ANALES de Edificación

Received: 08/12/2022
Accepted:13/12/2022

Anales de Edificación
Vol. 8, Nº3, 40-46 (2022)
ISSN: 2444-1309
Doi: 10.20868/ade.2022.5097

Evaluación de la capacidad de resistencia al fuego de una estructura de hormigón armado tras un terremoto.
Post-earthquake fire capacity assesment of a reinforced concrete frame.

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Resumen-- **Un terremoto es uno de los peligros más desastrosos que puede sufrir un edificio. Puede ir seguido de incendios, cuyos efectos pueden ser mayores que los del propio seísmo en las zonas urbanas. Las normas de construcción actuales no tienen en cuenta esta doble acción, que actúa de forma secuencial y descuida la probabilidad de que se produzcan incendios después de un terremoto. Los edificios no están suficientemente diseñados para este tipo de acciones que pueden conducir al colapso. El objetivo de este artículo es presentar una evaluación de la capacidad de resistencia al fuego tras un terremoto (PEF) de un armazón de hormigón armado que se ha seleccionado de un edificio, diseñado de acuerdo con los códigos de diseño de edificios de Argelia. Se realiza un análisis sísmico no lineal del armazón para evaluar su capacidad portante frente al efecto del terremoto. El armazón dañado se someterá a altas temperaturas debidas al fuego y se analizará numéricamente mediante el software ANSYS APDL, incluyendo las no linealidades geométricas y de materiales. Los resultados muestran que cuando una estructura, previamente dañada por la acción sísmica, se expone a un incendio posterior al terremoto, su vulnerabilidad se ve influida en comparación con la de la estructura expuesta únicamente al fuego. Se discute el modo de colapso global o local de la estructura sometida a un incendio post-terremoto o a un incendio solo.**

Palabras clave— Estructura de CR; incendio post-terremoto; modelización numérica; análisis no lineal; RPA99v2003.

Abstract— **Earthquake is one of the most disastrous hazards that a building can suffer. It can be followed by fires, the effects of which may be greater than those of the earthquake itself in urban areas. The current building standards do not take this double action into account, which acts in a sequential manner and neglects the probability of fires occurring after an earthquake. Buildings are not sufficiently designed for such actions which can lead to collapse. The aim of this article is to present an assessment of the post-earthquake fire (PEF) capacity of a reinforced concrete frame that has been selected from a building, designed according to Algerian building design codes. A non-linear seismic analysis of the frame is carried out in order to assess its bearing capacity against the effect of the earthquake. The damaged frame will be subjected to high temperatures due to fire and numerically analysed using ANSYS APDL software including material and geometric nonlinearities. The results show that when a structure, previously damaged by seismic action, is exposed to a post-earthquake fire, its vulnerability is influenced compared to that of the structure exposed to fire alone. The mode of global or local collapse of the structure subjected to post-earthquake fire or to fire alone is discussed.**

Index Terms— RC frame; post-earthquake fire; Numerical modelling; non-linear analysis; RPA99v2003.

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I. Introduction

FIRES that follow a major earthquake have always been a significant danger and risk for urban areas. Major earthquakes are often followed by post-earthquake events, such as soil liquefaction, landslides, etc., and the damage caused by these events may even be worse than that of the earthquake itself. Among these is the post-earthquake fire which is one of the most catastrophic earthquake-related events to have occurred in urban areas (Benham et al., 2017). Current design codes do not take into consideration the occurrence of post-earthquake fires. The building design is not adequate to deal with this double action which acts in a sequential manner leading to collapse (Benham et al., 2013).

