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Lateral load behavior of RC buildings exposed to fire: case study

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ABSTRACT

Fire damaged reinforced concrete buildings are often made usable by performing non-structural and cosmetic repairs. These types of repairs do not guarantee the safety of such buildings under the effect of future lateral loads, such as earthquakes. In this paper, an evaluation of the lateral load response of an RC fire damaged – building in Libya is presented. The building was used as a courtyard for the public and the fire visibly damaged some of its columns and beams. Material tests were conducted in situ and in the laboratory to evaluate the material strength of the building after the fire. Then, lateral load carrying capacity of the building before and after the fire was evaluated. Analysis results show that up to one-hour fire duration the building does not lose a considerable amount of its lateral strength; however, at about two and half hours fire duration, it lost about 30 percent of its lateral load carrying capacity. Lastly, analytically predicted residual strength material properties are in good agreement with material test results.

ARTICLE HISTORY

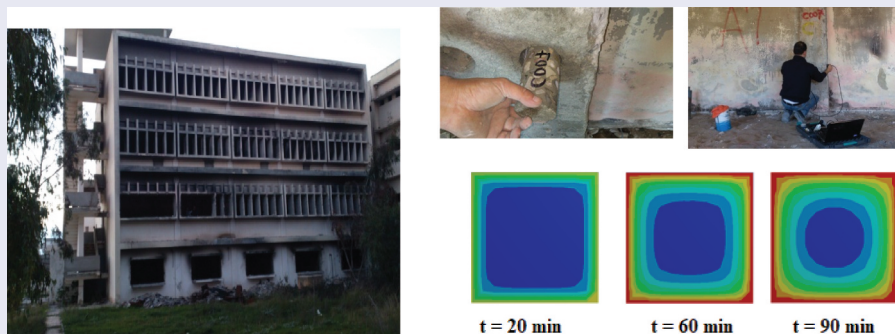
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KEYWORDS

Fire-damaged buildings;
seismic performance;
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RC building exposed to fire: Experimental and theoretical work



Conclusions: up to one hour fire duration the building does not lose a considerable amount of its lateral strength, however, at about two and half hours fire duration it lost about 30 percent of its lateral load carrying capacity.

1. Introduction

Buildings in urban regions need to be designed in a way that provides a “safe and sound” environment for the inhabitants, for the design life of the building. Whilst almost all codes guarantee safety under load combinations representing the extreme loading possibilities, pre and post earthquake fire loadings have not received adequate attention in the available codes. The occurrence of fire inside a reinforced concrete building is an unexpected phenomenon and can break out at any time due to various reasons. Although reinforced concrete is high fire resistance compared with other building materials, when exposed to fire, concrete degraded,

and its strength decrease. Heat also causes the concrete to crack, which further degrades its stiffness. In general, the strength of concrete and reinforcement steel decrease with increase of heating temperature (Arioz 2007; Raut and Venkatesh 2012; Tao and Wang 2013). Restoration of fire-damaged reinforced concrete buildings is of great interest both for the owners and insurance company in terms of both reducing the capital cost and restarting the business due to earlier reoccupation of the buildings. Columns are the primary structural elements that transfer the loads of a building vertically to the foundation. Several studies and experimental works in the literatures show that when RC columns are exposed to fire, the material properties of

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concrete and the reinforcing steel change as a result of the temperature increases. The decreases in yield strength and modulus of elasticity reduce the overall strength of the column. A study on the Effect of fire on in-plane and out-of-plane behavior of reinforced concrete frames with and without masonry infills was conducted by P. Kumar, and G. Srivastava. They develop a finite element-based numerical model to compute the fire response of RC frames and infill-frames. It was found that the maximum degradation in in-plane stiffness of the frame with infill was about 17% lower than that of the frame without infill (Kumar and Srivastava 2018). Fire-damaged reinforced concrete buildings are often made usable by performing non-structural and cosmetic repairs without proper seismic evaluation and retrofitting. Prediction of lateral load-carrying capacity of a fire-damaged RC building is an important step in seismic performance evaluation of the building under future earthquakes effects. To investigate the effect of fire on a RC structure, modeling strategies typically have three components: the fire model, the heat transfer model, and the structural model (Kumar and Srivastava 2017). The fire model provides the temperature–time relationship of the fire to which a given structure is exposed to. Typically, the fire model is derived from one of the following: standard fire curves, realistic fire curves, and equivalent fire severity curves. In this study standard fire curve was used. The heat transfer model is utilized for the estimation of the thermal gradients or temperature profiles within the structural members corresponding to the given fire exposure. It is an essential component of the analysis, as the load-carrying capacity of the structural elements depends upon the internal temperatures. Researchers have developed empirical relations to facilitate quick estimation of temperatures within the structural members under certain conditions. One such method is the 500

isotherm method prescribed by the Euro code (EN 1992-1-2 2004). When greater accuracy is desired, FE software such as ANSYS Inc (2013), ABAQUS (2011), and OpenSees can be utilized. In this study ANSYS Inc software was used. The structural model is used to gauge the effect of a rise in internal temperature on the load-carrying capacity or strength of the structure. The temperature profile obtained from the heat transfer model along with applied loads and temperature-dependent material properties are given as input, and corresponding stresses and serviceability limit states are determined to evaluate the failure criteria. The residual load-carrying capacity computed from the structural model is compared with the required load capacity during the fire, to assess whether the design is safe and is performing satisfactorily. In this paper an evaluation of the seismic performance of a RC building subjected to pre-earthquake fire is presented. The building was used as a central court building of Al-Baida city in Libya. The fire occurred in the right wing of the ground floor of the building, and as reported by witnesses, it lasted approximately for an equivalent continuous time of 2–3 hours, involving a lot of paper documents, furniture, and other materials. More reliable information about temperatures was obtained during on-site surveys. All furniture, windows, doors, infrastructure are severely affected by fire. Most of the structural elements are grey to black in color. Examinations of rubble show all floor tiles are dismantled and burned, window glass is shattered but not melted. The electric wires are burned however the copper in electric wires had not been softened. These observations indicate that the temperature is in the range of 400 to 600°C (Fire Safety of Concrete Buildings 2010; Bernard, William, and William 1972). This is also assured by the grey to black color of the structural concrete elements surfaces. Concrete color provides a broad, general guide of temperatures



Figure 1. General view of the building after the fire.

