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PERFORMANCE AND DAMAGES OF R.C. SLABS IN FIRE

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Abstract

Contrary to a common misconception, concrete structures are particularly vulnerable to fire, as witnesses by several cases of fire-induced collapses of buildings with a primary concrete structural system. Even when no collapse occurs, concrete elements are permanently damaged by the fire and may need to be repaired or substituted. Despite the economic losses that this implies, a procedure for assessing the level of fire damage is not indicated in the codes and this aspect is generally not considered in the design praxis.

This paper presents a methodology for investigating the response of floor slabs with complex geometry exposed to fire and assessing the entity of the damage on the basis of the decrement of the load bearing capacity at the end of the fire. By considering this quantity for different time of exposure to a standard fire, a curve is obtained that provides important information on the vulnerability of the slab to the fire action and can be used for optimizing the design on the basis of the required class of resistance or for choosing between different slab alternatives.

Keywords: Structural fire safety; R.C. slabs in fire; voided biaxial deck; numerical modelling; nonlinear response; residual load bearing capacity; structural vulnerability to fire.

1 Introduction

Contrary to a common misconception, concrete structures are particularly vulnerable to fire, as witnesses by several cases of fire-induced collapses of buildings with a primary concrete structural system, such as the collapse of Windsor Tower, Spain, in 2005 or of the Architectural faculty building of Delft University, Netherlands, in 2009), as well as buildings with concrete or composite deck system, such as the WC7 collapse, New York, US, in 2001. These events have emphasized the importance of a more careful consideration of the effects of a fire on concrete structures and have renewed the attention to the advanced modeling and investigation of the behavior of concrete structural elements during and after a fire event.

A particularly critical occurrence during a building fire is represented by the failure of a floor slab, which may trigger the vertical propagation of both the fire and the structural failures. The severity of the consequences of such an occurrence increases with the height of the building. In tall or high-rise buildings, concrete slabs lightened by voids or with hollow tiles are frequently used, as they allow for lighter and more economical structures. However, due to the complex geometry, simple methods for fire design are hardly applicable to these types of slabs. Hence, the prediction of their response during fire is affected by higher uncertainties.

Even when the slab does not fail, significant costs may be incurred for their reparation. Contrarily to structural steel, which generally regains the original mechanical properties after cooling, ordinary concrete suffers permanent damages when heated over 300°C: at this temperature micro-cracks develop as a consequence of the material dehydration and the thermal expansion of the aggregates, so that the loss of mechanical properties becomes permanent (Hertz, 2005). For this reason, in addition to the evaluation of the response during a fire, the residual performances of concrete elements after a fire are also important criteria to be considered in view of an efficient and sustainable design of concrete structures (Shipp, 2007), (ISO 15392, 2008). Nevertheless, a procedure for assessing the level of fire damage is not indicated in the codes and this aspect is generally not considered in the design praxis.

This paper presents a methodology for investigating the response to fire of a R.C. slab with cavities and assessing the entity of the damage on the basis of the decrement of the load bearing capacity at the end of the fire. By considering this quantity for different time of exposure to a standard fire, a curve is obtained that provides important information on the vulnerability of the slab to the fire action, to be used for optimizing the design on the basis of the required class of resistance or for choosing between different slab alternatives.

2 Object and method

2.1 Objects of the study

A hollow core slab with slack reinforcement is taken as case study. The slab spans a length of 7 m and is simply supported on the edge beams and free to expand. The slab section, which is designed to sustain a ULS occupancy load of 2.5 kN/m^2, is illustrated in Fig. 1 and has the following characteristics:

- The concrete deck is 40 cm high and is made of concrete C30/37 with the following mechanical and thermal properties at 20°C: characteristic strength f_{cu} = 30 MPa; stiffness E_c = 16.9 MPa; density ρ = 2300 kg/m^3; thermal conductivity λ = 1.64 W/(m·K); thermal expansion coefficient $\alpha = 6 \cdot 10^{-6}$ K^{-1}; specific heat capacity c_p = 900 J/(kg·K).
- The bottom and top reinforcement bars are made of B450C steel with a stiffness E_s = 210 GPa and a steel strength fy = 440 MPa and are spaced 30 cm along both the longitudinal and transversal directions. The top bars have a diameter of 14 mm, while the diameter of the bars at the bottom is 16 mm. Both bottom and top bars have a concrete cover 3 cm.
- The cavities have an elliptic shape with axes of 22 and 27 cm for the vertical and horizontal direction respectively. They are spaced 30 cm from one another, which leaves 8 cm as minimum thickness of concrete between the cavities. The minimum depth of the concrete below and above the cavities is 4.2 and 8.8 cm respectively.

