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Assessment of post-earthquake fire capacity of RC frames without seismic design

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A R T I C L E I N F O

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A B S T R A C T

To study the importance of post-earthquake fire events, 500 reinforced concrete (RC) frames were designed without seismic considerations. These consist in three-bays frames with a variation in the number of floors from two to six (100 frames each). Numerical analyses were performed in SAFIR to investigate the influence of different types of damage, damage location, fire location and fire curves on the post-earthquake fire resistance of the structures. It was observed that the seismic damage can significantly decrease the fire resistance of the frames. In an earthquake scenario it can be expected an increase in the response times of the firefighters as well as malfunctioning and damage of active and passive firefighting measures, which means that the use of parametric fire curves without active firefighting measures can be a better approach instead of the use of the fire curve ISO 834. It was observed, for the analysed cases, that parametric fire curves without active firefighting measures led to lower collapse times when compared to the fire curve ISO 834. On the other hand, when the active firefighting measures are considered, there is a very significant number of structures that do not collapse, which indicates the importance that these firefighting measures can have in a fire after an earthquake. The development of these analysis emphasizes the impacts of applying different fire curves on the damaged structure which consequently brings attention to an adequate definition of the fire curves.

1. Introduction

Earthquakes can cause fires that can turn into major conflagrations as was observed in the 1906 San Franscisco and the 1923 Kanto earthquakes [1]. The 1906 San Francisco conflagrations burned for three days [2–4]. The earthquake and fires destroyed more than 28,000 buildings and killed at least 3,000 people [2,5,6–9]. The Kanto earthquake generated 277 fires and almost half of these fires spread [1,6,10]. About 40 % of Tokyo and about 90 % of Yokohama were destroyed by the earthquake and fire [1,11]. There was a fire destruction of approximately 447,000 buildings over an area of 38.3 km^2 [2,6,9,10,12]. These two examples indicate the possible destructive consequences that post-earthquake fires can have in the built environment.

In some numerical works that researched the post-earthquake fire in RC structures it was perceived that structures with earthquake damage have lower fire resistance when compared with structures with no damage [13–17]. Specifically, some studies indicated that

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the difference in the time until collapse between structures with and without earthquake damage can be higher than two hours [18–20]. This reduction in the fire resistance can be related to the residual lateral displacements, the degradation on strength and stiffness, and the heating of the steel reinforcement due to the removal of the cover [13,15,16,21]. Most of the studies regarding the evaluation of the post-earthquake fire resistance of reinforced concrete structures consider the fire curve ISO 834, but there are other fire curves that can also be considered [22]. In a study it was compared the post-earthquake fire resistance of a three-bay three-story RC frame when subject to the fire curve ISO 834 and when subjected to iBMB (Institute of Building Materials, Concrete Construction and Fire Safety) fire curve [23]. The iBMB fire curve is a parametric natural fire model developed in [24]. It was observed that, based on the load bearing criteria, the post-earthquake fire resistance of the structure subjected to the fire curve ISO 834 was around 120 min while the post-earthquake fire resistance of the structure subjected to the fire curve iBMB was around 80 min [23]. This aspect shows the impact of considering different fire curves and indicates that despite the fact of the fire curve ISO 834 being a curve where the temperature always increases with time it does not necessarily means that will lead to lower fire resistance when compared to a fire curve with a cooling phase, such as the iBMB fire curve.

The development of some experimental works showed that severe loss of the column's concrete cover caused a faster heat penetration into the core of the concrete section in the damaged region [25]. In other experimental studies developed in RC frames it was observed that higher levels of cyclic loading leads to wider cracks which consequently can lead to higher temperatures in the elements [26,27]. On the other hand, in some studies it was observed that small cracks do not have a substantial impact on the thermal evolution through concrete [28,29]. It was also observed that the location of the damage may not coincide with the location of the fire, and that the openings and the development of the fire in the compartment have a significant influence in the temperature evolution in the elements [30]. The tests showed the vulnerability of thin elements (e.g. slabs and shells) of non–ductile RC frames to spalling in post-earthquake fire events [31,32]. The brick infill walls provide an insulation to the RC structural elements that lead to a slow transmission of heat to these elements. This is a beneficial aspect of masonry walls that should be considered while designing the columns and beams which are integrated in masonry walls [31]. The cracks that appeared in the different elements of the test structure seem to indicate that the details of the reinforcement may have implications for the global behaviour of the structure when exposed to fire [26,27]. The experimental test reflects the better performance of the RC frames with ductile detailing [27,30,32]. The recommendations that are generally used for the seismic design are also important in increasing the fire resistance of the structure [27,30].

The aim of the present work is to comprehend the influence that different types of damage and fire can have on the fire resistance of RC frames, and the novelty of the work primarily lies in the comparison of various fire curves, including the standard fire curve ISO 834 and parametric fire curves with and without active firefighting measures. In this study, 500 three-bay RC frames were designed. These frames encompass 100 frames with two, three, four, five and six floors. Then, based on these frames, several studies were developed with different damage and fire scenarios. It was observed that seismic damage can significantly reduce the fire resistance of the RC structures. Furthermore, it was also observed that different fire curves can lead to significant differences in the fire resistance of the RC structures.

2. Simplified methodology for post-earthquake fire assessment of RC frames

2.1. Numerical modelling assumptions

The designed frames were analysed under fire conditions with SAFIR, which is a software that can perform analysis of structures at elevated temperatures, where the thermal and mechanical analysis are developed individually and in sequence [33,34]. A fibre model is used to define the cross-section geometry of the elements, where each fibre can have a different material allowing the consideration of composite sections, such as reinforced concrete [34,35].

The concrete model is based on the laws of EN 1992-1-2 and incorporates an explicit formulation [36]. This refinement of the model also considers the non-reversibility of transient creep strain when the stress and/or temperature is decreasing. During cooling, the mechanical properties of strength and strain at peak stress are not reversible [37].

The numerical analyses end at a time defined by the user, if a state of equilibrium is not found (no convergence) or if a numerical problem is found at the material level [35,38]. The adopted strategy for the numerical modelling of the post-earthquake fire assessment of RC frames involves the definition of the sections of the columns and beams with different damages and fire curves. The developed sections are then used in the frames with different characteristics. In the end, the time until collapse of each frame is analysed to ascertain the impacts of damage and fire.

2.2. Seismic damage

It was considered a reduction of the concrete cover in the RC elements (columns and beams) in the plastic hinge region to simulate the earthquake damage. In the current study, there were considered three types of damage, namely, damage D0/D1, D2 and D3. Based on the option on experimental observations, it was assumed that moderate seismic damage can be associated with extensive large cracks and spalling of concrete and severe damage with extensive crushing of concrete and disclosure of buckled reinforcements [39]. Due to the number of analyses performed, it was necessary to establish simplified criteria to represent the different levels of physical earthquake damage in the models. As so the damage D0/D1 relates to an intact section (D0) and a section with minor cracks (D1). The damage D2 relates to a section with slight damage, with a removal of 50 % of the exterior fibres representing the cover of the section. Finally, the damage D3 relates to a section with severe damage, with the removal of the entire concrete cover. Studies exploring alternative methodologies for incorporating damage representation in such analyses could be pursued in future research.

It was considered a combination of the models D0 and D1 because according to the state-of-the-art, small cracks do not have a substantial impact on the thermal evolution through concrete [28,29]. It was considered that all sides of the column cross-section have damage and in the cross-section of the beam, the damage is only considered in the lateral sides and in the bottom because it was assumed that the detachment of concrete in the top side of the beam due to the earthquake damage is restraint by the slab. The sections of the columns and beams with the three types of damage are represented in Fig. 1. A scheme of the simplified modelling strategy is represented in Fig. 2.

3. Post-earthquake fire assessment in RC frames without seismic design

3.1. Introduction

In Fig. 3 it is presented an overview of the several steps developed for the post-earthquake fire assessment in RC frames without seismic design and in Fig. 4 it is presented a diagram of the models developed in SAFIR. In the beginning there is a sampling of the properties of the frames which is followed by the design of the frames. Then, the designed frames are modelled in Seismostruct, where a displacement-based adaptative pushover is developed and the capacity curves and drifts are obtained [40]. These results allow to establish a relation between the drifts and the damage in the frames. Then there is the modelling of the frames in SAFIR, where different scenarios and fire curves are considered in the analyses [33,34]. The results obtained in SAFIR allow the development of cumulative distributive function curves of the time until collapse of the frames. The following sections explain in detail each component presented in Fig. 3.

3.2. Properties and characteristics of the frames

There were developed 500 three bays RC frames without seismic design that are representative of a building stock. These frames include 100 frames with two, three, four, five and six floors. The characteristics and properties of the frames were obtained from a report that statistically describes the geometric and material mechanical properties of RC buildings designed without seismic detailing [41].

The variables used for the design of the frames are presented in Table 1 and the properties of concrete and reinforcing steel are respectively presented in Table 2 and Table 3. The variable H_1 relates to the height of the first floor, $H_{>1}$ is the height of the other floors, L_{beam} is the length of the beams, f_{cm} is the mean value of concrete compressive strength, G is the self-weight, P_l is the percentage of longitudinal reinforcing steel present in the columns, f_{yk} is characteristic yield strength of the reinforcing steel and c is the cover of the columns and beams. The parameters A and B are the truncation values of the distributions considered for the development of the frames, which means that values lower than A and higher than B were not used in the numerical analyses.

The design of the frames was developed according to the Portuguese regulation for RC structures [42–44]. At the time of these regulation there were no seismic requirements for the design of structures. The chosen values of the cover are based on the prescriptions presented in these regulations. The thermal conductivity values used in the numerical analysis correspond to the mean value, between the upper limit and the lower limit presented in EN 1992-1-2 [36].

