# A Parametric Study of Fire-Damaged Reinforced Concrete Columns under Lateral Loads

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**Abstract-Columns are the structural members of buildings that ensure structural stability. A fire can severely affect the columns' structural performance by degrading the properties of their constituent materials, thereby reducing the strength capacity, stiffness, and stability. In seismic zones, the knowledge of the post-fire behavior of these elements is a fundamental requirement for a realistic seismic performance assessment. This study utilized numerical analysis using the parametric fire model of Eurocode-1 to estimate the post-fire axial and lateral performance of reinforced concrete columns. In the first step, the axial load-bearing capacity was evaluated from a parametric study for cantilever columns. In the second step, the lateral load capacity, force-displacement behavior, stiffness, ductility, energy dissipation capacity, and residual displacements were estimated to determine the impact of fire damage on the behavior of columns under lateral loads. The results showed that both the lateral load capacity and the ductility of the reinforced concrete columns decreased significantly due to fire exposure. This also indicated that fire damage decreases the vertical load-bearing capacity, and the reduction in lateral capacity was attributed to the loss of concrete's compressive strength. The column characteristics that significantly influence the residual response behavior were identified as section size, column height, axial load ratio, and concrete's compressive strength.**

***Keywords-natural fire; columns; post-fire behavior; fire damage; residual strength; lateral load capacity***

## I. INTRODUCTION

Fires and earthquakes are among the most severe conditions buildings may face. In the case of fire, the structural and nonstructural elements may undergo different changes depending on the intensity and exposure duration. The fire safety of Reinforced Concrete (RC) structures highly depends on their fire resistance, which relies on the thermal conductivity and the resistance of load-bearing elements such as walls, columns, and beams. Fire damage leads to a decrease in the strength and deformation properties of structures. A Post-Fire Earthquake (PFE) is an eventual disaster, but its damage to RC members is complicated and usually affected by uncertain factors. The reinforced concrete structure can remain standing after a fire, or after an earthquake. The full or partial collapse of concrete buildings during a fire is rare, however, earthquakes following a fire can cause significant damage depending on the residual properties of the structural elements. In post-fire performance assessment, it should be decided whether to repair and strengthen or demolish and rebuild the entire structure, taking into account the earthquake loads during its remaining service life.

The behavior of a structure after a fire is related to its residual bearing capacity, so it is necessary to determine, quantify, and compare it with the safety levels. The study of the impact of earthquakes on structures damaged by real fires is a fundamental issue in assessing concrete's structural behavior in severe load combinations. Although the sequential application of these extreme loads may be too severe in most cases, this strategy may be appropriate for the design of important structures, where cost and technology are no barrier. However, current regulations do not consider fire and earthquake hazards [1]. Several studies investigated the influence of fire on the mechanical strength of RC members and structures [2-4]. Post-fire material tests showed that concrete's mechanical properties degrade after fire exposure and do not fully recover after cooling [5-7]. Different models were proposed in [8, 9] on the post-fire full stress-strain response of fire-damaged concrete and the residual stress-strain relationship after exposure to high

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temperatures. Numerical investigations in [10, 11] evaluated the impact of fire duration and intensity on the residual load-bearing capacity and delayed failure of reinforced concrete columns. Experimental research and test results on the post-fire behavior of RC columns were presented in [12, 13]. A new modified finite element model was developed in [14] to predict the behavior and strength of concrete-filled steel tubular subjected to axial compression, considering the influence of some determinant parameters. A numerical study examined the fire behavior of thermally insulated strengthened RC beams subjected to fire exposure in [15]. The effect of fire on the flexural behavior of RC beams was studied in [16, 17].

An experimental study on the seismic behavior of high-performance concrete frames after a fire was presented in [18], showing that fire exposure can transform the failure mode of a frame subjected to reverse-cyclic loads from strong-column-weak-beam to strong-beam-weak-column with poor cyclic performance. Another study was conducted in [19] on the seismic performance of reinforced concrete beam-column joints after a fire and their practical strengthening. The performance of RC frame structures in post-earthquake fire loading and their failure times were analyzed in [20, 21]. In [22, 23], the seismic performance of RC short columns with light transverse reinforcement and RC beam-column joints under varying axial forces was investigated. Several studies on fire resistance and post-fire seismic behavior of concrete shear walls were carried out experimentally and numerically [24-27].