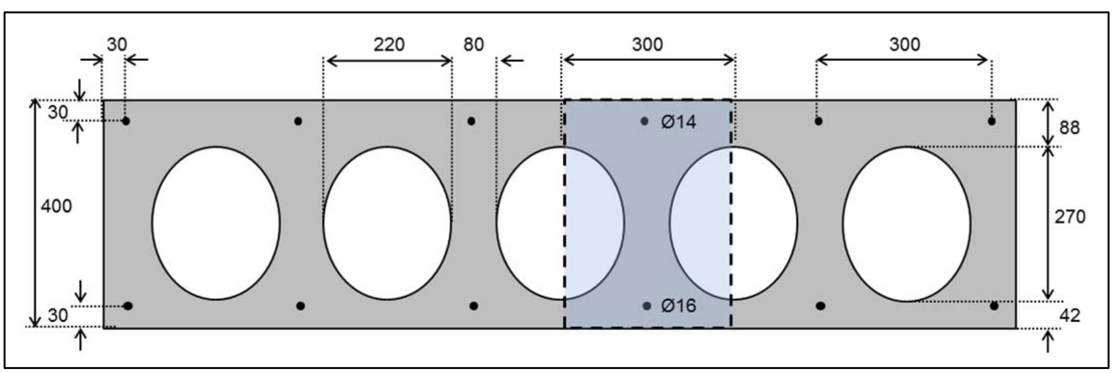


Fig. 1. Geometry and dimensions of the slab cross section.

The self-weight of the slab and the weight of the slab superstructure give a characteristic permanent weight G_k = 8.6 kN/m^2. The design load at the ultimate limit state (ULS) results to be equal to p_{sd} = 13.2 kN/m^2 when the combination on the left of Equation 1 is used with the safety coefficient γ_G = 1.1 and γ_Q = 1.5 prescribed by the Danish action code for the permanent and variable actions respectively. Simplified hand calculations of the flexural resistance give a load bearing capacity p_{ud} = 14.9 kN/m^2 at 20°C, when the ULS material safety coefficients $\gamma_{M,s}$ = 1.2 and $\gamma_{M,c}$ = 1.5 are used for the steel and the concrete, respectively. The corresponding design utilization factor at the ULS is therefore equal to μ_d = 89% and the residual capacity of the slab, intended as the additional load the slab can sustain without failing, is $(p_{ud}\text{-}p_{sd})$ = 1.7 kN/m^2.

$$\text{ULS:}\ \ p_{sd} = \gamma_G \cdot G_k + \gamma_Q \cdot Q_{k1} \qquad\qquad \text{FLS:}\ \ p_{sd} = \gamma_G \cdot G_k + \psi_1 \cdot Q_{k1} \tag{1}$$

The imposed load for fire design calculations is obtained by using the load combination for the fire limit state (FLS) shown on the right of Equation 1, where the reduced safety coefficients γ_G = 1.0 and ψ_1 = 0.4 for office occupancy are used, as prescribed by the Danish action code for the fire design (DS/EN 1991-1-2 DK NA, 2011). This load results to be p_s^{fi} = 9.6 kN/m^2. The same code allows the use of the characteristic values of the steel and concrete mechanical properties at 20°C for the calculation of the load bearing capacity during fire. If the condition at the beginning of the fire is considered (corresponding to a time of exposure t = 0 min), the load bearing capacity obtained by means of simplified hand calculation is equal to p_u^0 = 18.2 kN/m^2. The utilization factor at the beginning of the fire results to be significantly lower that the design one for the ULS and is in particular equal to μ^0 = 53%. The residual load bearing capacity of the slab $(p_u^0\text{-}\ p_s^{fi})$ = 8.6 kN/m^2. An overview of the calculated values is given in Table 1.