(Yüzer, Aköz, and Öztürk 2004). The fire caused readily visible surface damage, as shown in Figure 1. According to fib bulletin 46, 2008 fire damage can be classified as Class 3 (FIB Task Group 4.3 2008). Material tests were conducted in situ and in the laboratory to evaluate the material strength of the building after the fire. Furthermore, a finite element model was developed to evaluate the distribution of temperature within the cross-section of each structural member during a fire. Then the results were used to estimate the residual strength of each fire-damaged structural member in the building. For each different fire exposed time, the lateral load-carrying capacity of the building was predicted using nonlinear pushover analysis that takes into account the reduction in the strength of each fire-affected member. The seismic performance of the building before and after the fire was evaluated according to the Turkish earthquake code TERDC-2007 (Ministry of Publicworks and Settlement 2007). The following sections of this paper elaborate on the details of building description, material tests, heat transfer analysis, residual strength prediction, nonlinear pushover analysis, and seismic performance results.

2. Architectural plans and structural system

The building consists of a ground floor and upper three typical floors, built in the early 1970s; each story is 3 m clear height and 1460 m² plan area. The structural framing is made of conventional cast-in-situ reinforced concrete as it is shown in Figure 2. The framing system consists of a grid of columns, mainly of a 40 cm square size, as shown Figure 2. The columns are reinforced with 8 ϕ 18 longitudinal mild steel plain bars and ϕ 8 lateral reinforcement spaced at 15 cm center to center. The maximum spacing between the columns is 6 m center to center in short direction and 4.5 m spacing in

long direction. The slabs of the building floors are hollow block type with total thickness of 25 cm comprising (approximately) 5 cm topping and 20 cm thickness hollow block. The ribs are of 15 cm thickness and spaced at 55 cm center to center with 2 ϕ 18 mm bars to resist negative and positive bending moments. The ribs are mainly constructed continuously in 3 spans 6 m, 2 m and 6 m long, respectively, except at entrance lobbies where it is 6 m long single spans. The main beams supporting the ribs are embedded beams with a 25 cm thickness and 10 mild steel plain bars combination of ϕ 18 and ϕ 20 top and bottom provided with ϕ 8 double stirrups spaced at 15 cm center to center. Conventional local type hollow block of 20 cm thickness is used for partitions.

3. Material tests

Tests were conducted in situ and in the laboratory to determine concrete material properties. The nondestructive *rebound hammer* and *pulse velocity* tests were conducted in situ according to the recommendation given in ACI 228-R10 and ASTM C 805 (ACI Committee 228.1R-03 1994). In the rebound hammer test, all affected structural members are divided into grids with designation numbers and labels, then the hammer test is performed, and the rebound number is recorded. An average of at least ten rebound readings are taken for one test, see Figure 3(c). In the pulse velocity test method, the estimation of the compressive strength of concrete is based on measuring the time it takes for a pulse of vibration energy to travel through a concrete member. The tests were performed according to ASTM C 597 standard (ASTM C597 2009). Some test results are shown in Table 1 and Figure 3. Twenty core samples have been extracted from the different structural elements affected by the fire. Locations of the concrete

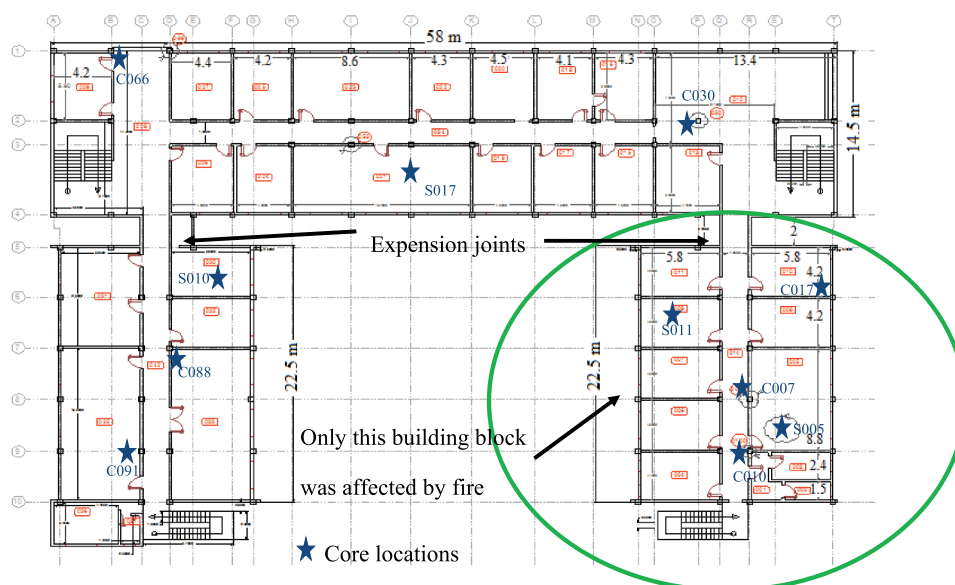


Figure 2. Plan view of the ground floor of the building's blocks.

