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STRUCTURAL BEHAVIOR OF STEEL STORAGE RACKS UNDER DIFFERENT FIRE SCENARIOS

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Abstract

Steel solutions commonly used in logistic to store goods and products (i.e. racks) are assembled by thin-walled cold-formed components and their competitiveness on the market depends essentially on the total weight of the framed system. Despite great efforts have been done on seismic and static design, very limited attention was given to the problem of the fire design and to the robustness of such structures. Due to the limited thickness of the structural elements, it is well-known that their resistance to the fire load is quite limited. No specific design rules or design procedures have been developed till now. Nowadays, the only way to protect these frames against fire is the use of sprinkler or more complex solutions, like isolated chambers. However, fire hazard cannot be always eliminated and when these strategies were not effective, global structural collapse of the racks, during a fire, have been observed.
In this paper a parametric analysis is proposed to deeply understand the behavior of racks against fire. In particular, starting from an existing rack configuration, different fire scenarios were modeled by changing the fire position along the fame and the fire design curve. Moreover, the progressive collapse of the rack is discussed proposing also two reinforcing strategies.
Final results discuss about the best way to design these slender structures against fire, trying to prevent the global failure or, at least, to guarantee a safe collapse mode.

Keywords: Fire design, thin-walled members, steel racks, global instability.

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1 INTRODUCTION

An important area of the steel construction industry is represented by structural systems made by thin-walled cold-formed components [1], which are extensively employed in logistic to realize frames used for the long- and medium-term storage of goods and products. One of the most common types of storage solutions is the so-called adjustable selective storage pallet rack (in the following simply identified as *rack*), which is the core of the present paper. As shown in Fig. 1, the vertical elements (uprights) are jointed with diagonals and struts in the transversal (cross-aisle) direction forming a set of trussed columns connected in the longitudinal (down-aisle) direction to each other by pairs of pallet beams directly supporting the stored goods. At each beam end, a shop welded bracket with hooks is accommodated into special slots regularly pitched along uprights allowing for a rapid assemblage of the skeleton frame [2]. Generally, due to logistic reasons, bracing systems (spine bracings) are rarely used in the down-aisle direction.

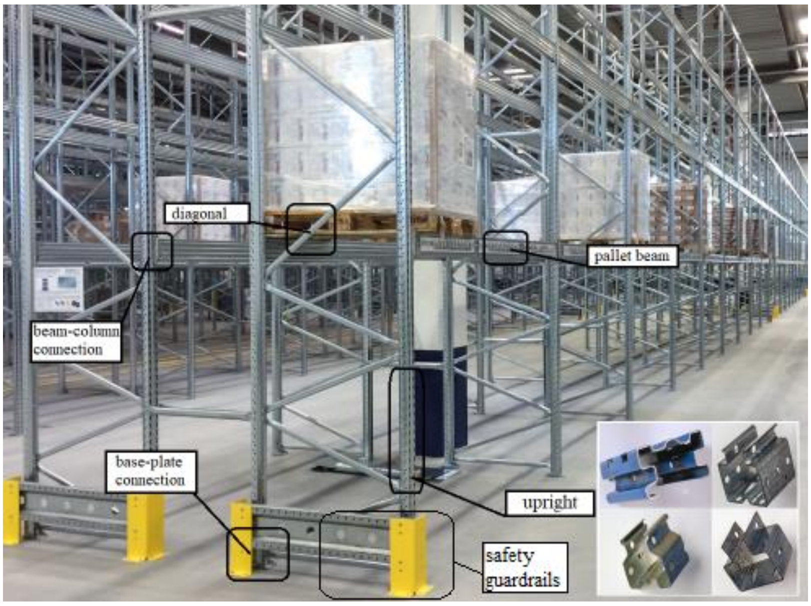


Figure 1: Example of unbraced rack(a) and its main components (b-d) [3].

Racks are widely adopted in warehouses, and they are loaded with tons of valuable goods. The loss of goods after a structural collapse may represent, for the owner, a very large economic loss, much larger than the cost of the whole rack on which the goods are stored, as highlighted by the quite recent collapses [4]. Racks are also frequently adopted in supermarkets and shopping centers, in areas open to the public. A global structural collapse, in this case, may endanger the lives of the customers, as well as that of the workmen and employees, involving not only Civil and Penal Right considerations about the liability of the owners, but also economic considerations related to insurance coverage.

