FIRE RESISTANCE OF A STEEL STRUCTURE UNDER DIFFERENT FIRE-AFTER-EARTHQUAKE SCENARIOS CONSIDERING BOTH STRUCTURAL AND NON-STRUCTURAL DAMAGE

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1. SUMMARY

This paper addresses numerically the behavior of steel structures under Fire after Earthquake loading. The study is focused on a four-storey library building and takes into account the damage that is induced due to earthquake to both structural and non-structural members. The basic objective is the assessment of both the fire behavior and the fire-resistance of the structure in the case where the structure is damaged due to earthquake. Initially, the temperature distribution in the fire compartment is determined, considering different levels of non-structural earthquake induced damage, using the principles of Computational Fluid Dynamics. The various fire-scenarios result to different time-temperature histories developing on the structural members and this affects the fire resistance of the building. In order to study the behavior of the structure for the various temperatures histories, a 3D 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 fire resistance of the structure is determined using limits for the rotation of the structural components that are subjected to fire. These limits are dependent on the temperature level as well as on the level of the structural damage at the end of the earthquake. In this paper a review of the steps and the numerical techniques that are followed in order to address the aforementioned issues are presented, whereas the detailed study can be reached in [1].

2. INTRODUCTION

The main problem addressed in this study is the assessment of the behaviour of steel frame structures at elevated temperatures, considering that the starting point is a state of permanent damage caused by a prior seismic event, using numerical methods. It is noted that the seismic damage to both structural and non-structural members is taken into account. The study focuses on the determination of the fire-resistance (in time domain) of structures during post-earthquake fire event. In this way the fire-resistance calculated depends on the damage caused by the earthquake.The study is focused on a steel-frame building that is used as a library. However, the developed methodology can be applied to any structural typology. Initially, the basic steps of the well-known seismic and gravity loading design are followed. The idea is to study the fire-performance of the structural system that is damaged due to seismic actions that are scaled-up, with respect to the earthquake intensity, in order to represent more severe earthquake with lower probability of occurrence. Moreover, it is assumed that the building can suffer different levels of damage (induced to both structural and non-structural components) depending on the earthquake probability of existence. To this end, first the verification of the performance of the structure is conducted through the comparison of the available capacity with the required one [2]. The objective is to study the behaviour of the structure for different Post-Earthquake Fire (PEF) scenarios that are related to scaled seismic actions. *Here, the term Post-Earthquake Fire Scenario is related only to the non-structural damage*. Different PEF scenarios are generated, taking into account various levels of non-structural damage, resulting to different boundary conditions for the thermal problem. As a result, different time-temperature distributions in the fire-compartment are obtained.
Another goal is to evaluate the reduction of the fire-resistance of the structure due to the earthquake induced damage. The reduction is referred with respect to the case where the structure is not damaged. In the latter case, the fire resistance is indicated as the *reference* one in this study. The comparison of the results concerning the Fire after Earthquake (FAE) scenarios with the *reference* case indicates the reduction of the fire resistance due to the induced damage. It is noted that in this study the term FAE is related to both structural and non-structural damage and thus is different from the term PFE. The FAE scenarios include scaled seismic actions (directly connected to structural damage) and different PEF scenarios (directly connected to non-structural damage).
This paper is actually a brief description of the doctoral dissertation of the first author [1], summarizing the methodology that is included in this study. Due to the great volume of the results, only the most important ones are presented here.

3. THE CASE STUDY – DEFINTITION OF FIRE-AFTER-EARTHQUAKE SCENARIOS

The building that is studied here houses a library and is presented in Fig. 1. The first step of the study is the definition of both PEF scenarios and *reference* scenarios. The PEF scenarios are used in order to study the effect of the non-structural damage to the fire evolution in the enclosure. In this study the PEF scenarios are defined through the non- structural damage that is induced due to earthquake. Specifically, two different issues are considered which are the functionality of the fire-sprinkler system and the breakage of the windows. The *reference* scenarios are defined to be the cases where the sprinkler-system is entirely functional and the one where none of the windows is broken due to earthquake. Finally, ten different PEF and two *reference* scenarios are generated.