Table 1. Analytical calculation of the slab loada and capacity for the design (ULS) and fire case (FLS t = 0)

<table>
<tr><th>Design</th><th>γ_G</th><th>G_k</th><th>γ_Q</th><th>Q_k</th><th>p_s</th><th>$\gamma_{m,s}$</th><th>$\gamma_{m,c}$</th><th>p_u</th><th>p_u - p_s</th><th>μ</th></tr>
<tr><td>ULS</td><td>1.1</td><td rowspan="2">8.6</td><td>1.5</td><td rowspan="2">2.5</td><td>13.2</td><td>1.2</td><td>1.5</td><td>14.9</td><td>1.7</td><td>89%</td></tr>
<tr><td>FLS (t = 0)</td><td>1.0</td><td>0.4</td><td>9.6</td><td>1.0</td><td>1.0</td><td>18.2</td><td>8.6</td><td>53%</td></tr>
</table>

2.2 Analysis and assumptions

The fire is considered in the form of a standard fire exposure of the bottom surface of the slab for a total duration of 90 min. The choice of a nominal fire curve, such as the standard fire (ISO834-1, 1999) has been preferred over a natural fire, because it does not depend on the compartment, where the slabs may be placed. As such, the results depend on the sole intrinsic characteristics of the slab and can be used for comparing different slabs as alternative design solutions for the same building.

The limitation of the fire exposure to the sole bottom surface is due to the fact that the slab element is assumed to be in place as part of a building deck system. Therefore, in case of fire, only the bottom surface of the slab is exposed to the thermal load, while the sides are protected by the adjacent slab elements. This condition means that the vertical axes passing through the centre of the cavities are symmetry lines for the temperatures and allows limiting the thermal analysis to the part of the cross-section comprised within two consecutive cavity axes. This area is highlighted in Fig. 1 by means of a grey rectangle with dotted line borders. Due to the uniformity of the mechanical load and the unidirectional framework of the slab, also the study of the structural response can be limited to the longitudinal strip having the cross-section highlighted in the box.

The investigation of the load bearing capacity for a given time of fire exposure is performed by performing two separated analysis:

a. a thermal analysis, which provides the temperature histories at each node of section mesh for 90 min of fire;
b. a structural nonlinear static analysis, which is divided into three steps:
    i. first the imposed mechanical load is applied to the top surface of the slab and a static incremental analysis is carried out;
    ii. then the temperature histories obtained from the thermal analysis are applied to the respective mesh nodes and a transient static analysis is performed;
    iii. finally, the mechanical load is incremented up to the failure of the slab, by means of a static incremental analysis with arc-length displacement control: the temperatures and corresponding mechanical properties of the elements of the mesh are the same as those reached at the end of the previous analysis step.

The load bearing capacity is taken in correspondence of the peak of the load-displacement (pushover) curve obtained as analysis output and is assessed at the beginning of the fire (t = 0), and at 30, 60, and 90 min of fire exposure, by means of four different pushover analyses. The analysis at the beginning of the fire is used for validating the structural model against the analytical calculation of the load bearing capacity. The validation of the thermal analysis is also accomplished by comparison of the temperatures with two different simplified analytical formulas giving the temperatures at a given depth one-side exposed concrete slab without cavities. This comparison in illustrated with more details in the following paragraph.

2.3 Numerical models

Two-dimensional thermal and structural models have been implemented in Abaqus (Simulia Corp., 2011). The thermal model represents the transversal cross-section, while the structural model represents a longitudinal strip of the slab. As mentioned in the previous section, both models refers only to the part of the slab within the centreline vertical axes of two subsequent cavities highlighted in the grey box of Fig. 1.

- Thermal model

The thermal model of the cross-section assumes adiabatic conditions for the top and side surfaces. The adiabatic top surface is used in order to model the insulation provided by paving and possible sound insulation on the top of the slab, which limits the dissipation of the heat through the top surface. A fully adiabatic boundary gives the highest temperatures in the slab and is therefore the most conservative situation to assume, in lacking of more detailed information on the thermal properties of the slab superstructure.

The adiabatic boundaries on the vertical sides of the strip are used to simulate the symmetry of the heat flux across those boundaries, when the slab is part of the floor system of a building. The air in the void has not been modelled and therefore the surface of the cavities is regarded as an insulating surface during the whole duration of the fire. This assumption is valid in a first phase of the fire, when the air in the void hinders the transport of heat to the upper part of the section, but may lead to a slight overestimation of the temperatures of the bottom concrete and a slight underestimation of the temperatures of the upper concrete at a later stage of the fire, when the radiation trough the cavities becomes significant (Schiermacher & Poulsen, 1987). However, given the limited time of fire exposure considered, this difference is expected to be minimal and the contribution of the radiation through the cavities has been neglected.

