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NON LINEAR FINITE ELEMENT MODEL FOR POST-EARTHQUAKE FIRE PERFORMANCE EVALUATION OF STEEL PORTAL FRAMES

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ABSTRACT

Post-earthquake fires (PEF) especially in densely populated urban areas have been catastrophic in recent seismic events. It appears to be an important design load which has not been considered critical by most design standards. Moreover, current performance-based seismic design philosophy permits certain level of damage to a structure based on the assumed design seismic hazard. These damaged structures are extremely vulnerable to post-earthquake fires. Even after the outbreak of fire, the structural integrity of the damaged structure must be intact for sufficient duration enabling the firefighters to evacuate and extinguish the fire in the affected building. The recent performance-based design, necessitates evaluation of the fire resistance level of earthquake damaged building with or without the outbreak of post-earthquake fire. In this study an integrated seismic and thermal analysis model was developed using the sequential thermal–structural analysis scheme using the finite element program, ABAQUS. A simple portal frame was considered to investigate the global behaviour of the frame and determine post-earthquake fire resistance. A 2D transient heat transfer analysis was conducted and the transient nodal temperatures across the structural elements cross sections were stored for subsequent thermal structural analyses. The state of earthquake inflicted damage, corresponding to desired performance level was realized using pushover analysis. The results of the simplified 2D model matched reasonably well with that of 3D finite element model considered for validation study. The developed model is being used for subsequent study to investigate the multi-story moment resisting frames with fire scenarios resulting in asymmetric heating of the frame.

Keywords: Performance-based design, post-earthquake fire, seismic hazard, fire resistance, performance evaluation.

1. INTRODUCTION

Construction of steel moment resisting frames (SMRFs) are common in seismically active regions. SMRFs have been preferred because of their fast and economical construction with factory controlled production. However steel structures have an inherent weakness of severe loss of strength and stiffness at elevated temperatures and are susceptible to damage in such extreme loading scenarios. Therefore, these structures are invariably fire protected with sprayed fire resistant material (SFRM) coatings to prevent the temperature rise in their structural elements. Though SFRM coatings have been experimentally proven to be effective in normal fire scenarios, in the event of earthquake these protective layers are severely damaged making the steel structures vulnerable to post-earthquake fire. Studies showed that even a 4% loss of protective layer could significantly decrease the fire resistance of columns by up to 40% (Tomecek and Milke. 1993). Ryder et al. (2002) reported similar results from an experimental study on effect of loss of fire protection materials on the fire resistance of steel columns. In addition damaged active fire protection measures such sprinklers, fire and smoke detection system along with severely impaired city infrastructure such as blocked roads, loss of water supply and diminished information network in the event of earthquake make the mitigation of multiple post-earthquake fire extremely difficult (Mousavi et al. 2008).

Owing to the enormous loss of life and economy in such multi-hazard scenarios, researchers and designers highlighted the need for the development of performance based design (PBD) for post-earthquake fire events in line with PBD for earthquake. In view of the current performance based seismic design approach, where a certain extent of damage in case of severe earthquake is acceptable (Della Corte et al. 2003), the analysis of the behavior of structures under the effect of fires in combination or following an earthquake is a significant research field, which is not yet fully explored (Faggiano and Mazzolani 2011). Although the fire and earthquake resistances of structures have been extensively studied independently, there are very limited studies on the combined effect of post-earthquake fire on steel structures.

