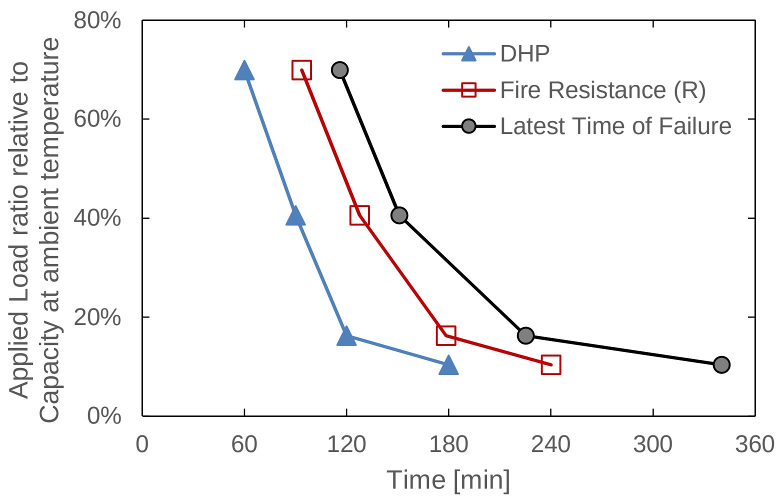


Figure 4. Evolution of R and DHP indicators and latest time of failure as a function of the applied load, for the RC beam.

To illustrate this, Figure 5 shows the temperature evolution in the steel rebars located at the lower corners of the section, for the ISO fire and for the natural fire with 90 minutes of heating phase. A maximum temperature of 634°C is reached after 128 minutes of ISO fire exposure, corresponding to failure for a 41% load ratio. The maximum temperature reached in the course of the natural fire is equal to 623°C and occurs after 148 minutes, which corresponds to the time of failure observed in Figure 4. The slight difference between the temperatures of the corner rebars at failure (623°C versus 634°C) is due to the influence of the other constituents in the section such as other rebars and, to a lower extent, concrete.

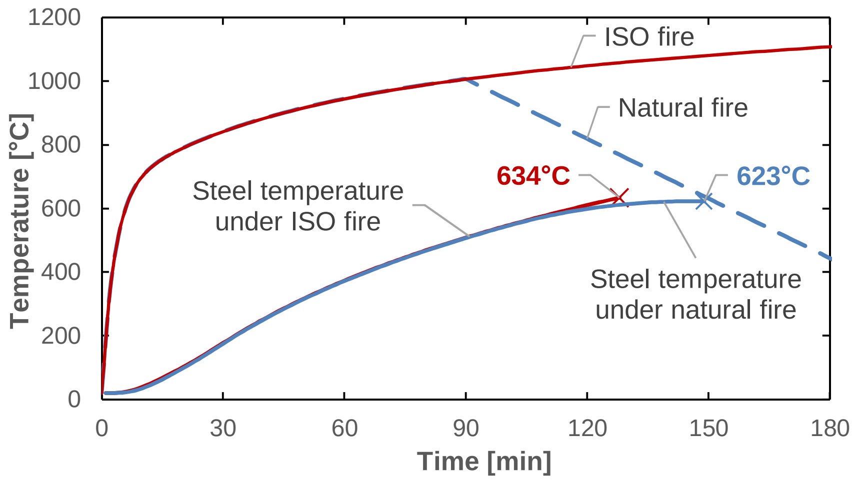


Figure 5. Temperature evolution in the lower corner steel rebars of the RC beam for the ISO fire and for the 90 minutes natural fire.

4.4 Steel Column with and without Thermal Protection

A HEB 400 steel column in S355 is subjected to fire on its four sides. The column height is 4 m and it has a sinusoidal imperfection with amplitude of L/300. It is simply supported at both ends and submitted to a vertical load applied at the center of the top section. The load bearing capacity at the beginning of the fire is 6256 kN.