The design of the frames starts with a random selection (not correlated) of 100 values from each variable presented in Table 1,

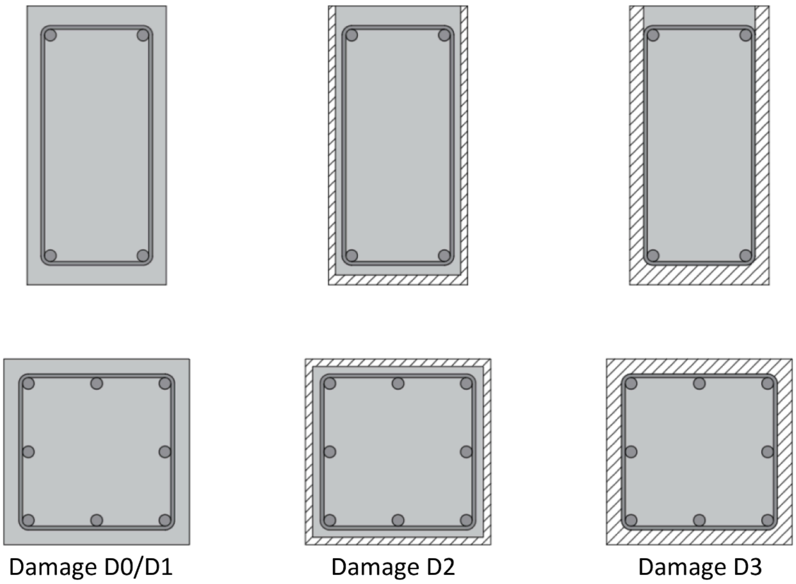

Fig. 1. Schematic of the sections with different types of damage considered in the analyses.

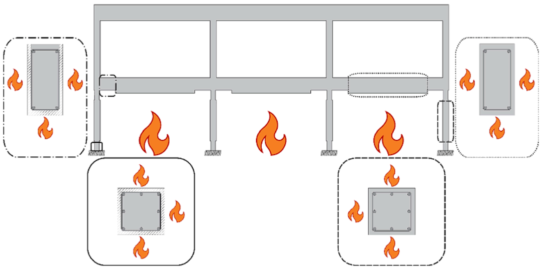

Fig. 2. Scheme of the simplified modelling strategy.

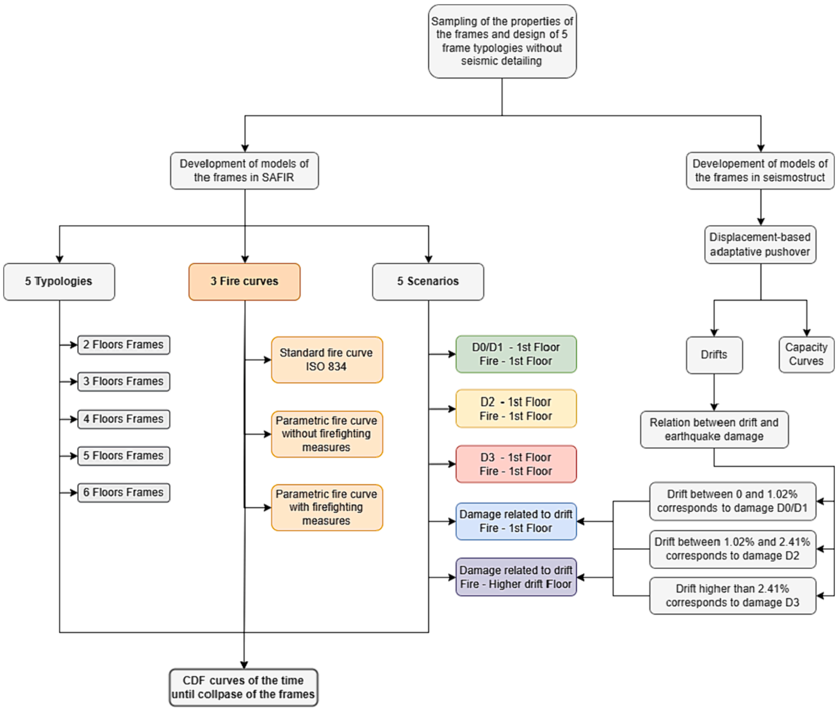

Fig. 3. Diagram of the several steps developed for the post-earthquake fire assessment in RC frames without seismic design.

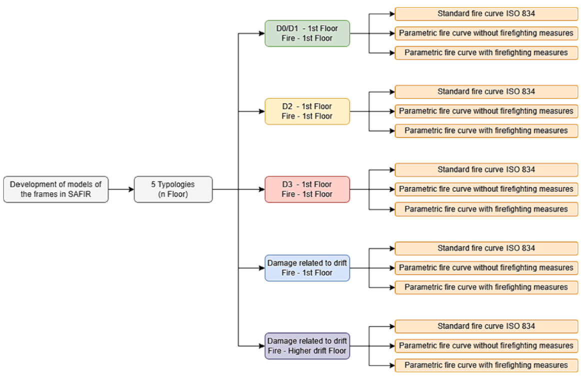

Fig. 4. Diagram regarding the models developed in SAFIR.

Table 1
Parameters of the established distributions for the considered properties of the frames.

Variables	Mean	CV (%)	A	B	Distribution
H_1 (m)	3.2	10	2.5	5	Lognormal
$H_{>1}$ (m)	2.8	6	2.5	4	Normal
L_{beam} (m)	4.4	16	2.5	6.5	Lognormal
f_{cm} (MPa)	23.8	49	18	36	Gamma
G (kN/m^2)	8	12.5	6	10	Normal
P_l (%)	1	40	0.3	3.5	Lognormal
f_{yk} (MPa)	235/400/500	–	–	–	Uniform
c (mm)	10/15/20/25	–	–	–	Uniform

Table 2
Concrete properties used in the numerical analyses.

Concrete properties [36]	
Concrete model	Siliceous aggregates [36]
Specific mass of concrete	2300 kg/m^3
Water content	26 kg/m^3
Emissivity	0,7
Tensile strength	0
Poisson's ratio	0,2

Table 3
Reinforcing steel properties used in the numerical analyses.

Reinforcing steel properties [36,45]	
Steel model	Hot rolled. class B [36,45]
Modulus of elasticity	210 GPa
Poisson's ratio	0,3
Emissivity	0,7

where there is an equal probability of selection of values of the characteristic yield strength of the reinforcing steel (f_{yk}) and the cover.

These variables were used to design each set of 100 frames with two, three, four, five and six floors. These values were then used to carry out a simulated design procedure considering conventional rules adopted in the past for gravity loads only. In this process it was considered that the column sections of the first floor are the same as the second floor, the sections of the third floor are the same as the sections of the fourth floor and the sections of the fifth floor are the same as the sections of the sixth floor. Additional properties of the concrete and steel were set constant in all frames, as their variation is essentially independent of the associated mechanical properties and period of manufacturing.

3.3. Parametric fire curves

To comprehend the differences of applying the parametric fire curve and the standard fire curve ISO 834 on the frames there were developed 200 parametric fire curves (two for each RC frame since each frame have two different compartments) where active firefighting measures are considered and 200 parametric curves where no active firefighting measures are considered [22]. Damage in the passive and active fire protection systems and delays in the firefighter's response are probable situations in a post-earthquake scenario, which can be represented by the parametric fire curves where no active firefighting measures are considered. This situation can be compared with the situation where the active firefighting measures are considered, which can represent a fire that occurs at any given time. The parametric fire curves correspond to compartment fires in dwellings with characteristic value of the fire load density of $q_{f,k} = 948\ MJ/m^2$. It was assumed that the compartment has two openings in the walls, one window (0.4L × 0.4H) and one door (0.8 × 2.0 m). The size of the window is 40 % of the size of the compartment walls. This is a common type of configuration of the openings in masonry infill walls [46]. Regarding the area of the compartment fire, it was considered that all compartments are

Table 4
Properties of the material in the fire compartments.

Material	Properties			
	Thickness (cm)	Unit mass (kg/m^3)	Conductivity (W/mK)	Specific Heat (J/kgK)
Normal weight concrete	20	2300	1.6	1000
Brick 1	15	625	0.36	840
Brick 2	11	654	0.38	840

quadrangular, so the area is obtained with L_{beam}^2.

The material properties of the fire compartments are presented in Table 4 and the active firefighting measures are presented in Table 5 and in Table 6 to be used according to EN 1991-1-2. All the curves developed are represented in Fig. 5 and Fig. 6.

3.4. Scenarios analysed in each frame typology

As mentioned earlier, there are five frame typologies studied in this work. The frames have always three-bays and then there is a variation in the number of floors, from two floors to six floors. For each frame typology, five different scenarios were analysed. The first scenario corresponds to frames with damage D0/D1 and fire only in the bottom floor, which can basically be translated in frames without the consideration of damage and with fire in the bottom floor. The second scenario corresponds to frames with damage D2 and fire only in the bottom floor, and the third scenario corresponds to frames with damage D3 and fire only in the bottom floor. In these three scenarios the damage and fire are only considered in the bottom floor. For the fourth and fifth scenario it was assumed a correspondence between the drift of the floors and the type of damage considered in those floors. The difference between the fourth and the fifth scenarios is that in the fourth scenario the fire is always considered in the bottom floor while in the fifth scenario the fire is considered in the floor with higher drift. A drift between 0 and 1.02 % corresponds to damage D0/D1, between 1.02 % and 2.41 % corresponds to damage D2 and higher than 2.41 % corresponds to damage D3 [47].

The values of the drifts of the floors of each frame were obtained from numerical analyses developed in the software SeismoStruct [40]. The characteristics and properties of the frames developed in SAFIR were replicated in SeismoStruct and the drifts were obtained with displacement-based adaptive pushover (DAP) analyses. In these analyses the lateral load profile was initially defined proportional to the fundamental mode shape and is progressively adapted as a function of the stiffness degradation accumulated in the different elements of the frames. The frames were subjected to increasing loads until the structure loses the equilibrium or the base shear reduce to 80 % of the maximum base shear. The elements were modelled with force-based elements with 5 integration points in the columns and beams based on the study by Sousa et al [48]. The concrete was modelled with the constitutive relation proposed by Mander et al, and the steel reinforcement was defined with the one proposed by Menegotto and Pinto [49,50].

