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EFFECTS OF FIRE FOLLOWING EARTHQUAKE ON STEEL MOMENT RESISTING COVERED VERSUS UNCOVERED STRUCTURES

M. Ziaei[1], E. Peyghaleh[1] and M. R. Zolfaghari[2]

ABSTRACT

Fire Following Earthquake (FFE), as an indirect seismic hazard, threats many population centers and mega-cities around the world. There are examples of catastrophic fires following earthquakes such as the 1906 San Francisco earthquake, the 1994 Northridge and the 1995 Kobe earthquake, representing the importance of further studies and investigations on this phenomenon. According to current seismic design codes, ductile structures are designed to suffer damages to some extent during strong earthquakes in order to avoid total collapse and therefore, safeguard human lives. However, damaged steel structures exposed to fire following earthquakes show higher vulnerability to fire than those with no structural damages. Structural elements in ductile structures, once exposed to sever ground motions may lose parts of their fire protection coating which could escalate the penetration of heat flow in damaged structural elements. This in turn may result in significant reduction of structural fire resistance. In this paper the performance of a single-storey steel structure is modeled 3 dimensionally using a finite element computer software. All the structural elements in this frame are assumed to be fire coated. The structure is first analyzed based on gravitational loads. In the second step, the structure is analyzed against seismic peak ground displacement. This step allows identification of structural elements for which fire protection coatings are damaged. In the final step, thermal-mechanical analyses are performed in order to assess the effect of fire on uncoated structural elements. The analysis results are finally compared with those from structures with no coating protection.

Introduction

Experiences from past earthquakes showed that urban structures are vulnerable to most seismic hazards such as fault rupturing, liquefaction, landslide and above all strong ground motions. Excessive damages to structures and urban utilities can themselves pose secondary hazards such as flooding, fire, environmental pollution, etc. Fire Following Earthquake is the major threat for cities with dense concentration of timber buildings or buildings with no fire resistant protection. Besides cities with high pressured natural gas distribution network or air-drawn electricity distribution network are also vulnerable to fire following earthquake. During an earthquake simultaneous fire ignitions, multiple structural fires and high density of urban areas can generate wide spread fire. Fire Following Earthquake (FFE) consists of many simultaneous and catastrophic fires which could result in widespread economic damages and loss of life (Scawthorn et al, 2005, Chen et al, 2003). Example of such historical cases is the 1906 San Francisco Earthquake with destructive consequences (Scawthorn et al, 2005, Chen et al, 2003). More recent examples

[1] PhD student in structural engineering, Dept. of Civil Engineering, K.N.Toosi University of Technology, Tehran City, Iran.
[2] Assistant Professor, Dept. of Civil Engineering, K.N.Toosi University of Technology, Tehran City, Iran.

are the 1994 Northridge and the 1995 Kobe Earthquakes (NFPA, 1995). Like other fire pattern, the FFE process consists of three main phases; ignition, spread and suppression. Most of researches in recent years have focused on modeling fire spread and macro modeling of FFE and less attention is paid to study the effects of fire following earthquake on buildings from structural point of view. With regards to FFE risk modeling, statistical correlations made between strong ground motions and ignition frequencies are mostly used as a mean to simulate this phase of an FFE model. Mizuno et al (1978) developed the first IFE models based on statistical analyses of FFE damage data from earthquakes in Japan. Scawthorn (1986) followed this approach and expanded this concept to develop probabilistic post-earthquake fire ignition and spreading model. Using statistical data, Tokyo Fire Department (1997) developed some curves which show the ignition mean rates as functions of PGA. Zolfaghari et al (2009) made an attempt to analytically model intra-structure ignitions following earthquakes using a probabilistic approach. Most of these researches are conducted for being used in FFE risk modeling. One of the most important methods in this subject is the method proposed by Hamada (1951). The most advanced model is proposed by Consins et al (2002) in New Zealand that is based on cellular automates method and considers the building material type. Since these models are developed for risk and insurance purposes, the structural behavior in fire and especially in earthquake is not considered.

From structural points of view, structural behavior under seismic load is more important when structures enter the plastic range due to large earthquakes which could result in sever reduction of structures' load carrying capacity. For such a structure exposed to fire, the likelihood of structural instability rises while the structural survival time decreases. This is especially important in steel structures, since the mechanical characteristics of steel is rapidly deteriorate when temperature is increased and the steel members that have been entered into the plastic range during earthquake, lose their load carrying capacity more quickly.

Few researches have been carried out on the behavior of steel structures under seismic load as well as effect of fire following earthquake. Examples include the study conducted by Della Corte et al (2003). They tried to obtain some quantitative information about post-earthquake fire resistance of moment resisting steel frames. In their study, a simplified modeling of structural damage caused by earthquake is proposed and numerical analyses are performed with reference to a single-bay single-storey frame structure, allowing the main parameters affecting the problem to be identified. Another analytical study was conducted by Yassin et al (2008) on the performance of steel frames structures under fire following earthquake. Their analyses on a two-dimensional steel frame concluded that FFE performance of steel frames is affected by the lateral deformation caused by seismic ground motions. Alderighi et al. (2009) conducted a numerical investigation for the assessment of structural fire performance for buildings made of earthquake resistant composite steel-concrete frames. They tried to identify some key structural parameters that make it possible to correlate the predictable performance under seismic and fire loadings when these two are considered as independent events.

Fire-resisting coatings are frequently being used in steel structures in recent years. There are some experimental and analytical researches which study the effect of fire proofing cover on the performance of steel structures under fire. Kirby et al (1998) conducted some experimental tests to study the behavior of protected and unprotected steel members under natural fire. Extensive experimental tests carried out in Cardington (1993, 1998) to investigate the performance of protected vs. unprotected steel structure under fire. Wald et al. (2004) studied the temporal variation of temperature of steel columns under natural fire and provided time-temperature curves along different points of protected columns.