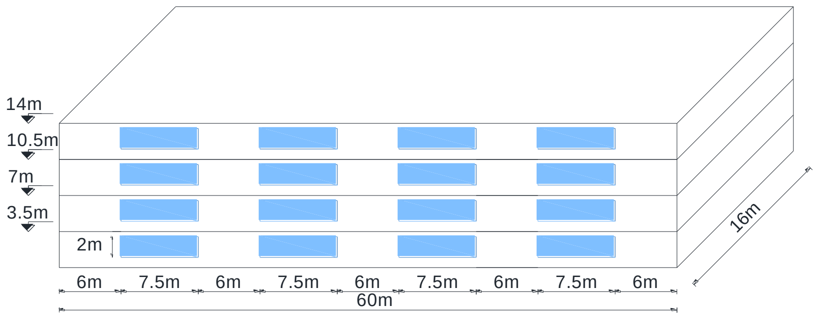

Fig. 1 The library building.

4. CFD MODELLING

The next issue is the development of the numerical model that is used for the numerical simulation of the natural PEF scenarios in the first floor of the library building. The simulation is conducted in Fire Dynamics Simulator (FDS) [3] using the principles of Computational Fluid Dynamics (CFD). At this point the study is focused on the effect of non-structural damage to the fire evolution in the enclosure. It is observed that as the area of the damaged (open) windows is increased, the maximum value of the total Heat Release Rate (HRR) is amplified and this indicates that the peak value that is recorded depends on the available oxygen in the fire enclosure. Moreover, it is concluded that the position of the damaged windows does not affect the evolution of the total HRR with time and that the active percentage of the fire-sprinkler system affects strongly the total HRR in the fire-compartment.

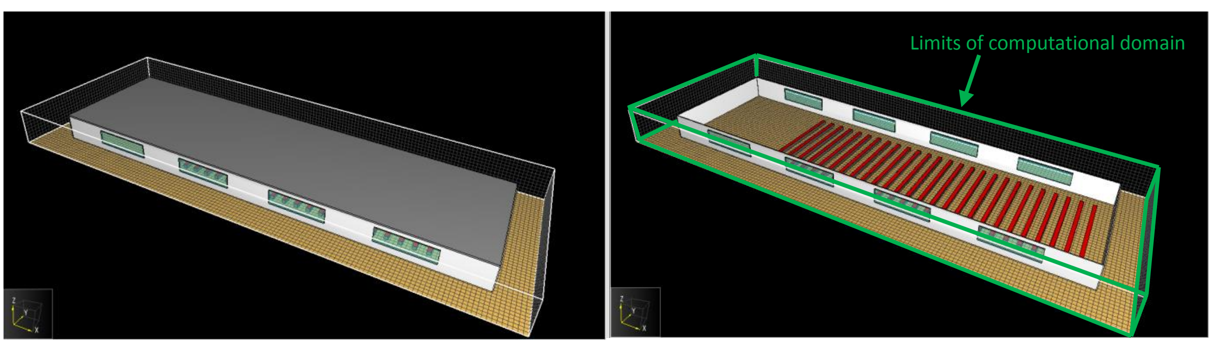

Fig. 2 The numerical model for the first level and the limits of the computational domain.