The column is first analyzed with no thermal protection. The column is then added a thermal protection of 20 mm thickness all around the section, see Figure 6, in order to investigate the effect of thermal protection on the Fire Resistance and on DHP. The thermal properties of the insulating material are: thermal conductivity 0.3 W/mK, specific heat 800 J/kgK, specific mass of the dry material 550 kg/m³, water content : 16.5 kg/m³, coefficient of convection 35 W/m²K, emissivity 0.8.

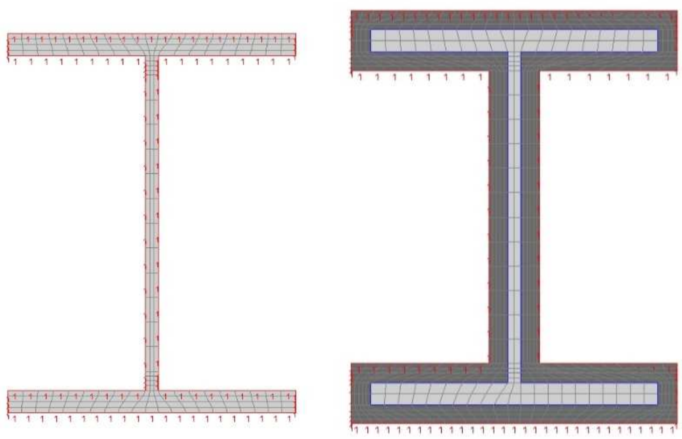


Figure 6. HEB400 steel column with / without thermal protection subjected to fire on 4 faces.

The results are given in Table 3. With no thermal protection, the temperature in the column increases very quickly, leading to a quick decrease of the steel mechanical properties and failure of the column. For load ratios between 60% and 30%, the Fire Resistance ranges from 12 to 17 minutes, as expected for this type of steel element with no thermal protection. The DHP range shows a shift of 5 minutes with respect to the Fire Resistance. This is due to the slight delay between the times of maximum temperature in the compartment and maximum temperature in the section, caused by the thermal inertia of the steel flanges and web.

A thermal protection can be applied on such steel structural component to improve the fire performance. In this example, the thermal protection allows for improving the Fire Resistance from 14 minutes to 61 minutes under a 50% applied load ratio. This performance improvement is obtained by delaying the temperature increase in the steel; however the side effect is that the difference between R and DHP is increased up to 19 minutes, indicating a higher propensity for delayed collapse. This increased difference between R and DHP is the result of the increased delay between the maximum temperature in the compartment and in the steel section.

Time in min	No thermal protection		Thermal protection	
Load ratio	R	DHP	R	DHP
60%	12	7	54	35
50%	14	9	61	43
40%	15	10	69	50
30%	17	13	79	60

Table 3. Indicators R and DHP for the 2 steel columns under different load ratios.

Finally, Table 4 gives the maximum temperature in the center of the flange at time R and the maximum temperature reached during the natural fire corresponding to the DHP. It is important to note that this latter temperature is not reached at time DHP, it is reached during the cooling phase that follows. The table gives the values of maximum temperature, or critical temperature, for the columns with different load ratios and with and without thermal protection. As expected, the critical temperature increases with a decrease in the load ratio. Also expected, the thermal protection does not affect the critical temperature; it only affects the time of occurrence of this temperature in the section. This can be seen by comparing the columns “T @ R” for the component with and without thermal protection, the slight differences in temperature observed being mainly due to the influence of temperatures in other parts of the section that has a non-uniform temperature distribution. The temperatures corresponding to R and to DHP are also nearly identical. This indicates that for such simple structural component the delayed failures under natural fires arise when the steel temperature reaches the critical temperature, in the sense of the temperature reached at the Fire Resistance time. This occurs during the cooling phase and the most significant the thermal inertia, the most significant the delayed effect. Of course, this behavior is directly linked to the hypothesis of the steel constitutive model used here that the strength and stiffness of steel are only dependent on its temperature; true creep effects are not explicitly considered. A more sophisticated constitutive model could yield a critical temperature that is lower for a protected component than for an unprotected one. For characteristic fire durations, the difference is expected to be small.