A limited number of studies attempted to estimate the post-fire lateral capacity performance of concrete columns. Although the behavior of reinforced concrete columns at elevated temperatures has been extensively investigated under service loads after cooling, studies on post-fire seismic behavior of RC structural elements are extremely rare. The results of [28, 29] evaluated the post-fire seismic performance of reinforced precast concrete columns damaged by fire to determine the impact of fire damage on force-displacement behavior, moment-curvature relationship, and residual displacements. An experimental study investigated the seismic performance of shear critical post-heated RC columns that had been repaired [30]. In [31], the results of a study on the seismic resistance of strengthened concrete members after fire exposure were presented. Furthermore, a numerical investigation of the response simulation of RC columns under lateral loads was conducted in [32]. In [33], the residual strength and lateral/seismic load capacity of RC columns after fire exposure were evaluated using numerical analysis.

This paper presents a numerical analysis that investigates the behavior of RC columns damaged by fire under vertical load and evaluate the lateral bearing capacity under horizontal loads, including a parametric study to identify the influential column characteristics. This study employed the structural analysis software SAFIR [34], which is capable of 3D simulation of building structures in fire. The parametric natural fire curves of Eurocode-1 were selected to perform the simulations.

## II. EVALUATION OF VERTICAL RESIDUAL LOAD-BEARING CAPACITY OF COLUMNS

Each column was first subjected to constant loading, and then a section of the column was exposed to natural fire to assess its residual characteristics. Loading was applied in successive simulations in a decreasing and monotonous manner, and the time of the collapse was calculated for each loading level. The corresponding loading level represents the load-bearing capacity of the column to resist natural fire, including the heating and cooling phases.

### *A. Time-Temperature Curves Used*

The fire curves were taken from the parametric fire model of Annex (A) of EN 1991-1-2(2002) [35] that represents the action of a natural fire, including the cooling phase. Figure 1 shows the various used fire curves, differing by the duration of the heating phase of 15, 30, 60, 90, and 120 minutes.

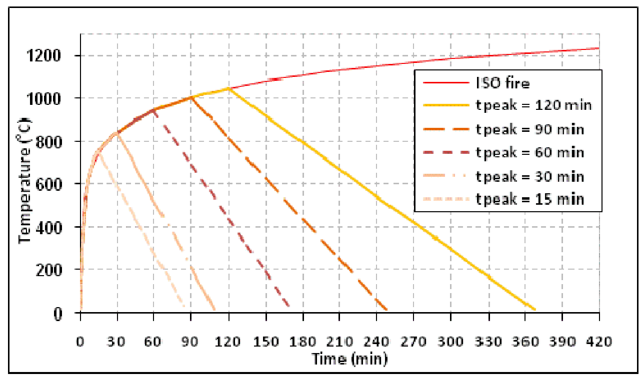



Fig. 1. Time-temperature curves.

### *B. Parametric Studies*

The following parameters were considered to evaluate the total residual capacity:

- The section size of the column: 20×20cm, 30cm×30cm, 40×40cm, 60×60cm, and 80×80cm.
- The duration of the heating phase of the fire ($t_{peak}$): 15, 30, 60, 90, and 120 minutes to assess the influence of the maximum temperature on the residual characteristics.
- The height of the column: 3, 4, and 5m.
- The columns were cantilever, fixed in rotation at the bottom and free at the top.

### *C. Results and Discussions*

#### *1) Influence of the Effective Height of the Column*

Table I shows the results for columns, considering their height, before and after fire exposure. A 30×30cm section was chosen to perform the simulations. For the same height, the influence of the maximum temperature during fire was considerable: a loss of 70% of vertical capacity ($N_r$) was found for the fire of $t_{peak}$=60min for 3m columns. Considering the variation in column height for the same fire, the residual vertical capacity ($N_r$) of the 3m column was found to be greater than the 5m column's (about 90%).

TABLE I. VERTICAL LOAD BEARING CAPACITY BEFORE ($N_{20°C}$) AND AFTER FIRE EXPOSURE ($N_R$) FOR COLUMNS

| Height (m) | Before fire $N_{20°C}$(kN) | $N_r$ (kN) after exposure | | | | |
|---|---|---|---|---|---|---|
| | | Fire of $t_{peak}$=15min | Fire of $t_{peak}$=30min | Fire of $t_{peak}$=60min | Fire of $t_{peak}$=90min | Fire of $t_{peak}$=120min |
| 3 | 1978 | 798 | 669 | 589.5 | 559.5 | 540 |
| 4 | 1446 | 589.5 | 498 | 441 | 420 | 406.5 |
| 5 | 972 | 396 | 339 | 304.5 | 292.5 | 283.5 |

### 2) *Influence of Section Size*

Table II presents the results of different-sized sections having the same height (4m). For the 40×40cm section, a loss of 75% was found for a 60min fire and a loss of 85% was found for the section of 80×80cm. The maximum temperature during fire exposure and cooling time was among the most important parameters responsible for the degree of concrete damage. The greater the section size, the higher the temperature gradient during the cooling phase and therefore, the higher damage induced in the concrete. With natural cooling, the interior temperature gradient within the concrete can be higher than during the heating phase.