Much research has been carried out on reinforced concrete structures to assess their performance under the effect of post-earthquake fire. Behnam et al. (2013) investigated the behaviour of reinforced concrete structures subjected to post-earthquake fire. They studied two three-storey reinforced concrete frames with identical geometry but designed for two different earthquake levels. The frames underwent a seismic analysis by the Pushover method followed by an ISO834 standard fire according to two different fire scenarios. They concluded that reinforced concrete structures damaged by seismic loads have less fire resistance than undamaged structures. They also found two types of collapse mechanisms during the fire analysis. While the global collapse occurred in the frames subjected to post-earthquake fire, the local collapse occurred for the fire case only. Another research work was conducted by Vitorino et al. (2020) which presents the evaluation of the post-earthquake fire capacity of reinforced concrete elements. They created different sections for beams and columns, with different types of damage and the number of fire frontiers. Thermomechanical analysis of the elements was performed to determine the influence of the damage and the fire frontiers on the temperature of the steel reinforcements. It was concluded that the damage due to earthquake as well as the fire frontiers greatly influence the resistance of reinforced concrete members to post-earthquake fire.

In this article, numerical modelling of a reinforced concrete frame was established to study its behaviour and to evaluate its post-earthquake fire capacity. The numerical analysis was performed using the ANSYS APDL software including geometric and material nonlinearities.

II. Methodology

The procedure of the analysis carried out in this study consists firstly in the application of the vertical loads due to the dead and live loads. Seismic analysis of the reinforced concrete frame is then carried out by the non-linear static pushover analysis while keeping the vertical loads constant (Elnashai, 2001). To determine the target displacement of the structure, the N2 method provided by Eurocode 8 (CEN, 1998) was used. The inelastic response spectrum in acceleration-displacement format is determined from the elastic response spectrum (Fajfar, 2000). After the target displacement is determined, the frame will be subjected to ISO834 standard fire sequentially (Behnam, 2014). The multilinear isotropic von-Mises plasticity and the bilinear isotropic von-Mises plasticity are used here to model the concrete and steel reinforcement materials plasticity in ANSYS APDL (2015) respectively.

III. Materials properties at elevated temperatures

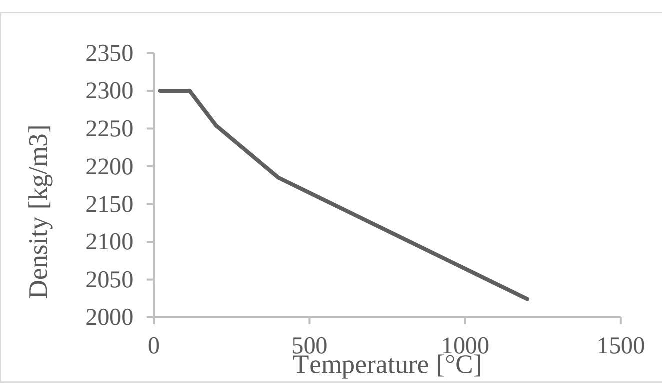
A. *Thermal properties*

The most necessary thermal properties to perform heat transfer and to assess temperature distribution in concrete members are density, thermal conductivity, and specific heat (Schneider, 1988).

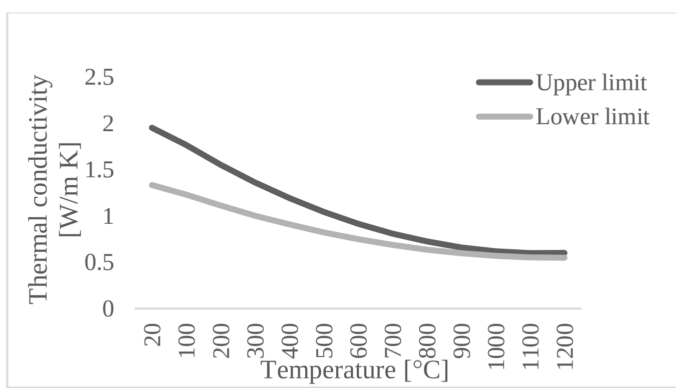
The density of concrete is slightly affected by temperature which is mainly due to moisture loss during heating (Schneider, 1988).

Moisture content, type of aggregate, and mix proportions are the most important parameters for the thermal conductivity. This thermal property of concrete can be determined between two limits: upper and lower. The lower limit was used in this study. Fig. 1 shows the variation of density and the thermal conductivity as a function of temperature of concrete respectively by referring to the EN 1992-1-2 (CEN, 2004).