Although steel profiles covered with fire resistant cover have desirable behavior under common fires (not FFE) which could postpone structural collapse, these covers may not protect steel profiles subjected to seismic motions. In case of FFEs, steel members are entered in plastic ranges due to the earthquake load and plastic hinges are formed in beams and columns which could result in the development of cracks in fire resistant coating at locations of plastic hinges. Thus, steel members with cracked fire resisting coating are vulnerable to fire which in turn results in faster decline of mechanical characteristics. There seems to be a gap in researches on FFE from structural point of view and researches on the structural damage related to FFE have shown less attention to such behavior.

In this paper, an analytical approach, based on finite element model is used in order to study the performance of steel frames. To compare the results, a steel frame under common fire and post-earthquake fires as well as covered vs. uncovered conditions are considered; in total eight cases. For the frames under seismic load and subsequent fire, the term EFS is used. If the frame is just under fire loading, the term FS is used. The moment resisting steel frames are designed based on Eurocode 8 and are analyzed using a finite element computer software. In EFS frames, seismic loading is applied by a lateral displacement. Fire loading in the frames without cover is simulated by increasing temperature in the bottom beam flange. In the case of covered steel frames, the fire load is applied by increasing temperature in the location of plastic hinges which might take place in the beam for some cases and in the beam and the columns in other cases.

Seismic loading

Performance of structures under fire following earthquakes depends on several factors and large uncertainties are associated with prediction of structural mechanical behavior under such conditions. The main problem in such situation is reliable estimation of structural state following earthquakes. Such structural states represent the initial conditions for fire loading following earthquakes. The behavior of a specific structure under earthquake loading can be predicted using complicated numerical models. There are high degree of variation of estimated seismic damages to structures due to high uncertainties associated with both structural properties and earthquake ground motions. A structure under seismic load could be subjected to mechanical as well geometrical damages. Mechanical damages are the degradation of mechanical properties of those structural components engaged in the plastic range of deformation during earthquakes. Geometrical damages on the other hand, are the results of changes in structural geometry, due to plastic excursion during earthquakes, such as permanent displacements and rotations.

In this paper, the effects of both geometrical and mechanical damages are considered by applying earthquake loading in the form of specific lateral displacement. Although it is more appropriate to apply acceleration or displacement time-history records, a simplified method for simulating earthquake induced damage is to apply a specific lateral displacement, representing structural maximum displacement caused by earthquake record. In this way, geometrical damages are considered by the permanent deformations of the structure after unloading. Furthermore, mechanical damages are considered since the mechanical properties of material are changed in the locations where plastic areas are formed.

To calculate the maximum lateral displacement, the concept of maximum allowable displacement which is suggested in most valid seismic design codes is used herein. The maximum allowable displacements mentioned in these codes are estimated based on the design structural performance under desired hazard level. For example, in the Iranian 2800 earthquake code (1999) for ordinary structures with life-saving performance under 475 years ground motions, the maximum allowable displacement is equal to 5% of structure height. Most of this displacement (about 90%) is elastic displacement. The summation value of dead and live load is 5300 kg/m^2 and is applied as a pressure on beam top flange.

Fire loading

In order to study the effects of fire on steel structures, fire resistance capacity of different structural elements should be taken into account. There are several methods for applying fire load on a structure. In this paper, fire is modeled by applying increasing temperature on the structural elements. Bare members or members with cracked covers due to earthquake motions, are directly exposed to fire. Therefore, for such members, temperature is assumed to increase uniformly and monotonically, in accordance with the time-temperature curve suggested by the standard ISO-834 (1975) which is also provided by Eurocode 1 (Fig.1(a)). As it is well known, the ISO curve is based on a purely conventional fire action model, which does not represent any particular fire that could develop in real buildings. Many researches have been conducted on this subject and are still in progress.

In the cases where fire proofing cover is not used, the temperature is applied according to ISO-834 time-temperature curve as uniformly increasing temperature in columns, bottom flanges of beam and part of beam web that is placed out of concrete slab. In the cases where fire proofing cover is used, the temperature is applied according to ISO-834 time-temperature curve as uniformly increasing temperature where fireproofing covers are cracked at plastic hinges in columns, bottom beam flange and part of beam web that is placed out of concrete slab. In addition, for those areas with non-cracked fireproofing covers, the temperature is applied according to time-temperature curve provided by Wald et al. (2004) (Fig.1 (b)). The time-temperature curve is developed for covered columns as shown in the photo and area section in figure 2. In this example the cover material is vermiculite cement spray with thickness of about 18 mm. The typical locations of cracked fireproofing cover induced by earthquake loading are shown in figure 3(a), where the temperature loading is applied. The regions where the temperature loading is applied in uncovered frames is shown in figure 3(b) that is the bottom beam flange and part of beam web out of concrete slab.