TABLE II. VERTICAL LOAD BEARING CAPACITY BEFORE ($N_{20°C}$) AND AFTER FIRE EXPOSURE ($N_r$) OF SECTIONS

| Section | Before fire $N_{20°C}$ (kN) | $N_r$ (kN) after exposure | | | | |
|---|---|---|---|---|---|---|
| | | Fire of $t_{peak}$=15min | Fire of $t_{peak}$=30min | Fire of $t_{peak}$=60min | Fire of $t_{peak}$=90min | Fire of $t_{peak}$=120min |
| sec20×20cm | 295.5 | 123 | 108 | 99 | 96 | 93 |
| sec30×30cm | 1446 | 589.5 | 498 | 441 | 420 | 406.5 |
| sec40×40cm | 1725 | 652.5 | 537 | 466.5 | 439.5 | 421.5 |
| sec60×60cm | 8988 | 2847 | 2185.5 | 1776 | 1621.5 | 1521 |
| sec80×80cm | 16410 | 4689 | 3433.50 | 2658 | 2365.50 | 2175 |

## III. Lateral Load Capacity Evaluation of Fire-Damaged RC Columns

The horizontal load capacity was also evaluated to examine the column's behavior under combined vertical and horizontal loads, which is a real situation that a column must survive in frame structures. The lateral load was assumed to be a seismic load. This process utilized a numerical SAFIR model in a cold situation to investigate the lateral load response of the columns. The axial load was 25% and 50% of the residual vertical capacity $N_r$ and was held constant throughout the analysis. The lateral load was applied horizontally at the top of the columns in displacement-controlled mode and incremented from zero up to the column failure, as shown in Figure 2. It was possible to estimate the maximum horizontal load $H_u$ that the column can support. The horizontal load $H_u$ represents the shear force that an earthquake can induce in a column.

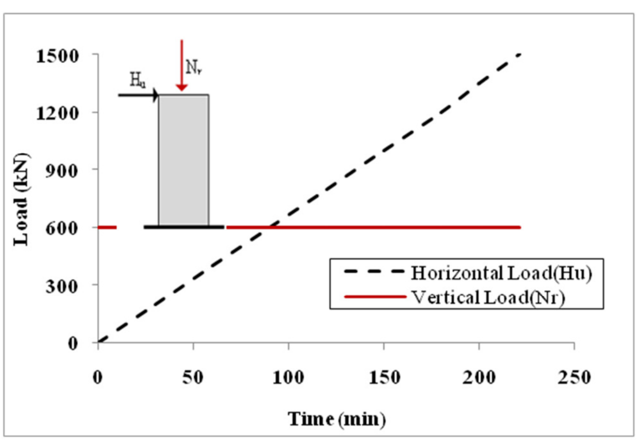



Fig. 2. Evolution of $H_u$ and $N_r$.

### A. *Horizontal Load-Bearing Capacity ($H_u$) with Vertical Load*

The column was subjected to both axial load and lateral deformation. The axial load used was the vertical load capacity of the column after fire exposure. Two loading rates were used: 25 and 50% of the residual bearing capacity.

### 1) *Effect of Effective Height*

For each fire and different heights, 44% and 42% loss of lateral capacity were found when applying 25% and 50% axial load of $N_r$ respectively, as shown in Figure 3.

(a)

(b)

