FIRE RESISTANCE OF STEEL FRAMES UNDER DIFFERENT FIRE-AFTER-EARTHQUAKE SCENARIOS BASED ON SCALED DESIGN ACCELEROGRAMS

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Abstract

The scope of this paper is to examine the performance of a steel framed structure under fire conditions after earthquake events. Taking into account this combined scenario, it is clear that the fire behavior of steel structures depends on the intensity of the ground motion and the study should be based on the performance based design philosophy. For this purpose a three-dimensional beam finite element model is developed using the non-linear analysis code MSC Marc. The combined scenario involves two different stages: during the first stage, the structure is subjected to the ground motion record while in the second stage the fire occurs. Several time-acceleration records are examined, each scaled to multiple levels of the Peak Ground Acceleration (PGA). The objective is to relate, the level of damage of the structural members occurring due to earthquake, to the fire-resistance of the structure.

Keywords: fire-after-earthquake, steel structures

INTRODUCTION

The design of structures according to the current codes is performed individually for the seismic and the thermal actions. Despite the significant progress of the worldwide scientific research on the earthquake response and on the fire-performance, the research concerning the combined scenario is rather limited. It is expected that the damage induced by earthquake can be present to both structural and non-structural members. The seismic damage to non-structural members can be related to different fire-after-earthquake (FAE) scenarios that should be considered at the fire design of the structure. Moreover, it is expected that the fire performance of structures will be different, depending on the level of damage caused to the structural members by the seismic loads.

Recently, some studies have been conducted, for the evaluation of the performance of structures under combined scenarios of FAE. For example, the post-earthquake fire resistance of steel moment resisting frames is evaluated in Zaharia et al (2009). Moreover, two different moment resisting steel frames are considered in Della Corte et al (2003) for the evaluation of the FAE resistance.

1 DESCRIPTION OF THE PROBLEM

The aim of the current study is to evaluate and quantify the behaviour of the four-storey steel frame, which is illustrated in Fig. 1, for the combined scenario of fire after earthquake. Taking into account the fact that the behaviour of the structure for the combined scenario is strongly dependent on both structural and non-structural damage induced due to the seismic action, it is evident that the problem should be approached through the performance based philosophy. For this purpose, the multi-level design approach of FEMA which considers four different seismic performance levels (Operational, Immediate Occupancy, Life safety and Collapse Prevention), is adopted. Following the principles of the performance based seismic design, the intensity of the earthquake is scaled up in order to represent more severe seismic actions. For this reason seven different accelerograms are selected, according to the criteria that are defined in Section 1.2. The earthquake records are scaled to multiple levels of the Peak Ground Acceleration (PGA). Each FAE scenario consists of two different stages. The first stage is defined from the time-acceleration record while in the second stage the fire is described using the ISO fire curve. It is noted that in the current study, the damage induced to the non-structural members due to earthquake (e.g. breakage of windows, broken sprinkler system etc.) is not taken into account.

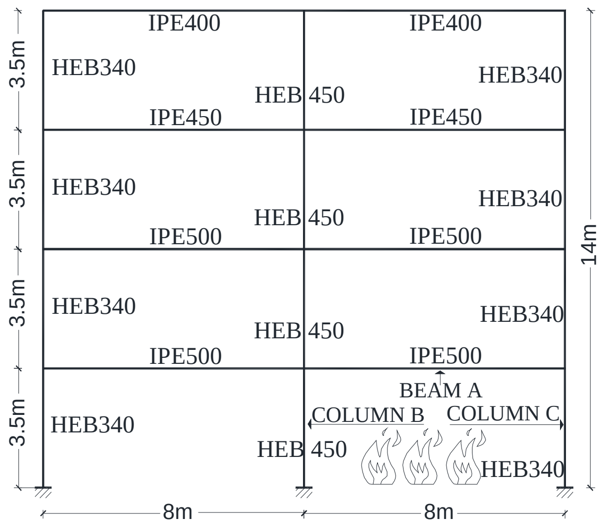


Fig. 1 The considered steel frame

The study here is focused on new buildings where the regulations of the current codes are applied. The first task is to design the steel frame for the ultimate limit state (ULS) combination of actions for the gravity loading. Regarding the design for the seismic actions, the principles of EN 1998-1-1 (2004) are followed and the lateral force method of analysis is used. Specifically, Type 1 elastic response spectrum is considered with a_g = 0.36 g and soil type D (S=1.35). The behaviour factor q was taken equal to 4.