Table 2. Typical result of compressive strength testing.

Element no.	Height mm	Diameter mm	Ratio	Volume cm ³	Weight gr	Density gr/cm ³	Equivalent compressive strength MPa	Ultrasonic velocity m/s
C192	169	67.8	2.49	610.4	1345.5	2.204	11.03	3773.00
S111	165	67.8	2.43	595.9	1407.4	2.362	13.25	4014.00
S142	121	67.8	1.78	437.0	979.7	2.242	16.51	4187.00

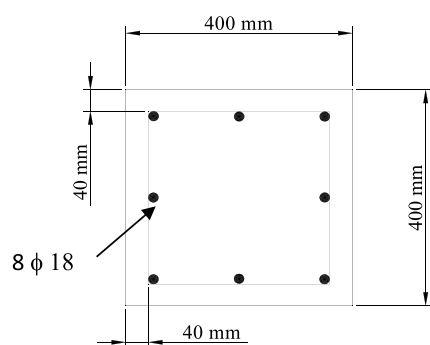
Table 3. Rebound values versus core compressive strength values.

Fire effect	Element no	Rebound number (RN) average	Core compressive strength MPa
Fire exposed	C192	16	11.03
	C007	12	9.6
	S111	19	13.25
	S005	18	12
	C 010	15	10.6
	S011	18	12.7
	S110	25	15.4
	C199	18	12.5
Fire non exposed	C100	17	11.3
	C091	33	17.1
	C088	35	18.2
	S336	32	16.85
	S142	32	16.51
	S017	35	18.5
	C030	34	16.8
	C066	30	15.7
	S010	36	20.2

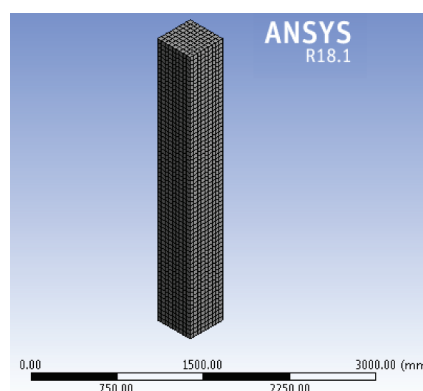
column was modeled using 8-node linear heat transfer brick elements as shown in Figure 4(b). Modeling of reinforcement bars is neglected, and it is assumed that the reinforcement bar temperature is equal to the surrounding concrete temperature. The developed FE model takes into account the nonlinear temperature dependent on concrete material properties, such as specific heat, and thermal conductivity. The variation of concrete thermal conductivity with temperature specified by EN 1992-1-2:2004 code was used in the model (see Figure 5; XTRACT software). The concrete density is assumed to be constant and equal to 2300 kg/m³. The analysis results of a column exposed to ISO834 standard fire curve from four sides are shown in Figure 6.

To illustrate the thermal predictions from the model, the temperature variation is plotted as a function of fire exposure time at various locations of the column cross-section in Figure 7. The temperature at various depths of concrete, as well as in reinforcement bar locations, increases with fire exposure time. As expected, the predicted temperature decreases with increasing distance from the fire-exposed side.

The cross-sectional temperatures generated from thermal analysis are used as input to the strength analysis (i.e. generation of moment–curvature relationship and axial force–moment interaction diagram). In the strength analysis, a finite element model was developed using XTRACT software as shown in Figure 8(a) (XTRACT software). In the analysis, it is assumed that plane sections before bending remain plane after bending and there is no bond-slip between steel reinforcement and concrete. The fire unaffected concrete compressive strength of 17 MPa which is obtained from material tests was used as initial value. The initial value of reinforcement bars yield strength was taken to be equal to 420 MPa. As the fire duration increases, the temperature will increase inside the column cross-section, causing a reduction in the compressive strength of each concrete layer of the XTRACT model and also causing a reduction in the reinforcement bars stress–strain relationship. The reduction in the material properties of concrete and reinforcement steel due to elevated temperatures was taken into account using reduction factors proposed by the Eurocode (see Figure 8(b); EN 1992-1-2 2004). The proposed reduction factors are in good agreement with the experimental material test results. This can be shown from the results in Figures 7 and 8(b) as



(a)



(b)

Figure 4. (a) Column section details, (b) FE model.

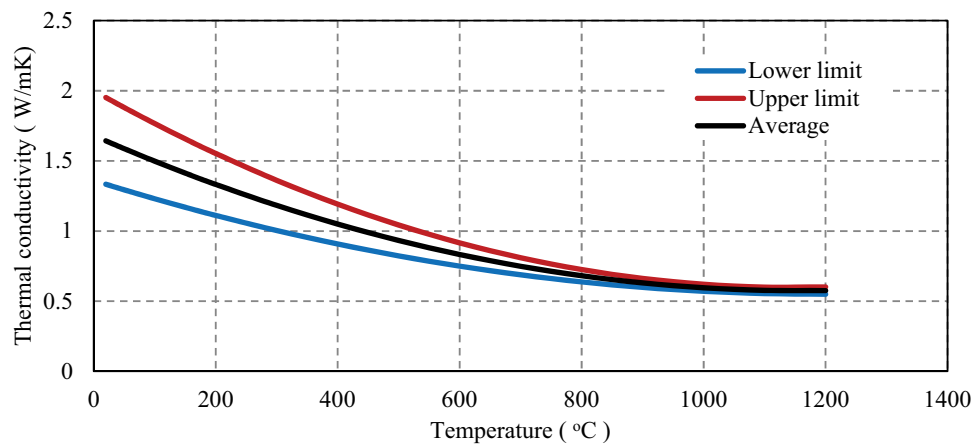


Figure 5. Variation of concrete conductivity with temperature (EN 1992 standard).