In one of the initial numerical investigations on the PEF performance of steel structures Della Corte et al. (2003) investigated unprotected steel moment-resisting frames. Elastic perfectly plastic behaviour for steel was assumed and P-Δ effect was included to simulate geometric nonlinearity. In addition, reduced strength and stiffness was used to simulate the sustained seismic damage. Numerical model was used for thermal-structural analysis on pre- and post-earthquake scenario to investigate the effect of earthquake induced damage on the fire resistance of the structure. It was observed that seismic design philosophy can significantly affect the performance of steel moment resisting frames in PEF event. In another study, Faggiano et al (2007) conducted, nonlinear pushover analysis and 3D coupled thermal structural analysis in ABAQUS to investigate unprotected steel portal frames exposed to PEF. The results showed that the fire resistance for frames were approximately the same as that of the undamaged frame in case performance level is within the operational level. It was concluded that higher the seismic damage lesser will be the fire resistance rating of the damaged structure in PEF. In line with the Federal Emergency Management Agency (FEMA) 356 performance levels for earthquake design, Faggiano et al (2011) suggested fire after earthquake performance levels as Operational fire (Of), Life Safe fire (LSf), Local Collapse fire (CLf), Section Collapse fire (CSf), Global Collapse fire (CGf). Further, Faggiano et al. (2011) developed a robustness assessment framework, to evaluate the performance of buildings subjected to earthquake, and suggested fire performance levels for various conditions of fire.

Behrouz and Hamid (2013) conducted sequential analysis to investigate the PEF resistance of a 10-story moment-resisting steel structure designed to meet the Life Safety (LS) level of performance according to FEMA 356. Nonlinear pushover analysis was conducted in SAP (2000) and for sequential thermal- structural analysis, SAFIR was used. Three scenarios of vertical fire spread initiating on 1^{st}, 4^{th}, and 7^{th} floors with time delay of 5 to 25 minutes were considered. PEF resistances of the frame exposed to the concurrent fire and the fire with 5 minutes lag were observed to be much lower than that with a lag of 25 minutes. The results indicate that subjecting the frame to a delayed fire spread caused the frame failure in cooling phase, unlike the other scenarios where the frames failed during heating phase.

Mehrdad et al. (2013) developed a multi-resolution finite element model to assess both local behaviour of the reduced beam section connection and global behaviour of the frame. The frame was with reduced beam section connection commonly used in strong column and weak beam design concept in earthquake resistant design. The ABAQUS model included 3D elements for only one connection of interest to investigate the local behaviour and line element for the rest of the multi-storey frame model. The time history analysis and sequentially uncoupled thermal-structural analysis was conducted in ABAQUS. In general the symmetric PEF scenarios mostly resulted in smaller IDRs when compared to those resulting from the earthquake, therefore the likelihoods of global failure because of symmetric fire scenarios were not imminent. However the unsymmetrical PEF scenario may give rise to excessive P-delta effects, leading to collapse (Mehrdad (2003)) which is presently investigated by authors and will be reported in subsequent studies.

Roben et al. (2010) studied the structural behaviour of multi-storied composite frame subjected to vertically traveling fire between three floors by varying the inter storey time delays of fire travel and floor beams sizes. The study considered the natural fire model and highlighted the importance of the cooling regime of the fire and the importance of relative axial stiffness of the beams especially in cooling regime. It was observed that the fire travel time affects the global structural behaviour significantly however their research with limited fire scenarios could not suggest worst-case rate of vertical fire spread. One of the critical findings in case of travelling fire was the cyclic movement in columns at each floor level due to alternate heating and cooling regime on adjacent floors which puts significant ductility demand on the connections. This requires extensive numerical and experimental investigations for the various connections in steel structures under travelling fire scenarios for a robust connection design.

Though limited studies have been done on post-earthquake fire performance of structures, there is a compelling need to develop a computationally efficient and commercially viable analysis and design tool for investigating the behaviour of steel structures to encompass wide spectrum of earthquake damage followed by innumerable realistic fire scenarios. The objective of this study is to develop a simplified 2D finite element model capable of modelling the sequence of events in post- earthquake fire events. A brief review is presented on modelling of fire hazard, heat transfer, and sequential thermal structural analysis. The results of the validation study for heat transfer and thermal structure analysis models are presented.