The results concerning the temperature "near" the structural members indicate that the temperature is highly non-uniform along the members and that it changes rapidly during the fire-exposure. An FSI (Fire-Structure Interface) model is proposed in order to overcome the difficulties which arise due to the strong spatial non-homogeneity and the intense time variation of the gas-temperature profile in the fire-compartment. The FSI model is developed in order to "condense" the results (concerning the spatial and temporal gas-temperature evolution) and the fire-compartment is divided into eleven virtual zones. The model is based on the idea of the "two-zone" models, as they are proposed in EN-1991-2 [4], and adopts the assumption that each virtual zone is further divided into two layers of uniform temperature:

the upper (hotter) layer and the lower (colder) layer. It is concluded that the "dual-layer" post-processing FSI model can describe effectively the temperature distribution in the fire-compartment. The advantage lies in the fact that the phenomenon is depicted in a simple way through three different variables which are the temperature of the upper and the lower layers and the layer height. In the sequel, the temperature profile of the structural members can be calculated for both the *reference* and PEF scenarios, using the gas-temperature that results from the dual-layer model, depending on the virtual zone and the layer in which they are located. The calculation of the temperature profile of the structural members is based on the simplified models that are proposed in EN 1991-1-2 [4].

5. SEISMIC – GRAVITY DESIGN / NON-LINEAR STRUCTURAL ANALYSIS FOR SEISMIC LOADING

The design of the structural system for the gravity and earthquake loadings follows. The numerical model and the techniques that are used for the simulation of the structural behavior under the FAE loading, is the next issue. The behavior of the structure for the design earthquake is verified through the results of pushover analysis which indicates that the system follows the capacity design rules for the formation of the plastic mechanism. Moreover, the behavior of the structural system is studied in detail through non-linear dynamic analyses, based on the direct integration of the equations of motion. The seismic action is modelled using seven different time-history accelerograms which are compatible with the design spectrum. The accelerograms are further scaled in order to represent more severe earthquakes using scale factors for the Peak Ground Acceleration (PGA). The results of the Incremental Dynamic Analysis (IDA) indicate that the maximum inter-storey drift angles are recorded at the second level of the structure. Only in the case where the most severe earthquake is used, the first level becomes the critical one. Finally, the response of the structural system is verified with respect to the limit values for the drifts that are recommended in FEMA 356 [5]. Taking into account both the permanent and the transient drifts, the response of the structure is found to be bound between the Intermediate Occupancy (I.O.) and the Life Safety (L.S.) performance levels.

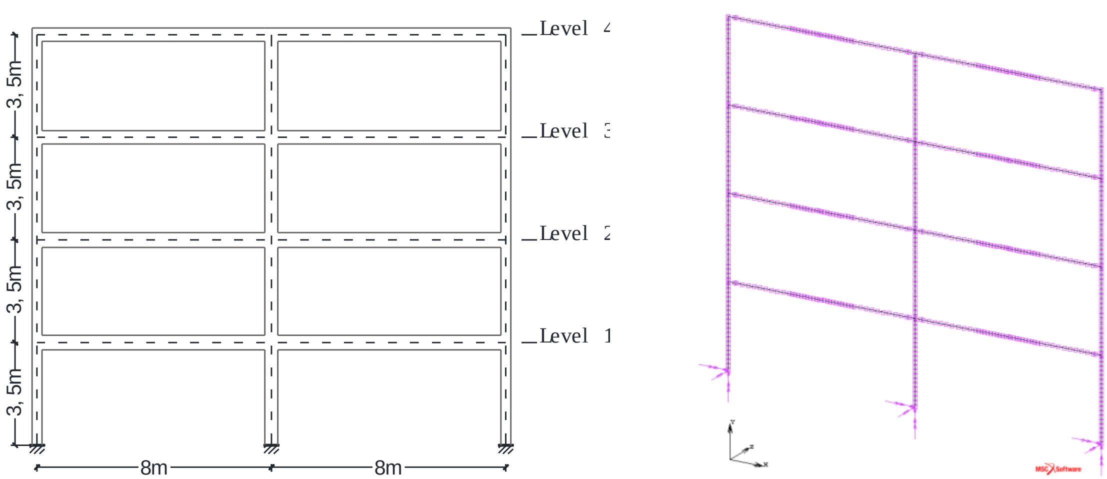

Fig. 3 Typical frame of the structural system and the corresponding numerical model.