Max Temp. in °C	No thermal protection		Thermal protection	
Load ratio	T @ R	T for DHP	T @ R	T for DHP
60%	475	499	480	491
50%	537	552	525	542
40%	565	578	571	583
30%	613	634	621	633

Table 4. Maximum temperature in the flange of the steel columns.

The analyses are repeated for the protected steel column of 4 m height when attacked by the fire on three sides and for the protected steel column attacked by the fire on four sides which height is increased to 6 m and 8 m. The initial strength is 5913 kN and 5486 kN for the 6 m and 8 m height columns, respectively. The Fire Resistance and DHP for the four configurations of the protected HEB 400 steel column are plotted in Figure 7 for load ratios from 30% to 60%.

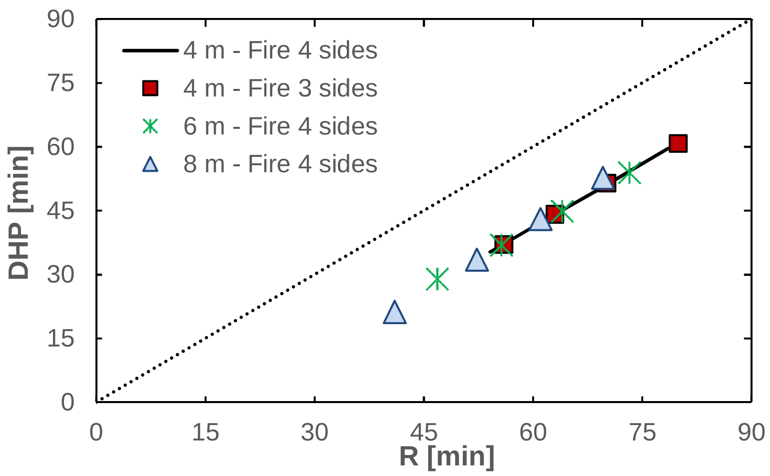


Figure 7. R-DHP indicators for the protected steel column for different heights and fire conditions.

The results in Figure 7 are located on a line parallel to the bisector in the R-DHP space, i.e. the time interval between the Fire Resistance and the DHP is constant. This indicates that the fire boundary conditions and the height of the column do not influence its sensitivity to delayed failure. Obviously, when the column height is increased (i.e. increased slenderness), the DHP is reduced but this reduction appears to be in the same magnitude as the reduction in Fire Resistance. Hence, the sensitivity to delayed failure (in the sense of the time interval between R and DHP) is approximately independent on the slenderness. For this steel column, the main parameter governing the delayed effects is clearly the delayed temperature increase in the section.

The analysis of the slenderness influence on the parameters R and DHP is affected by the hypothesis of the steel constitutive model of Eurocode 3, in which creep effects are not explicitly considered. For steel heated above approximately 400°C for prolonged time periods, the creep effects may influence the fire behavior of steel members [26]. The creep effects tend to reduce the critical temperature of steel columns (or, which is equivalent, the column failure time) and this reduction is more pronounced for slender columns than for stocky columns, since the additional creep strains lead to additional second order effects. Therefore, it is expected that the high-temperature creep would influence the parameters R and DHP to a different extent depending on the column slenderness. Adopting the recommendation of Eurocode 3, the effects of thermal creep have not been given explicit consideration in the present research; however it might be interesting for future works to investigate these effects especially during the cooling phase.

4.5 Timber element

In a timber structural component subjected to natural fire exposure, the charring process in timber will continue during the cooling phase. This delayed charring process is generated by the energy received from the radiative and convective heat flux still produced by the cooling surrounding fire and from the combustion of the structural component itself [27]. As a consequence of the resulting increase in charring depth, the effective cross-section continues to decrease. In addition, the delayed heat transfer in the uncharred core of the section leads to an additional decrease of the mechanical properties. According to Eurocode 5 [28], the strength and stiffness of softwood start to decrease as soon as the temperature exceeds 20°C and they reduce to zero at 300°C. Due to the combination of delayed charring process and delayed decrease in mechanical properties, it is expected that timber structural components are prone to delayed failure.