From Figs. 7–11 are represented the capacity curves obtained for the five typologies. As expected, it is possible to observe an increase in base shear and horizontal displacement with the increase in number of storeys, resulting from the increase in robustness of the base columns and increase in total height of the frames. The results also show the large dispersion in the curves within each typology resulting from the variability in the parameters considered in the simulated design (see Table 1), but also due to the presence of soft storey mechanisms at different storeys, as described hereafter.

Then, for each of the five scenarios mentioned it is considered three different fire curves, which are the standard fire curve ISO 834, parametric fire curves without active firefighting measures and parametric fire curves with active firefighting measures. With the development of these scenarios, it is possible to compare the results and observe the effects of the damage and different fire curves on the time until collapse of the frames.

4. Results for the scenarios under study

4.1. Two-floor frames

In this section, it is analysed the time until collapse of the three bay two-floor frames. The frame configurations developed are those shown in Fig. 12. In Fig. 13 are represented the drifts of the three-bay and two-floors frames and the correspondent damage and fire location. For the three-bay two-floor frames the fire is only considered in the bottom floor since the higher drifts are always observed in the bottom floor. The drifts observed in most of the frames correspond to damage D2 in the bottom floor. The percentage of exceedance of the time until collapse of the developed frames are presented in Fig. 14, and the mean and standard deviation are presented in Table 7. The results show that the frames with severe damage (damage D3) have significantly lower times until collapse when compared with undamaged frames (damage D0/D1), which indicates the high impact that the damage can have on the time until collapse of the frames. From the scenarios analysed, the scenario that leads to lower times until collapse is when it is considered damage D3 and parametric fire in the bottom floor.

The standard deviation of the fire resistance of the frames with damage D0/D1 is higher than the standard deviation of the fire resistance of the frames with damage D2 which is higher than the standard deviation of the fire resistance of the frames with damage D3. These differences in the variability of the fire resistance of the frames can be related with the cover of the beams and columns.

The failure of the frames with damage D3 is expected to happen in the extremities of the beams and/or columns, where the steel reinforcement is exposed to fire. The frames with damage D0/D1 have a variability in the cover which can explain the variability in the

Table 5
Factors considered in the "No Active Fire Fighting Measures" scenario [22].

Automatic fire suppression		Automatic fire detection		Manual fire suppression				
Automatic water extinguishment system	Independent water supplies	Automatic fire detection and alarm	Automatic alarm transmission to fire brigade	Work fire brigade	Off site brigade	Safe access routes	Fire fighting devices	Smoke exhaust system
1.0	1.0	1.0	1.0	1.0	1.0	1.5	1.5	1.5

Table 6
Factors considered in the "Active Fire Fighting Measures" scenario [22].

Automatic fire suppression		Automatic fire detection		Manual fire suppression				
Automatic water extinguishment system	Independent water supplies	Automatic fire detection and alarm	Automatic alarm transmission to fire brigade	Work fire brigade	Off site brigade	Safe access routes	Fire fighting devices	Smoke exhaust system
0.61	1.0	0.73	1.0	1.0	0.78	1.0	1.0	1.5

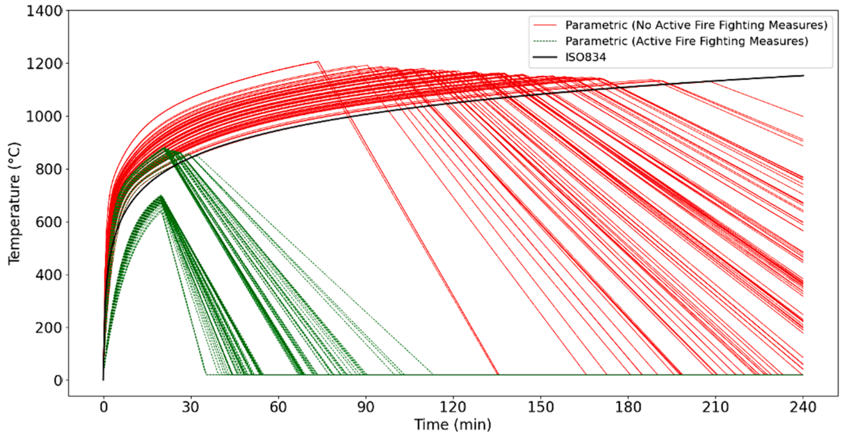

Fig. 5. Parametric fire curves considered on the first floor of the frames.

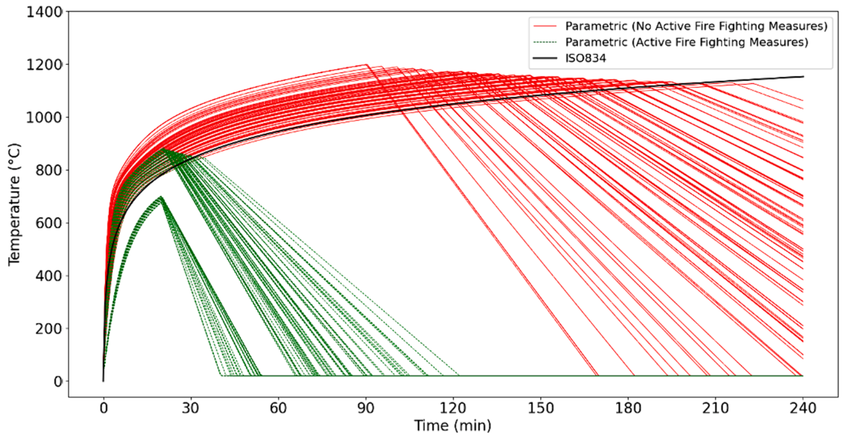

Fig. 6. Parametric fire curves considered on all the floors of the frames except on the first floor.

results of the frames with damage D0/D1.

The results of the scenario where there is a correspondence between the drifts and the damage show a high similarity with the results of the scenario where it is considered damage D2 in the bottom floor. This aspect can be explained by the high number of frames where the drift corresponds to damage D2 in the bottom floor, as is observed in the Fig. 13. The results also show that the use of parametric fire curves with no active firefighting measures leads to, approximately, a 10-minute reduction in the time until collapse of the frames when compared with using the fire curve ISO 834. In Table 8 it is presented the number of frames that collapsed and did not collapse when considering the parametric fire curves with active firefighting measures.

When the parametric fire curves with active firefighting measures are used there are some frames that collapse after the maximum temperature peak on the respective parametric fire curve. When the peak temperature is reached there is still a temperature increase in the RC sections, which can lead to the collapse of the structure in the cooling phase of the parametric fire curve.

This type of information is important because the cooling phase can correspond to the aftermath of the fire and there is the possibility of the rescue teams and firefighters being inside the building. It is vital to bear in mind that the structures can collapse in the cooling phase.

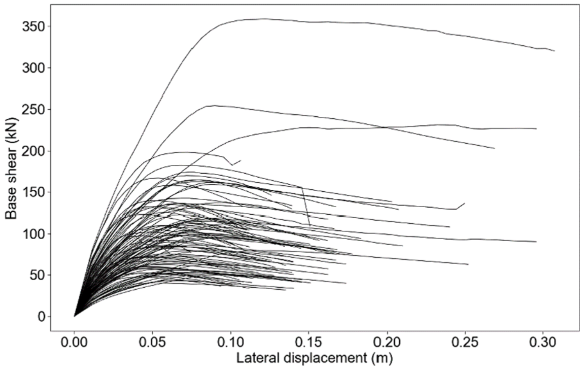

Fig. 7. Capacity curves of the two-floors frames.

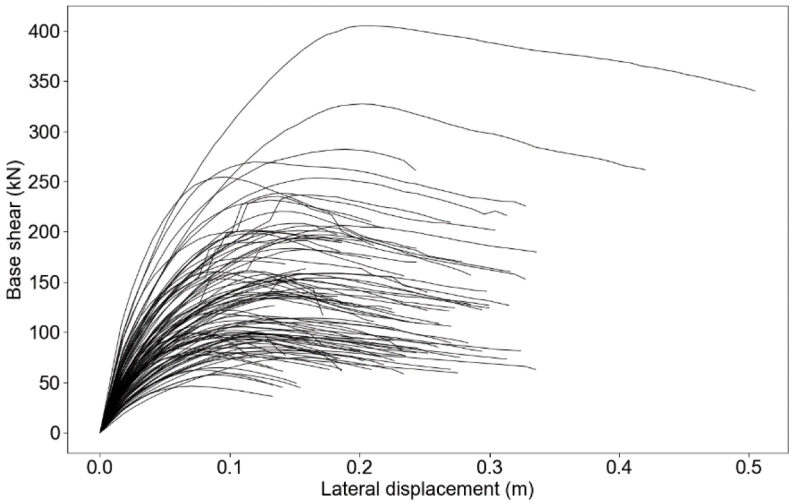

Fig. 8. Capacity curves of the three-floors frames.

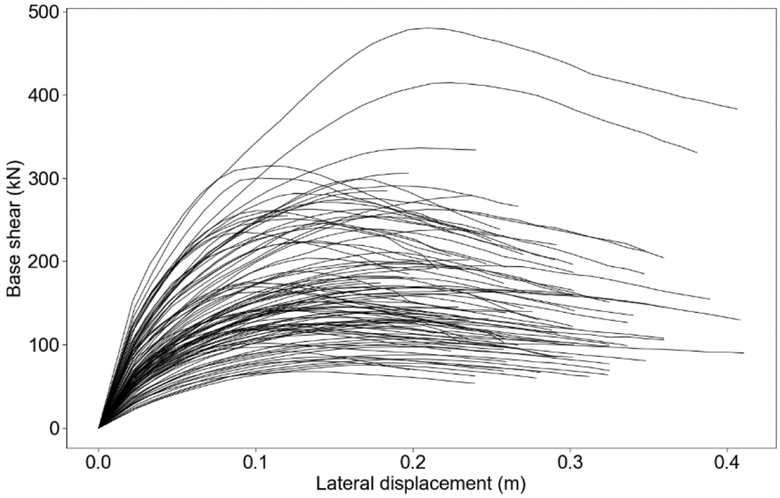

Fig. 9. Capacity curves of the four-floors frames.

