FIRE-AFTER-EARTHQUAKE ANALYSIS OF STEEL FRAMES

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INTRODUCTION

Today the design of structures is performed, according to the current design codes, individually for the seismic and the thermal actions. The significant progress of the worldwide scientific research on the earthquake response and on the fire performance of structures has been consolidated to the current design rules for both cases respectively. On the contrary, the research concerning the combined effects of seismic and thermal actions on structures is rather limited. The current fire design codes are based on the assumption that at the beginning of the fire event the structure is in the elastic region of the material behaviour. This is not valid in the case of a fire outbreak after earthquake, since the structure is damaged due to the seismic loads.

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 (breakage of windows which allow free airflow, blocking of fire proof doors, sprinklers failure etc.) can be related to different fire-after-earthquake 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. The induced damage can be related to the plastic deformation of the structural members and to the partial loss of their fire protection. Various studies [1], [2], [3] have been conducted at the past years considering the effect of the damage induced in the fire protection materials on the load-bearing capacity of structural steel members. These studies reveal the reduction of the fire resistance due to the partial loss of the fire protection but they do not take into account the mechanical damage that may be caused by the seismic loading.

It is evident that the fire resistance of structures is strongly related to the expected level of damage caused by the design earthquake. Taking all these into account, it is clear that the design of structures for fire-after-earthquake scenarios should be approached through a performance based design framework, relating the desired fire performance with the expected damage level (to structural and non-structural members) due to the seismic actions.

Recently, some studies have been conducted, for the evaluation of the performance of structures to combined scenarios of fire-after-earthquake. For example, the post-earthquake fire resistance of steel moment resisting frames is evaluated in [4]. Moreover, two different moment resisting steel frames are considered in [5] for the evaluation of the fire-after-earthquake resistance.

The aim of the present study is to evaluate the post-earthquake fire performance of steel frames. In contrast with similar studies, the here attention is focused on the behaviour of a single beam element in which it is assumed that damage has been induced at the ends due to the action of seismic forces. The estimation of the fire resistance is based on a detailed three-dimensional numerical model which takes into account the inevitable geometrical imperfections of steel members as well as the unfavourable lateral-torsional buckling mode. As a result, interesting tables are produced comparing the fire resistance in the time domain of damaged and undamaged members.

1 DESCRIPTION OF THE PROBLEM

The study here is focused on new buildings where the capacity design rules of Eurocode 8[6] have been applied. The plastic hinges are expected to be formed at the end of the beams and at the bases of the columns. Taking into account the latter, the fire performance of the damaged beams is studied in fire-after-earthquake scenarios. In particular, in this study a numerical model is used to assist the evaluation of the fire performance of both laterally restrained and unrestrained steel beams, considering different fire-after-earthquake scenarios, leading actually to different levels of damage at the end of the beams. Various numerical analyses are conducted in order to quantify the reduction of the fire resistance, in the time domain, due to the damages that are caused by the seismic loading. The analyses take into account the geometric initial imperfections of the steel beams and the non-linear stress-strain relationship of structural steel in elevated temperatures. Parametric analyses are conducted considering various amplitudes of initial imperfections and different loading levels, in order to study the effect of these parameters to the fire behaviour of the damaged steel beams.

1.1 Design for gravity loading

The first task is the design of the steel frame according to the current codes. For this purpose the seven-storey steel frame of *Fig. 1a* is considered. The steel frame is designed for the ultimate limit state (ULS) combination of actions for the gravity loading. The ULS combination can be simplified to $1.35G + 1.50Q$. The dead load G that is used for the design is equal to 27kN/m while the live load Q taken as 30kN/m for all the storeys except the last level where the live load is considered to be 12kN/m.