This section analyzes the behavior of a softwood timber beam exposed to natural fire. The analysis deals with a simply supported beam of 4.0 m span subjected to fire on 3 sides. Two different rectangular cross sections are considered, with a height equal to 0.40 m and 0.60 m and a width equal to 0.20 m and 0.30 m, respectively. The applied load is uniformly distributed on the beam and maintained constant during the fire. Moisture content and density at ambient temperature are assumed as 12% and 450 kg/m³. Young's modulus and characteristic bending strength are taken as 11 GPa and 24 MPa at ambient temperature, respectively.

Modeling the response of a timber component during the cooling phase is complex, in particular because it requires to quantify the additional energy generated through combustion of the component during cooling. Two methods currently available in Eurocode 5 are first considered in this section. This requires to make a few assumptions that are discussed hereafter. Meanwhile, it is noted that the focus of this research is on the development of the novel concept of DHP rather than on the detailed investigation of material models. When refined models for wood that incorporate the effects of cooling are made available, the concept presented in this paper can be applied with these models following the same procedure as illustrated here.

Annex A of Eurocode 5 gives a method for calculating the charring depth under natural fire exposure (referred to as the "simple method" hereafter). This method is empirical, based on the position of the 300°C isotherm during a fire. The charring depth can then be used as part of a reduced cross section calculation. The application of Annex A is limited by certain conditions, in particular regarding the time period with a constant charring rate (which depends on the fire load density and the opening factor). The reduced cross section method assumes that the core of the section is not affected by the elevated temperatures. However, this hypothesis might become questionable in the context of a residual load bearing capacity assessment, because the heat has time to propagate through the entire cross section.

For instance, the simple method is applied for calculation of the charring depth for the 0.60 m x 0.30 m section subjected to a natural fire with a 30 minutes heating phase. Considering a design charring rate of 0.7 mm/min, the charring depths at the end of the heating phase and at the end of the fire are respectively equal to 21 mm and 32 mm. A further 7 mm layer has to be subtracted for the reduced cross section calculation in order to account for material property degradation in uncharred portions, see Figure 8.

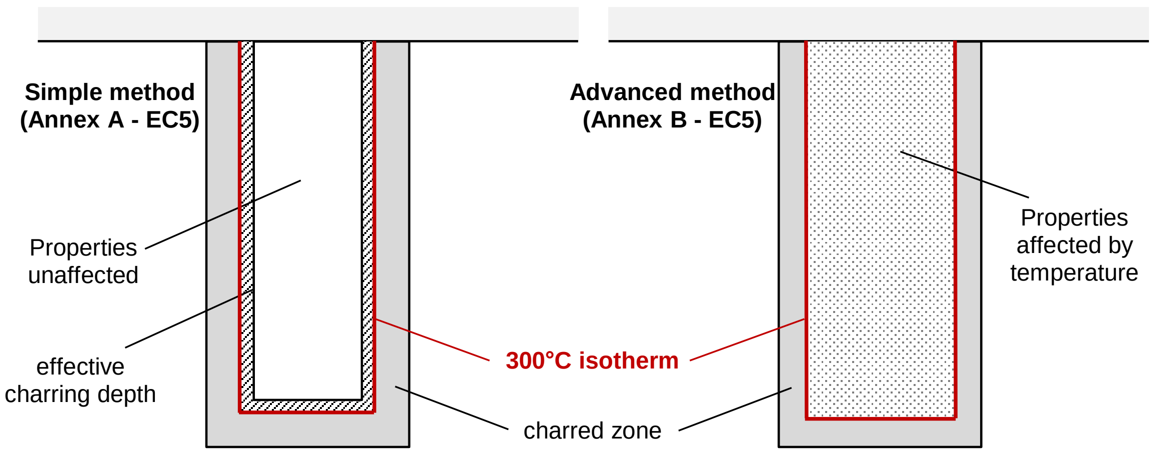


Figure 8. Effects of temperature on a timber section according to the simple (reduced cross section) method and the advanced calculation method.