A large number of research have been carried out on the behavior of racks, paying attention mainly to the seismic and static response. In fact, it is possible to assert that while the seismic behavior of these structures has been widely investigated [5], very limited attention has been focused on the problem of fire safety and structural robustness. Robustness is generally defined as the ability of a structure to withstand out-of-the-scheme events, such as impacts or fire without being damaged to an extent disproportionate to the original cause [6]. In case of fully loaded racks, a local damage generally propagates to the entire rack system and, in the worst case-scenario, to the neighboring ones causing the so-called *domino effect*.

Prediction of the structural performance of these structures is cumbersome because it is affected by many parameters like the non-standard geometries of their structural components which are thin-walled elements, the strong nonlinearity of the joint response and the great dead-to-live load ratio. For these reasons, the only way to design racks is to follow the *design assisted by testing* procedure: the numerical models used for the structural analyses must be based on *ad-hoc* components tests [7].

Fire effects on steel structures can be very dangerous: the variations of mechanical properties due to temperature variations can strongly reduce the bearing capacity and the induced thermal deformation can affect the global structural behavior [8]. For this reason, in case of fire, both mechanical and geometrical non linearities should be taken into account. The understanding of the fire effects on such structures is even more complex. The different protection strategies and methodologies proposed in the past for regular steel structures [9-12] cannot be directly extended to rack frames due to the limited thickness of the sections. The very thin thickness of zinc that is generally present on the structural members can guarantee only a poor protection [13] and the intumescent painting is not an efficient solution. In fact, the only effective strategy, which is in use nowadays, is the use of sprinklers.

In past research only local elements of rack under fire have been proposed: Ren et al [13] investigated the effect of localized fire on the uprights, proposing updated buckling curves for design purposes and Shah focused on the response of beam-to-column connections [14]. On the contrary, in the current paper, the whole structural system is investigated, by performing an extensive parametric analysis on an existing rack by varying the fire type, its position and intensity. In particular, attention is focused on the type of collapse induced by the fire position and different reinforcement strategies were investigated.

2 HYPOTESES FOR FIRE ANALYSIS

In this study, the fire load is represented by a temperature time history curve applied directly to a single span (Fig. 2a) or to a single beam element (like an upright), rather than to the entire structure. This simplification of the problem is considered acceptable for the purposes of the paper objectives. This procedure allows for the identification of most critical zone of the rack when subjected to a fire; on the contrary, if the same temperature were applied to the whole frame, only a global and simultaneous collapse would occur. Furthermore, numerous studies in the literature adopted a similar strategy for the application of the temperature curve [15].

In addition, it should be noted that the ISO temperature curves, used in the following, are representative of the time-temperature curve of a compartment and not of a single element. Indeed, it should be necessary to also consider convention and radiation of the heat to evaluate the element temperature. The increment of temperature can be represented by equation 1) reported in EN1993-1-2 [16]:

$$\Delta\theta_{a,t} = k_{sh} \cdot \frac{A_m}{V} \cdot \frac{1}{c_a \rho_a} \cdot \dot{h}_{net} \cdot \Delta t \tag{1}$$

where, $\Delta\theta_{a,t}$ is the increment of the temperature in the steel in a specific time Δt, A_m/V is the section factor of the structural element under consideration, c_a is the specific heat, ρ_a steel density, h_{net} heat flux and k_{sh} factor for the shadow effect.

By applying eq. 1) to the elements of the considered rack, it should be noted that when thin-walled steel profiles are considered, the temperature of the compartment (blue line in Fig.

2b) can be considered the same of the one evaluated in the structural element (grey line in Fig. 2b).