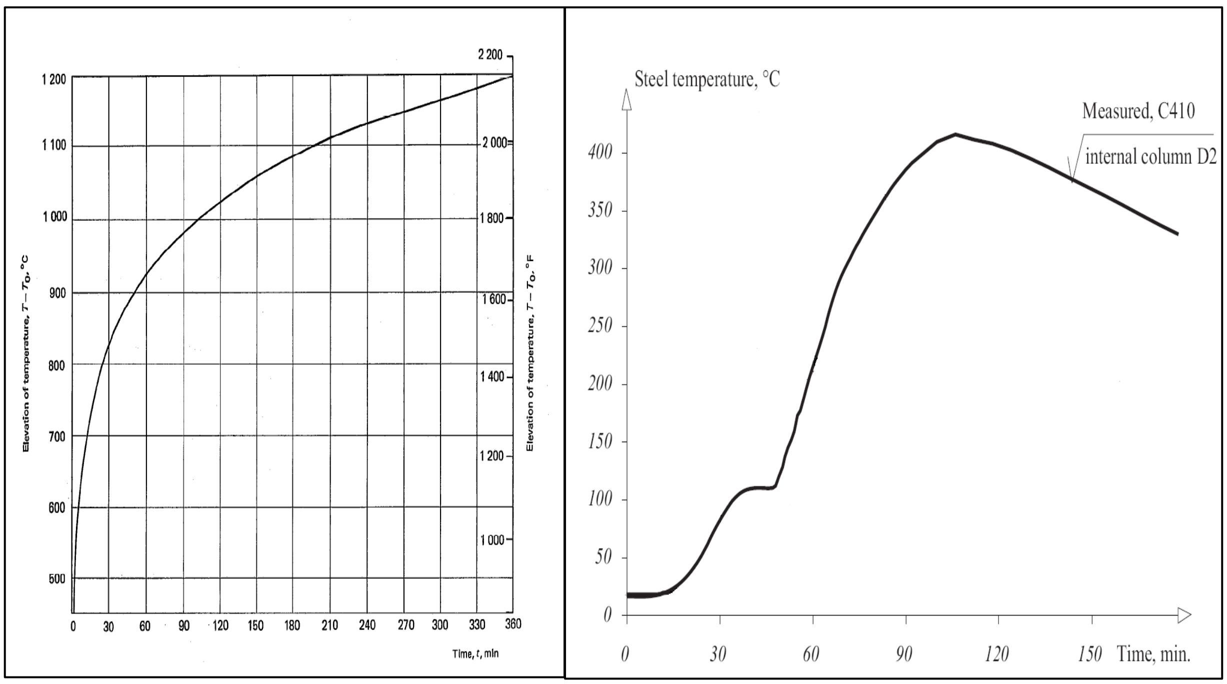

Figure 1. (a) ISO standard time-temperature curve (b) Wald et al. (2004) time-temperature curve

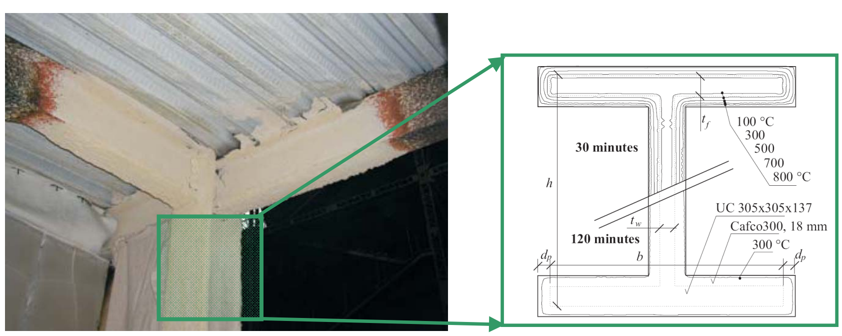

Figure 2. Photo and area section of covered steel column presented by Wald et al. (2004)

Table 1: member geometrical properties

Member	Profile	Steel	Length (cm)	M/M_p
Beam	IPEB400	ST37	600	0.1(End) 0.05 (middle span)
Column	IPB350	ST37	350	-

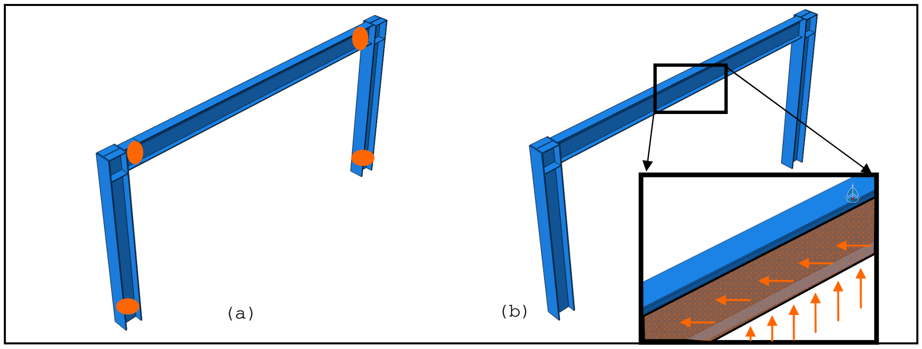

Figure 3. (a) Locations of cracked fire proofing cover where the temperature loading is applied in EFS frames. (b) Beam regions where the temperature loading is applied in uncovered frames.

Finite element modeling

In this paper, finite element models are used to study the performance of steel frames in eight cases. These cases are shown in Table2. For the FS frames, the analyses consist of two steps. Firstly, the gravitational load is applied on the top flange of the beam in a static general step. Then, the fire loading is applied as explained earlier and a coupled thermal-mechanical analysis is performed to simultaneously consider the effect of mechanical and temperature loadings. For the EFS frames, the analyses consist of three steps. Firstly, the gravitational load is applied on the top flange of the beam in a static general step. Secondly, the seismic load is applied in the form of a lateral displacement at the top of frame columns in a static general step. Finally, the fire loading is applied in the same way as explained for the FS frames. Since large displacements may occur during the analysis, large displacement option is activated in the analysis to account for large displacements.

Table 2: The model cases

Model name	Seismic load	Fire load	Uncover	Cover-column	Cover-Beam	Time-Temperature Curve in Beam	Time-Temperature Curve in column	Column hinge considered for temperature loading	Beam hinge considered for temperature loading
FS-C	-	√	-	√	√	Wald et al	Wald et al	-	-
FS-NC-Beam	-	√	-	√	-	ISO	Wald et al	-	-
FS-NC-Frame	-	√	√	-	-	ISO	ISO	-	-
EFS-C-Beam	√	√	-	√	√	Wald et al	Wald et al	-	ISO in Beam plastic hinge
EFS-C-Frame1*	√	√	-	√	√	Wald et al	Wald et al	ISO in Column plastic hinge	ISO in Beam plastic hinge
EFS-C-Frame2**	√	√	-	√	√	Wald et al	Wald et al	ISO in Column plastic hinge	ISO in Beam plastic hinge
EFS-NC-Beam	√	√	-	√	-	Wald et al	ISO	-	-
EFS-NC-Frame	√	√	√	-	-	ISO	ISO	-	-

* The columns are assumed to be interior ones and temperature is applied to whole part of column hinges.