On the other hand, Annex B of Eurocode 5 (referred to as the "advanced method" hereafter) allows for the use of advanced calculation method based on the theory of heat transfer and evolution of the properties with temperature. Hence, numerical simulations can be used to assess the thermo-mechanical response of timber components. However, the evolution of thermal and mechanical properties with temperature is only given for standard fire exposure. It is assumed here that these properties are not recovered during cooling; they keep the value corresponding to the maximum reached temperature. Another limitation is the fact that these properties are not able to capture the additional energy generated through combustion of the member during cooling.

For instance, for the 0.60 m x 0.30 m section subjected to a natural fire with a 30 minutes heating phase, the thermal analysis based on the advanced method leads to charring depths of 23 mm and 28 mm at the end of the heating phase and at the end of the fire, respectively. In the numerical simulations, the position of the 300°C isotherm is considered to define the charring depth (Figure 8). The emissivity and convection coefficient were taken equal to 0.8 and 35 kW/m², respectively, in accordance with Eurocode 5 and Eurocode 1.

At the end of the fire, the charring depth predicted by the simple (empirical) method of Annex A (32 mm) exceeds the one obtained by the advanced method (28 mm). However, comparison of the charring depths is not sufficient to compare the residual load bearing capacity predicted by the two methods.

Considering the simple method, the reduced cross section after cooling is obtained by subtracting 39 mm (32 + 7) to the three exposed sides of the section. For the remaining cross-section material (beyond the effective charring depth), this method assumes that the mechanical properties are unchanged. As a result, the residual load bearing capacity is equal to 65% of the initial capacity at ambient temperature.

In contrast, the advanced method computes the temperatures in the entire cross-section. Then, it applies temperature dependent reduction factors for the material mechanical properties according to Annex B of Eurocode 5. According to this advanced method, the material strength is reduced to zero only on a depth of 28 mm where the temperature has exceeded 300°C (instead of 39 mm predicted by the empirical method). However, the entire section is affected by the temperature, see Figure 8. This has a significant effect on the load-bearing capacity evolution during cooling because the core of the section may experience a decrease in strength and stiffness long after the end of the heating phase of the fire. In this example, the temperature at the center of the section reaches 60°C approximately 7 hours after the end of the fire; yet at 60°C the reduction in mechanical properties is already significant. Given the conservative assumption that the properties are not recovered, the effects of maximum temperature reached in each fiber of the section during the course of the fire are accounted for in the calculation of the residual load bearing capacity. As a matter of fact, the residual capacity for the considered component is found equal to 47% of the initial capacity at ambient temperature.

As a conclusion, for this example, the advanced method of Annex B predicts a lower residual capacity at the end of a natural fire (47%) compared with the simple method of Annex A (65%). This is due to the assumptions in the former method that the mechanical properties of the entire section are affected by temperature and that these properties are not recovered at all during cooling. This is important in case of natural fire because the heat has time to propagate through the section. Most likely, the simple method was not developed for assessment of residual capacity several hours after a fire. In fact, the delayed heat transfer leads to significant reduction of the properties of the core of the section. This effect more than compensate for the fact that the charring depth is a bit underestimated with the advanced method compared with the simple (empirical) method.

It is interesting to note that the two methods lead to load bearing capacity at the end of the 30 minutes heating phase of 74% (simple method) and 73% (advanced method) of the value at ambient temperature. The charring depths were found equal to 21 mm (simple method) and 23 mm (advanced method). Hence, the significant level of discrepancy observed for natural fire exposure is not observed for standardized fire exposure (because in the latter case, the heat has not propagated much further than the char front). The two methods fortunately lead to similar results for the Fire Resistance rating of timber components.

Based on the discussed limitation of the simple method for residual load bearing capacity assessment, the advanced method was adopted to evaluate the response of the softwood timber beam under natural fire. Numerical simulations were conducted for the timber beams under natural fire exposure. For timber components, it is particularly important to run the simulations for a long time after the fire, because even a slight increase in temperature affects the material properties significantly.