The specific heat of concrete varies with temperature and with moisture content. Fig. 2 represents the variation of specific heat as a function of temperature at 3 different moisture contents.



(A)



(B)

Fig. 1. (A) Variation of concrete density as a function of temperature, (B) Variation of thermal conductivity of concrete as a function of temperature (CEN, 2004)

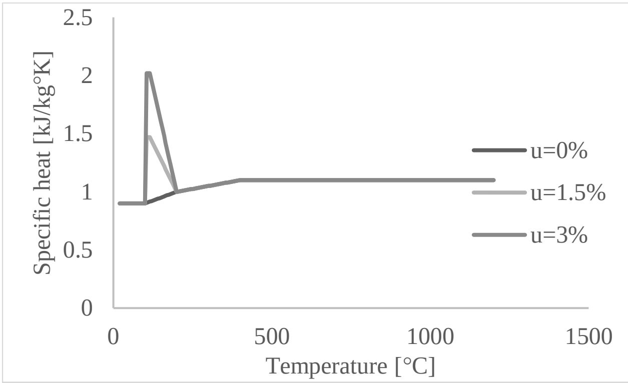


Fig. 2. Variation of specific heat as a function of temperature for moisture content 0%, 1.5% and 3% (CEN, 2004).

Fig. 3 shows the variations of the thermal conductivity and the specific heat of steel reinforcement respectively as a function of temperature evaluated according to EN 1994-1-2 (CEN, 2005).

B. *Mechanical properties*

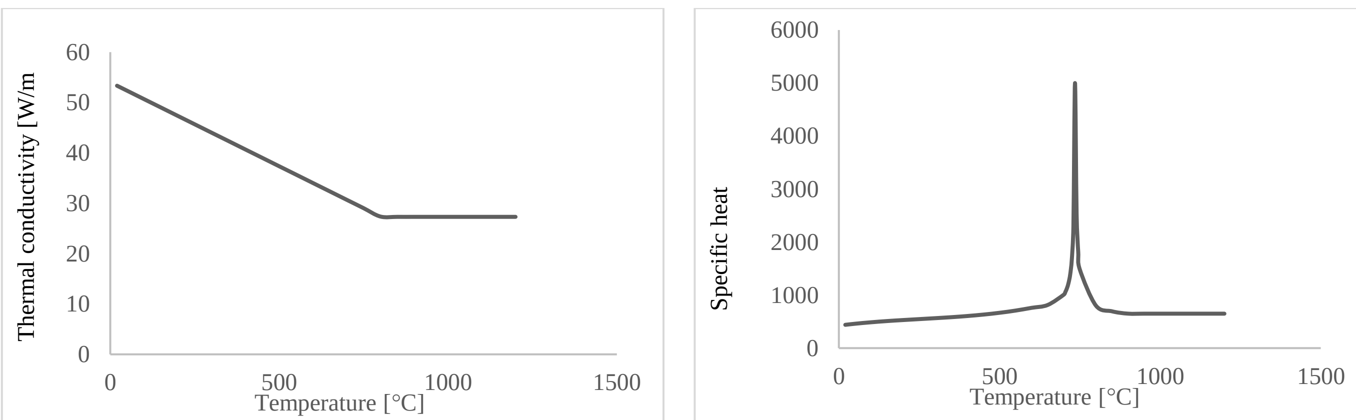
Exposure to high temperatures affects the mechanical properties of concrete and steel (Haouachet al., 2021; Kada al., 2019) materials.
The tensile strength of concrete should typically be ignored, according to EN 1992 1-2 (CEN, 2004). The thermal elongation changes with the cement paste percentage, the aggregate type, the heating rate, and the stress level (Benham et al., 2017). Fig. 4 represents the compressive stress-strain relationship of concrete for different temperatures and the evolution of thermal elongation of siliceous aggregates concrete as a function of temperature.