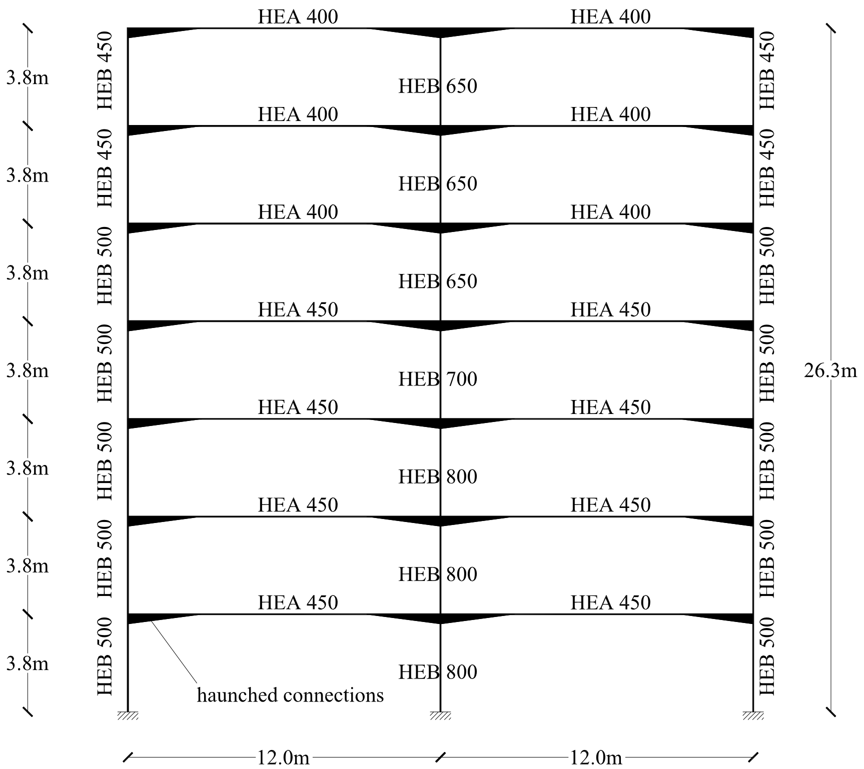

Fig. 1a. The considered frame

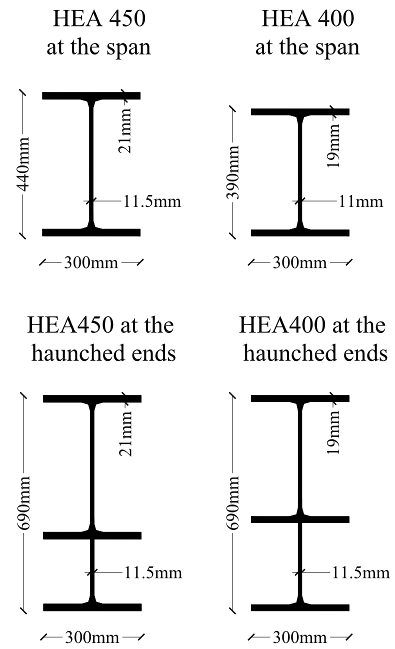

Fig. 1b. The beam cross sections

1.2 Seismic design

The design for the seismic actions is performed according to EN 1998-1-1 [6] and the lateral force method of analysis is followed. For the design, Type 1 elastic response spectrum is considered with a_g=0.16g and soil type A (S=1.00), according to Eurocode 8. The behaviour factor q was taken equal to 6. The steel frame is designed for the ultimate limit state (ULS) combination of actions for three different cases which correspond to different values of the combination coefficient $\psi_{2,i}$. For the three different ULS design combinations of actions, the value of $\psi_{2,i}$ is considered to be equal to 0.3, 0.5 and 0.8 respectively. According to the previous considerations for the seismic design, the calculated cross sections of the beams and columns are given in *Fig. 1a*. The beams are haunched for a length L_h =1.2m at both ends. The cross sections at the ends and in the middle are given in *Fig. 1a.*

The evaluation of the seismic behaviour of the steel frame structures is conducted through push-over analysis. The results of the analysis indicate that the formulation of the plastic hinge mechanism follows the provisions of the code for the local and global ductility of the structure. All the plastic hinges are formed at the beams and at the base of the frame structure. This indicates that the most vulnerable structural members of the structure when the fire follows the seismic loading are the damaged beams, since the columns (apart from their base) are expected to remain in the elastic region during the earthquake. Thus, the present study is focused only to the assessment of the fire performance of the steel beams.