The modelling of the seismic and the fire hazard follows. The seismic action is modelled through different time-history acceleration records.To this end, seven time-history acceleration records are selected from the European strong motion Database. These records are scaled to match the design spectrum using Rexel v3.5 (Ambraseys et al, 2002) following the guidelines of par. 3.2.3.1.2 of EN 1998-1-1 (2004).The considered records are summarized in the headings of Tab. 1. The fire action is represented through the ISO fire curve and it is supposed that the fire breaks out in the first level of the frame structure as it is illustrated in Fig. 1. In order to simplify the problem, it is assumed that the temperature inside the fire compartment is uniform. The temperature-time curves are calculated according to the guidelines of EN 1993-1-2 (2003), depending on the cross-section characteristics of the structural members. Finally, the FAE scenarios are defined. Regarding the seismic action, the time-history acceleration records, which were already modified as explained earlier, are further scaled to three levels of the Peak Ground Acceleration (PGA) using the scale factors 1, 1.5 and 2. It is noted that the reference fire scenario corresponds to the case where the structure is not damaged due to earthquake.

2 NUMERICAL SIMULATION

The numerical analysis is carried out using the nonlinear finite element code MSC-Marc (2011). The model for the simulation of the behaviour of the steel frame is developed using element 98 of the library of MSC Marc (2011). This is a straight beam in space which includes transverse shear effects. The cross-section of the finite element used for the numerical modelling, is a user-defined solid numerically integrated one. Four different branches are defined for the sections of the structural members: the upper flange, the web (which is divided into two parts for more accurate results) and the lower flange branch. Depending on the order of the numerical integration that is selected for every branch of the cross-section, the stress strain law is integrated through solid sections using a Newton-Cotes rule. The output results are exported to different layers, corresponding to the position of the integration points. The results are exported to different layers corresponding to positions of integration points.

The yield stress of the structural steel is equal to 275 MPa at room temperature. All the material properties are supposed to be temperature dependent according to EN 1993-1-2 (2003). It is underlined that the strain hardening of the steel for temperature range 20 °C – 400 °C is neglected in order to simplify the problem.

The problem is solved through dynamic transient analysis with direct integration of the equations of motion and the Newmark-Beta operator is used.

The numerical analysis is divided into two different parts. In the first analysis the steel frame is subjected to the seismic action while in the second one the analysis restarts and the deformed structure is exposed to the standard fire ISO curve for 60 minutes.

3 RESULTS OF THE NUMERICAL ANALYSIS

The results of the time history analysis for the earthquake loading indicate that the steel frame fulfils the demands of the capacity design rules and the plastic hinges are formed at the beam ends and to the columns bases, as it is expected. The study is focused on the structural members which are exposed to the fire in the next stage of the analysis (beam A, column B and column C), as it is illustrated in Fig. 1. At the starting point of the fire analysis plastic hinges have already been formed at the ends of beam A. The fire-after earthquake analysis indicates that the failure occurs when one more plastic hinge is formed at the mid-span of the beam. It is evident that this is a local type failure. In this study it is considered that the global failure of the steel frame will follow immediately after the local failure.

In order to study more systematically the behaviour of the steel frame under the combined scenario of fire after earthquake, the results of the analyses are classified according to the amplitude of the maximum equivalent plastic strain that is developed to the plastic hinge locations, at the end of the seismic analyses. This classification characterizes the level of damage induced in the structural members. Tab. 1 summarizes maximum equivalent plastic strain at both ends of Beam A for the various FAE scenarios. It is noted that Beam A appears the higher degrees of damage, compared with all the structural members of the steel frame.

It is observed that the results corresponding to the scale factors (S.F.) 1 and 1.5, are reasonable. On the contrary, when the scale factor 2 is implemented in the analysis, the values of the equivalent plastic strain become very high. The results that produce values of equivalent plastic strain higher than 0.2 are not considered acceptable and the corresponding FAE scenarios are not further studied.