follows: it is shown from Figure 7 that at 120 min fire duration and at a depth of 100 mm from the face of the column, the concrete temperature was equal to 300°C; and at this temperature according to Figure 8(b), the reduction factor of concrete strength is about 0.8; that is the concrete strength becomes ($17 \times 0.8 = 13.8$ MPa) and this value are in good agreement with the experimental results given in Tables 1 and 2. The axial force-moment capacity diagrams for the column at various time durations were obtained and presented in Figure 9. From the figures, it is clear that both the moment and axial load carrying capacities of the column decrease with increasing time of fire exposure. This is due to the deterioration in the material strength and stiffness as a result of increased temperatures in concrete and steel. Furthermore, the analysis results show that up to 60 min of fire duration the column does not lose a considerable amount of its strength, however at 120 min fire duration it will lose about 35 percent of its axial load and moment carrying capacities.

5. Earthquake hazard

Soil boring and laboratory tests were conducted to determine the geotechnical soil properties at the building site. From these tests, the allowable bearing

strength was 1.14 kg/cm^2 , and the soil type according to the Turkish Earthquake Resistant Design Code (TERDC-2007) was Z3. While no definitive probabilistic seismic hazard assessments (PSHAs) have been undertaken for Libya, several studies investigating the seismicity and the seismotectonic do exist. Early seismic zonation maps were presented by Mallick & Morghem and Kebeasy. More recent studies by Suleiman & Doser, Al-Heety & Eshwehdi and Al-Heety provide a more comprehensive account of the seismicity and seismotectonic and benefit from locally obtained strong-ground motion data from the Libyan Digital Seismological Network (LDSN) (Al-Heety and Eshwehdi 2006; Al-Heety 2013). According to these studies and the seismic map of the world shown in Figure 10(a), the maximum peak ground acceleration at the building site can be taken as 2 m/s^2 . And according to the Turkish earthquake code and the site soil conditions, a 5% damped elastic response spectrum was obtained and plotted in Figure 10(b).

6. Seismic performance evaluation

To predict analytically the seismic performance of the case study building before and after the fire, a finite element model was established using

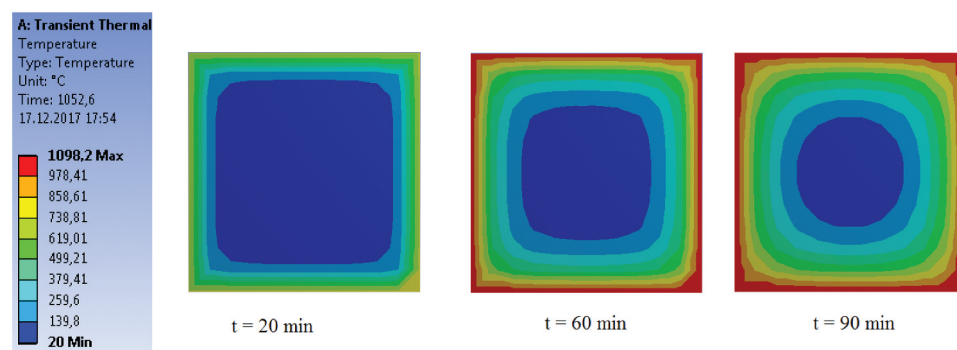


Figure 6. Temperature distribution.

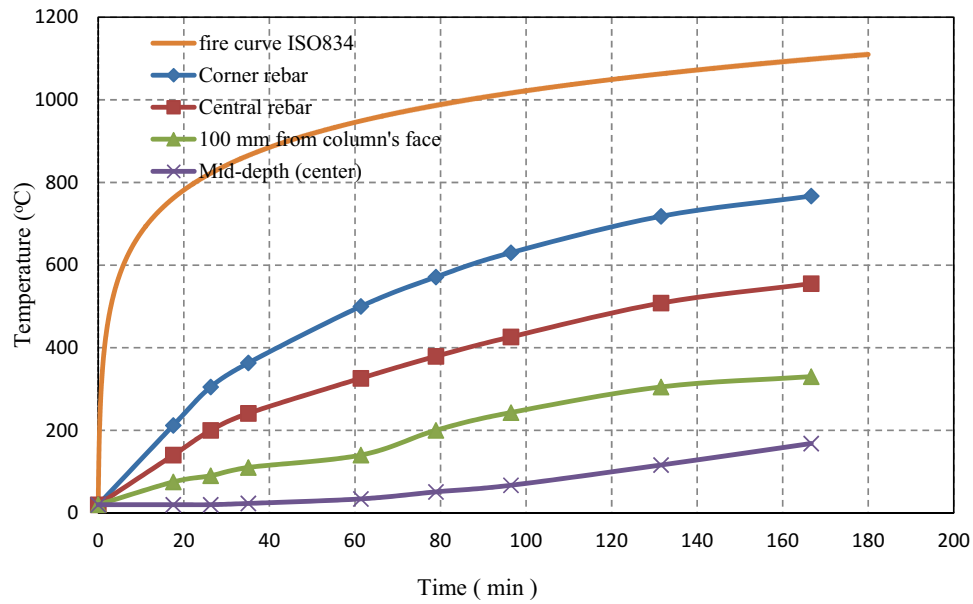


Figure 7. Temperature as a function of time at different locations of column cross section.