1.3 Fire Design

The fire design is based on the loading combination for accidental design situations which is given in EN 1991-1-2 [7] and it can be simplified to $G+\psi_{2,1}Q$. Three different cases are considered again, using the three different cases for the quasi-permanent value of the combination factor $\psi_{2,1}$ ($\psi_{2,1}$=0.3,0.5,0.8). In this study the fire resistance of the steel beam results from the subsequent numerical analysis. It must be noticed that in the numerical analysis, the steel beams are submitted to the standard ISO fire curve and the temperature profile of the beam during the fire exposure is calculated according the provisions of EN 1993-1-2 [8].

2 CASE STUDIES – PARAMETRIC ANALYSES

This study is focused on the parametric analysis of a steel beam of the second level. Two case studies are considered. In the first case the beams are supposed to be laterally restrained (*Beam LR*) while in the second case the beams are laterally unrestrained (*Beam LU*). Two different fire-after-earthquake scenarios are considered for both case studies, considering different levels of damage caused to the steel beams by the earthquake. Additional parametric analyses are conducted considering the amplitude of the initial imperfections and the level of the imposed loading for both case studies. Three different loading combinations are considered, as indicated before, for different values of the combination factor $\psi_{2,1}$ ($\psi_{2,1}$ =0.3, 0.5, 0.8) corresponding to the utilization of the 40%, 50% and 58% of the yield stress of structural steel at room temperature. All the different parameters that are considered are summarized in *Table 1*.

Table 1. Parametric analyses

Fire-after-earthquake scenario	Beam LR (laterally restraint)						*Beam LU* (laterally unrestrained)		
	Loading level: % of the yield stress						Loading level: % of the yield stress		
	40		50		58		40	50	58
	Maximum amplitude of initial imperfections (mm)						Maximum amplitude of initial imperfections (mm)		
A	2	5	2	5	2	5	2	2	2
B	2	5	2	5	2	5	2	2	2

3 NUMERICAL ANALYSIS

The numerical analysis is carried out using the nonlinear finite element code MSC-MARC [9]. The models for the simulation of the fire behaviour of the steel beams are developed through four-node shell elements (*Fig. 2*). The yield stress of the structural steel is equal to 235MPa at room temperature. The numerical model takes into account the non-linear stress-strain relationship of the steel in elevated temperatures. Additionally, the yield stress, the proportional limit and the elastic modulus are supposed to be temperature dependent according to EN 1993-1-2 [8].

Both the ends of the beams are rotationally restrained, while the longitudinal displacement is not restrained. In the case study of *Beam LR* the out-of-plane displacements are restrained while in the case of *Beam LU* there is no lateral restraint at the nodes.

3.1 Analyses

The numerical analysis for the simulation of the behaviour of steel beams under seismic and thermal actions is based on the finite element method. It must be noticed that initial imperfections are incorporated in the geometry of the steel beam for a more realistic assessment of its behaviour.

There are many different ways to introduce initial geometric imperfections in the structural members. A simple way in the context of finite element analysis is to extract the buckling eigenmodes and introduce them as imperfections with fixed amplitude. More specifically, the normalized buckling mode is multiplied by a scale factor leading to certain maximum amplitude and the resulting displacements are added to the initial coordinates of the structural member. For the case study of *Beam LR* the eigenmodes that are used are related with the local buckling at the upper flange of the beam at the mid-span and at the haunched ends (see details in Fig. 2). The buckling eigenmode that is used as initial imperfection for the *Beam LU* is the one corresponding to the lateral torsional buckling.