2. FIRE SCENARIOS AND MODELLING

The thermal action of fire is mostly simulated by a single compartment time temperature relationship. Post flashover i.e. when the fire has fully developed (Buchanan 2001) a uniform temperature is assumed inside the compartment, at this momentthe heat release rate (HRR) is maximum affecting the transmission of heat to adjoining structural elements. The design codes such as Eurocode (1) or ASCE (2006) suggests two types of time temperature curves (a) Nominal or standard fire curve or (b) Parametric fire curves. Standard fire curves such as those given by ISO834 (ISO 834 International Standard, 1999) are extensively used in Europe whereas American Society for testing and Materials (ASTM) E119 (ASTM, 2006) are widely used in North America. Figure 1 shows the time temperature variation for hydrocarbon fire, ISO834, ASTM E119, and external fires respectively. For comparison purpose a typical parametric fire is superimposed. The hydrocarbon fires attains high temperature in the range of 1100^{0}C after 30 minutes and maintains a constant value thereafter. External fire curves are used in case of investigating the effect of compartment fires on external structural elements.

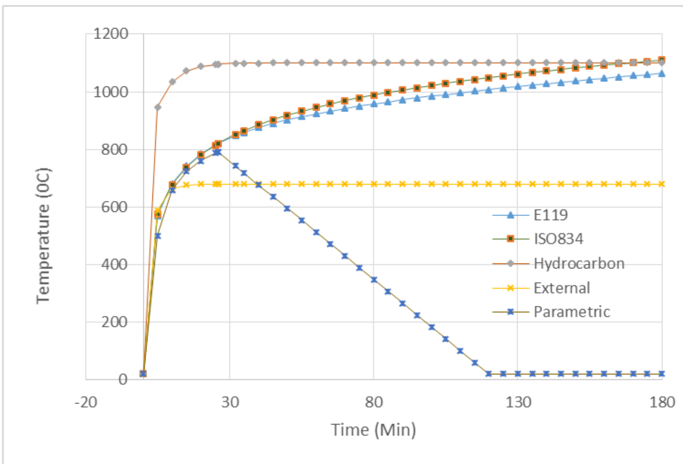

Figure 1: Comparison of Standard and parametric Time Temperature Curves

Standard temperature-time curves though straightforward to use are not suitable for describing the fire phenomena because these curves neither take into account the environmental parameters of the compartment nor its ventilation conditions. Additionally fire according to standard fire curves never decay. On the other hand parametric fire curves are more realistic since these take into account compartment's ventilation conditions and thermal properties of its surroundings. Parametric fire curves, furthermore, consider all the three phases of fire namely, growth, decay and ambient phase, thus allowing for a temperature decrease once the fire load has been exhausted. One of the limitations of the standard or parametric curves is its applicability only in moderate size compartments.

3. HEAT TRANSFER ANALYSIS

The exposed surfaces of a structural member surrounded by hot gases transfer heat to the inner core by the conduction of energy through the solid. Equation 1 expresses the one dimensional heat transfer rate known as Fourier's law. The heat flux, q''_{Tot} (W/m^2) is the heat transfer rate across the depth per unit area perpendicular to the direction of transfer, and it is proportional to the temperature gradient ∇T. The parameter k is a transport property known as the thermal conductivity (W/m.K) and is a characteristic of the material. The rate "Equation 1" is solved with the convective and radiative boundary conditions as given by "Equation 2 and 3" respectively. Where T_g and T_s are gas and surface temperature respectively. The "Equation 3" provides the difference between thermal energy that is released due to radiation emission and that gained due to radiation absorption. The total heat flux at the surface of the steel element due to combined effect of convection and radiative heat transfer is given by "Equation 4", where ε is the emissivity and σ = 5.67E-8W/m^{2}K^4 is Stephan Boltzmann's constant. The heat flux by radiation can also be expressed similar to that of the convective heat flux in terms of temperature difference between gas and the surface concerned. However it is important to note that the h_r, which is coefficient of heat transfer by radiation, is highly temperature dependent and hence needs to be considered accordingly in the numerical model (Agrawal et al. 2104). Eurocode 1 suggests a constant emissivity, ε = 0.5 and coefficient of convection as h_c =25 W/m^{2}K for modelling heat transfer due to radiation and convection boundary conditions from standard fire (Faggiano et al. 2007).