IV. Finite element model development and validation

The numerical analysis in this study was performed using the ANSYS APDL software including material and geometric nonlinearities. As aforementioned, the analysis has to be performed in sequentially which requires two models with identical geometries, structural and thermal. While the former is to be used in the seismic analysis as well as in transient mechanical analysis considering mechanical and thermal loads, the latter is used in the transient thermal analysis. (Dzolev et al., 2018)

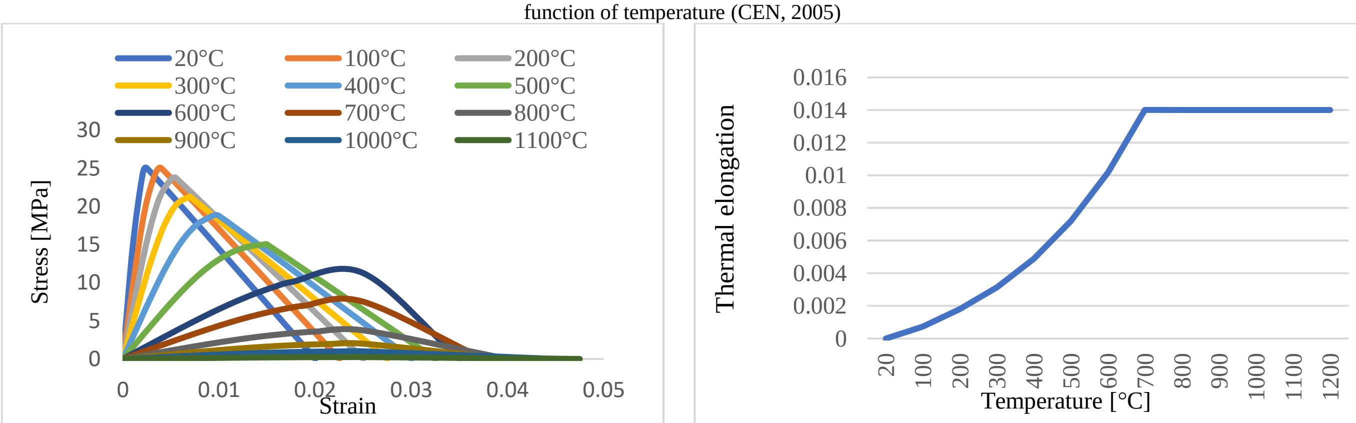
A. *Thermal and Structural finite element models*

SOLID70 and LINK 33 elements are used in thermal analysis. The eight-node 3D SOLID70 with a single degree of freedom at each node is used to model the concrete material. For modelling the steel reinforcement, the linear element LINK 33 of two nodes, each node containing only one degree of freedom of temperature, is used. For exposed and unexposed surfaces, convection coefficients of \alpha_c=25 Wm-2°C-1



(A) **(B)**

Fig. 3. (A) Variation of thermal conductivity of steel reinforcement as a function of temperature, (B) Variation of specific heat of steel reinforcement as a function of temperature (CEN, 2005)



(A) **(B)**

Fig. 4. (A) A) Stress-strain curves of siliceous aggregates concrete at elevated temperatures, (B) Thermal elongation of siliceous aggregate concrete (CEN, 2004)

and \alpha_c=4 Wm-2°C-1 are used, respectively, whereas emissivity related to concrete surface is ε=0.7.

For structural analysis, SOLID65 and LINK180 elements are used. SOLID65 with eight nodes, three degrees of freedom at each node: translations in the three orthogonal directions, is used to model the concrete material. This element is frequently used for concrete discretization due to its capability of plastic deformation, cracking and crushing (Wolanski et al, 2004). For modelling the steel reinforcement, LINK180 element, with two nodes, three degrees of freedom of translation at each node is used. This element is also capable of plastic deformation. A perfect bond is assumed between concrete and reinforcing steel indicating that there is no bond-slip since it can be neglected when the aim of the analysis is to obtain the global response of the structural elements (Gao et al., 2013). Hence, no specific contact element is assigned.