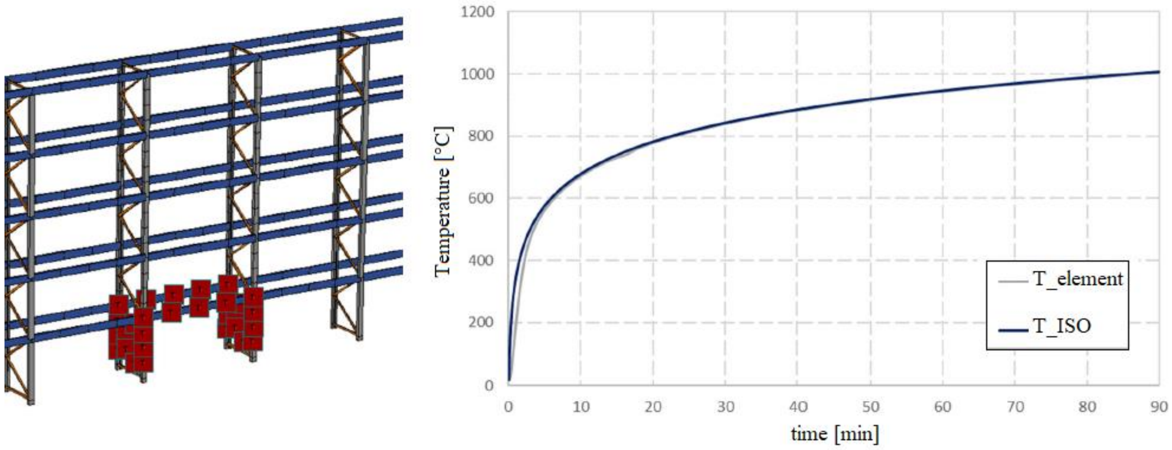


Figure 2: hypothesis on the application of the temperature curve.

3 THE CONSIDERED CASE STUDY

The case study consists of an existing rack having 9 bays with 2780mm of span and 5 storage floors, with 1330mm of interstorey, for a total height of 6650mm. In the transversal direction, the upright frame is composed by a k-bracing type with a step of 665mm and having 1000mm of width (Fig. 3). In the standard (S) configuration all the columns are made by a 75x115x50 lipped C-section with 2mm of thickness, all the beams are 40x160 rectangular hollow profile with a thickness of 1.3 mm and the diagonals are hollow square section of 35mm with a thickness of 2mm. The same geometric layout was considered with standard elements or with two different reinforcement hypotheses (R and RR). The material is the same for all the elements: steel S350GD with a Young modulus equal to 210 GPa and ν=0.3.

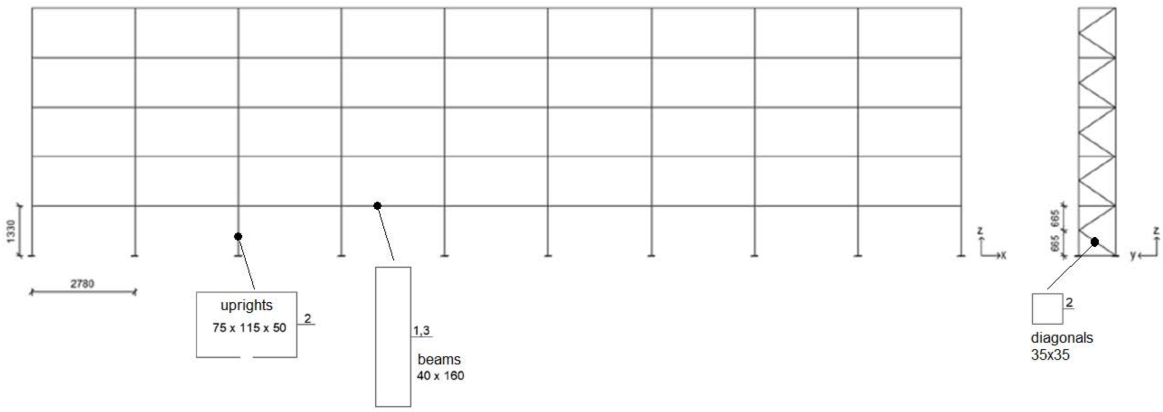


Figure 3: The considered rack geometry (dimensions in mm).

In the first reinforced model (R) the thickness of all the beams was increased up to 2mm; while in the second reinforced configuration (RR) together with the increase of the beams thickness, the first and the second storage level column thickness were increased too (4mm). The load was considered as uniform distributed on all the beams and equal to 4.0 N/mm to simulate the presence of 3 pallets of 800kg each. Beam-to-column and internal and external base-plate connections have been assumed as non-linear, i.e. characterized by a moment-rotation rule, and their response have been calibrated based on previous cyclic and monotonic tests. The variability of the connection properties with the temperature were not considered in the paper. For all the configurations two different fire type were considered (ISO834, named ISO, and the fire hydrocarbon curve, named HYD [16]). Moreover, the fire position was varied considering the internal (C) or the external (L) position and from the first to the fifth storage levels. The fire was considered acting on a single column (SC) or on a complete bay (CB). The layout of all the considered analyses is depicted in Fig. 4.