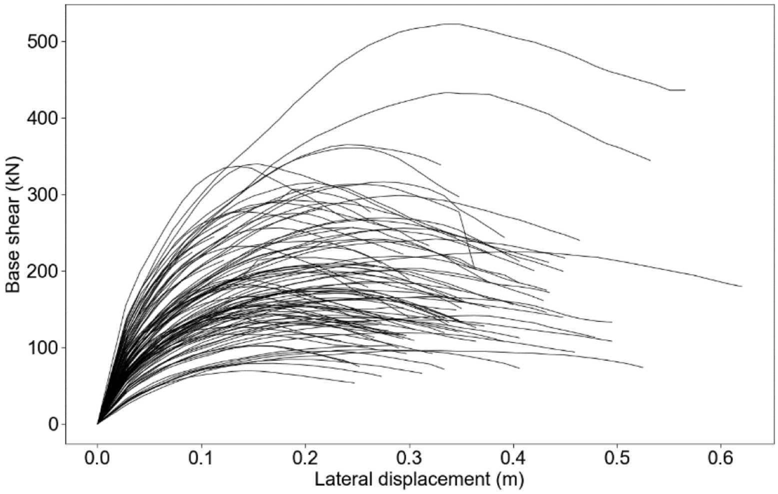

Fig. 10. Capacity curves of the five-floors frames.

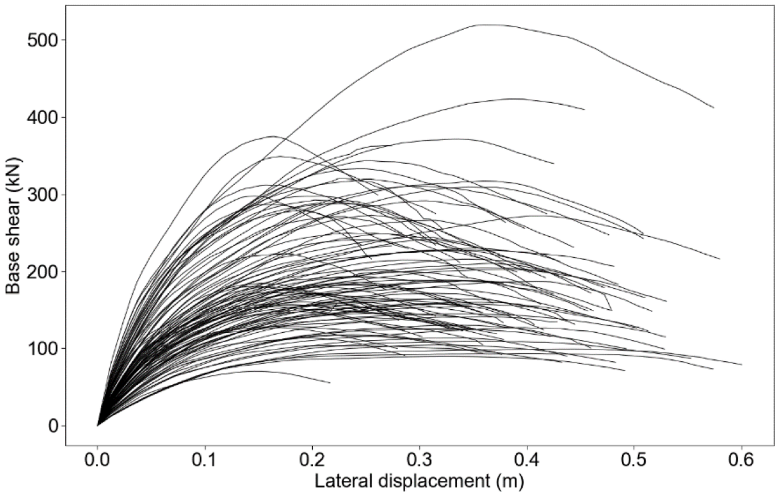

Fig. 11. Capacity curves of the six-floors frames.

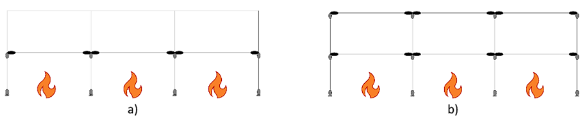

Fig. 12. Three-bay, two-floors frame configurations, a) Damage and fire in the bottom floor, b) Fire in the bottom floor and damage related with the drift.

4.2. Three-floors frames

In this section, it is analysed the time until collapse of the three bay three-floor frames. The frame configurations developed are those shown in Fig. 15. The percentages represented in Fig. 15 b) consist in the number of frames (since there are 100 frames in each typology) that have fire in a specific floor. For example, for the fifth scenario (where the fire is in the floor with higher drift) there are 39 frames with fire on the first floor, 3 frames with fire on the second floor and 58 frames with fire on the third floor. In Fig. 16 are represented the drifts of the three-bay and three-floors frames and the correspondent damage and fire location. In the scenario where the location of the fire is related to the drifts, for the three-bay three-floor frames the fire is considered mainly in the bottom floor and on the third floor since the higher drifts are observed in these floors. The drifts observed in most of the frames correspond to damage D2. The percentage of exceedance of the time until collapse of the developed frames are presented in Fig. 17, and the mean and

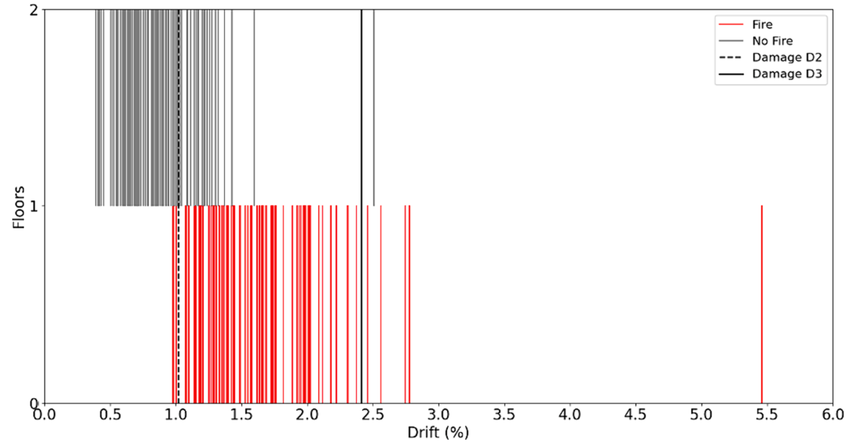

Fig. 13. Drifts of the three-bay and two-floors frames and the correspondent damage and fire location.

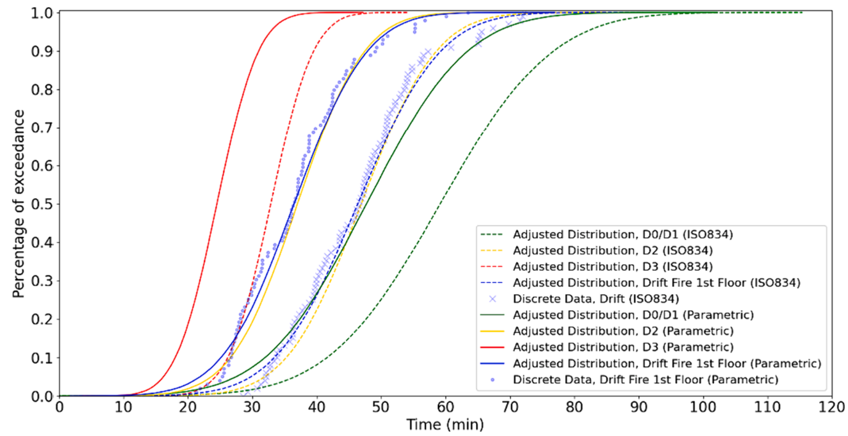

Fig. 14. Percentage of exceedance of the time until collapse of the three-bay two-floor frames considering the fire curve ISO 834 and parametric fire curves with no active firefighting measures.

Table 7
Median and Standard Deviation of the time until collapse of the three-bay, two-floors frames.

Model	Damage	Fire	Median	Standard deviation
ISO 834	D0/D1	First floor	59.09	13.71
	D2	First floor	46.87	9.10
	D3	First floor	32.81	5.23
	Related to drift	Higher drift floor	46.36	10.07
Parametric with no active firefighting measures	D0/D1	First floor	47.75	12.29
	D2	First floor	36.99	8.19
	D3	First floor	24.51	5.06
	Related to drift	Higher drift floor	36.55	8.92

standard deviation are presented in Table 9. The previously observations regarding the impact of the damage on the time until collapse, the variability in the results, the differences between fire curves and the scenario that leads to lower times until collapse are also observed in the three bay three-floor frames. Between the two scenarios where there is a correspondence between the drifts and the damage, the scenario that leads to lower times until collapse is where the fire is in the floor with higher drift. This situation is expected since the higher drifts correspond higher levels of damage in the floors of the frames, which are, consequently, more susceptible to have lower times until collapse. Same as before, there is a relatively similarity of the results of the scenario where there is a correspondence between the drifts and the damage and the scenario where damage D2 and fire is considered in the bottom floor. This can also be explained by the significant number of frames where the drift corresponds to damage D2. In Table 10 it is presented the

Table 8
Number of three-bay, two-floors frames that collapsed and that did not collapse when considering parametric fire curves with active firefighting measures.

Model	Damage	Fire	No Collapse	Collapse	Collapse (cooling phase)
Parametric with active firefighting measures	D0/D1	First floor	97	3	2
	D2	First floor	95	5	4
	D3	First floor	73	27	15
	Related to drift	Higher drift floor	94	6	0

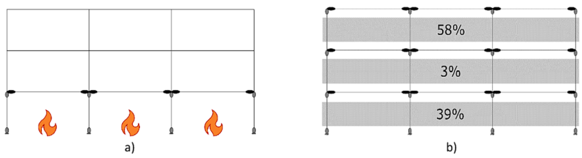

Fig. 15. Three-bay, three-floors frame configurations, a) Damage and fire in the bottom floor, b) Damage related with the drift and fire in floor with higher drift (percentage of fire on each floor).