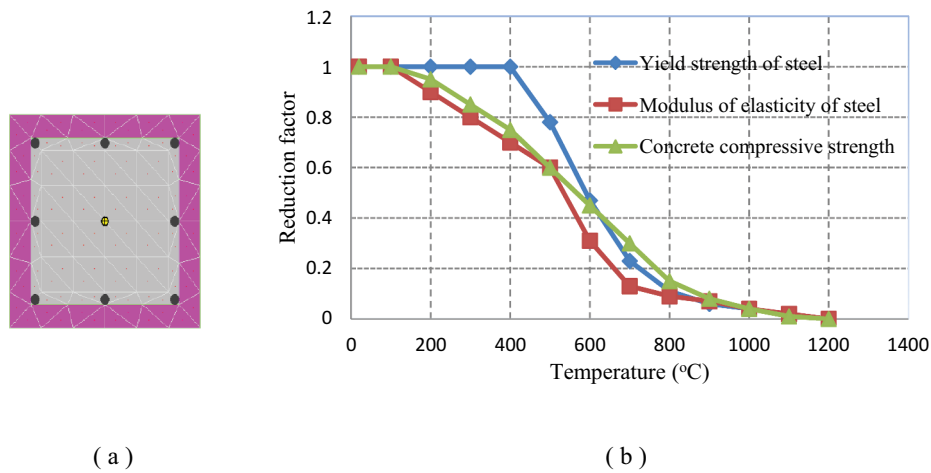


Figure 8. (a) XTRACT FE model, (b) Strength reduction factor according to Euro code (Kumar and Srivastava 2018).

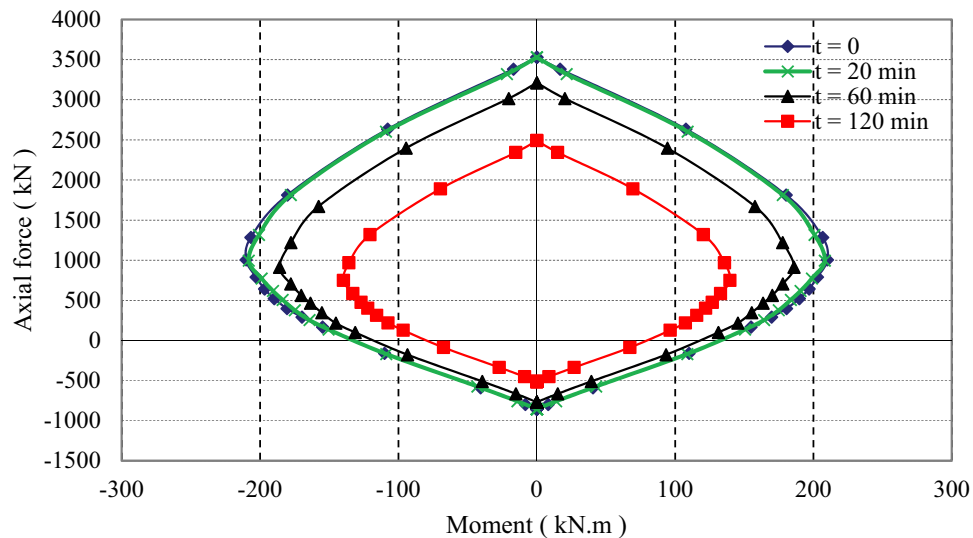


Figure 9. Axial force–moment interaction diagrams at various times for the RC column under fire exposure.

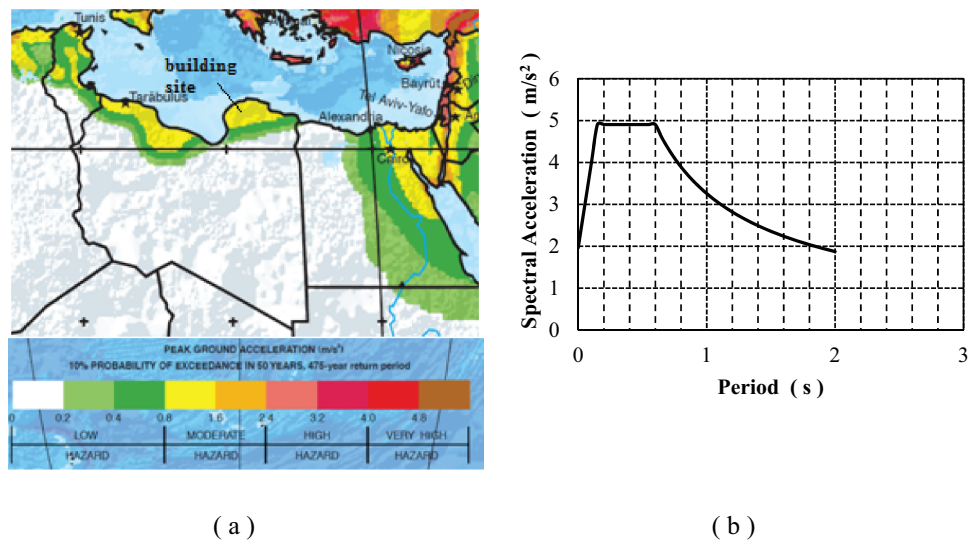


Figure 10. Building site: (a) seismicity and, (b) design response spectrum.