The modelling of the reinforcement has been neglected in the thermal model and the temperatures of the two steel bars have been assumed equal to those of the concrete at the nodes pertinent to the bar positions. This simplification is justified by the small, local effect that the presence of the steel has on the temperature field of the section. On the other side, the modelling of the reinforcement in the thermal model would imply a significantly higher computational effort, due to the necessary refinement of the mesh around the bars in both the thermal and structural models.

The degradation of the concrete thermal properties follows the indications given in the Eurocodes (EN 1992-1-2, 2004), while the thermal properties of the tiles have been considered constant at the initial 20°C value specified in section 1.1.

- Structural model

The slab strip has been modelled by means of rectangular plane elements oriented in the direction of the longitudinal axis of the slab and having a rectangular section of variable width along the vertical axis of the slab cross-section. In order to ensure a sufficiently smooth variation of the section that could approximate the curved surface of the voids, seven different thicknesses have been assigned to the plane elements along the section height.

The reinforcement has been modelled by means of one-dimensional rod elements embedded in the concrete. An elastic-plastic constitutive relation which accounts for the degradation of the stiffness, elastic strength, and ultimate strength has been considered for the steel, in compliance with what specified in the Eurocodes (EN 1993-1-2, 2005).

A damage plasticity model has been adopted for the concrete, which requires the definition of a compressive hardening and a tension-stiffening model. It is well known (Feenstra & De Borst, 1995) that the presence of softening in the constitutive relation causes a dependency of the results on the mesh size, due to strain localization (Bontempi & Malerba, 1997). The objectivity of the solution has therefore been ensured by preserving the fracture energy with a proper calibration of the material softening relation to the mesh, which respects the size limits for the consideration of the interaction contribution (Cervenka, et al., 1990) with the given reinforcement ratio. As noted by Carstensen et al. (2013), the validity of this calibration could be weakened during fire, as the fracture energy and the interaction contribution may degrade with the temperature. However, in lack of experimental knowledge on the variation of the tensile fracture energy with temperature, the fracture energy is usually kept constant during fire. Furthermore, the stiffness of the tensile branch at a given temperature is assumed to be equal to the initial stiffness of the compressive branch. From these assumptions, the degradation of the tensile strength and strain with the temperatures has been derived. The temperature dependency of the stiffness and strength of the compressive stress-strain relationship has been directly taken from the Eurocodes (EN 1992-1-2, 2004).

Due to the presence of geometric nonlinearities, the thermal expansion is expected to affect the vertical displacement of the slab. As a result, a temperature dependent expansion coefficient has been considered for the steel and the concrete, according to the specifications of the Eurocodes (EN 1992-1-2, 2004; EN 1993-1-2, 2005).

3 Results of the investigation

3.1 Thermal analysis

The results of the thermal analysis of the decks are reported in Fig. 2. In the left part of the figure, the variation of the temperatures along the vertical centreline of the section is reported for 30, 60, and 90 min of fire. In the right part of the figure, the thermal map of the section at 90 min is shown.

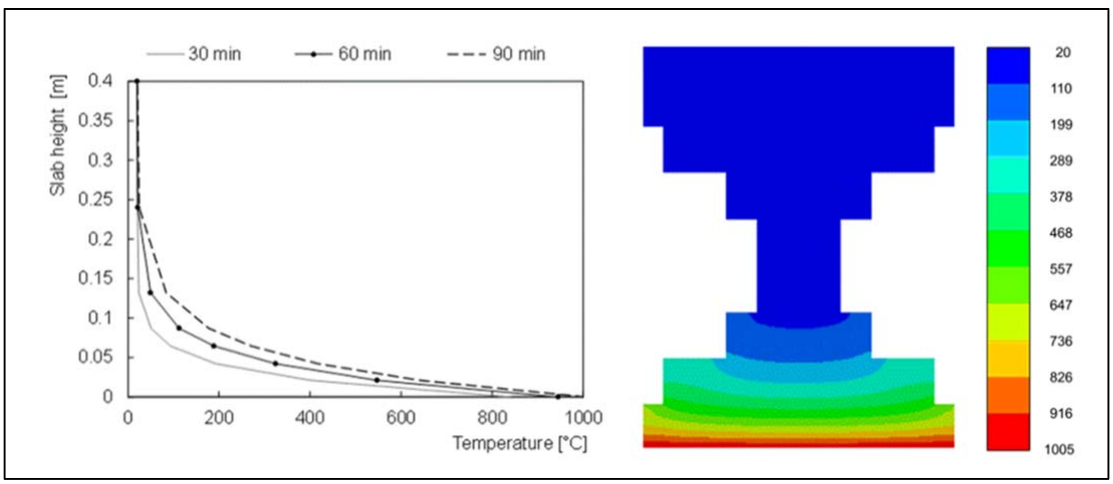


Fig. 2. Results of the thermal analysis: variation of the temperatures at the vertical centerline of the cross-section at 30, 60 and 90 min (left) and thermal map of the section at 90 min of fire exposure (right).