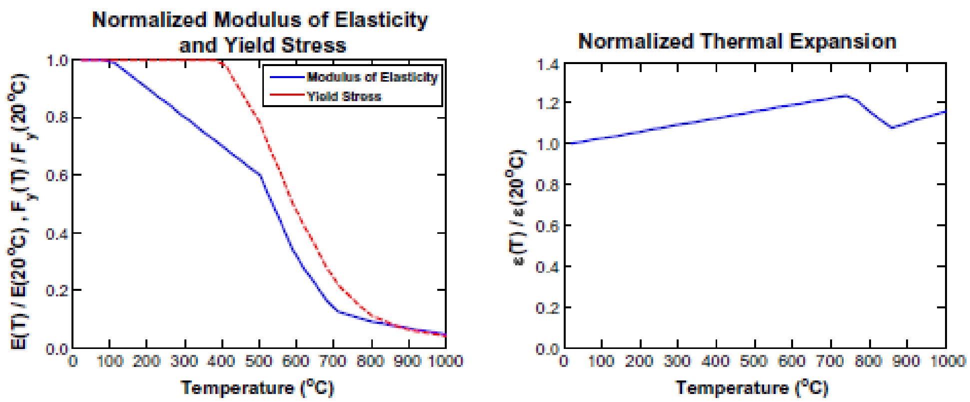

Figure 2: Temperature dependent thermal properties of Steel Structures (Eurocode 3)

$$[1] \quad q''_{Tot} = -k\,\nabla T$$

$$[2] \quad q''_{conv} = h_c(T_g - T_s)$$

$$[3] \quad q''_{rad} = \varepsilon\sigma(T_g^4 - T_s^4) = \underbrace{\varepsilon\sigma(T_g^2 + T_s^2)(T_g + T_s)}_{h_r}(T_g - T_s) = h_r(T_g - T_s)$$

$$[4] \quad q''_{Tot} = q''_{conv} + q''_{rad} = h_c(T_g - T_s) + h_r(T_g - T_s)$$

In the present study, a 2D nonlinear finite element model for transient heat transfer analysis was developed. Figure 2 shows the Eurocode-3 specified temperature dependent normalised specific heat and conductivity of structural steel considered in the present study. A constant value of emissivity of 0.5 was considered to simulate the radiative heat transfer from the ambient to structure. The beam and column cross section were modelled using 2-dimensional 4-noded rectangular heat transfer element DC2D4. The results of the heat transfer analysis were stored in the form of the time temperature curves corresponding to the section points at each node and serve as thermal load during the thermal structural analysis.

4. STRUCTURAL ANALYSIS

The finite element model based on sequentially coupled thermal structural analysis procedure in ABAQUS was developed. The steel portal frame was modelled using beam element, B21. The time temperature curves corresponding to the section points at each node of the beam serve as thermal load during the PEF event. Figure 3 shows temperature dependent strength and stiffness degradation and variation of normalised thermal expansion of steel as suggested by Eurocode 3. The reductions in strength and stiffness are particularly significant in the temperature range of 400°C to 700°C.

Three step analysis procedure based on the framework proposed by Mousavi et al. (2008) was performed. These steps of analysis simulate the actual sequence of loads from initial gravity loads to final thermal loads. The inelastic damage in the moment resisting frame due to the earthquake corresponding to desired performance level was simulated by nonlinear pushover analysis in the second step. The frame was pushed to the target displacement corresponding to the desired performance level. Subsequently, it was unloaded to simulate residual inelastic damage sustained during the earthquake. This was followed by thermal structural analysis to simulate the effect of the fire following earthquake. The analysis results in the form of forces and deformation due to combined effect of thermal and mechanical effects were obtained. The gravity loads were acting throughout the analyses. The structural response in terms of the displacement and rotation were observed at critical sections. The fire resistances for both the undamaged and the post-earthquake damaged frames were considered to have reached at the time instance when one of the plastic hinge attained a rotation of 0.05 radian (Table 1) at this instant the rundown failure was observed.