** The columns are assumed to be exterior ones and temperature is applied just to column interior flange in plastic hinge areas.

Table 3 illustrates the variation of mechanical and thermal properties of steel material with temperature. 3D solid elements are used to model columns and beam profiles in for these frames. The beam-to-column connections are assumed to be rigid. The bases of the columns are restrained in all transitional and rotational directions. Since the concrete slab prevents the beam to have horizontal movements in direction perpendicular to the beam axis, the beam is restrained in out of plane horizontal movement (Figure 4(a)).

Table 3: Variation of mechanical and thermal properties of steel material with temperature

Temperature (‘C)	**Ultimate strain**	**Yield strain**	**Yield stress (kg/cm2)**	**Thermal expansion**	**Specific heat (J/Kg’C)**	**Conductivity coefficient (W/m‘C)**
0	0.15	0.0014	2750	0	380	54
200	0.15	0.018	2750	0.0025	500	47
400	0.15	0.02	2750	0.0055	650	41
500	0.15	0.021	2200	-	-	34
600	0.15	0.022	1300	0.0085	800	30
730	0.15	0.022	600	0.0115	850	28
800	0.15	0.022	306	0.0115	800	28
900	0.15	0.022	200	-	-	28
1000	0.15	0.022	100	0.014	700	54
1200	0.15	0.022	50	0.0175	700	47

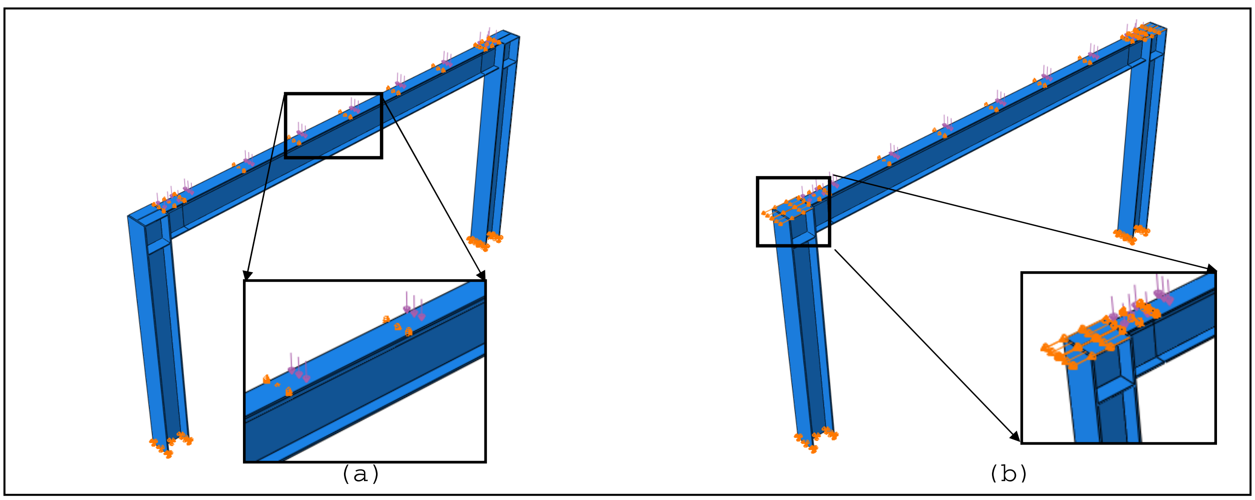

Figure 4. (a) First step: boundary conditions and gravitational load (b) Second step: boundary condition, gravitational load and seismic load.

RESULTS AND DISCUSSION

The analysis results for each given load case are shown in the figures 5 to13. The typical displacement in the vertical direction under gravitational load is shown in figure 5(a). The maximum displacement is occurred at mid span of the beam which is equal to 1.66 millimeters. Lateral displacement of EFS models subjected to seismic load is equal to 15.05cm. The frame conditions after earthquake loading with plastic strain is shown in figure 5(b). As it was anticipated, the plastic hinges are formed at the two ends of the beam and near the bases for two columns. As it was mentioned before and can be seen in this figure, the cracks in the fireproofing covers start to develop in these locations.

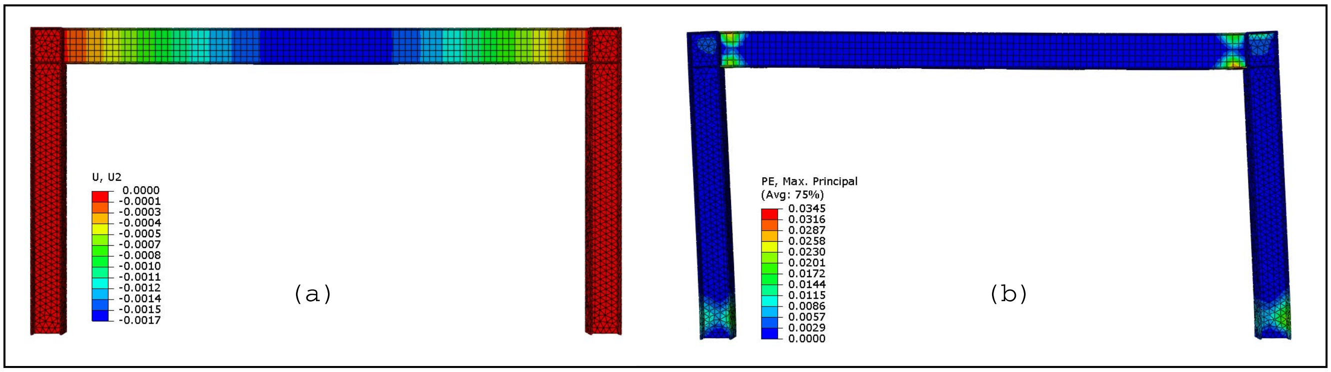

Figure 5.(a) typical displacement in vertical direction under gravitational load, (b) plastic strain after earthquake loading.