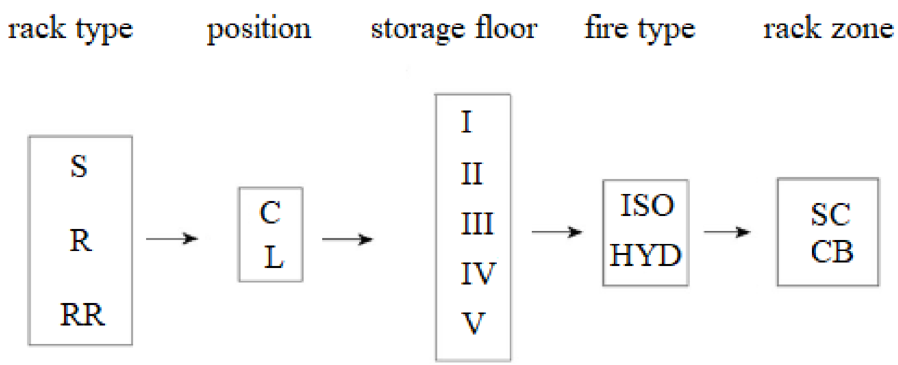


Figure 4: Layout of the considered cases.

As an example, the case S_C_III_ISO_CB case is depicted in Fig. 5. which represent the standard rack subjected to a fire (represented by the ISO curve) on a central bay at the third storage level.

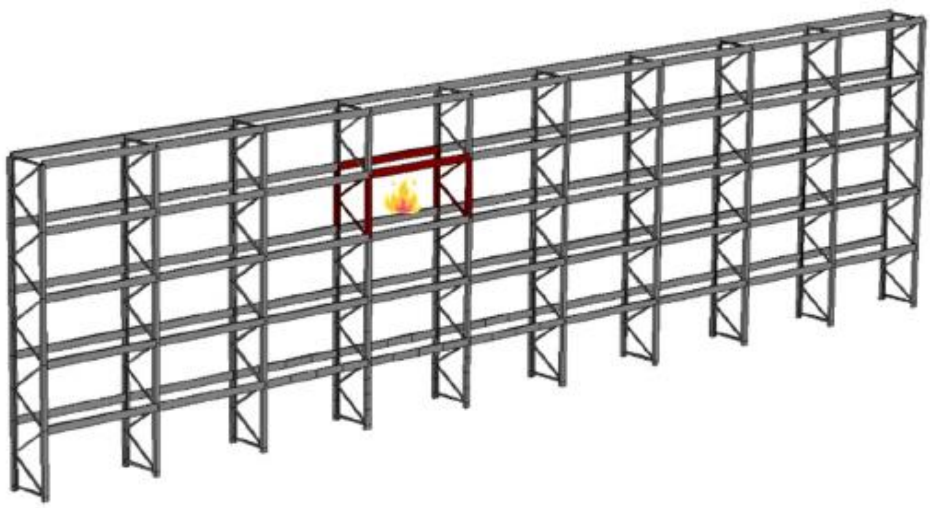


Figure 5: Example of the S_C_III_ISO_CB case (in red the elements in which the ISO curve was assigned).

Nonlinear thermomechanical analyses were performed by using the finite element software Straus7 [17], following two main steps: i) thermal analysis, which gives as output the temperature on each structural node for each time interval; ii) non-linear mechanical analysis by considering second order effects and mechanical nonlinearities. According to [16] the Young modulus, the yielding stress, the conductivity and the thermal expansion of the material were modeled as non-linear and dependent on the temperature value.