Tab. 1 Level of damage for the FAE scenarios (end of the earthquake)

	Fire-after-earthquake scenarios													
	FAE 290xa		FAE 293ya		FAE 612xa		FAE 1726xa		FAE 1726ya		FAE 5850xa		FAE 6142ya	
S.F.	x=0	x=8m	x=0	x=8m	x=0	x=8m	x=0	x=8m	x=0	x=8m	x=0	x=8m	x=0	x=8m
1.0	0.020	0.036	0.044	0.036	0.032	0.028	0.046	0.046	0.034	0.022	0.016	0.022	0.080	0.090
1.5	0.100	0.138	0.124	0.134	0.200	0.200	0.158	0.162	0.086	0.104	0.070	0.058	0.220	0.222
2.0	0.308	0.346	0.262	0.294	0.528	0.564	0.278	0.266	0.154	0.192	0.158	0.152	0.406	0.442

The distributions of the equivalent plastic strain along the height of the cross-section of Beam A, for both the beam ends, are presented in Fig. 2. These distributions correspond to the starting point of the fire analysis. It is observed the plastic hinges are "fully" developed at both beam ends at the end of the earthquake event, and this holds for all the FAE scenarios.

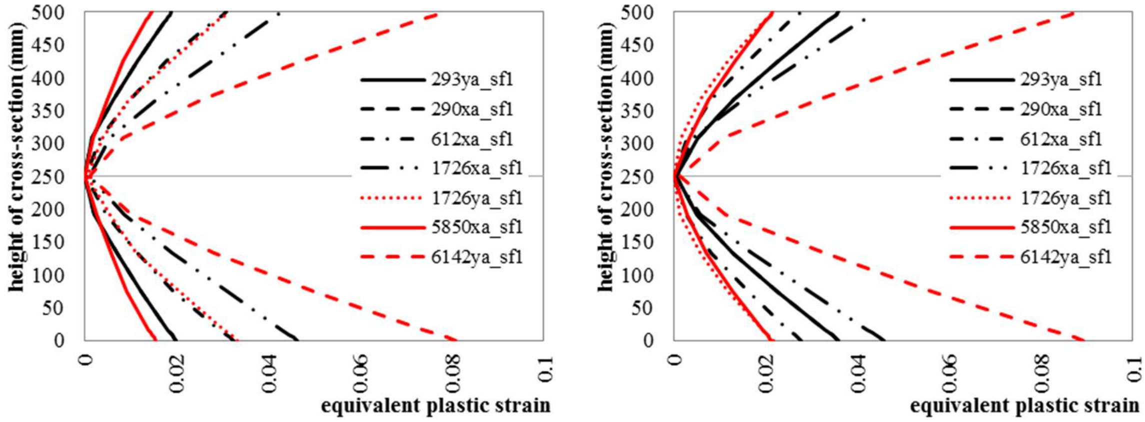


Fig. 2 Equivalent plastic strain distribution at x=0 and x=8m for Beam A (S.F. 1.5)

The evaluation of the outcomes of the analyses, taking into account the equivalent plastic strain is a qualitative approach, for thoroughly understanding the behaviour of the structure during the combined scenario, but is not an objective criterion for the determination of the fire resistance of the steel frame. In this study the criterion that is proposed in order to assess the fire-resistance of the structure, is based on the rotational capacity of the structural members at elevated temperatures.