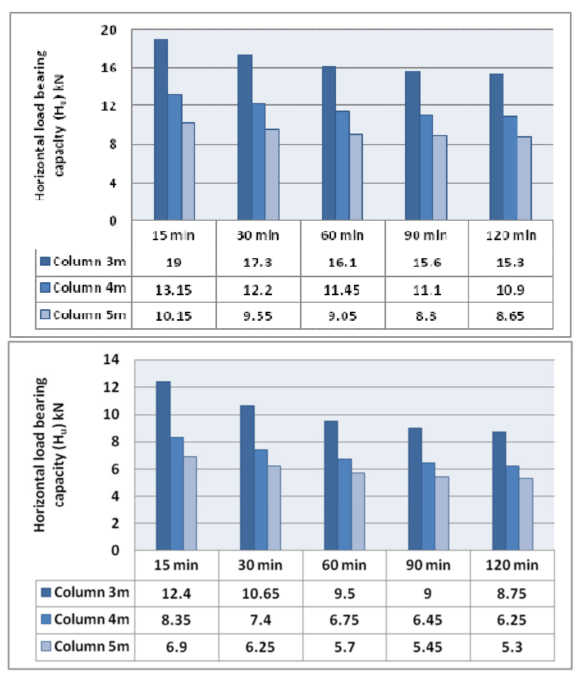


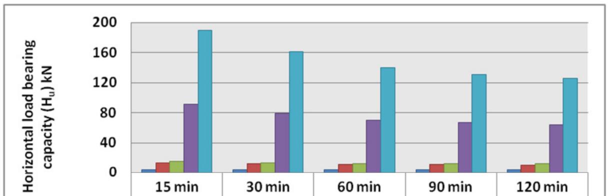

Fig. 3. $H_u$ as a function of various heights for (a) 0.25 of $N_r$, (b) 0.5 *of* $N_r$.

However, as can be observed in Figure 3, the reductions in the lateral load capacities for the same height were less than the reductions in the compressive strength, which minimized the impact of the compressive strength on the column behavior.

### *2) Influence of the Size of the Sections*

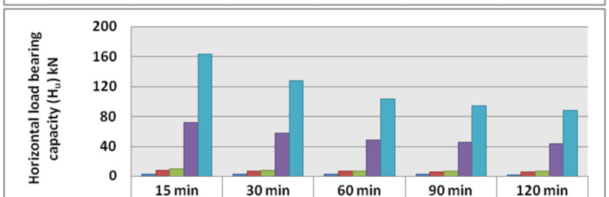
Figure 4 shows the evolution of $H_u$ considering the effect of the section and the properties of the residual material after fire exposure. As the duration of fire increases, the residual properties of the material decrease, and a loss of 10% to 35% was evaluated for all columns considering the same height (4m). This degradation was due to the additional loss of the concrete's compressive resistance caused by the continuity of the evolution of temperatures in the massive sections. For the same fire, the horizontal load-bearing capacity of the column can increase by 97% depending on the section's dimensions.

(a)




| | 15 min | 30 min | 60 min | 90 min | 120 min |
|---|---|---|---|---|---|
| Sec 20x20 | 4.2 | 4.15 | 4.1 | 3.9 | 3.8 |
| Sec 30x30 | 13.15 | 12.2 | 11.45 | 11.1 | 10 |
| Sec 40x40 | 15.15 | 13.7 | 12.65 | 12.2 | 11.95 |
| Sec 60x60 | 91.3 | 79.15 | 70.45 | 66.85 | 64.4 |
| Sec 80x80 | 189.45 | 161.2 | 139.75 | 131.3 | 125.5 |

(b)



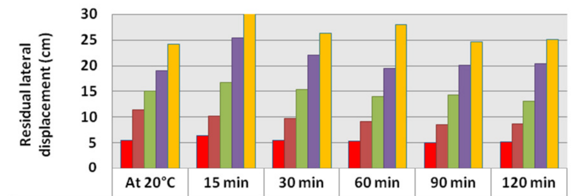

| | 15 min | 30 min | 60 min | 90 min | 120 min |
|---|---|---|---|---|---|
| Sec 20x20 | 3.05 | 2.9 | 2.65 | 2.55 | 2.5 |
| Sec 30x30 | 8.35 | 7.4 | 6.75 | 6.45 | 6.25 |
| Sec 40x40 | 10.1 | 8.6 | 7.55 | 7.15 | 6.9 |
| Sec 60x60 | 71.95 | 58.15 | 49.15 | 45.65 | 43.35 |
| Sec 80x80 | 163.95 | 127.8 | 103.8 | 94.45 | 88.35 |

Fig. 4. $H_u$ as a function of section size: (a) 0.25 of $N_r$ (b) 0.5 of $N_r$.

## B. *Evolution of the Maximum Lateral Displacement*

### *1) Influence of the Height of the Column*

Figure 5 shows the maximum lateral displacements at the top of the column for heights of 3, 4, and 5m using the same section of 30×30cm. Considering the residual resistance of the materials after the fire, the maximum lateral displacement was calculated for each fire. The results showed that the displacement value increased about 36% at the top of the 5m column compared to the reference column of 3m. The largest lateral displacement was 16cm for the 5m column exposed to fire with a heating phase ($t_{peak}$) of 15 minutes. Fire exposure has a greater effect on the residual compressive strength of concrete, and the structural response is governed by the geometrical properties of the column, such as inertia and height.