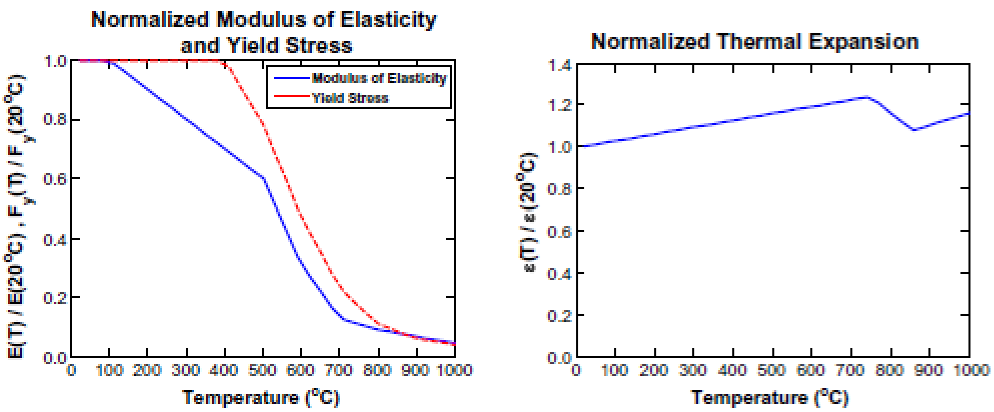

Figure 3: Temperature dependent mechanical and deformational properties of Steel Structures (Eurocode 3)

In this study, earthquake performance levels as per SEAOC 1995 have been considered which are, Fully Operational (FO), Operational (O), Life Safe (LS) and Near Collapse (NC). The acceptable performance limits in terms of inter-storey drift and plastic rotations corresponding to four performance objectives are tabulated in Table 1.

Table 1. Seismic performance levels of structures (SEAOC Vision 2000)

Performance level	Inter-story drifts (δ/h)	Plastic rotations (θ)
Full Operational	0.002 – 0.005	0 (elastic range)
Operational	0.002 – 0.01	≈ 0 (negligible damage)
Life Safe	0.01 – 0.02	0.01 – 0.03
Near Collapse	0.02-0.04	0.02-0.05

5. VALIDATION OF NUMERICAL MODELS: RESULTS AND DISCUSSION

A simple portal frame shown in Figure 4 (Faggiano et al. 2007) is selected for the validation of the uncoupled thermal structure analysis model. The frame is of steel grade S235 (Eurocode 1) having span and height of 3.5m each. The bay width of 3m is considered in the other direction. The dead and live load considered in the analysis are 4.5kN/m^2 and 2kN/m^2 respectively. The cross sections of the beams and column are IPE180 and HEB180 respectively. The chosen frame has been analysed by Faggiano et al. (2007) using fully coupled temperature-displacement transient analysis scheme in ABAQUS. They used solid brick element in their analysis which may not be computationally viable for large scale multi-storeyed structural post- earthquake fire investigation. However this study considers computationally efficient simple beam element (B21), which is extensively used for structural analysis especially for investigating the global structural behaviour. The beam element B21 has an additional temperature degree of freedom per node and is capable of simulating the temperature gradient across the cross-section. In order to simulate the temperature gradient across the cross section ABAQUS allows inputting the temperature at multiple section points (3 section points for planer beam and 5 section points for beams in 3-D frame) per node of the beam. Figure 4 shows three section points along the depth of the 2-D planer beam element cross section, where the gradient across (direction -1) the width is neglected.

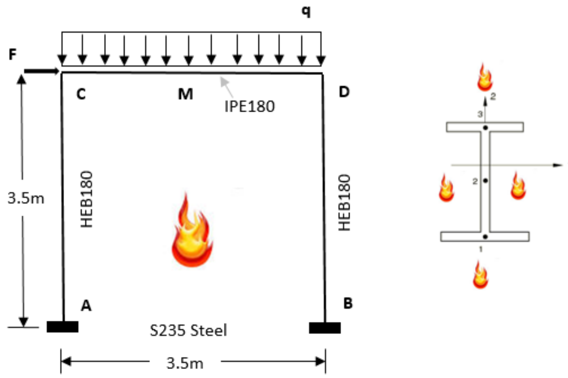

Figure 4: Simple Portal Frame Configuration (LH1 S235) (Faggiano et al. 2007)