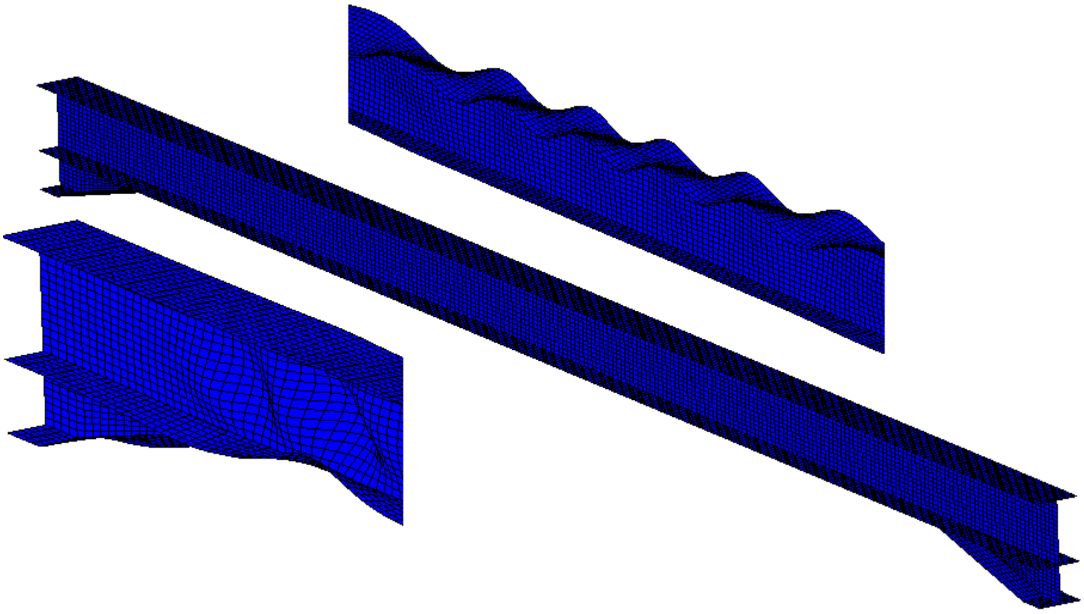

Fig. 2. The numerical model and the initial imperfection of local buckling

3.2 Fire analysis

First, numerical analyses are conducted in order to find the fire resistance of the steel beams for both case studies. The beams are exposed to the standard ISO fire curve for 60 minutes. It is noted that no thermal analysis is conducted. The temperature profile of the cross section of the beam is assumed to be known and is defined according to the procedure that is proposed in EN 1993-1-2[8] for the fire exposure. Moreover, it is noted that there is no thermal gradient in the cross section and that the temperature is supposed to be constant along the beams. At this stage, the numerical analysis is conducted for both perfect and imperfect steel beams.

3.2.1 Fire-after-earthquake analysis

In the sequel, the analysis for the fire-after-earthquake scenarios takes place for both case studies. The seismic actions are simulated through the storey drift displacements that are introduced during the earthquake excitation. More specifically the storey drift is connected to the rotation of the column nodes. The beams of the frame structure are supposed to rotate in the same direction at both ends, following the rotation of the columns due to the earthquake forces. The rotation which is introduced at the end of the beams due to the seismic actions and the deformed shape are demonstrated in *Fig. 3*. The total imposed rotation θ_{tot} has two components: the elastic (θ_{el}) and the plastic (θ_{pl}) one. After the earthquake excitation, the remaining part of the total imposed rotation at the beams, is not necessary equal to the plastic part of the rotation but is strongly related to the deformation response of the structure for the specific ground motion. In this study it is supposed that after the earthquake excitation, the plastic part of the total imposed rotation will remain i.e. the columns will no more be vertical.

Taking into account the previous, the numerical analysis of the steel beams for the seismic and thermal actions has three different stages. At the first stage the steel beams are submitted to the same rotation at both ends until the target rotation is reached. The unloading stage follows, where the reduction of the rotation is equal to the elastic part θ_{el}. At the third stage the steel beams are submitted to the standard fire ISO curve for 60 minutes. The temperature profile of the steel beam is supposed to be constant along the beam and at the cross section and is derived from the provisions of EN 1933-1-2 [8].