Once obtained the internal stresses for each step of the analyses the safety index, SI, were evaluated on all the elements in accordance with EN15512:2020 [7], considering the stability equation for the element in bending and compression:

$$\left(\omega_{x,y}\frac{N_{Ed}}{\chi_y N_{Rd}}\right)^{\alpha_y}+\left(\omega_{x,LT}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT}M_{y,Rd}}\right)^{\beta_y}+\left(\frac{M_{z,Ed}+\Delta M_{z,Ed}}{M_{z,Rd}}\right)^{\delta_y}\leq 1{,}0$$

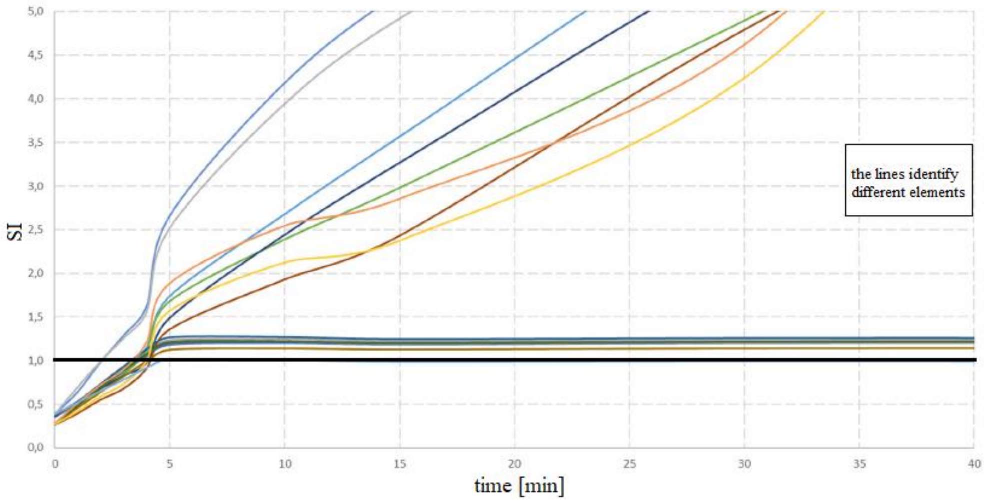
$$\left(\omega_{x,z}\frac{N_{Ed}}{\chi_z N_{Rd}}\right)^{\alpha_z}+\left(\omega_{x,LT}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT}M_{y,Rd}}\right)^{\beta_z}+\left(\frac{M_{z,Ed}+\Delta M_{z,Ed}}{M_{z,Rd}}\right)^{\delta_z}\leq 1{,}0 \tag{2}$$

where N_{Ed}, $M_{y,Ed}$, $M_{z,Ed}$ are the acting axial force and bending moments along y- and z- directions; N_{Rd}, $M_{y,Rd}$, $M_{z,Rd}$ are the resistant axial force and bending moments along y- and z- directions; χ_y, χ_z, χ_{LT} are the reduction factors for the global buckling and $\omega_{x,y}$ $\omega_{x,z}$ $\omega_{x,LT}$ are the interaction factors.

From a numerical standpoint, it should be noted that also when in a specific step one or more elements reached the SI equal to one, the step forward happened anyway, until an eventual convergence problem. On the contrary, from a physical point of view the low redundancy of the structure, suggests that when one element lose is carrying capacity (upright or beam) also the whole frame collapse. The results are hence proposed continuously in time, but the global collapse has been assumed in correspondence of the first SI = 1.

4 RESULTS

The result obtained for the two considered temperature curves are really close to one another and, for the sake of brevity, only the ones associated with the ISO curve are discussed in the paper. for the S_C_I_ISO_CB case is proposed in Fig. 6a, in terms of SI versus time. In the figure the SI of all the columns and beams at the first storage level are proposed grouped together. This figure is representative of all the case, in particular it can be noted that before 5 minutes a great number of beams and columns reached the collapse, generating a global collapse of the whole frame. Due to the increase of the axial force in the beams and to their limited thickness, some of them reached the SI = 1 also within 1 minute, as showed in Fig. 6b for the same S_C_I_ISO_CB case. It can be concluded that for the specific case the collapse of the frame is reached in 1 minute.



a)

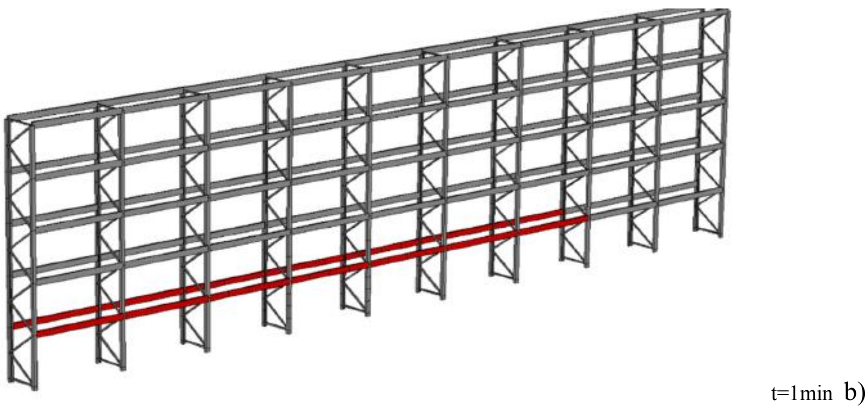


Figure 6: S_C_I_ISO_CB case, (a) SI results versus time and (b) element collapsed after 1 minute (in red).