6. FIRE DESIGN/NON-LINEAR STRUCTURAL ANALYSIS FOR FIRE LOADING

The next issue is the detailed study of the fire behavior of the structural system for both the ISO and the natural *reference* scenarios. At this point of the study, the structure is considered to be undamaged. The study of the temporal variation of the axial force and vertical deflection at the mid-span of the heated beam indicate clearly the different stages that take place during the fire exposure. Three different stages are identified which are the *restrained thermal expansion*, the *increase of curvature* and the *catenary action* stages. It is concluded that the structural system fails due to the formation of a local unstable mechanism at the heated beams.

7. NON-LINEAR ANALYSIS FOR FIRE-AFTER EARTHQUAKE LOADING

The study continues with the presentation of the numerical simulation of the behavior of the structural system under the FAE loading. The first issue is the definition of the FAE scenarios. The scenarios are divided into two main categories, depending on the representation of the fire action: the ISO-FAE scenarios and the Natural-FAE scenarios. Totally, 28 different ISO-FAE scenarios are generated, depending on the accelerogram that is used and the scale factor of the earthquake intensity. The fire action is simulated through the ISO-fire curve in all the cases. The definition of the Natural-FAE scenarios is based on the idea that the structure can suffer different PEF scenarios as the earthquake becomes stronger. Specifically, it is considered that as the intensity of the seismic action is scaled-up, using the appropriate scale factors for the PGA, different levels of non-structural damage are expected and these are directly connected to the different PEF scenarios. Thus, each Natural-FAE scenario is defined by a scaled accelerogram and the expected PEF scenario. Totally, 49 different Natural-FAE scenarios are defined. The results of the numerical analyses indicate the failure mechanism of the structural system under the FAE scenarios. In all the cases that are studied, the same failure mechanism took place (local unstable mechanism at both beams that were exposed to fire) as the one that is observed in the *reference* fire scenarios. The results are presented in terms of mid-span deflection and rotation at the support locations of the heated beams. It is observed that the maximum recorded values of the compression forces are not influenced by the ‘level of damage’ that is induced during the earthquake but they are affected from the PEF scenario that is used. Moreover, the temporal evolution of the equivalent plastic strain fields and the Von Mises stresses are studied in detail in order to understand thoroughly the formulation of the unstable mechanism at the heated beams. In general, it is concluded that numerical analysis stops due to convergence failure which is a result of the unstable kinematic mechanism that develops. Finally, it is concluded that the fire resistance of the structural system cannot be easily defined and that it is demanding to define integrated criteria in order to calculate the fire-resistance of the structural system during fire exposure.

8. FAILURE CRITERIA FOR STRUCTURAL MEMBERS AT ELEVATED TEMPERATURES

Next, the study is focused on the definition of the failure criteria for the determination of the fire-resistance of the structure under fire and FAE loading. To this end, three-dimensional shell finite element models are proposed, for the simulation of the behaviour of I-beams at elevated temperatures. First, the models are validated against published experimental results. The ductility of the beams is obtained through three-point bending tests (monotonic loading), using the standard beam approach. In the sequel, parametric analyses are conducted, with respect to the amplitude of the initial imperfections, at elevated temperatures, mainly in order to define the ductility of the members, in terms of rotational capacity and rotational (failure) criteria. The criterion that is proposed in this study is determined through the value of the rotation that corresponds to the exhaustion of the available rotational capacity of the beam. The term *ultimate available rotation* is used to identify this rotation. It is concluded that, clearly, the criterion depends on the amplitude of the initial imperfections and that it is not strongly temperature dependent. On the other hand, it is observed that as the amplitude of initial imperfections increases, the *ultimate available rotation* is significantly decreased. The study is further extended to the evaluation of the rotational capacity of pre-damaged beams at elevated temperatures. Different cyclic loading patterns are introduced in order to induce a specified "level" of damage in the beam. Cyclic loading is used in order to simulate the damage that is induced at the ends of the beams that belong in the frame structure due to earthquake loading. In this case the ductility of the beams is obtained through virtual three-point bending tests (monotonic loading), which follow the cyclic loading stage. Parametric analyses were conducted with respect to the amplitude of the initial imperfections and to the different cyclic loading patterns, at elevated temperatures. The results indicated that as the temperature rises, the ultimate available rotation at first is reduced and in the sequel it is slightly increased. On the other hand, the 'level of damage' induced due to cyclic loading affects strongly the ultimate available rotation. Specifically, the aforementioned rotation is considerably reduced when the 'level of damage' is escalated and this becomes more important as the amplitude of the initial imperfections increases. Taking into account the previous, it is concluded that the failure criteria can be separated into two categories. In the first category the failure criteria are defined through monotonic loading and they can be used for the determination of the fire-resistance of the frame structure that is not damaged due to earthquake i.e. in the case of the *reference* fire scenarios. The criteria that depend on the 'level of damage' that is induced in the structural members during the cyclic loading belong in the second category. In this case, they are classified according to the 'level of damage' induced in the beam in order to be utilized for the evaluation of the fire-resistance of the damaged frame structures.