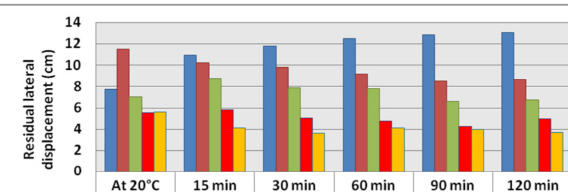



| | At 20°C | 15 min | 30 min | 60 min | 90 min | 120 min |
|---|---|---|---|---|---|---|
| Column 3m | 5.44 | 6.36 | 5.5 | 5.36 | 5.01 | 5.1 |
| Column 4m | 11.4 | 10.23 | 9.78 | 9.15 | 8.54 | 8.69 |
| Column 5m | 15.07 | 16.66 | 15.27 | 13.9 | 14.29 | 13.05 |
| Column 6m | 19.02 | 25.35 | 22.03 | 19.52 | 20.03 | 20.4 |
| Column 7m | 24.2 | 30.08 | 26.22 | 27.99 | 24.62 | 25.05 |

Fig. 5. Evolution of maximal lateral displacement.

### *2) Influence of the Sections of Columns*

Figure 6 shows the influence of the section's size on the maximum lateral displacement. For the 20×20cm sections, the displacement increased gradually and its maximum was about 13cm. This growth was due to fire damage to the mechanical residual property of the materials, and the strength of the columns decreased following the degradation of the strength. For the 80×80cm sections, the displacement of 5cm has a small variation, showing that a column's behavior depends on its geometrical properties.




| | At 20°C | 15 min | 30 min | 60 min | 90 min | 120 min |
|---|---|---|---|---|---|---|
| sec20x20 | 7.75 | 10.93 | 11.83 | 12.54 | 12.85 | 13.06 |
| sec30x30 | 11.51 | 10.23 | 9.78 | 9.15 | 8.54 | 8.69 |
| sec40x40 | 7 | 8.71 | 7.88 | 7.85 | 6.62 | 6.76 |
| sec60x60 | 5.53 | 5.8 | 5.08 | 4.74 | 4.27 | 4.95 |
| sec80x80 | 5.64 | 4.15 | 3.64 | 4.15 | 4.01 | 3.7 |

Fig. 6. Influence of section size on the maximum lateral displacement of a 4m column height.

## C. *Lateral Loads and Displacement Response Analysis*

Horizontal displacements were calculated using the incremental method, evaluating the displacement for each time step. Figure 7 shows the load-lateral displacement which traces the development of lateral displacement on the top of columns under the incremental loading.

### *1) Energy Dissipation*

The area enclosed by the curve and the horizontal axis in Figure 8 is defined as the energy dissipated by the columns, representing the ability to consume seismic energy through plastic deformations. Figure 7 shows that these curves are approximately straight lines before cracking. Within each curve, the decrease in secant stiffness caused by fire loading is somewhat insignificant, leading to small energy dissipation. However, they bend towards the displacement axis towards the end of loading. In other words, the top lateral displacements develop at accelerating rates before the failure of the column.

The curves indicate that the stiffness of an unheated column is higher than that of a heated one. This is because elevated temperatures cause more damage to the stiffness of the column subjected to greater temperature exposure (120min fire). Figure 8 also shows that the stiffness of the column exposed fires of 60, 90, and 120 minutes is very similar. This can be attributed to the damage caused by the high-temperature effect on the residual resistance of concrete. Among the curves presented, the stiffness of the column that has not been exposed to elevated temperature is the highest. The loading curves indicate that an elevated temperature has a significant effect under these conditions. Energy dissipation can be divided into two phases: pre-cracking and post-cracking. The energy dissipation capacity in the pre-cracking phase is very small.

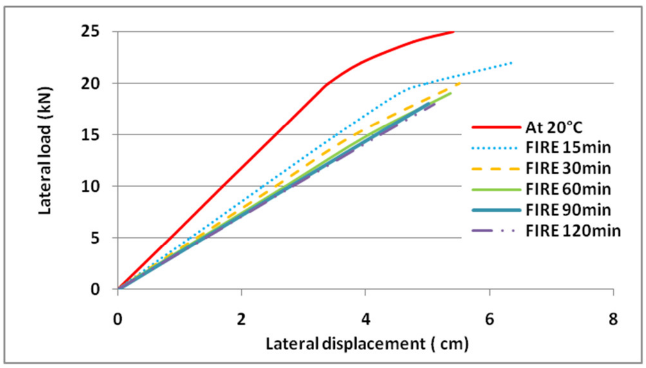



Fig. 7. Load-displacement curves of 3m columns.