In all cases, temperature and vertical displacements are plotted on deformed shape of the models after the fire loading. Lateral and vertical displacements at specific time are presented in table 3. The allowable beam deflection is estimated based on the following equation (Iranian National Building Code):

$$\beta_{allowable} = \frac{\Delta}{L_{Beam}} = \frac{1}{240} \qquad (1)$$

Where $\beta_{allowable}$, Δ and L_{Beam} are maximum allowable beam deflection to beam length ratio, maximum beam deflection and beam length respectively. For the frames modeled in this paper, the allowable beam deflection is equal to 2.5 cm which is corresponding to the $\beta_{allowable}$. Table 4 shows the ratios between vertical displacements at beam mid span to the allowable beam deflections (γ). For frames with no fire proof cover (NC), the failure mode takes place earlier than those of covered frames and therefore, analyses results are presented for 7200 seconds after the application of fire loading. For EFS-NC-Frame, since the frame collapsed before reaching the 7200 seconds, the analysis results are shown only for 7000 seconds after the application of fire loading. However, for covered frames, the analyses results are shown for 14400 seconds after the application of fire loading. For NC models, the ratio of beam mid span deflection to allowable deflection is very high, especially for EFS-NC models which imply beam destruction. In contrast, as it can be observed, the rise in the temperature for FS-C model has negligible effect on deflection ratio and structure remains in allowable conditions. Nevertheless, development of plastic hinges at beam and columns due to earthquake loadings reduce the effectiveness of fire resisting cover which in turn results in higher deflection ratios. In such cases, the plastic hinges are formed at the two ends of the beam and thus, the whole beam moves downward (Figure 9-11).

Table 4: Analysis results for each model

Model name	Max Deflection Recording Tine(Sec)	Lateral displacement After Fire (cm)	Max deflection in beam (cm)	γ	Time to reach 2.5 cm deflection in beam mid span	Time to reach 5 cm deflection in beam mid span	Result Figure
FS-C	14400	-	0.166	0.0664	-	-	Figure 6
FS-NC-Beam	7200	-	70	28	2966	3016	Figure 7
FS-NC-Frame	7200	-	206	82.4	2467	3092	Figure 8
EFS-C-Beam	14400	13.4	8.7	3.48	9023	10623	Figure 9
EFS-C-Frame1	14400	18.9	15.4	6.16	9124	10324	Figure 10
EFS-C-Frame2	14400	14.7	11.0	4.4	9032	10432	Figure 11
EFS-NC- Beam	7200	19.2	70	28	2861	2981	Figure 12
EFS-NC-Frame	7000	36	224	89.6	2176	2676	Figure 13

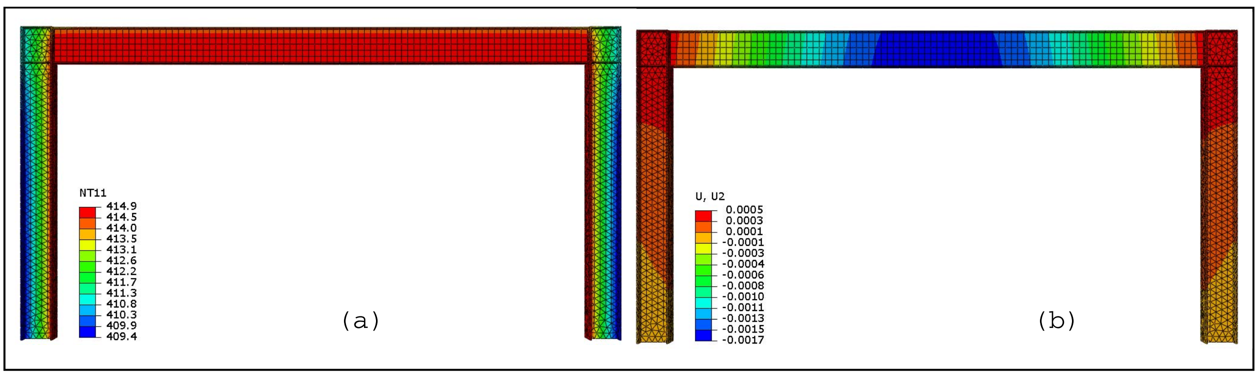

Figure 6. FS-C model: (a) Temperature distribution (b) Vertical displacement

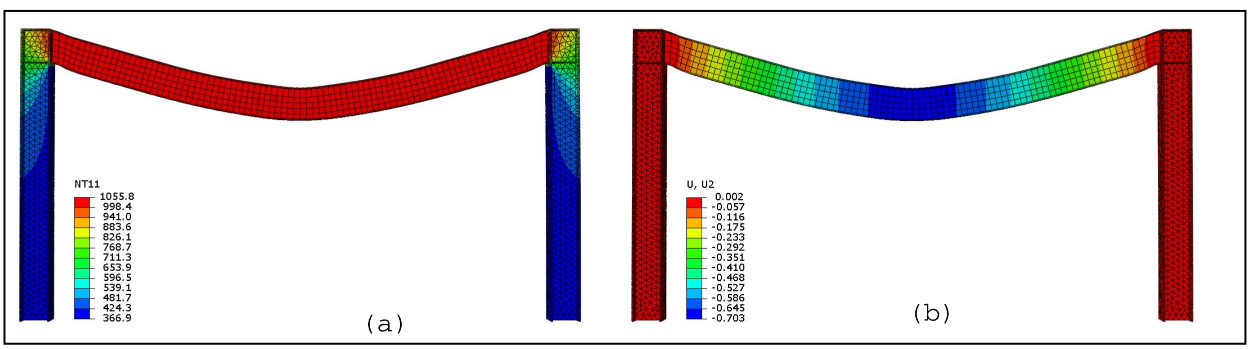

Figure 7. FS-NC-Beam model: (a) Temperature distribution (b) Vertical displacement