5.1 Heat Transfer analysis results

The 2D finite element heat transfer numerical model developed in this study was used to simulate the distribution of temperature across the depth of sections. The time-temperature results at three section points of the beam cross section were stored for the subsequent thermal structural analysis

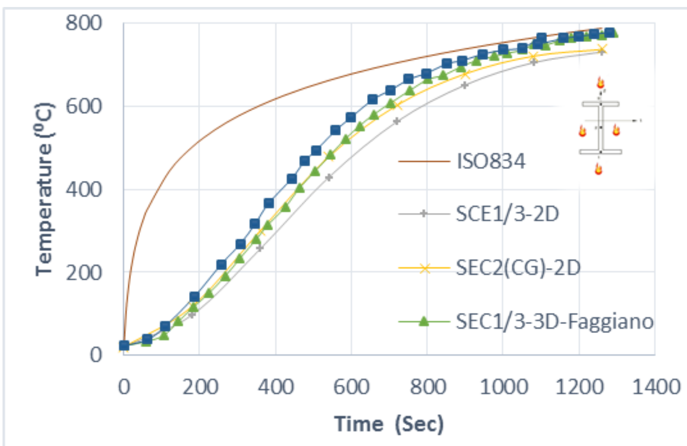

Figure 5: Time Temperature history in IPE180 Beam Section a comparison with Faggiano et al. (2007)

The emissivity equal to 0.5 was considered in the heat transfer analysis (Faggiano et al 2007). In this study the ISO 834 fire was applied on all four faces of the beams and columns, therefore section points 1 and 3 in the flanges have same temperature time history. The section points for the beam element are numbered from 1 to 3 across the depth of the beam or column as shown in Figure 4. The temperature growth at the section points obtained using the developed model is plotted for initial 20 minutes of fire in Figure 5 and compares reasonably well with that of the result obtained by 3D model used by Faggiano et al. (2007). The temperature growth initially lags the ambient gas temperature, ISO 834. Similar time temperature curves for 1hr fire for HEB180 column cross sections are stored which serve as input for thermal structural analysis.

5.2 Seismic Analysis Results

The damage due to earthquake in the frame under consideration was inflicted using pushover analysis in ABAQUS. Two dimensional frame with beam elements (B21) was used in the structural analysis. The frame was statically analysed initially subjecting only the gravity loads. Subsequently it was pushed to the target displacement corresponding to the desired performance levels and unloaded to simulate residual inelastic damage sustained during the earthquake. For the validation study three performance levels considered were – Fully Operational (FO): δ/h = 0.5%; Operational (O): δ/h = 1.0%; and Life Safe (LS): δ/h = 2.0%. The result for the pushover analysis is presented in terms of capacity curve as shown in Figure 6. The results of the pushover analysis are in good agreement with the three dimensional numerical study conducted by Faggiano et al. (2007).

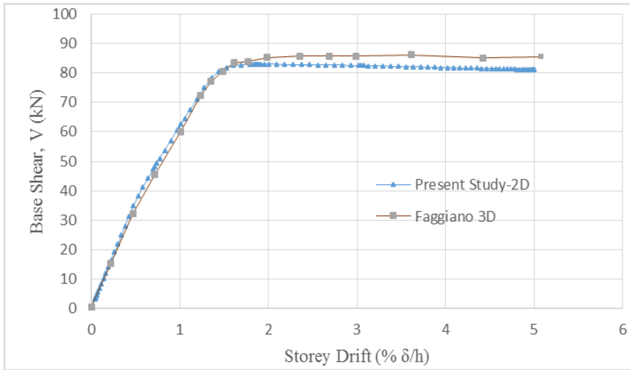

Figure 6: Pushover Curve for the 2-D frame

5.3 Thermal Structural Analysis Results

This section presents the results of the validation of the simplified thermal structural model developed in present study. The damaged state of the frame characterising the desired performance level was subjected to previously determined time-temperature load. The structural element B21 used in the finite element model is capable of incorporating the temperature gradient across the cross section. To this effect time -temperature histories previously saved from thermal analysis are input at three section points of the beam cross section at each node using a user subroutine UTEMP (ABAQUS).