B. *Thermal model validation*

In order to validate the thermal model, a reinforced concrete beam was taken from (Dwaikat et al., 2009). The beam, made of normal strength concrete was tested under the ASTM E119 (2008) standard fire for 3 hours. The sample is 3962 mm, 254 mm, and 406 mm long, width, and depth respectively. Fig. 10 illustrates the temperature variation with time obtained from the experiment and the developed model at the corner rebar as well as the mid-depth of the beam.

A similarity is observed in the temperature evolution between the experimental and the numerical results and are in good agreement. Small differences can be seen between 18 and 40 minutes at the corner rebar showing that the predicted temperatures are smaller than the experimental results. Other differences can be seen after 115 minutes at the mid-depth of the beam.

C. *Structural seismic model validation*

A one-story one-bay reinforced concrete frame tested under cyclic loading was taken from [20] for structural seismic model validation. The frame having a height of 1.625 m and a span of 2.1 m is made of normal strength concrete of C20/25 grade. The cross sections of the beam and columns are 15cmx20cm and 15cmx15cm respectively. The yielding stress for steel reinforcement is 400 MPa and 500 MPa for longitudinal reinforcement and stirrups respectively. The force-displacement diagram obtained from the envelop of the cyclic test and the developed numerical model is illustrated in Fig. 6. The initial stiffness, the ductility as well as the yield force are well estimated using the numerical model proposed in this study.

D. *Thermo-structural model validation*

The aforementioned beam was subjected in the same time to thermal loads through the ASTM E119 (2008) standard fire curve and to tow points constant mechanical loads. The test results are used for thermo-structural validation. Fig. 7 presents

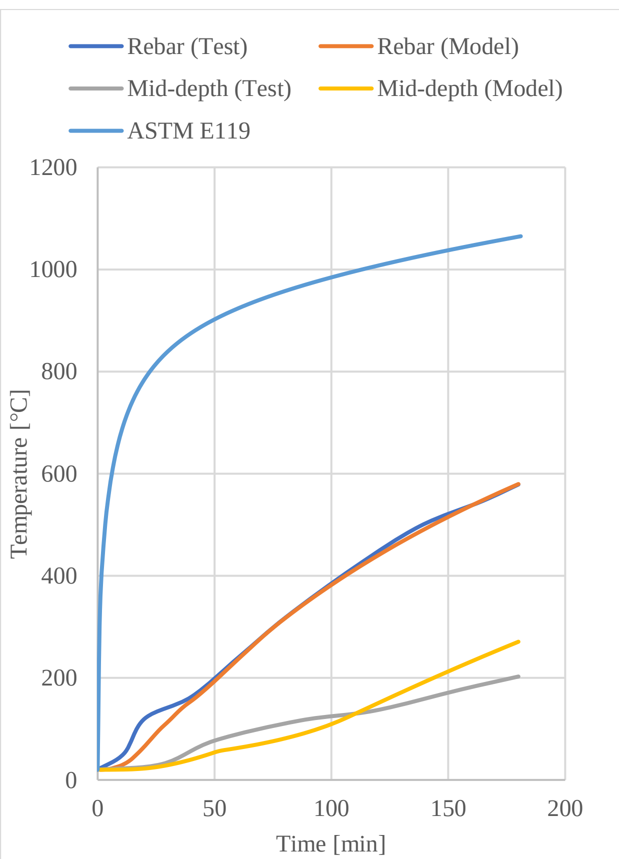


Fig. 5. Comparison of temperature evolution in the tested and calculated beam as a function of time

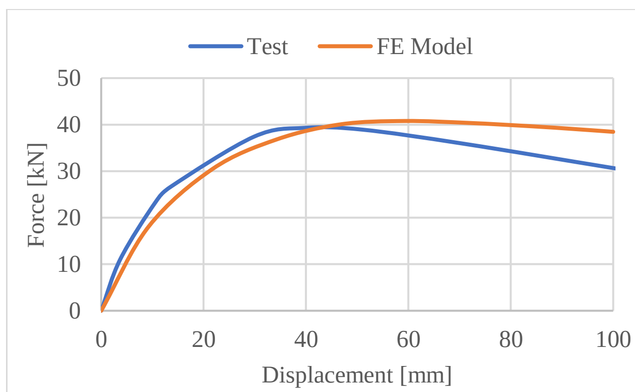


Fig. 6. Force-displacement curves obtained from the cyclic test and the numerical model.