The convexity of the isotherms in the thermal map reflects the insulating effect of the voids discussed in section 2.3 and indicates an overestimation of the temperatures at the edges of the void and an underestimation of the temperatures at the vertical centreline of the section for long fire exposure. However, a comparison with the analytical temperatures obtained for a full 40 cm height concrete slab without voids proves that this overestimation is negligible. These analytical temperatures have been calculated according to two different empirical formulas found by Wickström (1986) and Hertz (1981) for a unidirectional heat transfer through the depth of a concrete slab and reported in Equation 2 and 3, respectively. The comparison of the temperatures is shown in Fig. 3 with respect to the temperature along the centreline of the section at 90 min (left) and the temperature in correspondence of the bottom steel bar for the whole fire duration (right).

Wickström: $$T_c(z, t_h) = \left[0.18 \cdot \ln\left(\frac{\alpha}{\alpha_c} \frac{t_h}{z^2}\right) - 0.81\right] \cdot \left(1 - 0.0616 \cdot t_h^{-0.88}\right) \cdot T_g(t_h) \tag{2}$$

Hertz: $$T_c(z, t) = \frac{312}{345} \cdot (T_g - 20) \cdot e^{-1.9 \cdot k(t) \cdot z} \cdot \sin\left(\frac{\pi}{2} - k(t) \cdot z\right) \text{, with: } k(t) = \alpha \Big/ \sqrt{\frac{\pi}{750 \cdot t}} \tag{3}$$

where:

t_h is the time of exposure to a standard fire expressed in hours;
t is the time of exposure to a standard fire expressed in minutes;
T_g is the temperature of the fire at the time t_h or t;
z is distance from the exposed surface of the slab to the point of interest;
T_c is the concrete temperature at a depth z into the slab and at a time t_h or t;
α_c is the reference diffusivity, equal to $4.17 \cdot 10^{-7}$ m^2/s
α is the constant equivalent value of the slab diffusivity, assumed equal to $5.64 \cdot 10^{-7}$ m^2/s and calculated as:

$$\alpha = \lambda_{eq} / (\rho \cdot c_{eq}) \tag{4}$$

where:

ρ is the density of the concrete at 20°C, previously indicated as 2300 kg/m3;
c_{eq} represents the constant equivalent value of the specific heat, assumed equal to 1000 J/(kg·K), as suggested in literature (Hertz, 1981), (Buchanan, 2002).
λ_{eq} represents the constant equivalent value of the conductivity, assessed as 1.3 W/(m·K) as suggested by Buchanan for calcareous concrete (Buchanan, 2002).

It should be noted that the value of the diffusivity used differs from the reference values suggested by Wickström and Hertz in their methods, which are 4.17 m^2/s and 3.26 m^2/s respectively. The reason is that the suggested values have been calibrated on a concrete with a lower conductivity than the one considered in this study. In particular, a value of the conductivity of 0.75 W/(m·K) is suggested in Hertz's model, which corresponds the conductivity calculated at an average temperature of 600°C on the minimum curve indicated for the conductivity by the Eurocodes (EN 1991-1-2, 2002). This curve corresponds to a concrete with conductivity at 20°C equal to 1.33 W/(m·K), i.e. significantly lower than the one of the considered slab. Therefore, given the high sensitivity to the conductivity of both temperature models, the suggested values couldn't be used, and a higher value of diffusivity is used, which is based on the equivalent conductivity calculated on the basis of the temperature dependency of the actual conductivity of the slab.