The aim of the thermal structure analysis is to ascertain the fire resistance rating of the damaged structure and subsequently determine the effects of earthquake damage on the fire resistance rating. Three performance levels namely fully operational (FO), operational (O) and life safe (LS) have been considered. Each frame was subjected to thermal load and the time required to reach a previously defined collapse condition, i.e. achievement of 0.05 radian of plastic rotation in at least one hinge of the structure (Faggiano et al. 2007) is determined by dynamic implicit analysis scheme in ABAQUS. The variation of normalised fire resistance i.e. the ratio of the PEF fire resistance corresponding to a given performance level (drift) to the fire resistance of undamaged frame with the earthquake performance level for the frame is shown in Figure 7. The results show that for portal frames with uniform heating inside the compartment, the post-earthquake fire resistance decreases with increase in the earthquake damage. However for such frames the decrease in PEF resistance is very small. Moreover, Faggiano et al. (2007) reported the fire resistance for fully operational performance level to be the same as that of the undamaged structure. In the present study the undamaged structure has greater fire resistance as compared to that of the FO performance level. The analysis of the vertical displacement U_2 (Figure 8) at the beam midpoint provides a worthy insight into the behaviour of steel portal frame subjected to PEF. At the onset of fire the structures is in the deformed state and the deformation is downward as the temperature grows the beam undergoes thermal elongation and curves upwards. However the sudden strength and stiffness degradation of the cross section causes the rapid downward deformation of the beam called rundown failure.

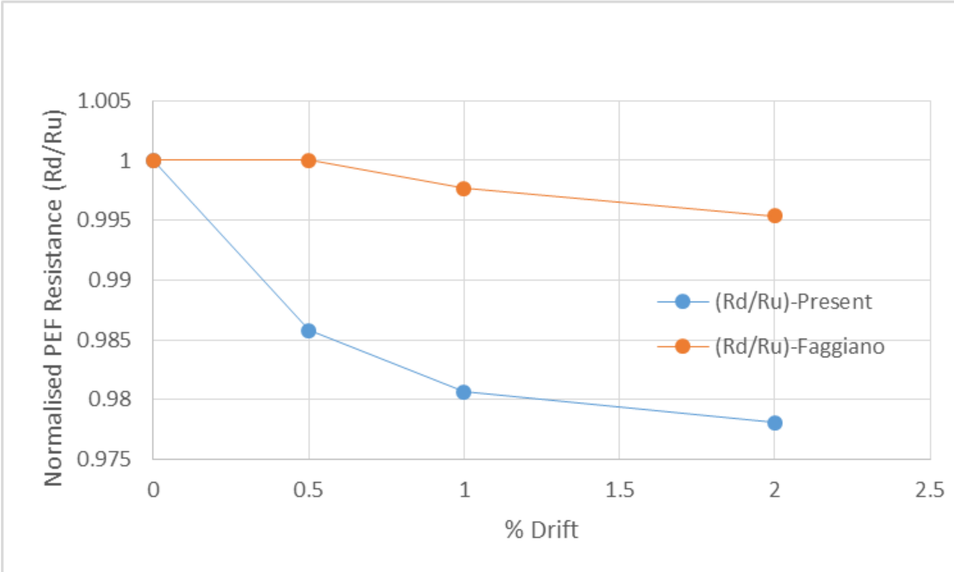

Figure 7: Variation of normalised PEF Resistance with earthquake performance level