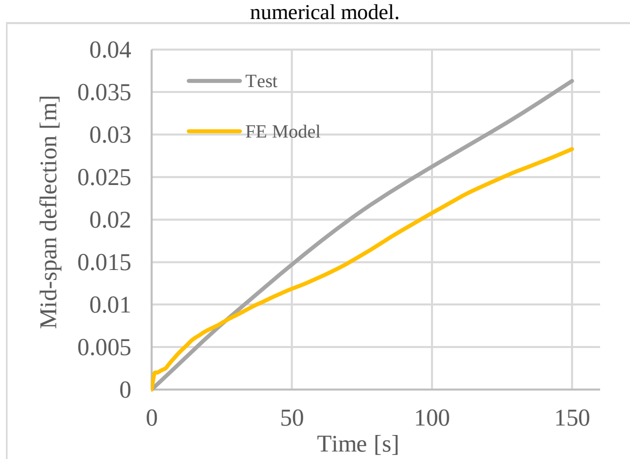


Fig. 7. Comparison of measured and predicted mid-span deflection.

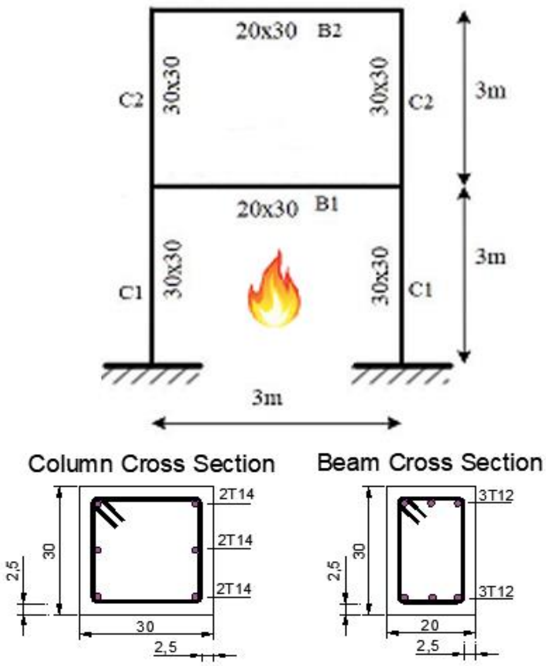


Fig. 8. Frame geometry.

a comparison of mid-span deflection as a function of time between test and numerical model. It can be seen that the mid-span deflection graphs obtained from the test as well as the generated model experience the same trend and show good agreement.

V. CASE STUDY

A two-storey reinforced concrete dwelling building is designed as per Algerian design codes (CBA, 1993; RPA99v2003, 2003). The edifice is assumed to be built in the city of Chlef, Algeria which is known for its severe seismicity. The structure is made of 20 MPa normal strength concrete and yielding strength of 400 MPa for longitudinal and transversal steel reinforcement. The Poisson’s ratio is assumed to be 0.2 and 0.3 for concrete and reinforcing steel respectively. The concrete cover was adopted equals to 2.5 cm even if it is crucial for the prevention of the steel mechanical properties (Haouach et al., 2021).

The structure is dimensioned for a dead load of 5.95 kN/m² and a live load of 1.5 kN/m². A frame is chosen from this construction in order to perform its structural analysis under the post-earthquake fire action using the ISO 834 standard fire (1999) for 4 hours. Fig. 8 illustrates the geometry of the frame as well as the fire scenario. Only three sides of the structural elements were exposed to fire.