9. RESULTS - CONCLUSIONS

In the last part of the study, the fire-resistance of the structure for the *reference* and the FAE scenarios is determined, in time and temperature domain, taking into account the failure criteria that were previously defined. In the case of the *reference* (ISO) fire scenario, the fire resistance is calculated to be equal to 1563 sec, 1388 sec, 1323 sec and 1291 sec for the case where the imperfections are not considered and for amplitude of initial imperfections 0.5 mm, 2 mm and 5 mm, respectively. In the case of the natural fire *reference* scenario the structure did not fail. Regarding the ISO-FAE scenarios, the results indicate that the reduction of the fire-resistance time (with respect to the fire-resistance time of the ISO *reference* scenario that is obtained using failure criteria which do not take into account initial imperfections) is considerable for all the amplitudes of initial imperfection that are studied, and lies between 14% (for the design earthquake) and 24% when the most severe earthquake is considered. In order to be able to compare the results of the FAE scenarios with the corresponding values that arise using the ISO curve, an "equivalent time" should be used in order to express the fire-resistance of the structures, It is clear that in the case of the natural FAE-scenarios it is difficult to determine the fire-resistance in time domain. The time that arises from the numerical analyses, is *real* time and it does not express the fire-resistance of the structure. In order to utilize the results of the natural-FAE scenarios, the critical temperature that defines the failure of the structure is "translated" into the equivalent ISO-fire time. In this way, the results are almost identical with the case of the ISO FAE-scenarios. The upper bound of the reduction of the fire-resistance time of the damaged structure (with respect to the fire-resistance time of the ISO *reference* scenario that is obtained using failure criteria which do not take into account initial imperfections) is approximately 25%.