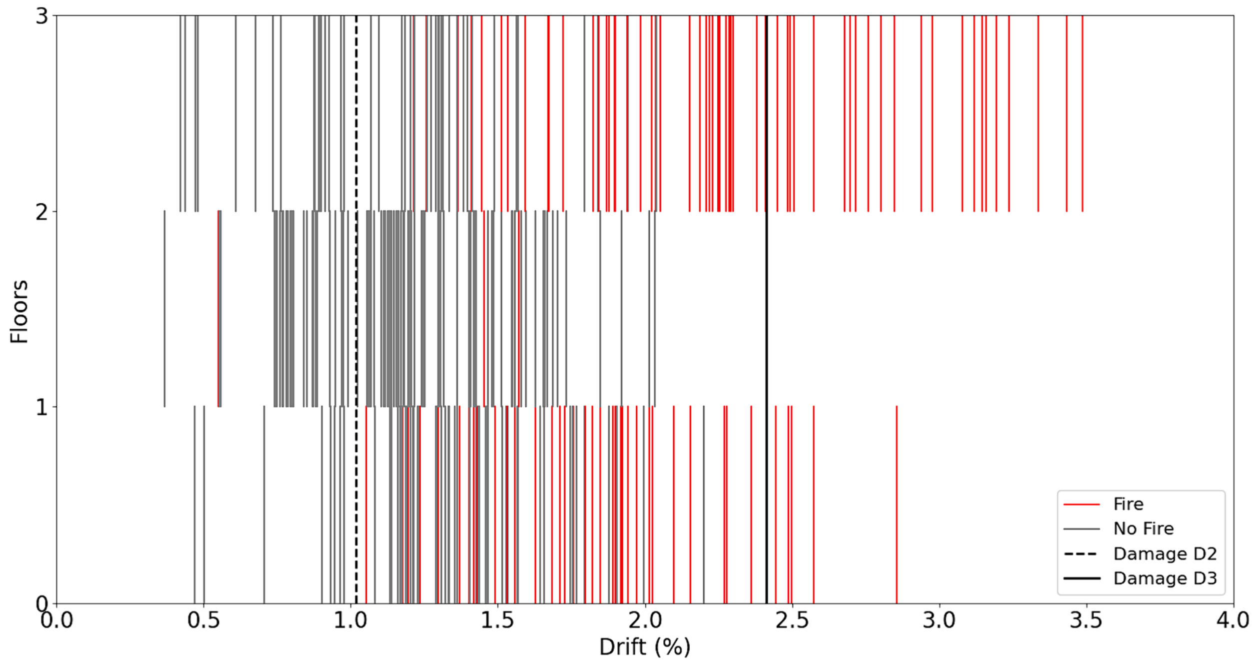

Fig. 16. Drifts of the three-bay and three-floors frames and the correspondent damage and fire location.

number of frames that collapsed and did not collapse when considering the parametric fire curves with active firefighting measures.

4.3. Four-floor frames

In this section, it is analysed the time until collapse of the three bay four-floor frames. The frame configurations developed are those shown in Fig. 18. In Fig. 19 are represented the drifts of the four-bay and four-floors frames and the correspondent damage and fire location. In the scenario where the location of the fire is related to the drifts, for the three-bay four-floor frames the fire is considered mainly in the bottom floor and on the third floor since the higher drifts are observed in these floors. The drifts observed in most of the floors of the frames correspond to damage D2 and there are a significant number of frames that have damage D3 on the third floor. The percentage of exceedance of the time until collapse of the developed frames are presented in Fig. 20, and the mean and standard deviation are presented in Table 11. The previously observations regarding the impact of the damage on the time until collapse, the variability in the results, the differences between fire curves, and the scenario that leads to lower times until collapse are also observed in the three bay four-floor frames. In Table 12 it is presented the number of frames that collapsed and did not collapse when considering the parametric fire curves with active firefighting measures.

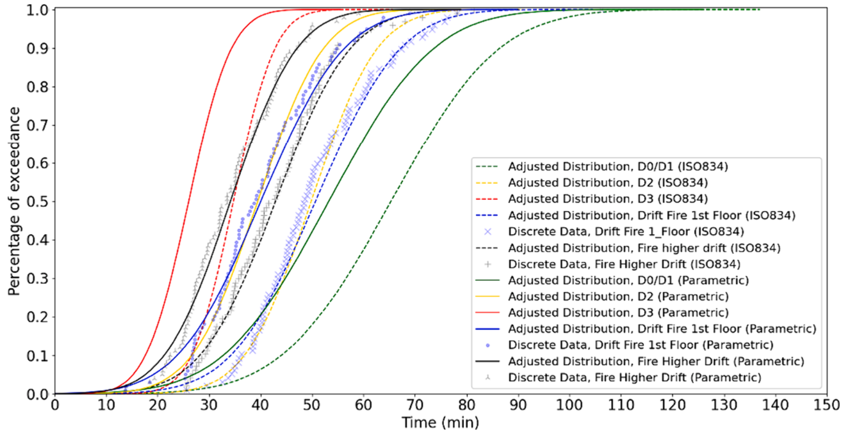

Fig. 17. Percentage of exceedance of the time until collapse of the three-bay three-floor frames considering the fire curve ISO 834 and parametric fire curves with no active firefighting measures.

Table 9
Median and Standard Deviation of the time until collapse of the three-bay, three-floors frames.

Model	Damage	Fire	Median	Standard deviation
ISO 834	D0/D1	First floor	65.28	16.63
	D2	First floor	49.87	10.15
	D3	First floor	34.70	6.35
	Related to drift	First floor	50.65	12.97
	Related to drift	Higher drift floor	42.88	11.43
Parametric with no active firefighting measures	D0/D1	First floor	53.33	16.05
	D2	First floor	39.49	10.00
	D3	First floor	26.04	6.54
	Related to drift	First floor	40.09	12.71
	Related to drift	Higher drift floor	34.22	10.08

Table 10
Number of three-bay, three-floors frames that collapsed and that did not collapse when considering parametric fire curves with active firefighting measures.

Model	Damage	Fire	No collapse	Collapse	Collapse (cooling phase)
Parametric with active firefighting measures	D0/D1	First floor	100	0	–
	D2	First floor	98	2	2
	D3	First floor	80	20	10
	Related to drift	First floor	97	3	2
	Related to drift	Higher drift floor	85	15	6

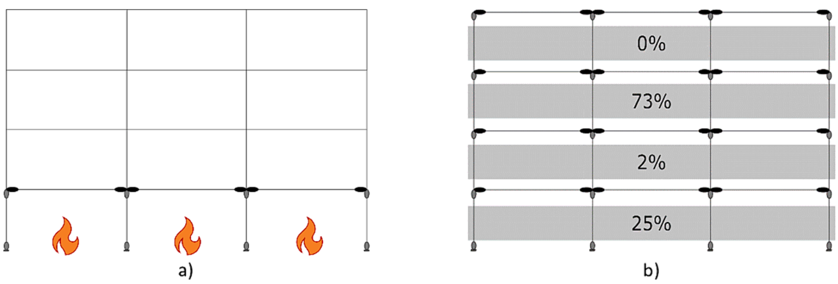

Fig. 18. Three-bay, four-floors frame configurations, a) Damage and fire in the bottom floor, b) Damage related with the drift and fire in floor with higher drift (percentage of fire on each floor).

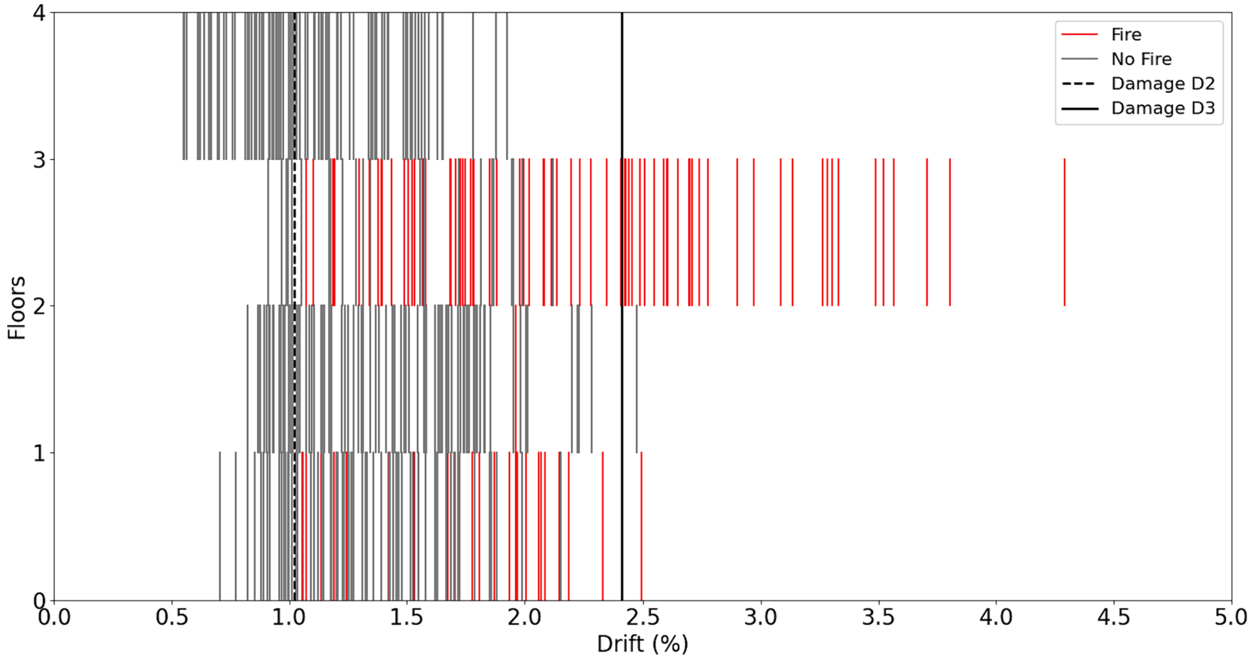

Fig. 19. Drifts of the three-bay and four-floors frames and the correspondent damage and fire location.