The Fire Resistance and DHP for the two timber beams are plotted in Figure 9 for load ratios from 30% to 60%. As was expected, the sensitivity to delayed failure is significant. For instance, the 0.60 m x 0.30 m cross section beam loaded at 40% of its ultimate capacity at ambient temperature has a fire resistance of 92 min and a DHP of 39 min. Contrary to the steel components of Section 4.4, the difference between R and DHP increases with R, indicating a higher sensitivity to delayed failure for larger cross-sections and for smaller load ratios.

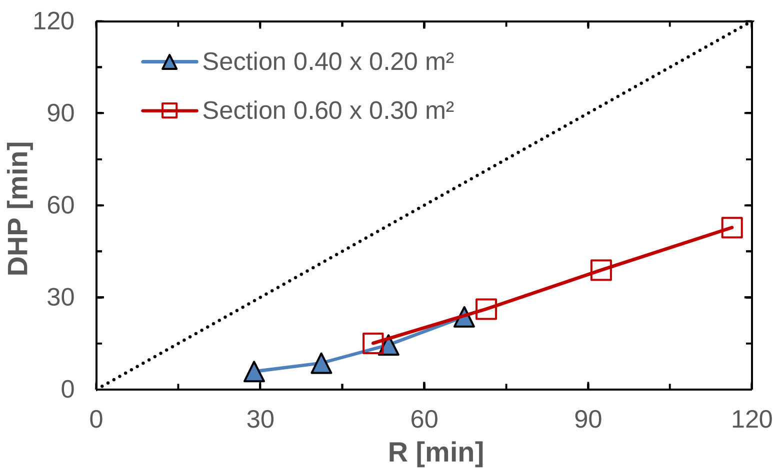


Figure 9. R-DHP indicators for the timber beam for different dimensions of cross-section.

5. Discussion

The numerical analyses conducted in the previous section have shown that the occurrence of failure during the cooling phase of a fire is a possible event for all studied configurations, including unprotected steel columns. Consequently, the DHP is always shorter than the Fire Resistance.

For simple steel elements, the delayed effects revealed by the DHP is directly linked to the delayed temperature increase in the sections. Yet for more complex configurations and other materials, additional effects may also take place in these delayed failures, such as the non-recovery of mechanical properties, the variations in the thermal gradients, or the delayed charring process in timber. The DHP allows for quantifying in a simple manner the impact of these complex effects on the possibility of delayed failure.

By considering the couple of indicators (R, DHP), the information provided about the fire performance of a structural component is more adapted to the design in a performance-based environment. It indicates whether a structure is prone to delayed failure or not. For instance, comparing in Figure 10 the RC column of Section 4.2 with the thermally protected steel column of Section 4.4 shows that the RC column is more sensitive to delayed effects and more prone to collapse during the cooling phase, as indicated by the difference between R and DHP. For load ratios lower than 45%, the RC column has a higher fire resistance and a higher DHP than the protected steel column at same load ratio. But for load ratios between 45% and 58%, the RC column, although it still has a higher fire resistance, has a lower DHP than the protected steel column at same load ratio. In other words, for high load ratios, a given natural fire exposure could result in a collapse for the RC column while not for the protected steel column with a lower fire resistance. This probably results (at least in part) from the fundamental differences in material behavior during cooling between steel and concrete, the latter experiencing additional strength loss during cooling. By considering only the Fire Resistance, the designer would neglect this effect, concluding that the safety level is higher for the RC column than for the steel column for any load ratio; however, this is not necessarily true anymore when natural fires are considered.