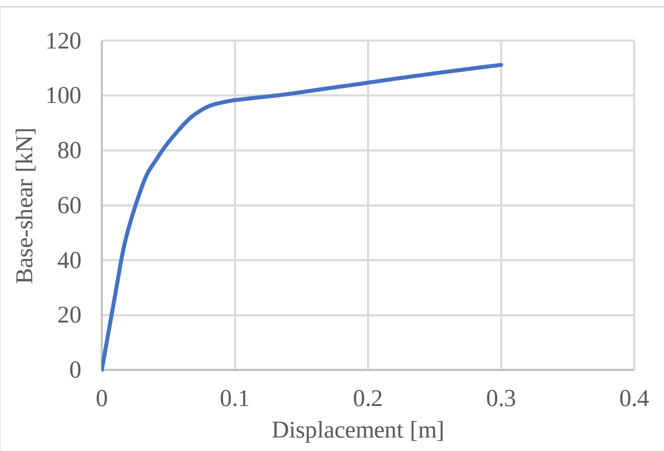


Fig. 9. Pushover curve of studied frame

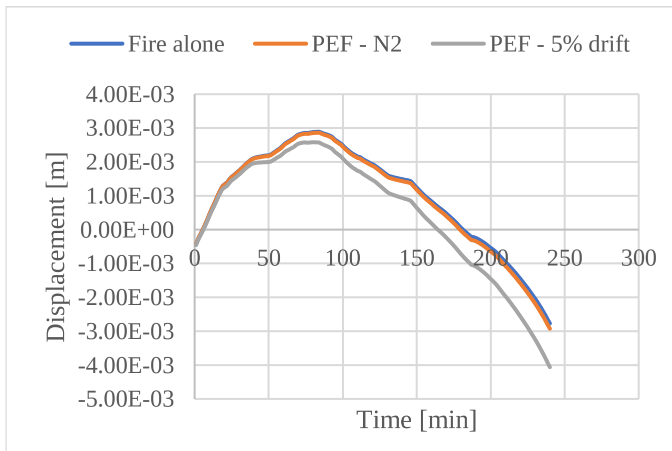


Fig. 11. Variation of the first-floor vertical displacement as function of time

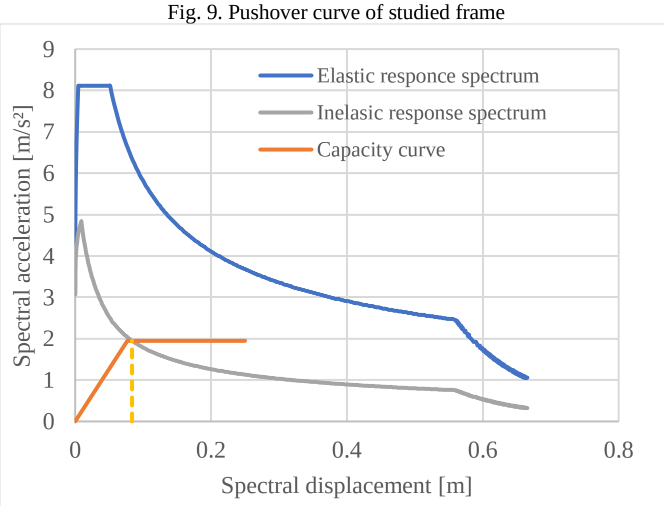


Fig. 10. Idealized capacity curve with response spectra

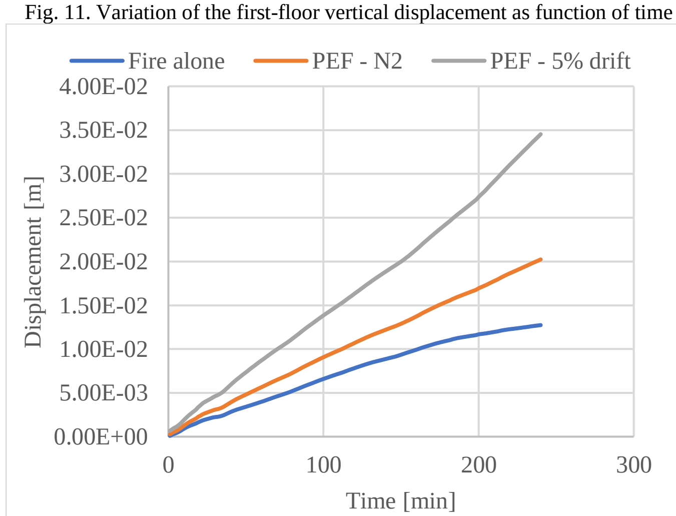


Fig. 12. Variation of the first-floor horizontal displacement as function of time

VI. Results

The response of the structure against the seismic load introduced by the response spectrum provided by RPA99v2003 (2003) was in the plastic range with a roof-displacement of 10 cm corresponding to a drift of 1.67% which is higher than the limit of 1% corresponding to the Immediate Occupancy performance level as per FEMA356 (2000).

The pushover curve as well as the performance point obtained using the N2 method are illustrated in the Fig. 9 and Fig. 10 respectively.

The post-earthquake fire behaviour of the reinforced concrete structure is presented in terms of displacement either vertical or horizontal as function of time. The frame is also subjected to fire alone. Fig. 11 and 12 present respectively the variation of vertical and horizontal displacements of the first floor as a function of time.

The vertical displacement in all cases experiences an upward trend up to around 2.90 mm for the fire alone and in the case of the LS performance level to around 2.15 mm in the case of the performance level corresponding to 5% of story drift after around 90 minutes. After that, the vertical displacement decreases until it reaches a deflection of around -2.90 mm for the case of fire alone and the LS performance level after 240 minutes while it reaches -4 mm in the case of 5% of story drift. It is worth mentioning that in the case of fire alone and the case of post-earthquake fire for the LS performance level, the vertical displacement is almost the same.