SAP2000 software. In the beginning, the structures were modeled without considering the fire effects, that is, the model represents the original state of the structure. Frame elements were used to model the beams, and columns and floor slabs were modeled by shell elements. Only the building block that affected by fire was modeled (i.e. the right-side block only, see Figure 2). The finite element model is shown in Figure 11. The total weight of the structure is 13,254 kN. The fire unaffected concrete compressive strength and yield strength of the reinforcement steel used in the analysis were 17 MPa and 420 MPa, respectively. These values were obtained from destructive and non-destructive material test results. To include the reinforcement details of each structural element in the model, reinforcement numbers, diameters, and stirrup spacing for each structural member were measured using a reinforcement

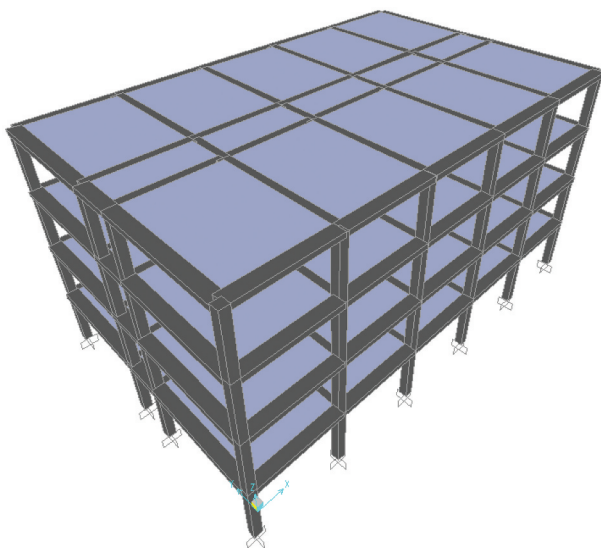


Figure 11. Finite element model of the building block.

radar device. These measured values were compared to the existing data of the project drawing details. At the end of this long process, the reinforcement details of each beam and column were included in the model.

The nonlinear static analysis procedure defined in the Turkish earthquake code TERDC-2007 was used to estimate the seismic performance of the building. This procedure is similar to the nonlinear static analyses procedure defined in FEMA356 standard. The procedure uses the philosophy of earthquake design based on performance, structural members are normally designed to satisfy various levels of performance, some of which are operational (O), immediate occupancy (IO), life safety (LS), and collapse prevention (CP). The procedure is based on a pushover analysis in which the inelastic variation of the base shear with respect to the top-story displacement (i.e. the pushover-curve) is obtained considering monotonic increments in the adaptive load patterns, such as the equivalent seismic loads or first mode shape. Then, the inelastic demand spectrum for the structure and the capacity spectrum, which is transformed from the pushover-curve, are compared, and inelastic demands are obtained from the intersection of the two curves. Based on the procedure of TERDC-2007, the capacity curve is established from the pushover curve by transforming the coordinates into modal displacement and modal acceleration (Ministry of Publicworks and Settlement 2007). The pushover curves of the fire unaffected building were obtained in X and Y directions. The X direction denotes the deformed shape of the building associated with its first vibration mode, and the Y direction denotes the deformed shape of the building associated with its second vibration mode.