At the same way, when the fire is applied to a single column (SC) instead of to the whole bay, the collapse of the first storage level beams is reached in 1 minute. In this case the frame exhibits a lateral global displacement of the central upright frame (Fig. 7b). The failure of the frame in the first case happens with a local collapse of the first storage level beams followed by a longitudinal global displacement. In the second case the global collapse starts with the local failure of the burned upright cause a lateral global deformation of the structure. The second ode is the most dangerous because in a real warehouse where there are multiple racks next to each other, it would lead to a *domino effect*.

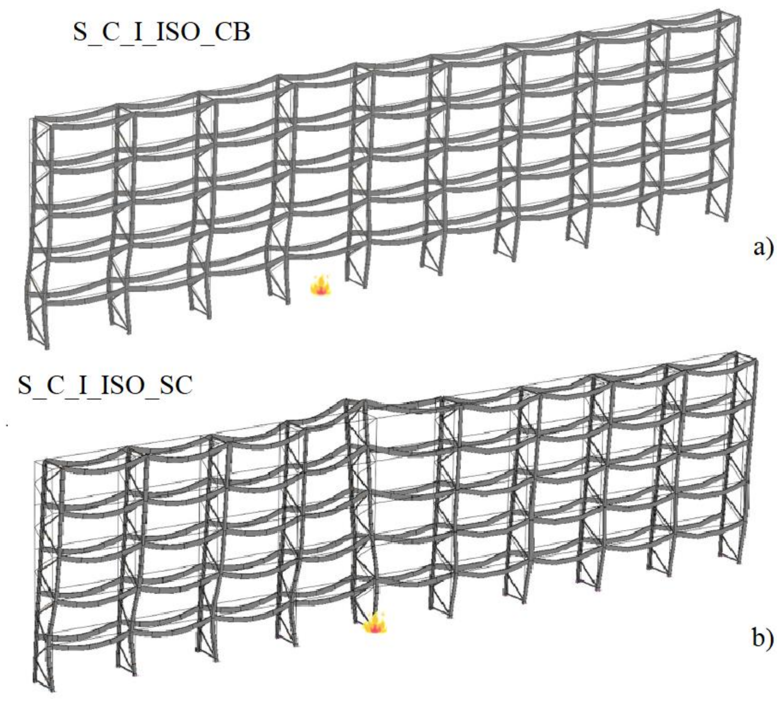


Figure 7: Global deformed shape after 1 minute of fire, (a) S_C_I_ISO_CB and (b) S_C_I_ISO_SC.

All the results obtained for the standard rack, are summarized in Fig. 8, in terms of number of collapsed elements versus the time of the analysis. The position of the fire has been varied by changing the storage floor and spatial zone (central or lateral). The ISO 834 curve is considered with a whole bay subjected to the fire action.