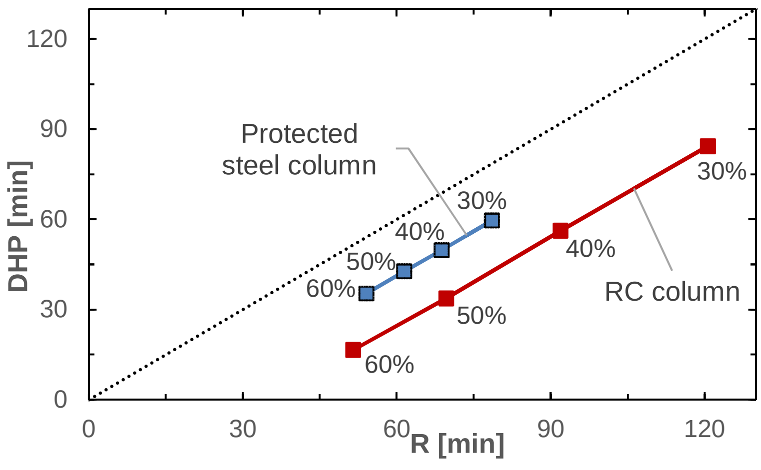


Figure 10. R-DHP indicators for a protected steel column and a concrete column under mechanical load ratios of 30% to 60%.

This single comparative example should not be used to draw general conclusions about the relative sensitivity to cooling of concrete versus steel structural components. This sensitivity depends on many factors other than the constituting material. As is the case with

Fire Resistance, the problem is complex and depends on the specificities of the design. For instance, an increase in the thermal protection thickness on the steel column is expected to result in an increase in the column sensitivity to delayed failure (i.e. the distance to the bisector in the R-DHP space would increase in Figure 10 for the steel column).

Nevertheless, this comparative example is important because it shows that, under natural fire, using the sole indicator R to compare the fire performance of two designs is not sufficient. A second indicator, namely the DHP, is needed to paint a full picture of the fire performance under a realistic (i.e. natural) fire. Indeed, a component that proved to perform better than another under standardized fire conditions (higher R) might in fact perform worse under a realistic fire (lower DHP).

6. Conclusion

Structural failure may arise during or after the cooling phase of a natural fire. These delayed failures concern all typologies of structural elements and constituting materials, as illustrated by numerical analyses conducted on reinforced concrete, steel and timber components. With the consideration of more realistic fire scenarios, the fire safety analysis of structures should necessarily address their response until complete burnout of a fire and beyond, in order to assess the safety during fire brigades intervention, building inspection, and possibly building rehabilitation.

This research proposes a method to characterize the behavior of structures under natural fire, including their sensitivity to delayed failure. The derived indicator (DHP) allows for quantifying the impact of different physical processes on the structural response. It constitutes a pragmatic measure that can be used to compare different structural systems in terms of their propensity to delayed failure. Several examples of applications on structural components have been presented in the paper. For future works, the research should be extended from the scale of the component to that of the structure, because particular characteristics of the latter such as joints between different components are expected to exhibit a significant sensitivity to cooling.

At the practical level, characterization of a structure by the couple of indicators (R, DHP) allows dividing the post-flashover time domain in three parts for a structure in fire, at least if the temperature development in the post-flashover phase is in the order of magnitude of the standardized fire. As long as the heating time passed after flashover is shorter than the DHP, the structure is theoretically safe; as soon as it exceeds the DHP, the structure has been affected to such an extent that it is expected to fail even if the fire stops thereafter; and when the post-flashover time of heating exceeds R the structure is theoretically collapsed. This information may be valuable for firefighters who can relate it with the information they can get about the duration of the fire when they arrive on site and use it for mitigating the risk during their intervention.

References

[1] Mao X, Kodur VKR. Fire resistance of concrete encased steel columns under 3- and 4-side standard heating. J Constr Steel Res 2011; 67(3):270-280.

[2] Pettersson O, Witteveen J. On the fire resistance of structural steel elements derived from standard fire tests or by calculation. Fire Safety J 1980; 2(1):73-87.

[3] Lie TT, Stringer DC. Calculation of the fire resistance of steel hollow structural section columns filled with plain concrete. Revue Canadienne de Genie Civil 1994; 21(3):382-385.

[4] Fike R, Kodur VKR. Enhancing the fire resistance of composite floor assemblies through the use of steel fiber reinforced concrete. Engineering Structures 2011; 33(10):2870-2878.

[5] EN 1991-1-2. Eurocode 1: Actions on Structures - Part 1-2: General actions – Actions on structures exposed to fire. Brussels: CEN, 2002.

[6] ASTM - American Society for Testing and Materials. ASTME119-00 –Standard Methods of Fire Test of Building Construction and Materials. West Conshohocken, PA, USA, 2007.