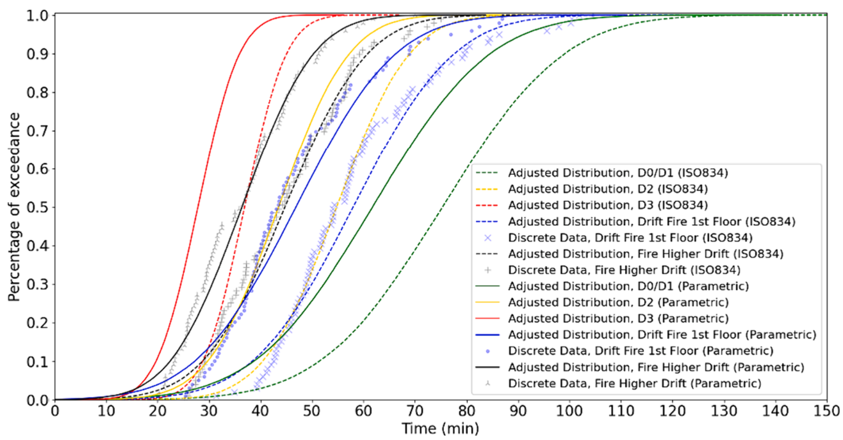

Fig. 20. Percentage of exceedance of the time until collapse of the three-bay four-floor frames considering the fire curve ISO 834 and parametric fire curves with no active firefighting measures.

Table 11
Median and Standard Deviation of the time until collapse of the three-bay, four-floors frames.

Model	Damage	Fire	Median	Standard deviation
ISO 834	D0/D1	First floor	75.38	18.68
	D2	First floor	54.78	11.13
	D3	First floor	36.90	6.38
	Related to drift	First floor	57.97	15.39
	Related to drift	Higher drift floor	44.79	12.51
Parametric with no active firefighting measures	D0/D1	First floor	61.79	17.98
	D2	First floor	43.49	10.77
	D3	First floor	27.80	6.33
	Related to drift	First floor	46.72	15.29
	Related to drift	Higher drift floor	36.39	10.89

Table 12
Number of three-bay, four-floors frames that collapsed and that did not collapse when considering parametric fire curves with active firefighting measures.

Model	Damage	Fire	No collapse	Collapse	Collapse (cooling phase)
Parametric with active firefighting measures	D0/D1	First floor	100	0	–
	D2	First floor	99	1	1
	D3	First floor	86	14	9
	Related to drift	First floor	100	0	–
	Related to drift	Higher drift floor	84	16	8

4.4. *Five-floor frames*

In this section, it is analysed the time until collapse of the three bay five-floor frames. The frame configurations developed are those shown in Fig. 21. In Fig. 22 are represented the drifts of the three-bay and five-floors frames and the correspondent damage and fire location. In the scenario where the location of the fire is related to the drifts, for the three-bay five-floor frames the fire is considered mainly on the third floor and on the fifth floor since the higher drifts are observed in these floors. The drifts observed in most of the floors of the frames correspond to damage D2. The percentage of exceedance of the time until collapse of the developed frames are presented in Fig. 23, and the mean and standard deviation are presented in Table 13. The previously observations regarding the impact of the damage on the time until collapse, the variability in the results, the differences between fire curves, and the scenario that leads to lower times until collapse are also observed in the three bay five-floor frames. In Table 14 it is presented the number of frames that collapsed and did not collapse when considering the parametric fire curves with active firefighting measures.

4.5. *Six-floor frames*

In this section, it is analysed the time until collapse of the three bay six-floor frames. The frame configurations developed are those shown in Fig. 24. In Fig. 25 are represented the drifts of the three-bay and six-floors frames and the correspondent damage and fire location. In the scenario where the location of the fire is related to the drifts, for the three-bay six-floor frames the fire is considered mainly in the third floor and in the fifth floor since the higher drifts are observed in these floors. The drifts observed in most of the floors of the frames correspond to damage D2 and there are a significant number of frames that have damage D3 in the third and fifth floor. The percentage of exceedance of the time until collapse of the developed frames are presented in Fig. 26, and the mean and standard deviation are presented in Table 15.

The previously observations regarding the impact of the damage on the time until collapse, the variability in the results, the differences between fire curves, and the scenario that leads to lower times until collapse are also observed in the three bay six-floor frames. In Table 16 it is presented the number of frames that collapsed and did not collapse when considering the parametric fire curves with active firefighting measures.

5. Global comparisons

5.1. *Comparison between the case studies*

In this section, the results of the different frames are compared with each other and are presented in Fig. 27. The graphs plotted on the left correspond to the results where the fire curve ISO 834 was considered, and the graphs plotted on the right correspond to the results where the parametric fire curves with no active firefighting measures where considered. The results of the scenarios where the damage is related to the drift and the fire is considered in the floor with higher drift are relatively similar with each other but the same

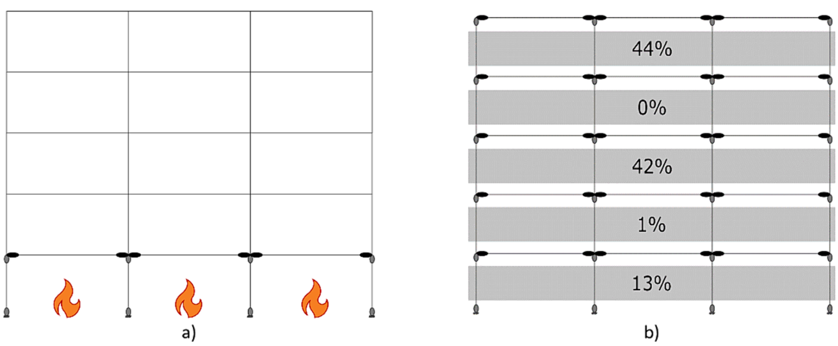

Fig. 21. Three-bay, five-floors frame configurations, a) Damage and fire in the bottom floor, b) Damage related with the drift and fire in floor with higher drift (percentage of fire on each floor).

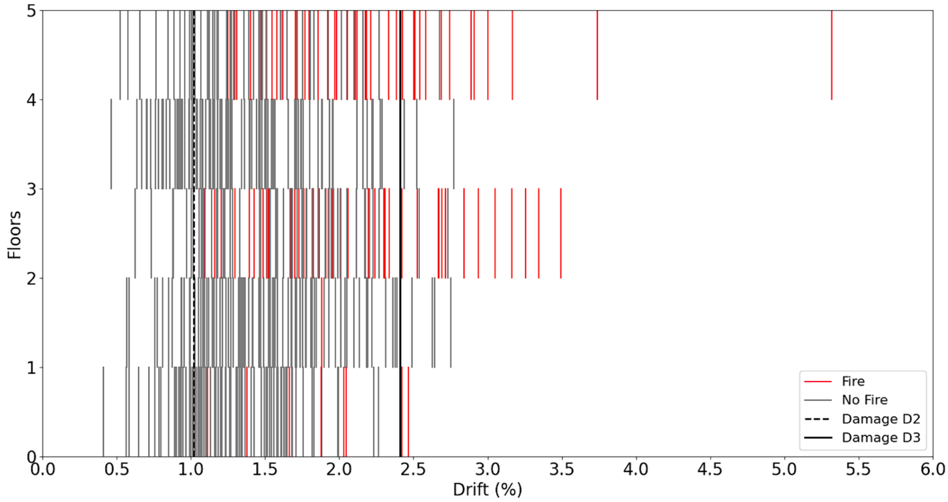

Fig. 22. Drifts of the three-bay and five-floors frames and the correspondent damage and fire location.

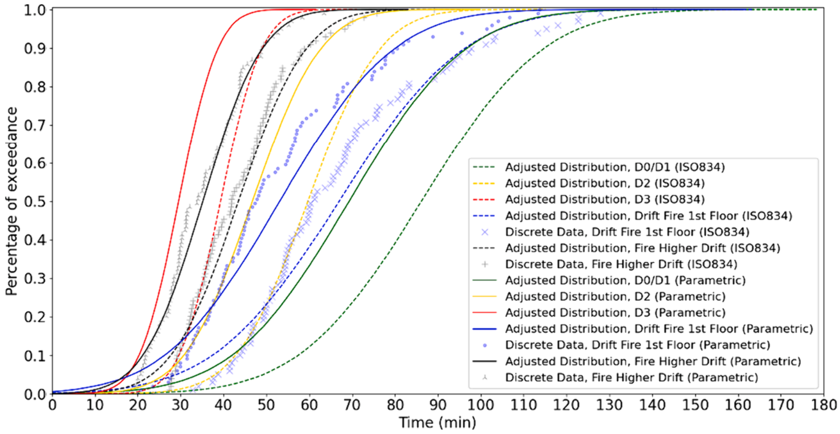

Fig. 23. Percentage of exceedance of the time until collapse of the three-bay five-floor frames considering the fire curve ISO 834 and parametric fire curves with no active firefighting measures.

Table 13
Median and Standard Deviation of the time until collapse of the three-bay, five-floors frames.

Model	Damage	Fire	Median	Standard deviation
ISO 834	D0/D1	First floor	86.17	22.32
	D2	First floor	59.66	13.23
	D3	First floor	39.36	7.58
	Related to drift	First floor	66.87	22.95
	Related to drift	Higher drift floor	43.57	12.37
Parametric with no active firefighting measures	D0/D1	First floor	69.35	21.42
	D2	First floor	46.72	12.51
	D3	First floor	29.51	7.21
	Related to drift	First floor	53.16	21.09
	Related to drift	Higher drift floor	35.22	10.92

it not observed in all the other scenarios, where there is an increase in the times until collapse with the increase in the number of floors. This aspect can be explained by the increase in the cross sections of the elements in the bottom floor with the increase of the number of floors.

Table 14
Number of three-bay, five-floors frames that collapsed and that did not collapse when considering parametric fire curves with active firefighting measures.