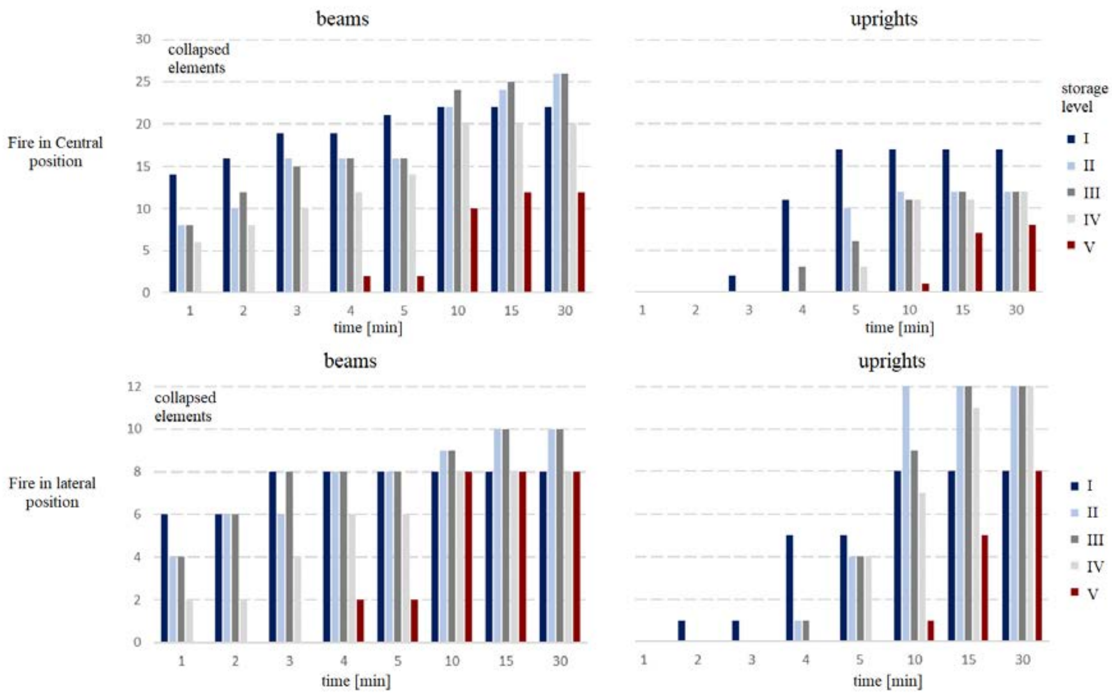


Figure 8: Number of collapsed element for S frame subjected to different frame scenarios.

For all the considered cases the time of collapse of the elements is always limited, in fact within the 5 minutes of exposition a non-negligible number of uprights and beams collapsed. For the other cases, independently on the location of the fire, the beams collapsed within the first minute of fire. The uprights are less sensitive to the fire and their collapse is more pronounced when the columns of the first storage level are directly exposed.

Finally, the comparison between the standard rack and reinforced ones is proposed in Tab. 1, for the case different scenarios.

Table 1: Collapse time of the first element for different scenario.

scenario	S	R	RR
C_I_ISO_CB	1 min	1 min	2 min
C_II_ISO_CB	1 min	1 min	2 min
C_III_ISO_CB	1 min	2 min	2 min
C_IV_ISO_CB	1 min	4 min	4 min
C_V_ISO_CB	4 min	10 min	10 min
L_I_ISO_CB	1 min	1 min	2 min
L_II_ISO_CB	1 min	1 min	2 min
L_III_ISO_CB	1 min	2 min	2 min
L_IV_ISO_CB	1 min	4 min	4 min
L_V_ISO_CB	4 min	10 min	10 min
C_I_ISO_SC	1 min	1 min	2 min
L_I_ISO_SC	1 min	1 min	2 min

As showed in the table, the proposed reinforcements are not effective because despite the thickness of the profiles have been increased more than 2 times, their value is still very low. As discussed in the introduction, also the application of painting or the zinc coating is not an effective solution to guarantee at least 15 minutes before the failure of 1 or more elements.

For this reason, the only effective solutions for such structures seem to be the use of *ad-hoc* well-designed sprinkler system or the use of special compartment with a controlled level of oxygen.

5 CONCLUDING REMARKS

In the paper, the response of a rack frame under different fire scenarios has been investigated, focusing attention on the safety of the structure during the time in which it is exposed. An extensive parametric analysis by varying the fire type, its position and intensity is discussed showing the very low level of reliability of such structures, mainly due to the very low thickness of the structural members. Nonlinear numerical thermo-mechanical analyses were performed by considering but geometrical and mechanical nonlinearities.

Two reinforcement strategies were discussed: i) the increasing of the thickness of the beams and ii) the thickness increment of both beams and columns. Both strategies were not really effective and are not able to satisfy a minimum level of performance. In fact, all the structures presented the failure of one or more structural element (beam or upright) in a time lower than 5 minutes, independently of the fire position and on the curve type.

For this reason, it can be concluded that the only effective solutions to design rack structures against fire seem to be the use of *ad-hoc* well-designed sprinkler system, which need of constant maintenance, or the use of special compartment with a controlled level of oxygen, which is quite expensive strategy and not always suitable. Both strategies will be investigated in the future.

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