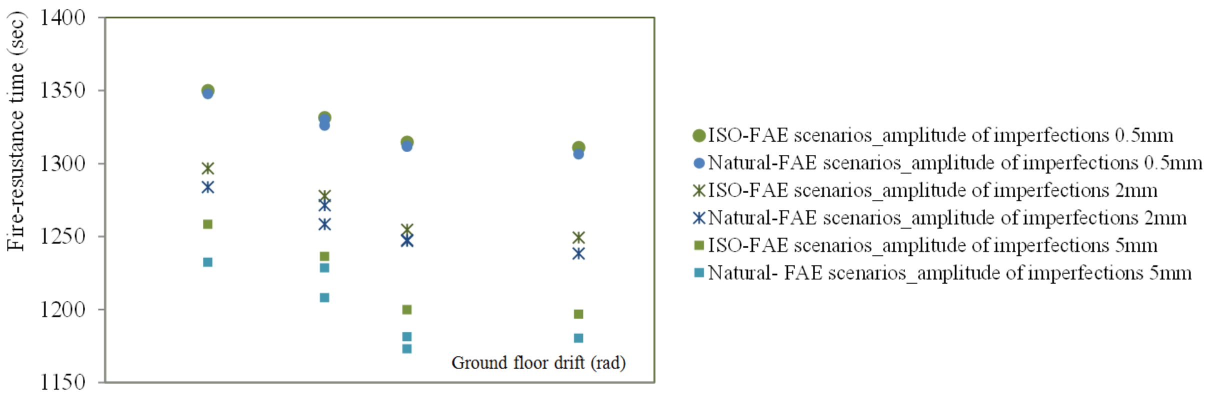

Fig. 4 Comparison of fire-resistance time for ISO and Natural FAE scenarios (in ISO-fire time).

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ΣΥΜΠΕΡΙΦΟΡΑ ΜΕΤΑΛΛΙΚΗΣ ΚΑΤΑΣΚΕΥΗΣ ΓΙΑ ΤΟ ΣΥΝΔΥΑΣΜΕΝΟ ΣΕΝΑΡΙΟ ΤΗΣ ΠΥΡΚΑΓΙΑΣ ΜΕΤΑ ΑΠΟ ΣΕΙΣΜΙΚΗ ΦΟΡΤΙΣΗ

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ΠΕΡΙΛΗΨΗ

Αντικείμενο της παρούσας εργασίας αποτελεί η διερεύνηση της συμπεριφοράς μεταλλικών πλαισιωτών κατασκευών σε υψηλές θερμοκρασίες, θεωρώντας ως σημείο έναρξης των υπολογισμών μια κατάσταση μονίμων βλαβών, που προέρχεται από πρότερη σεισμική καταπόνηση. Η μελέτη διεξάγεται μέσω αριθμητικών μεθόδων και επικεντρώνεται στον υπολογισμό του χρόνου πυραντοχής των κατασκευών στην περίπτωση που η πυρκαγιά εκδηλώνεται μετά από τη σεισμική φόρτιση. Η εργασία μελετά μια μεταλλική κατασκευή που χρησιμοποιείται ως βιβλιοθήκη. Αρχικά, η κατασκευή σχεδιάζεται για φορτία βαρύτητας και έναντι σεισμικής φόρτισης, σύμφωνα με τους ευρωπαϊκούς κανονισμούς. Στη συνέχεια μελετάται η συμπεριφορά του κτιρίου κατά τη διάρκεια της πυρκαγιάς, λαμβάνοντας υπόψη το γεγονός ότι το δομικό αλλά και το μη δομικό στοιχεία έχουν υποστεί διάφορα επίπεδα βλαβών λόγω του σεισμού, που κλιμακώνεται κατάλληλα ώστε να προσομοιώνονται σεισμοί με μικρότερη πιθανότητα εμφάνισης. Η μελέτη που διεξάγεται μέσω μεθόδων υπολογιστικής ρευστομηχανικής καταλήγει σε διάφορα σενάρια θερμικής καταπόνησης (θερμοκρασίες δομικών μελών), ανάλογα με το επίπεδο της βλάβης που έχει δημιουργηθεί από το σεισμό. Στη συνέχεια διεξάγεται θερμομηχανική ανάλυση, προσομοιώνοντας τη δυναμική συμπεριφορά της κατασκευής για τους διάφορους σεισμούς και την επακόλουθη θερμική καταπόνηση σύμφωνα με τα προαναφερθέντα σενάρια. Επίσης καθορίζονται όρια αποδεκτής βλάβης σε συνδυασμό σεισμικής και θερμικής καταπόνησης μέσω κατάλληλου προσομοιώματος πεπερασμένων στοιχείων που λαμβάνει υπόψη την ύπαρξη ατελειών. Τέλος, υπολογίζεται ο χρόνος πυραντοχής της κατασκευής, ο οποίος εξαρτάται από τον βαθμό βλάβης που εισάγεται στην κατασκευή λόγω της σεισμικής καταπόνησης.