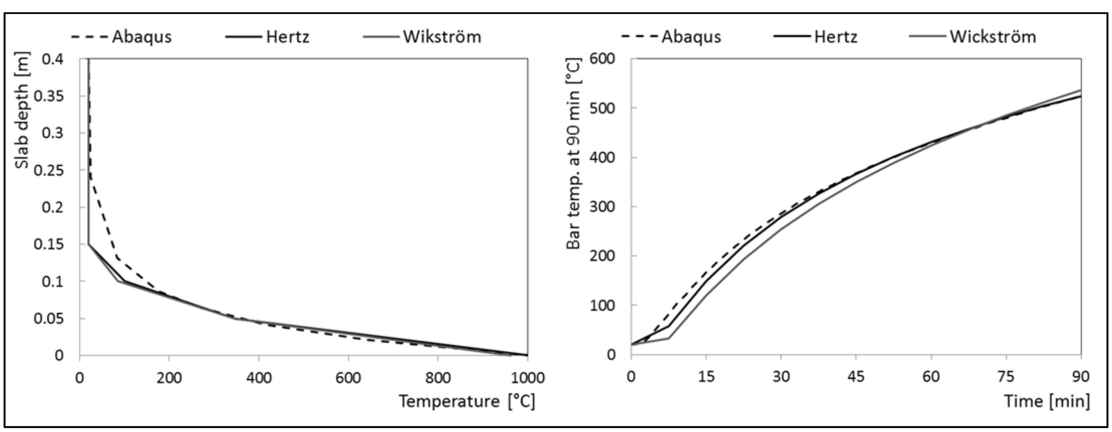


Fig. 3. Temperatures along the section vertical axis at 90 min (left) and of the bottom steel over time (right)

3.2 Structural analysis

The structural model has been validated against the analytical results obtained for the load bearing capacity at the beginning of the fire (t = 0), by running a nonlinear static incremental analysis without the thermal step for the temperatures. The calculated load bearing capacity p_u^0 = 20.2 kN/m2 results to be slightly higher than the analytically calculated one reported in paragraph 2.1. The difference is due to a small overestimation of the tensile resistance, which is needed to facilitate the convergence of the analysis. However, this effect is quite small and also limited to the very beginning of the fire, when the temperatures of the bottom concrete are low and the tensile resistance of the concrete is not significantly reduced by the thermal load.

The results of the structural analysis of the slab under fire are summarized in in Table 2: in the first column, the value of the solicitant load p_s^{fi} calculated in paragraph 2.1 for the fire case is reported for the convenience of the reader; in the second column labeled p_u^t, the load bearing capacities at t = 0, 30, 60, and 90 min are listed; the third column reports the residual capacities (p_u^t - p_s^{fi}) for the considered times of exposure; the fourth column μ^t shows the utilization factors for the considered times of exposure; finally, the fifth column shows the ratio between the residual capacity at the considered time of exposure and the residual capacity at the beginning of the fire. This index is calculated as shown in Eq. 4 and referred to as Fire Vulnerability Index (FVI). It assumes values in the interval (1, 0), where 1 corresponds to the nominal situation at the beginning of the fire and 0 corresponds to a collapse condition reached after sufficient fire exposure.

$$FVI^t = \frac{p_u^t - p_s^{fi}}{p_u^0 - p_s^{fi}} \qquad (5$$

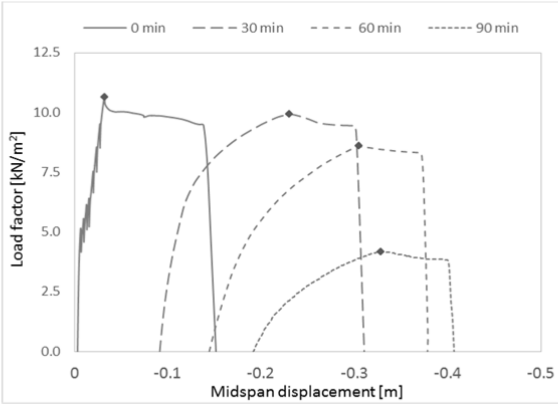
The use of such index seems preferable in order to extend this procedure to other slabs, which may have different loading conditions and different initial load bearing capacities. In this respect, an index based on the percentage decrement of the load bearing capacity is used in literature for the assessment of the permanent damage of R.C. beams after fire exposure and referred to as Damage Factor (Jayasree, et al., 2011). Here, the residual load capacity is instead used as reference parameter, as it better expresses the decrement of the safety level of the slab during fire.