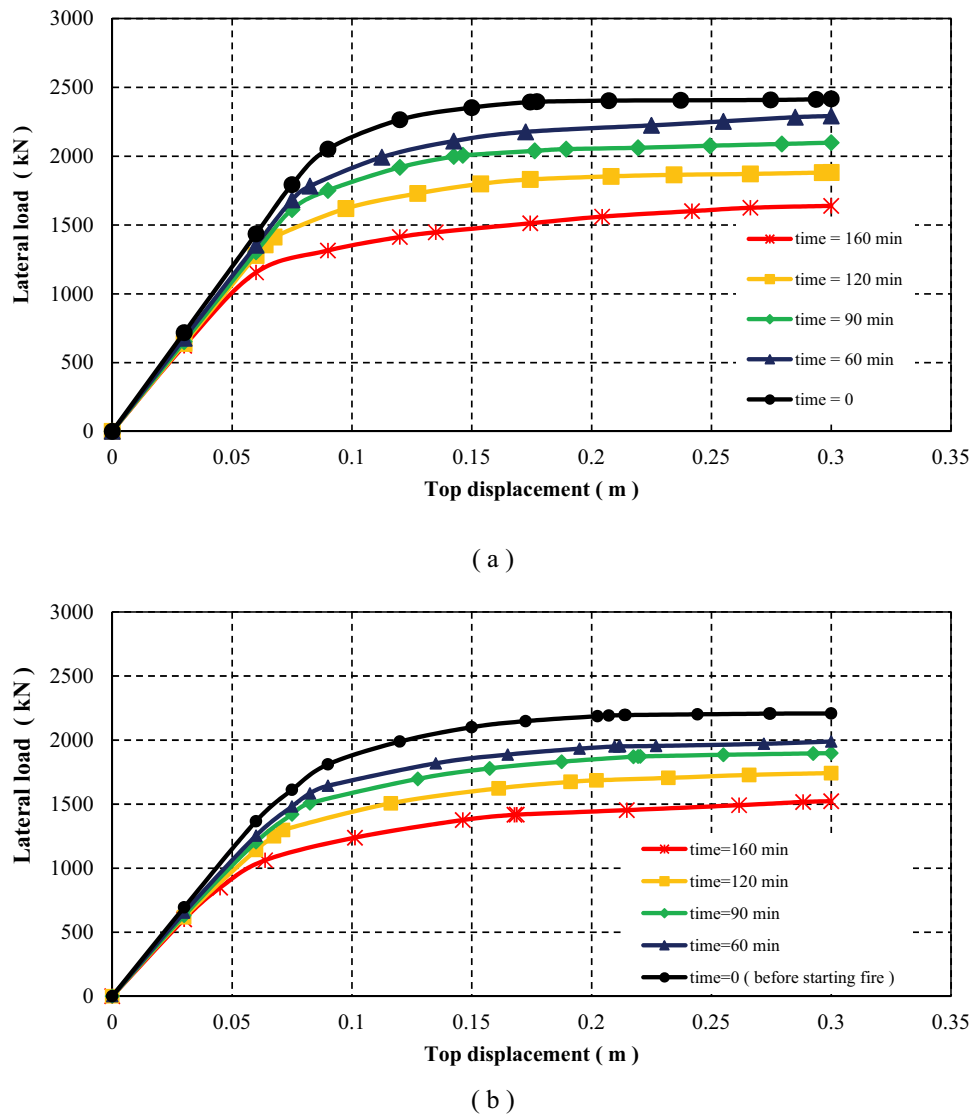


Figure 12. Pushover curves before and during fire (a) X direction, (b) Y direction.

To predict analytically the lateral load response of the building during different fire durations, the residual strength of each column and beam in the structure was calculated using the procedure presented in Section 4. And for each fire duration the calculated residual strengths were used to modify the finite element model of the building, and a pushover analysis was performed. The pushover curves for the building at different fire durations were obtained in the X and Y directions. Analysis results before the fire and at different fire durations are shown in Figure 12.

The seismic performance point of the building before and after the fire (i.e. at about two and half hours fire duration) in the Y direction was obtained using the TERDC-2007 code procedure and presented in Figure 13. At the performance point, the top displacement of the building before and after the fire was about 0.105 m and 0.13 m, respectively. To

evaluate the plastic hinge rotations in the structural elements, plastic rotation limits defined by FEMA356 were used. For immediate occupancy (IO), life safety (LS), and collapse prevention (CP) the plastic rotation limits are 0.003, 0.012, and 0.015 rad, respectively (FEMA356 2000). Analysis results show that before the fire, most of the columns and beams rotations lie below the life safety (LS) limit state, while after the fire, some columns and beams form plastic hinges that exceed life safety (LS) and collapse prevention (CP) limit states as it is shown in Figures 14 and 15. Furthermore, modal analysis results show that pre-fire vibration periods of the first, second and third modes are 1.04, 1.028, and 0.92 sec, respectively. And after the fire, the vibration periods of the first, second and third modes are 1.29, 1.27, and 1.15 sec, respectively.

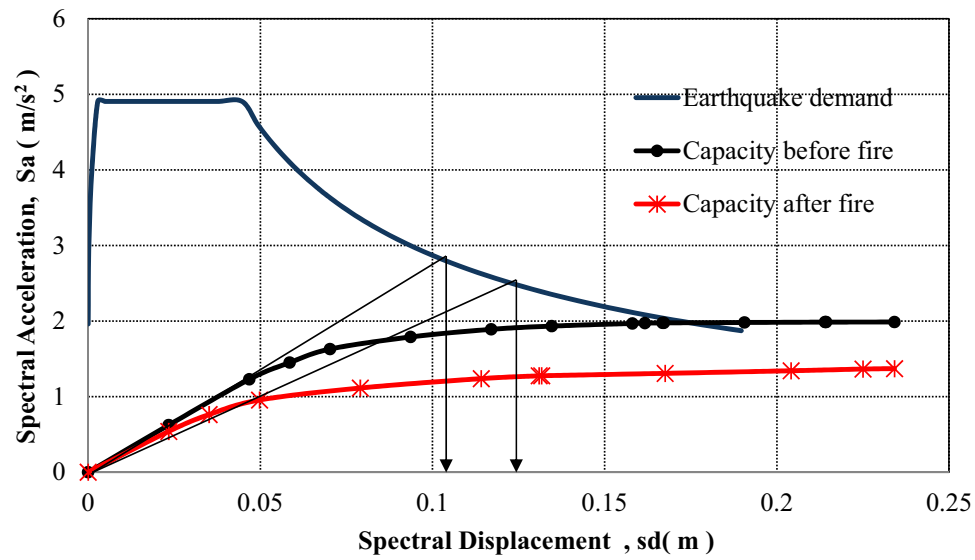


Figure 13. Performance point of the building before and after fire.