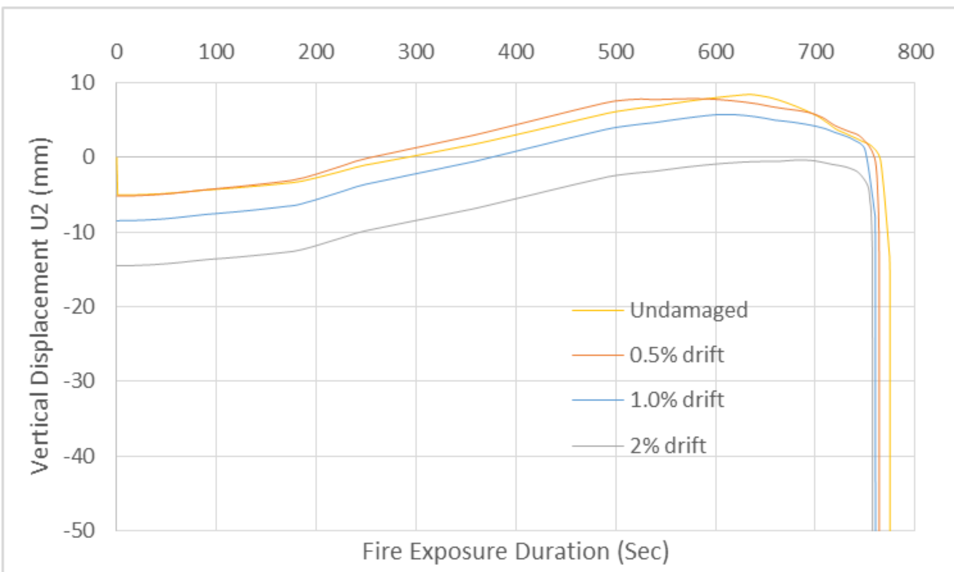

Figure 8: Vertical deformation of Beam mid-point, u_2 (mm)

Figure 9 shows the beam mechanism for the PEF structural thermal analysis of frame subjected to 1% drift. It shows the deflected shape of the frame subjected to post-earthquake fire. It is important to note that in all the four cases of the post-earthquake analysis considered in this study the collapse occurred by formation of beam mechanism.

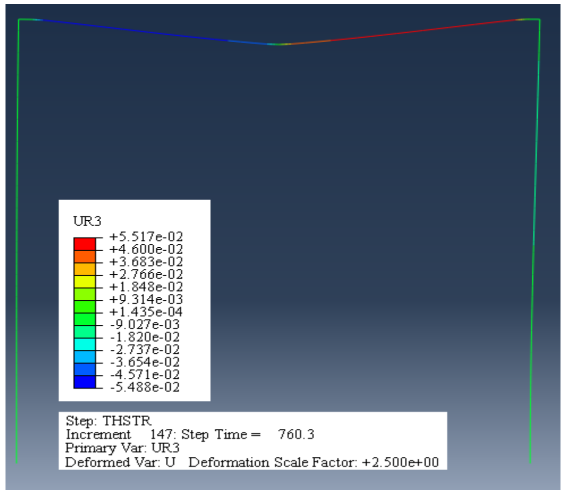

Figure 9: Beam Mechanism at the failure of the portal frame in PEF

6. SUMMARY AND CONCLUSION

A methodology for post-earthquake fire resistance investigation using sequentially coupled thermal structural analysis scheme in ABAQUS was developed. To this effect a simplified 2-D nonlinear finite element model for the post-earthquake fire behavior of steel portal frame was developed and validated based on the previous research by Faggiano et al. 2007. The results of the heat transfer analysis, seismic pushover analysis reasonably matched with the 3D model developed by Faggiano et al. 2007. Results from both the studies show a very small reduction in fire resistance with increase in the seismic performance level for such portal frames. Nevertheless the validated model can be used for evaluating the PEF resistance of 2-D or 3-D low, medium and high rise moment resisting framed structures firstly conducting non-linear time history analysis followed by asymmetric fire spread scenarios with time lag in fire spread. The analysis for the low rise moment resisting framed structure is in progress and the results of the study will be presented very soon. The analysis results will augment the much needed data for understanding the behaviour of SMRF subjected to post –earthquake fire and to correlate the extent of damage due to earthquake to the residual fire resistance of the MRFs.

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