The horizontal displacement of the first floor is increasing with time up to 1.27 cm in the case of fire alone, to 2.02 cm in the case of LS performance level, and to 3.45 cm in the case of 5% story drift.

It should be noticed that in all cases, no collapse was observed in the structure. Fig. 13 presents the deformed shape of the studied structure for the 3 cases. While beam mechanism is observed in the case of fire alone, sway mechanism is observed in the case of previously damaged structure.

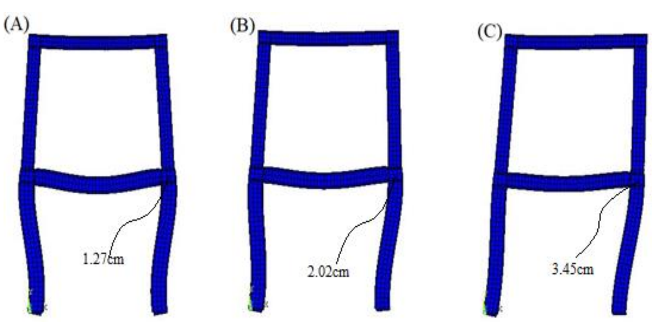


Fig. 13. Frame geometry.

VII. Conclusions

In this study, the post-earthquake fire capacity assessment of a reinforced concrete frame was performed including the material and geometric non-linearities. The reinforced concrete frame was previously dimensioned and designed based on Algerian design standards.

The vertical displacement due to fire action of a previously damaged structure can be affected, even if it was not observed for the LS performance level, it was observed in the case of 5% story drift. The horizontal displacement due to fire action is also affected by previous damage of the structure due to seismic loads.

Since no collapse was observed in the case of fire alone and the case of post-earthquake fire, it can be noticed that the reinforced concrete structures designed according to Algerian design codes are resistant to fire and post-earthquake fire actions. The oversizing provided by RPA99v2003 has induced this capacity of buildings in the cases of fire alone or post-earthquake fire.

Two deformed shapes were observed in the analysis, while a local mechanism was observed in the case of fire alone, a global mechanism was observed in the case of post-earthquake fire.

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