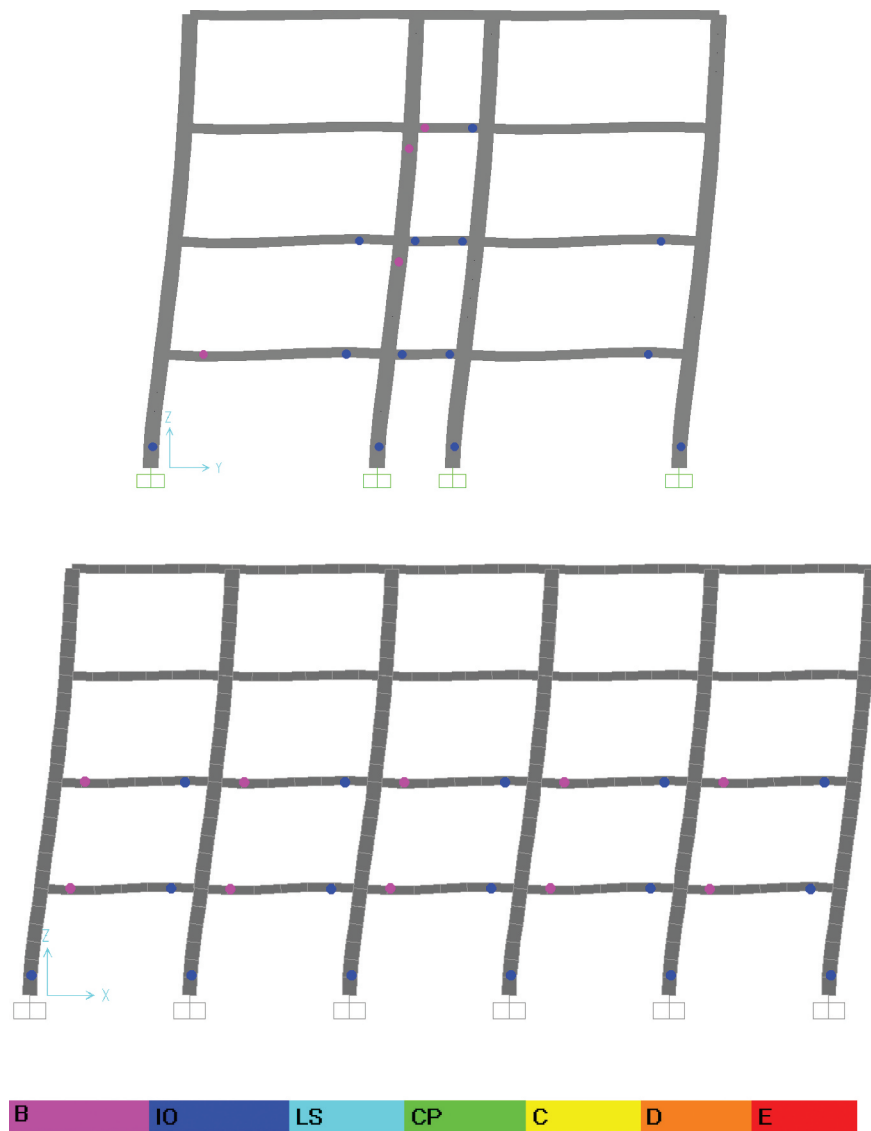


Figure 14. Columns and beams rotations before fire.

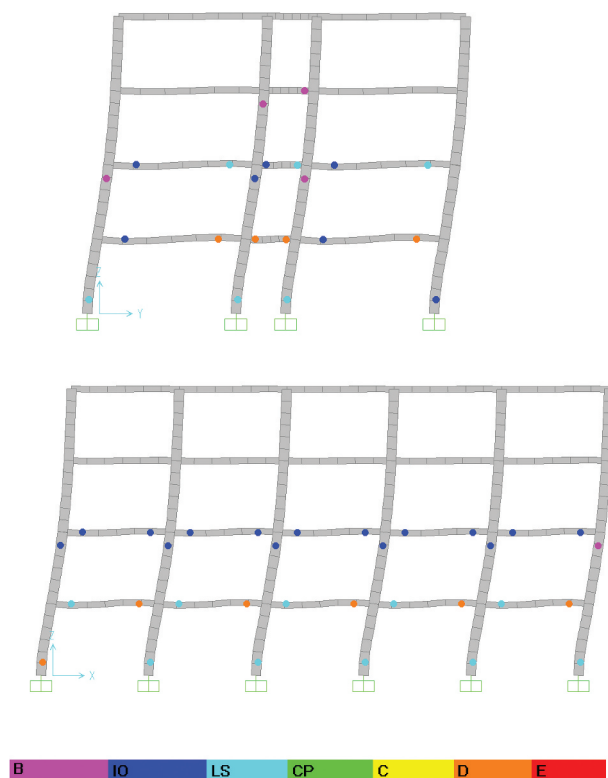


Figure 15. Columns and beams rotations after fire.

7. Conclusions

Based on the experimental and analytical results of this study, the following conclusions can be drawn:

- (1) The lateral load-carrying capacity of the case study building decreases with increasing fire-exposed time. Analysis results show that up to one-hour fire duration the building does not lose a considerable amount of its lateral strength, however at about two and half hour fire duration it will lose about 30 percent of its lateral load-carrying capacity. According to these results, it can be said that: if the building was exposed to fire for more than one hour, cosmetic repairs may not be suitable, and the building should be investigated for possible strengthening.
- (2) The seismic performance of the building before the fire was acceptable because at the performance point all beams and columns rotations were at the immediate occupancy (IO) limit state.
- (3) According to material test results, thermal analysis results and witnesses report it is expected that the building was exposed to fire for up to 2.5 hours. Furthermore, analysis results on the fire-affected building that exposed to fire up to 2.5 hours show that the rotations of some columns and beams

exceeded life safety (LS) and collapse prevention (CP) limit states. And this will form plastic hinges and the building needs to be strengthened.

- (4) Based on transient thermal analysis results shown in Figure 7, it is clear that the reinforcement bars at the column corners are more affected by fire than the central reinforcement bars.
- (5) At 120 min fire duration and at a depth of 100 mm from the face of the column, the concrete temperature was equal to 300°C; and according to Euro code this will reduce the concrete strength by about 20%; that is the concrete strength becomes ($17 \times 0.8 = 13.8$ MPa) and this value is in good agreement with the experimental test results given in Tables 1 and 2.
- (6) At 120-min fire duration, the average temperature of the reinforcement bars is about 550°C and according to Euro code this will reduce the strength and elastic modulus of steel by about 32% and 40%, respectively.
- (7) Pre- and post-earthquake fire loadings have not received adequate attention in the available codes. In future, more studies need to be conducted in order to obtain code formulation that estimates the residual lateral load carrying capacities of fire affected RC buildings.

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Disclosure statement

No potential conflict of interest was reported by the author(s).

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