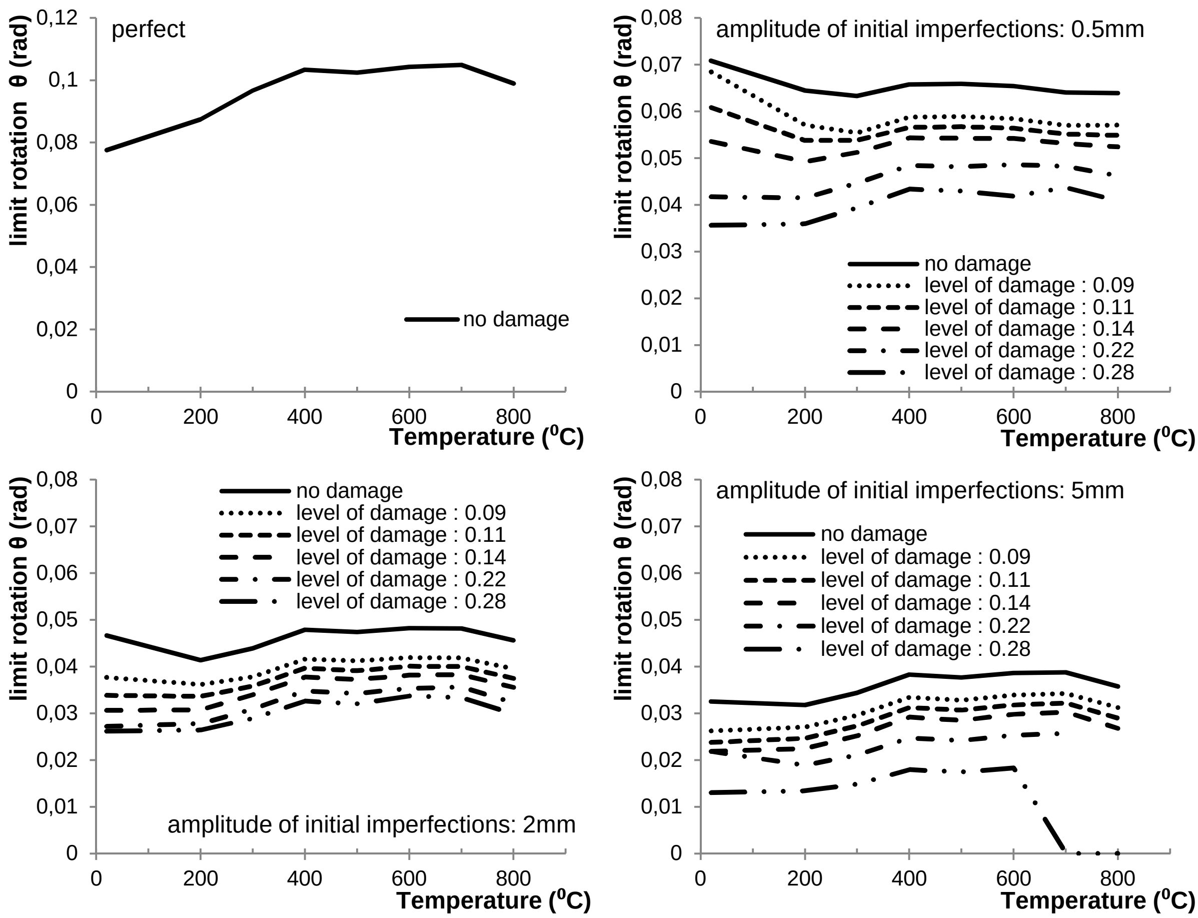


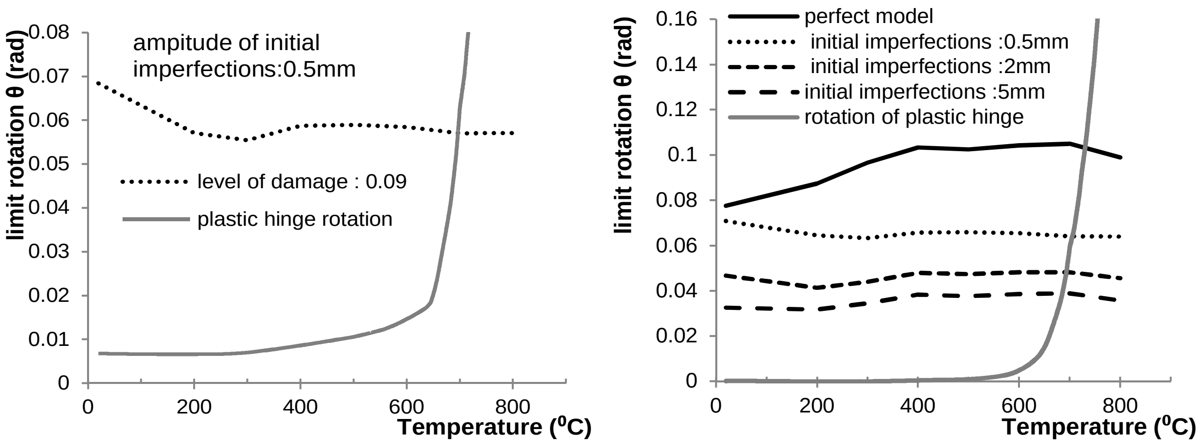
Fig. 3 Rotation limits

Advanced three-dimensional numerical models are developed for the determination of the rotational capacity of the steel beams under fire conditions (Pantousa D. et al, 2011). The three-dimensional models are based on shell finite elements and take into account the existing initial imperfections of the steel members. Parametric analyses are conducted with respect to the amplitude of the initial imperfections, in order to obtain moment – rotation curves for steel