[7] Fike RS, Kodur VKR. An approach for evaluating the fire resistance of CFHSS columns under design fire scenarios. J Fire Prot Eng 2009; 19(4):229-259.

[8] Garlock ME, Selamet S. Modeling and Behavior of Steel Plate Connections Subject to Various Fire Scenarios. J Struct Eng 2010; 136(7):897–906.

[9] Bukowski RW. Emergency Egress from Buildings, Part 1: History and Current Regulations for Egress Systems Design. NIST Technical Note: 1623. National Institute of Standards and Technology, Gaithersburg, USA, 2009.

[10] Gernay T, Dimia MS. Structural behaviour of concrete columns under natural fires. Engineering Computations 2013; 30(6):854-872.

[11] Kallerova P, Wald F. Fire Test on Experimental Building in Mokrsko. CTU: Prague, 2009.

[12] Hody P. Seven Swiss Firefighters Die in Collapsed Parking Garage. 2004. [online] Available at <http://www.firehouse.com/news/10514192/seven-swiss-firefighters-die-in-collapsed-parking-garage>. [Accessed 18 Aug 2014].

[13] Chen YH, Chang YF, Yao GC, Sheu MS. Experimental research on post-fire behaviour of reinforced concrete columns. Fire Safety J 2009; 44(5):741-748.

[14] Jin M, Zhao J, Chang J, Zhang D. Experimental and parametric study on the post-fire behavior of tubular T-joint. J Constr Steel Res 2012; 70:93-100.

[15] Kodur VKR, Raut NK, Mao XY, Khaliq W. Simplified approach for evaluating residual strength of fire-exposed reinforced concrete columns. Materials and Structures 2013; 46(12):2059-2075.

[16] Liu F, Gardner L, Yang H. Post-fire behaviour of reinforced concrete stub columns confined by circular steel tubes. J Constr Steel Res 2014; 102:82-103.

[17] Franssen JM. SAFIR: A thermal/structural program for modeling structures under fire. Engineering Journal – Americal Institute of Steel Construction 2005; 42(3):143-158.

[18] EN 1994-1-2. Eurocode 4 – Design of composite steel and concrete structures. Part 1–2: General rules – Structural fire design. Brussels: CEN, 2005.

[19] EN 1993-1-2. Eurocode 3 - Design of steel structures - Part 1-2: General rules - Structural fire design. Brussels: CEN, 2005.

[20] Kirby BR, Lapwood DG, Thomson G. The reinstatement of fire damaged steel and iron framed structures. London, UK: B.S.C, Swinden Laboratories, 1986. 0 900206 46 2.

[21] Franssen JM. Thermal elongation of concrete during heating up to 700°C and cooling. University of Liege, 1993. http://hdl.handle.net/2268/531.

[22] Gernay T. Effect of Transient Creep Strain Model on the Behavior of Concrete Columns Subjected to Heating and Cooling. Fire Technology 2012; 48(2):313-329.

[23] Gernay T, Franssen JM. A formulation of the Eurocode 2 concrete model at elevated temperature that includes an explicit term for transient creep, Fire Safety J 2012; 51:1-9.

[24] Yi-Hai L, Franssen JM. Test results and model for the residual compressive strength of concrete after a fire. Journal of Structural Fire Engineering 2011; 2(1):29-44.

[25] Torić N, Boko I, Peroš B. Reduction of Postfire Properties of High Strength Concrete. Advances in Materials Science and Engineering 2013. DOI: 10.1155/2013/712953.

[26] Torić N, Harapin A, Boko I. Experimental verification of a newly developed implicit creep model for steel structures exposed to fire. Engineering Structures 2013; 57: 116-124.

[27] Hopkin DJ. Predicting the thermal response of timber structures in natural fires using computational 'heat of hydration' principles. Fire and Materials 2013; 37:311-327.

[28] EN 1995-1-2. Eurocode 5 - Design of timber structures - Part 1-2: General - Structural fire design. Brussels: CEN, 2004.