By looking at the results reported in Table 2, it can be seen that the load bearing capacity of the slab decreases significantly after 90 min of fire. However, the slab does not fail and still retains 40% of the initial residual capacity (FVI^{90} = 0.4). Even if the slab resists the maximum duration assumed for the fire, the degradation of the slab performance is not proportional to the fire duration and the load bearing capacity has a drop after 60 min of exposure.

This is better highlighted in Fig. 4: on the left of the figure, the load-displacement (push-over) curves obtained as analysis output for the different fire durations are compared, while on the right of the figure the variation of the vulnerability index with the fire duration is shown. This curve is referred in the following as fire vulnerability curve and more clearly highlight a drop of the load capacity after ca. 60 min of fire (point of maximum curvature). This result is consistent with the fact that the fire performance of the slab is driven by the behavior of the steel reinforcement, which has a drop in the resistance when the temperature exceeds 400°C.

Table 2: Resulting capacities of the slab and vulnerability indexes for different fire exposure

t	p_s^{fi}	p_u^t	p_u^t-p_s^{fi}	μ^t	FVI^t
[min]	[kN/m^2]	[kN/m^2]	[kN/m^2]	[kN/m^2]	[kN/m^2]
0	9.6	20.2	10.6	48%	1
30		19.5	9.9	49%	0.93
60		18.2	8.6	53%	0.81
90		13.8	4.2	70%	0.40



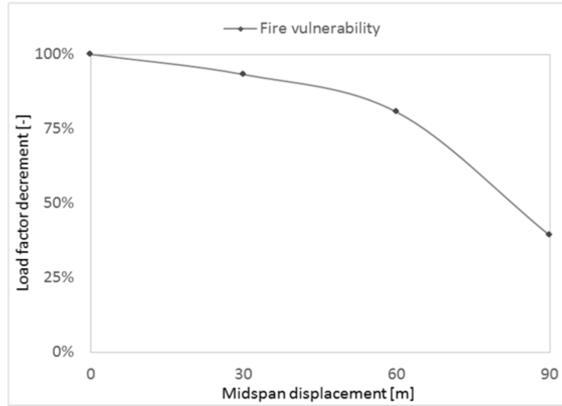


Fig. 4. Pushover analyses (left) and variation of the vulnerability index for 0, 30, 60, and 90 min of fire

4 Conclusions

The numerical modelling and analysis of a hollow core slab with slack reinforcement in fire have been presented. The analysis was aimed at highlighting the decrement of the load bearing capacity of the slab in bending for increasing severity of the fire and has not taken into account possible loss of anchorage of the reinforcement or spalling problems. In order to make the result independent from the building where the slab is used, the slab has been considered exposed from the bottom to a standard fire and its load bearing capacity has been assessed for 0, 30, 60, and 90 min of fire.

The results shows an overall good response of the slab to fire, as it does not fail and still exhibits some residual load capacity after 90 min of fire. However, the decrement of the capacity is not proportional with the time and has an abrupt drop after 60 min of fire.

In order to measure the effects of the fire on the slab, a vulnerability index is proposed, calculated as ratio between the residual capacity at the end and at the beginning of the fire. The index can be used as measure of the sensitivity of the slab to the fire, which is not always adequately represented by the degradation of the stiffness: the smaller the index, the higher the vulnerability of the element to the given fire. By diagramming the variation of this index with the time of fire exposure, a fire vulnerability curve is obtained, which provides more general information on the response of the element to increasing duration of the fire, such as: the maximum resistance time (e.g. the exposure time corresponding to null residual capacity) and the optimal time of exposure, after which the element exhibit a rapid decrement of the capacity (e.g. the time corresponding to the point maximum curvature).

The latter information can be useful in the view of a design aimed at ensuring not only the stability of the building for the required fire duration, but also limited economic consequences for the reparation of the concrete elements. In this case, the procedure could be extended to the consideration of the permanent loss of capacity after the element cooling.

The same procedure shown here for a hollow core slab can be followed for obtaining fire vulnerability curves of other slab typologies as well as precast beams with a complex geometry of the cross section. The curves can be used for comparing different design alternatives and optimize the design of concrete elements with respect to the required resistance class of the building. In this respect, it is important to recall that precast and R.C. sections are generally optimized with respect to service conditions. However, in case the element is particularly vulnerable to fire, significant modifications to the original design may be needed for compliance with the fire requirement, which may reduce or even nullify the effect of the initial optimization. The proposed analysis is particularly useful in these cases, as it allows for an optimization of the design with respect to the fire condition (Madsen & Lange, 2014).

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