IPE beams at elevated temperatures. The limit values for the rotation of the beam at the plastic hinges positions (beam ends) are represented in Fig. 3, for models that do not take into account initial imperfections (perfect models) and for models which include different levels of initial imperfections. In order to take into account the seismic damage, the limit values for rotation θ, are also obtained for beams which are pre-damaged due to cyclic loading. Each curve corresponds to different level of damage which is classified according to the level of the equivalent plastic strain that develops. The perfect models are examined only for the case where the beams are not damaged due to cyclic loading. Comparing the evolution of the rotation of beam A as the temperature increases with the limit values of Fig. 3, the fire-resistance of the steel frame is obtained.

In order to explain the procedure that was followed, the example of Fig. 4a) is presented. Two different curves are depicted. The first curve corresponds to the plastic hinge rotation (beam end at x=8 m) during the fire exposure in the case of FAE scenario 6142 ya, using S.F.=1, while the second curve represents the maximum acceptable rotation of the beam plastic hinge as a function of the temperature, for amplitude of initial imperfection equal to 0.5 mm. It is evident that this last curve corresponds to a specific level of damage induced to the end of the beam due to earthquake. In the specific case of Fig. 4a), as it is observed in Tab. 1, the level of damage for the beam end at the location x=8m, is equal to 0.09, thus the corresponding curve is selected in order to assess the fire-resistance time. The intersection of the two curves of Fig.4a indicates the temperature and the corresponding limiting rotation.

Taking into account the above, it is clear that the fire-resistance time of the steel frame depends on the level of initial imperfections. Tab. 2 summarizes the fire-resistance of the structure for the FAE scenarios, considering different amplitudes of initial imperfections. Concerning the reference scenario, the fire-resistance is obtained for different cases (Fig. 4b). First, the limit rotation curve that corresponds to the "perfect model" is used and the fire resistance is calculated equal to 1526 sec. Then, the limit rotation curves which are obtained through the analyses of the models that take into account initial imperfections, are used. Thus, the fire-resistance time is calculated equal to 1362 sec, 1526 sec and 1277 sec for amplitude of initial imperfections 0.5 mm, 2 mm and 5 mm respectively.



(a) illustrative example (b) reference scenario

Fig. 4 Calculation of the fire resistance

Finally, the reduction of the fire resistance time of the steel frame for the combined scenarios is obtained for two different cases (Tab. 2). In the first case (Case 1) the reduction is obtained taking into account the fire-resistance times of the reference scenario that correspond to different amplitudes of initial imperfection. In the second one (Case 2) the reduction is calculated with respect to the fire-resistance of the reference scenario that does not include the initial imperfections.

Tab.2 Fire resistance time (sec) and the corresponding reduction for the FAE scenarios

Fire-resistance in time domain (sec)						
	Amplitude of initial imperfections					
	0.5mm		2mm		5mm	
FAEscenario	scale1	scale 1.5	scale1	scale 1.5	scale1	scale 1.5
290xa	1325	1299	1274	1235	1234	1194
293ya	1331	1313	1281	1257	1244	1216
612xa	1333	1296	1285	1241	1248	1191
1726xa	1324	1293	1273	1230	1232	1186
1726ya	1328	1306	1278	1245	1238	1206
5850xa	1342	1320	1294	1267	1259	1227
6142ya	1321	1281	1269	1222	1232	1171
mean time	**1329**	**1301**	**1279**	**1242**	**1241**	**1199**
Reduction of the fire resistance time for the FAE scenarios						
Case 1	**2.41%**	**4.47%**	**2.28%**	**5.09%**	**2.82%**	**6.13%**
Case 2	**12.90%**	**14.74%**	**16.18%**	**18.58%**	**18.68%**	**21.45%**

4 CONCLUSIONS

In this paper the behaviour of a steel frame under the combined scenario of FAE is studied. The level of damage, induced due to earthquake is classified according to the values of the equivalent plastic strain in the plastic hinge locations. The fire-resistance of the frame, in time domain, is calculated using as criterion the rotational capacity of the structural members under fire conditions, which is obtained through appropriate 3D finite element models. The reduction of the fire resistance with respect to the undamaged steel frame increases as the level of damage induced by the seismic action becomes greater. The reduction of the fire-resistance strongly depends on the level of the initial imperfection that is taken into account.

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