Model	Damage	Fire	No collapse	Collapse	Collapse (cooling phase)
Parametric with active firefighting measures	D0/D1	First floor	100	0	–
	D2	First floor	99	1	0
	D3	First floor	94	6	3
	Related to drift	First floor	100	0	–
	Related to drift	Higher drift floor	82	18	13

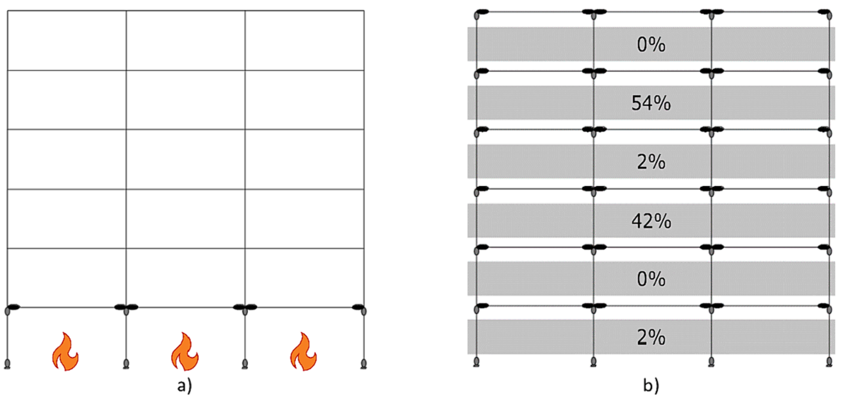

Fig. 24. Three-bay, six-floors frame configurations, a) Damage and fire in the bottom floor, b) Damage related with the drift and fire in floor with higher drift (percentage of fire on each floor).

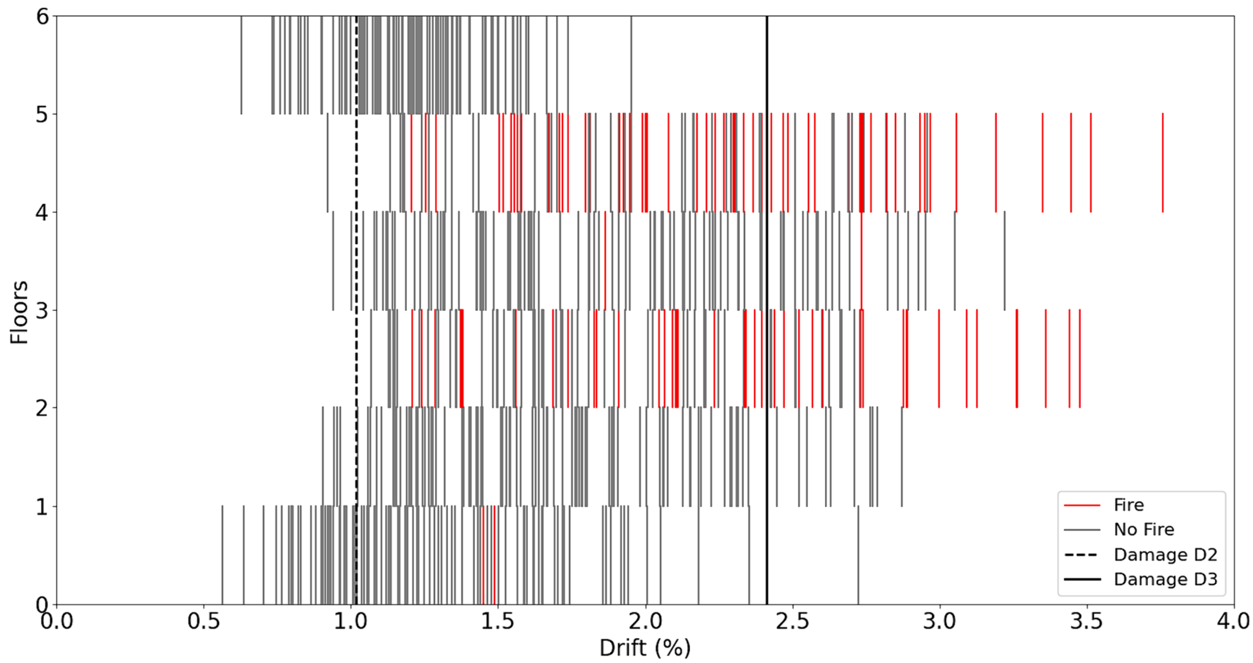

Fig. 25. Drifts of the three-bay and six-floors frames and the correspondent damage and fire location.

5.2. Low-rise and mid-rise frame buildings

The results of the frames can be grouped in low-rise frames (two and three floors) and mid-rise frames (four, five and six floors). In Fig. 28 are presented the low-rise frames and in Fig. 29 are presented the mid-rise frames. Since the results of the scenarios where the damage is related to the drift and the fire is considered in the floor with higher drift are similar with each other, there is not significant

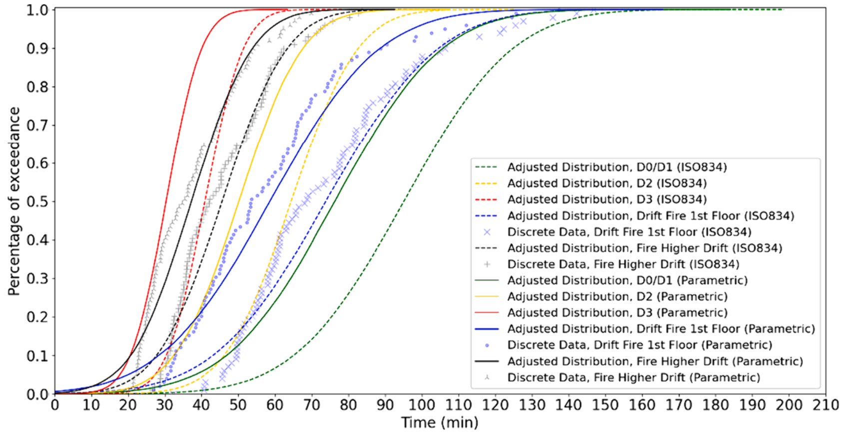

Fig. 26. Percentage of exceedance of the time until collapse of the three-bay six-floor frames considering the fire curve ISO 834 and parametric fire curves with no active firefighting measures.

Table 15
Median and Standard Deviation of the time until collapse of the three-bay, six-floors frames.

Model	Damage	Fire	Median	Standard deviation
ISO 834	D0/D1	First floor	95.07	23.42
	D2	First floor	64.48	14.27
	D3	First floor	40.92	8.27
	Related to drift	First floor	73.95	24.00
	Related to drift	Higher drift floor	45.62	13.85
Parametric with no active firefighting measures	D0/D1	First floor	76.49	23.15
	D2	First floor	50.17	13.76
	D3	First floor	30.38	7.66
	Related to drift	First floor	58.74	23.44
	Related to drift	Higher drift floor	37.30	12.98

Table 16
Number of three-bay, six-floors frames that collapsed and that did not collapse when considering parametric fire curves with active firefighting measures.

Model	Damage	Fire	No collapse	Collapse	Collapse (cooling phase)
Parametric with active firefighting measures	D0/D1	First floor	99	1	1
	D2	First floor	99	1	0
	D3	First floor	93	7	3
	Related to drift	First floor	99	1	1
	Related to drift	Higher drift floor	84	16	11

difference when they are grouped in low-rise and mid-rise frames and even when they are all grouped together (two to six floors), as is possible to observe in Fig. 30. This aspect is interesting because it shows that perhaps the post-earthquake fire resistance of a RC frame does not have a very significative relation with the number of floors of the structures. This observation can be made only when it is considered that the fire occurs always on the floor with higher drift. This is not an unrealistic approach when there is an assumption that higher drifts can lead to higher damage in electricity and gas lines, which consequently can lead to ignitions.

6. Conclusions

In this study there were designed 500 frames without considering seismic loading. These frames represent 100 frames of each typology considered, which are three-bay frames from two floors to six floors. Then, several analyses were developed to observe the impact of earthquake damage and different types of fire curves.

The impact of the damage on the fire resistance of the frames is noticeable when observing that the frames with damage (damage D2 and D3) have lower times until collapse when compared with undamaged frames (damage D0/D1). This reduction in the fire resistance is significant in the frames with severe damage (damage D3). Based on this observation, it could be beneficial to increase the seismic resistance of the structures, which can reduce the earthquake damage and consequently improve the fire resistance.