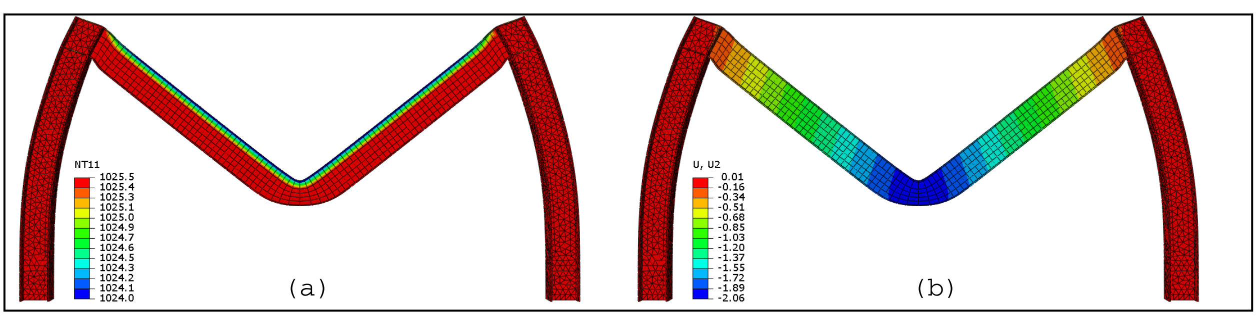

Figure 8. FS-NC-Frame model: (a) Temperature distribution (b) Vertical displacement

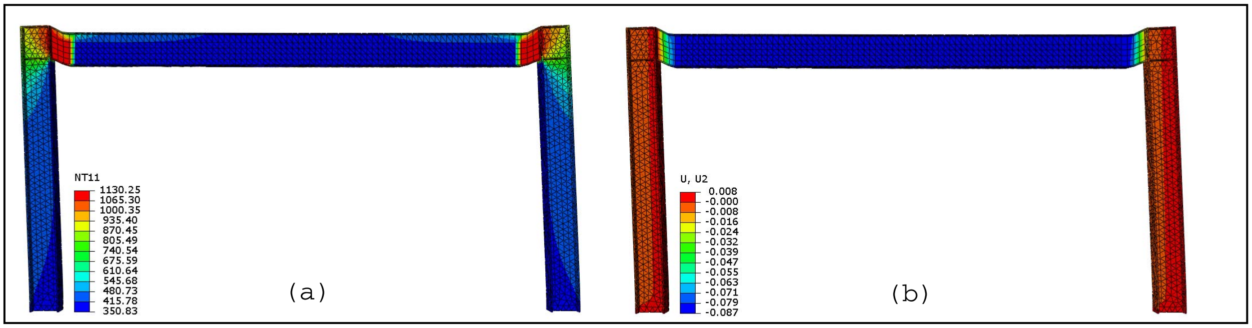

Figure 9. EFS-C-Beam model: (a) Temperature distribution (b) Vertical displacement

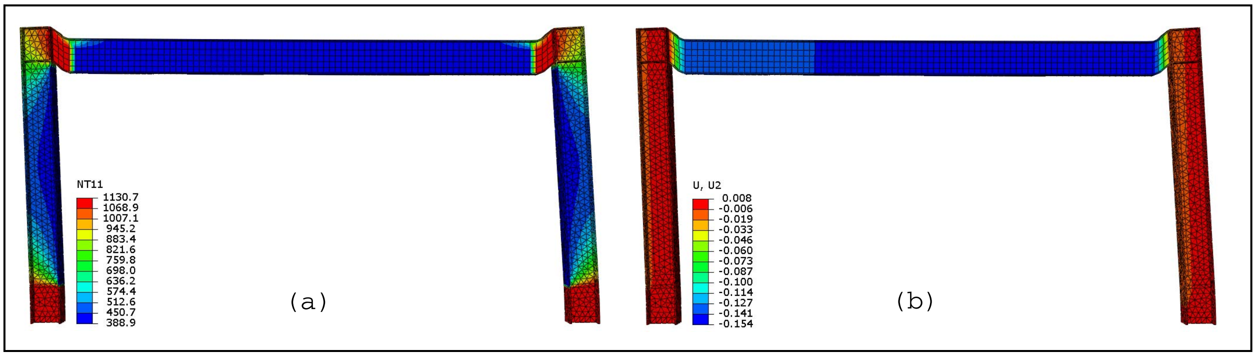

Figure 10. EFS-C-Frame1 model: (a) Temperature distribution (b) Vertical displacement