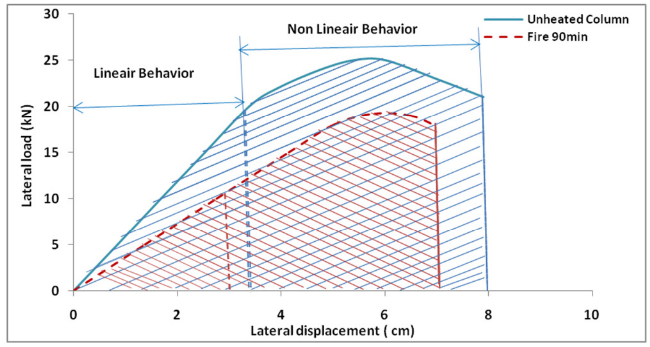



Fig. 8. Energy dissipated by the column.

### 2) *Ductility Degradation*

The displacement ductility factor $\mu_\Delta$, which is the ratio of the ultimate displacement $\Delta_u$ to the yield displacement $\Delta_y$, was calculated for each column to compare the performance in terms of sustained ductility. Under a particular drift level, Table III shows that the ductility of the unheated column is about 60% of the fire-damaged column. The differences in ductility mainly reflect the effect of fire on mechanical properties. Furthermore, the differences in ductility mainly reflect that elevated temperature exposure reduces the capacity of the column. It can be concluded that the exposing temperature has a remarkable effect on the load-bearing capacity of the columns. The ductility of the columns was not found to be affected by fire exposure up to 30min.

TABLE III. DUCTILITY PARAMETERS OF THE COLUMNS

| | **Before fire (at 20°C)** | **After fire of $t_{peak}$=15min** | **After fire of $t_{peak}$=30min** | **After fire of $t_{peak}$=60min** | **After fire of $t_{peak}$=90min** | **After fire of $t_{peak}$=120min** |
|---|---|---|---|---|---|---|
| **Lateral yield displacement $\Delta_y$(cm)** | 3.387 | 4.544 | 5.506 | 5.36 | 5.01 | 5.1 |
| **Lateral ultimate displacement $\Delta_u$(cm)** | 5.405 | 6.368 | 5.506 | 5.36 | 5.01 | 5.1 |
| **Ductility factor: $\mu_\Delta=\Delta_u/\Delta_y$** | 1.6 | 1.40 | 1 | 1 | 1 | 1 |

## IV. CONCLUSIONS

The following remarks can be extracted from this study:

- The results showed that the lateral load capacity and ductility decreased substantially as a result of fire exposure.
- Post-fire lateral load capacities were not considerably affected by the increase in fire duration up to 60min. After 60min of fire exposure, a little reduction in the lateral load capacity of the column was observed, which minimized the impact of the concrete compressive strength loss on the post-fire lateral capacity of the columns.
- The reduction in lateral load capacity appeared to be caused by the residual properties of the concrete after fire exposure. For low axial loads (25% of $N_r$), the effects of vertical compressive load on the behavior of the columns were considerable.
- The energy dissipation of the columns was not significantly affected by fires with duration up to 30min. For a column subjected to a 90min fire, the reduction was about 25% compared to an unheated one, which was attributed to the loss of compressive strength. The stiffness of the column after a 30min fire was not affected. The slope of energy dissipation curves was confusing after a 30min fire.
- Ductility reduction was significantly affected compared to an unheated column after a 15min fire. This reduction was attributed to the post-fire loss in the mechanical properties of concrete. This can be explained by the greater loss in concrete's compressive strength and thereby the lower lateral load-bearing capacity of the columns.
- The application of the available Eurocode model for concrete after fire exposure resulted in a good estimation of the residual capacity of fire-damaged columns compared to the available existing results.

## NOMENCLATURE

$N_r$ = Vertical load-bearing capacity of a column.
$H_u$ = Horizontal load-bearing capacity of a column.

$t_{peak}$ = Time corresponding to the end of the heating phase - duration of the heating phase of the fire.
$\Delta_y$ = Lateral yield displacement.
$\Delta_u$ = Lateral ultimate displacement.
$\mu_\Delta$ = Ductility factor.

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