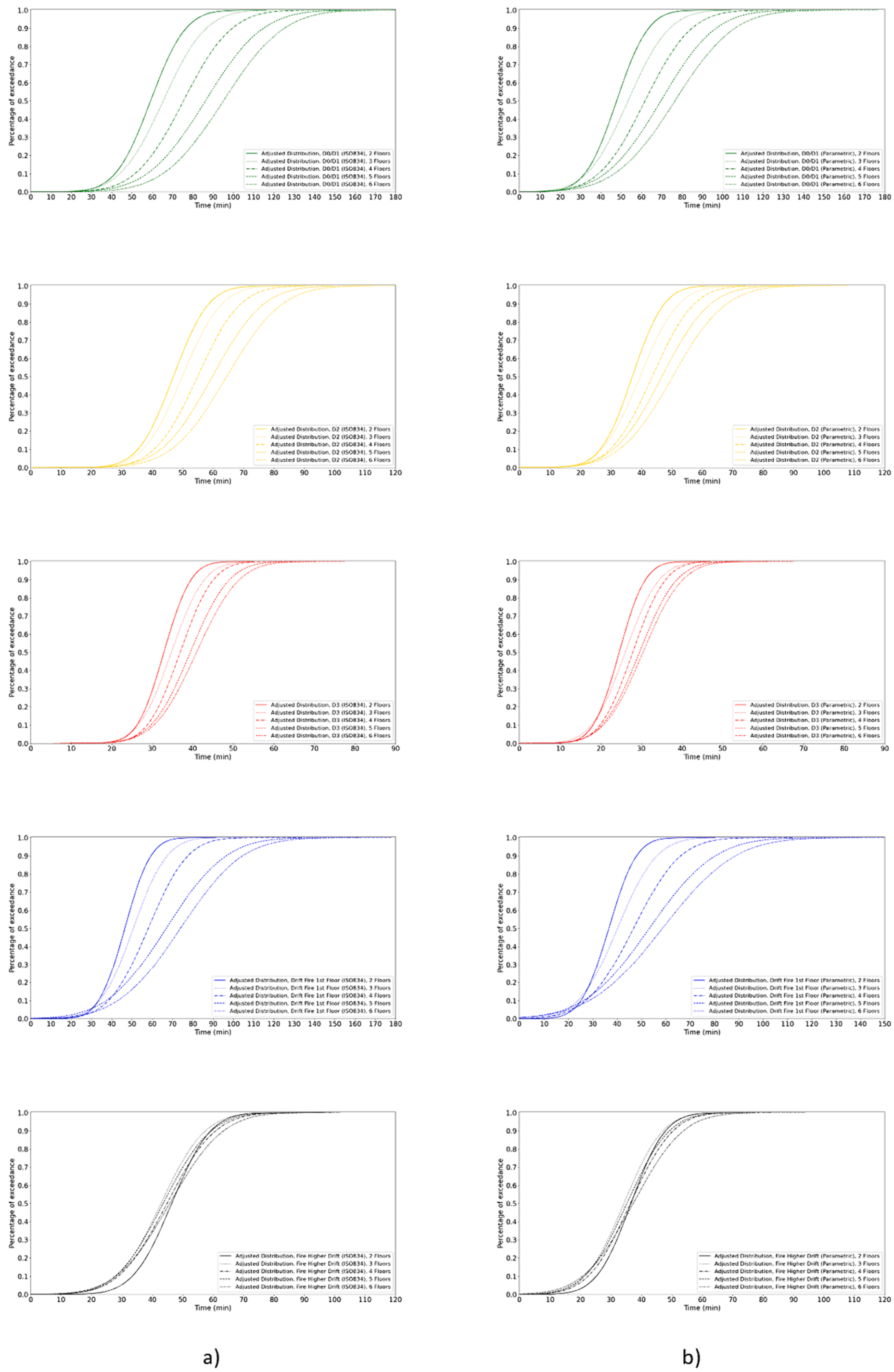

Fig. 27. Global comparison of the percentage of exceedance of the time until collapse of the frames. a) considering fire curve ISO 834; b) considering parametric fire curves with no active firefighting measures.

The variability of the fire resistance of the frames can be related to the variability in the cover of the elements, where there is a higher variability in the cover of the frames with damage D0/D1 and D2 when compared to the frames with damage D3.

The use of the fire curve ISO 834 leads to higher fire resistance of the frames when compared to the use of the parametric fire curves without active firefighting measures, which indicates that perhaps, considering parametric fire curves is a better approach to investigate the fire resistance of the structures of residential buildings in a post-earthquake event.

There was a considerable number of frames that did not collapse when the parametric fire curves with active firefighting measures

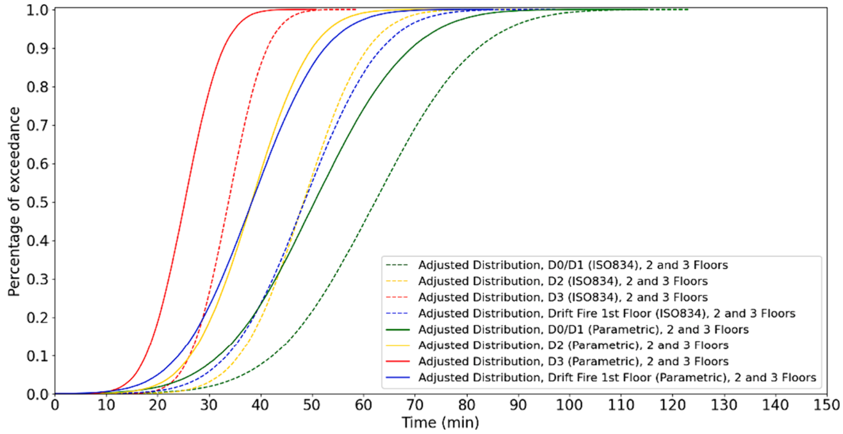

Fig. 28. Percentage of exceedance of the time until collapse of low-rise frames considering the fire curve ISO 834 and parametric fire curves with no active firefighting measures.

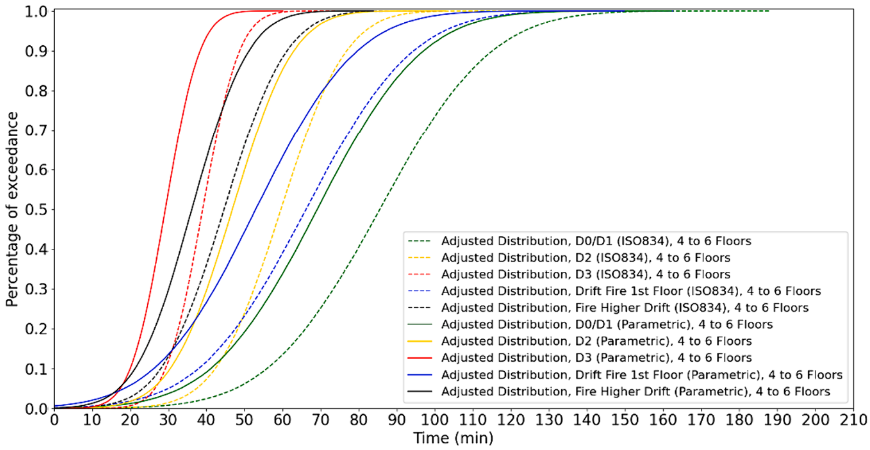

Fig. 29. Percentage of exceedance of the time until collapse of medium-rise frames considering the fire curve ISO 834 and parametric fire curves with no active firefighting measures.

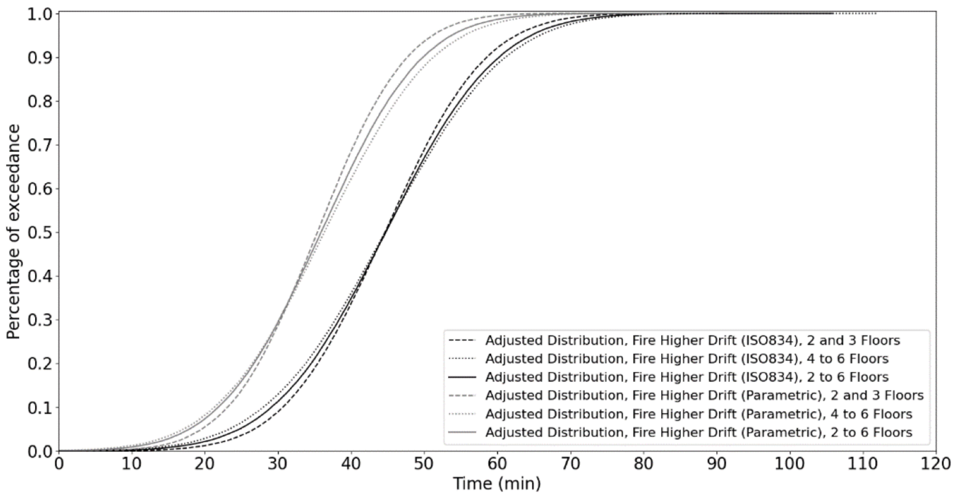

Fig. 30. Percentage of exceedance of the time until collapse of all the frames with damage related to the drift considering the fire curve ISO 834 and the parametric fire curves with no active firefighting measures on the floor with higher drift.

were considered, which shows the importance of adopting firefighting measures for the prevention of the collapse of structures due to fire. Beyond the adoption of active firefighting measures such as automatic water extinguishing system, automatic fire detection and alarm, offsite brigade, safe access routes and active firefighting devices, it also important that these measures work properly and are reliable in an earthquake scenario.

An overall conclusion obtained from the comparison between the different fire curves is that limiting the earthquake damage to the active and passive fire protection systems is of key importance for the fire resistance of the structures. Since in the cooling phase, the strength of the materials is non-decreasing, the structure continues to be safe if it is safe at higher temperatures [51,52], the behaviour of the structures can be conditioned by the damage and reduction of ductility due to the seismic damage. In the developed analyses it was observed that there are frames that collapse after the peak temperature of the respective parametric fire curve, which indicates that the failure can happen in the cooling phase of the fire. It should be noted that after reaching the peak temperature of the fire curve, there may still be an increase in temperature in the fibres. This increment can sometimes be sufficient to cause structural collapse.

A fire in a cooling phase or completely extinguished does not mean that the structure will not collapse, and rescue teams and firefighters should remain cautious in these situations.

The comparison between the results of the frames with different number of floors, where it was considered damage and fire on the bottom floor, showed that an increment in the number of floors also translates into an increment in the times until collapse. An explanation for this aspect is related to the increase of the cross-sections of the bottom floor with an increase in the number of floors.

For the scenario where the damage in the frames is related with the drift and the fire is in the floor with higher drift it was observed that the results are relatively similar, which seems to indicate that the fire resistance is not significantly related to the number of floors of the structure. This aspect is interesting and allows a global analysis of frames with different number of floors.

These types of studies are valuable because allow an improvement in the understanding of post-earthquake fire in RC structures and highlight the importance of adequate resistance of the active firefighting measures in seismic-prone regions.

CRediT authorship contribution statement

Hugo Vitorino: Writing – original draft, Visualization, Validation, Software, Methodology, Formal analysis, Data curation, Conceptualization. **Paulo Vila Real:** Writing – review & editing, Validation, Supervision, Methodology, Investigation, Formal analysis, Conceptualization. **Romain Sousa:** Writing – review & editing, Methodology, Investigation, Data curation, Conceptualization. **Carlos Couto:** Writing – review & editing, Supervision, Methodology, Investigation, Conceptualization. **Hugo Rodrigues:** Writing – original draft, Visualization, Validation, Software, Methodology, Formal analysis, Data curation, Conceptualization.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

Data will be made available on request.

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