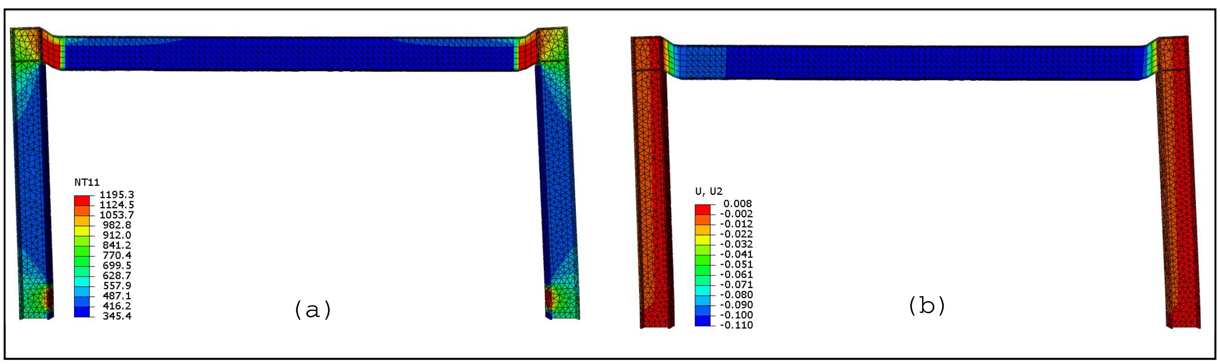

Figure 11. EFS-C-Frame2 model: (a) Temperature distribution (b) Vertical displacement

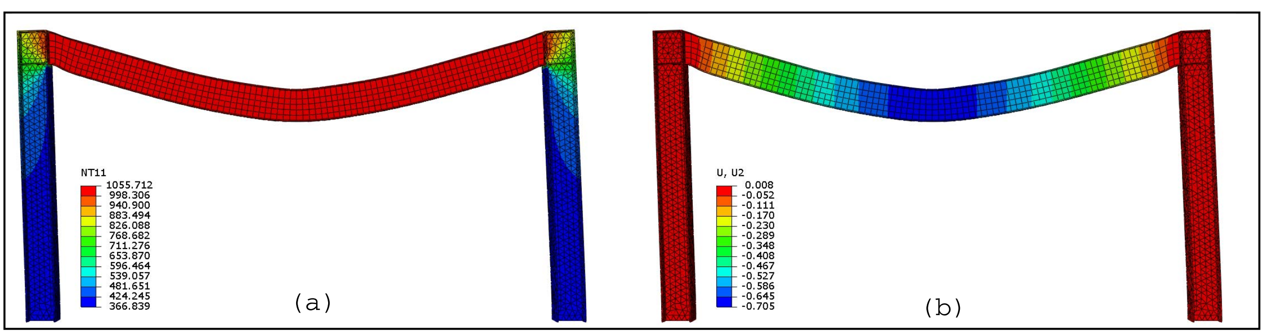

Figure 12. EFS-NC- Beam model: (a) Temperature distribution (b) Vertical displacement

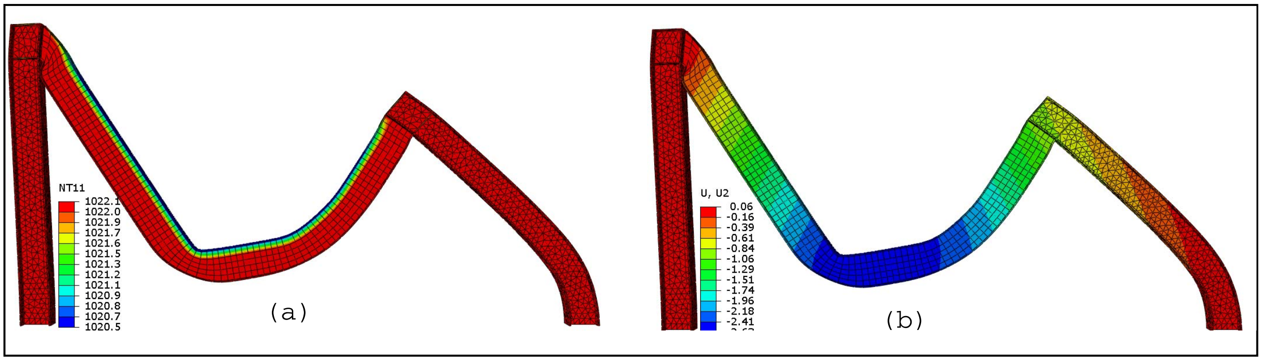

Figure 13. EFS-NC-Frame model: (a) Temperature distribution (b) Vertical displacement

As it is mentioned before, the EFS frames under earthquake loading deform laterally with maximum lateral displacement of 15.05 cm. The maximum horizontal displacement of the EFS-C-Beam frame after fire loading is less than the displacement applied to the frame to simulate earthquake loading. The reason is the relaxation of elastic deformation after earthquake unloading. In EFS-C-Frames the plastic hinges are also formed in column bases and thus, the horizontal displacement of the frame is larger than that for the EFS-C-Beam case as it was anticipated. For EFS-NC models the horizontal displacement of the frame is larger than those of fire proofed frames. As it can be seen in Figure 13, for EFS-NC-Frame one of the columns collapsed into inside direction in contrast to the FS-NC-Frame (Figure 8), for which the collapse took place laterally.

Table 3 also presents the time required to reach 2.5 cm and 5 cm deflection in beam mid span for each model. As it was anticipated, fire following earthquake has more destructive effect on structures than ordinary fires which results in reduction of structural survival time and sustainability, especially when the development of plastic hinges at columns are considered. As it can be seen in Figure 14 (a), the frame survival time is decreased in the case of fire following earthquake. The anticipated effect of fire proofing cover on survival time of structure under fire following earthquake is also shown in Figure 14 (b). However, as it can be seen, fire proofed frames have better performance even under fire following earthquake.