Two different fire-after-earthquake scenarios are defined connected with the level of the target rotation that is imposed at the ends of the beams. In the first seismic scenario (A) the total rotation is θ_{tot}=20mrad while in the second one (B), θ_{tot}=30mrad.

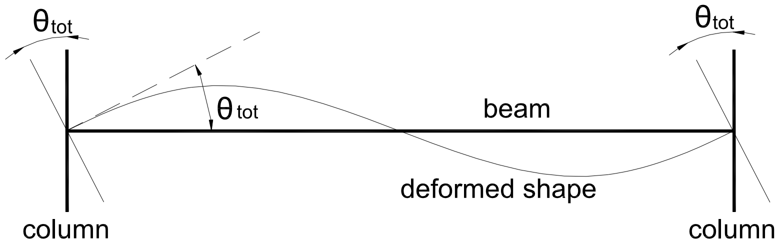

Fig. 3. The induced rotation at the beam ends

4 RESULTS OF THE NUMERICAL ANALYSIS

The results of the analysis regarding the fire resistance in the time domain for the first case study (*Beam LR*), are presented in *Table 2*. For the laterally restrained beam, the reduction of the fire resistance time in the case of fire-after-earthquake with respect to the reference case which is the fire situation, is rather limited. As it is expected, the reduction becomes greater as the amplitude of the initial imperfections increases, but the difference is not considerable. Also it is noticed that the fire resistance is reduced as the level of the applied loading (i.e. the coefficient $\psi_{2,i}$) increases. The reduction in the case of the fire-after-earthquake is similar for all loading levels.

Table 2. Parametric analyses results for the laterally restrained beam (*Beam LR*)

Analysis	Loading level: % of the yield stress					
	40		50		58	
	Maximum amplitude of initial imperfections					
	2mm	5mm	2mm	5mm	2mm	5mm
	Fire resistance in time domain (minutes)					
Fire	24.5	24.5	23.5	23	21.5	21.5
Scenario A	24	24	22.5	22	21	20.5
Scenario B	23	22.5	21.5	21	20	20

The reduction of the fire resistance in the case of fire-after-earthquake can be attributed to the available rotational capacity of the beams in elevated temperatures. Experimental and numerical studies (see e.g. [10]) indicate the considerable reduction of the rotational capacity of steel beams in elevated temperatures and this is due to the fact that the local buckling takes place at an earlier stage compared with the case of the room temperature. Moreover, in the case of fire-after-earthquake the rotational capacity is even more reduced because during the phase of the seismic loading, some parts of the members have already developed local buckling.

The results of the analysis regarding the fire resistance in the time domain for the laterally unrestrained beam (*Beam LU*) are presented in *Table 3.* The reduction of the fire resistance time in the case of fire-after-earthquake with respect to the reference case which is the fire situation, is remarkable. The reduction in the case of the fire-after-earthquake is increasing as the loading level becomes greater. The reduction of fire resistance ranges between 31% and 39% (for percentages of utilization of the yield stress of the structural steel at room temperature 40% and 58% respectively).

Table 3. Parametric analyses results for the laterally unrestrained beam (*Beam LU*)

Analysis	Loading level: % of the yield stress		
	40	55	58
	Maximum amplitude of initial imperfections		
	2mm	2mm	2mm
	Fire resistance in time domain (minutes)		
Fire	25.5	24.5	23
Scenario A	18.75	17	13
Scenario B	17.5	17	14

5 CONCLUSIONS

This paper presents a numerical model for the evaluation of the fire performance of steel frames for two different fire-after-earthquake scenarios. The results of the numerical analyses indicate considerable reduction (with respect to the reference case which is the fire situation) of the fire resistance time for laterally unrestrained beams in both the examined scenarios. However, further research should be conducted in order to obtain solid conclusions. More specifically, the effect of various parameters should be examined such as the type of the cross section of the beams, the restraint conditions, the type of the thermal loading (e.g. parametric fires), etc.

6 ACKNOWLEDGMENTS

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