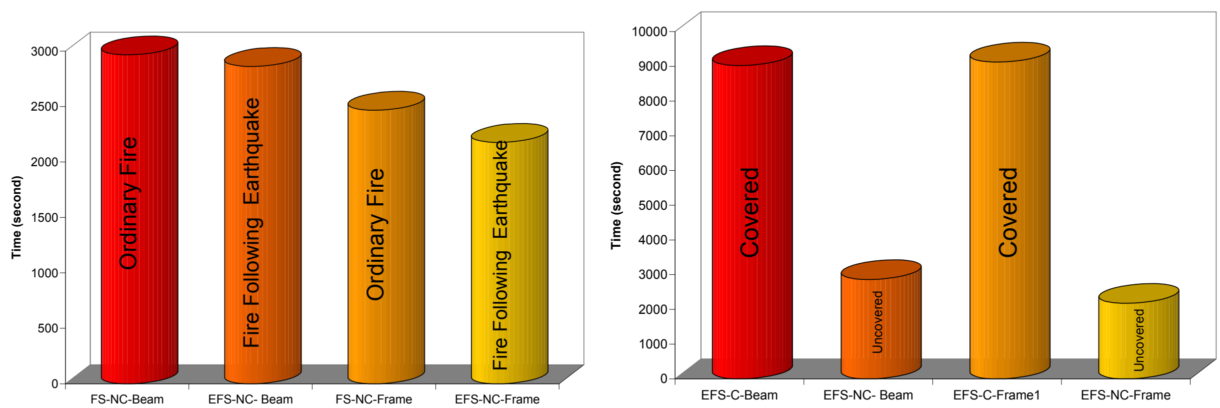

Figure 14. (a): Comparison between survival time of uncovered models under ordinary fires vs. post earthquake fire (b): Comparison between survival time of uncovered vs. covered frames under fire following earthquake

CONCLUSION

In this paper, steel frames are modeled using finite element computer software and the models are analyzed to study the differences between the performances steel structure under fire following earthquake versus ordinary fires. Furthermore, the efficiency of fire proofing cover for the case of fire following earthquake is investigated. The deflection of beam mid span is assumed as a measuring factor for structural deformation and determination of structural collapse. It is concluded that fire following earthquake has more destructive effect on structure than ordinary fires and the structure survival time is reduced, especially when the plastic hinge formation in columns is considered. For the case of covered frame under ordinary fires, no collapse is observed in the time period considered for the analyses of these models. However, in case of covered frame under fire following earthquake the effect of fire proofing cover is decreased and collapse can be observed in the time period considered for the analyses. Nevertheless, fire proofed frames have shown better performance, even under fire following earthquake in comparison with uncovered frames. As it can be seen in the case of fire following earthquake, the steel frame is collapsed laterally, whereas in the case of ordinary fire, the structure is collapsed internally. This is due to the lateral plastic deformation, formed in the structure during the earthquake. The collapse direction is important, especially for fire fighting operations.

REFERENCES

Alderighi E, Salvatore W, 2009, Structural *Fire Performance of Earthquake-Resistant Composite Steel-Concrete Frames*, Engineering Structures 31, pp 894-909 doi: 10.1016/j.engstruct.2008.12.001

Bravery PNR, 1993, *Cardington large building test facility*, Technical report, Building Research Establishment;

Building and Housing Research Center, 1999, *Iranian Code of Practice for Seismic Resistant Design of Buildings (Standard 2800)*, Edition 2, 31-57, Building and Housing Research Center, Iran

Cousins W, Heron D, Mazzoni S, 2002, *Estimating Risks from Fire Following Earthquake*, Client Report 60 (New Zealand: Institute of Geological and Nuclear Sciences)

Della Corte G, Landolfo R, Mazzolani F.M, 2003, *Post-Earthquake Fire Resistance of Moment Resisting Steel Frames*, Fire Safety Journal 38, pp 593-612doi: 10.1016/S0379-7112(03)00047-X.

Hamada M, 1951, *On Fire Spreading Velocity in Disasters (Tokyo: Sagami Shobo)*, in Japanese.

Iranian National Building Code, 2006, Chapter 10 *design and construction of steel structures*, Nashr & Tose-e Iran publication, ISBN: 964-7588-57-7

ISO-834 Fire resistance tests-elements, 1975, elements of building construction, International Standard ISO 834, Geneva

Kirby BR, 1998, *Behavior of a multi-storey steel framed building subjected to fire attack*, experimental Data, Technical report, British Steel

Martin DM, Kirby BR, O'Connor MA, 1998, *Behavior of a multi-storey steel framed building subjected to natural fire effects*, final report, Technical report, British Steel, Confidential report

Mizuno, H, 1978, *On Outbreak of Fires in Earthquakes*, Dissertation, Department of Architecture, Kyoto, University, Kyoto, Japan

National Fire Protection Agency (NFPA), *Earthquake Fire, Kobe, Japan, January 17, 1995*, NFPA, Quincy, Massachusetts, 1995

Scawthorn, C, 1986, *Simulation Modeling of Fire Following Earthquake*, Proc., U.S. National Conference on Earthquake Engineering, Charleston, South Carolina, 1675-1685

Scawthorn C, Eidinger J.M, Schiff A.J, 2005, *Analysis and modeling*, Fire Following Earthquake, 99-155, ASCE, USA

Tokyo Fire Department 1997, *Determinations and Measures on the causes of New Fire Occurrence and Properties of Fire Spreading on an Earthquake with a Vertical Shock*, Fire Prevention Deliberation Council Report, Page 42.

Wai-Fah, Chen, Charles Scawthorn, 2003, *Fire Following Earthquake;, Earthquake Engineering Handbook*, Edition 1, (29-1)-(29-65), CRC Press, Boca Raton.

Wald F, Studecka P, Kroupa L, *Temperature of Steel Columns under Natural Fire*, Acta Polytechnica, Vol. 44, No. 5-6, 2004

Yassin H, Iqbal F, Bagchi A, Kodur V.K.R, 2008, *Assessment of Post-Earthquake Fire Performance of Steel-Frame Buildings*, 14th World Conference on Earthquake Engineering, China

Zolfaghari M.R, Peyghaleh E, Nasirzadeh G, 2009, *Fire Following Earthquake, Intra-structure Ignition Modeling, Journal of Fire Sciences*, Journal of Fire Sciences, Vol. 27, No. 1, 45-79