Symmetric and Asymmetric Collapse Mechanisms of a Multi-Story Steel Structure subjected to Gravity and Fire

Sami And Kilic[1] and Serdar Selamet[2]

[1]Assistant Professor, Department of Civil Engineering, Bogazici University, Bebek, Istanbul 34342 Turkey; Phone: +90-212-359-7484; email: skilic@boun.edu.tr
[2]Assistant Professor, Department of Civil Engineering, Bogazici University, Bebek, Istanbul 34342 Turkey; Phone: +90-212-359-6430;
email: serdar.selamet@boun.edu.tr

ABSTRACT

Fire risks pose significant threats to the integrity and stability of the multi-story steel structures. The robustness of these structures against a fire hazard requires further attention. The progressive collapse of a high-rise structure is detrimental not only to the inhabitants in the building but also to the surrounding infrastructure. The collapse of World Trade Center Twin Towers showed that the impact location and the fire distribution could cause symmetrical as well as asymmetrical types of total collapse. This study investigates the uncoupled structural-thermal response of a 49 story steel high-rise structure. The structural load carrying system of the high-rise structure is assumed to be a moment resisting frame. The results show that the structural response and the progressive collapse differ depending on the fire spread and it does not significantly change due to the fire location as long as the fire is contained on a single floor. This study intends to provide a better understanding of the effect of fire loading leading to the collapse mechanism of a multi-story steel structure.

INTRODUCTION

The progressive collapse behavior of multi-story buildings has been intensively studied after the collapse of World Trade Center Towers in 2001 (Hoffman, 2004; Gross, 2005). A sensitivity analysis of a moment resisting and a dual system steel frame due to a column loss is conducted by Kim et al. (2011) and they concluded that the beam yield strength is the most critical design parameter. A detailed three-dimensional composite floor structure with shear connections is modeled by Sadek et al. (2008) and Alashker et al. (2010). One-floor composite structure is suddenly subjected to a column loss and it is observed that the tensile forces in the composite floor are carried by the metal deck. Further, the effect of shear connection capacity is found to be limited. A similar observation is made by Yu et al. (2009). A comprehensive three-dimensional model of the eight-story Cardington steel frame building is created by Kwasniewski (2010). The nonlinear dynamic behavior is analyzed due to column loss at different locations and due to increased gravity loading. The dynamic behavior of a 20 story steel building is investigated due to column loss with several different column locations by Fu (2009). Izzuddin et al.

(2008) and Vlassis et al. (2009) laid out a theoretical groundwork for steel frames due to a sudden column loss. A steel building might also collapse due to an extreme event such as fire. The underlying mechanisms of tall buildings when subjected to fire have been previously investigated by Lange et al. (2012). It was found that the main cause of collapse is the outward and inward deflection of the columns above and below the heated floor. Sun et al. (2012) investigated the vulnerability of steel braced frames and the development of plastic hinges at elevated temperatures.

In light of the described previous research on this subject, the main objective of this study is the assessment of the structural response of a 49 story steel high-rise structure due to ISO834 fire. It is intended to provide a better understanding of the effect of fire loading leading to the collapse mechanism of a high-rise structure. The commercial finite element code LS-Dyna is used as the analysis tool for the uncoupled thermo-mechanical dynamic response of the structure. Simulations are carried out using the Central Difference explicit time integration scheme. Geometric nonlinearity in terms of large displacements and rotations are taken into account. Material nonlinear response is considered through the use of a bi-linear constitutive model. The steel yield stress and stiffness of the material vary at elevated temperatures according to Eurocode provisions. The high-rise steel frame consists of steel beams and columns modeled with 2-node 1D (line) finite elements and composite floors modeled with 4-node shell 2D finite elements. In order to cut down the computational expense, the beam and columns are not modeled with shell elements, which could capture the local buckling effects during fire as discussed by Selamet and Garlock (2012). The structural load carrying system of the high-rise structure is assumed to be a moment resisting frame. Beams, columns, and the composite floor system are modeled with boundary conditions representing rigid connections. The secondary beams and the braces have pinned connections.

MULTI-STORY STEEL STRUCTURE MODEL

Definition
The multi-story steel structure is first modeled in ETABS, widely used structural engineering software. The steel frame sections (i.e. primary and secondary beams, bracings and columns) are designed with a combined earthquake and gravity load according to ASCE\SEI 7-10 seismic loading provisions [ASCE 2010]. The prototype composite floor layout is based on Chase Tower (formerly known as BankOne) in Chicago, IL and it consists of secondary beams, primary beams, lateral bracings and 10cm (4") thick concrete slab with full composite action with the secondary beams. The building has 49 floors with a total height of 179m (588ft). The model deflection calculations takes account the contribution of the concrete slab to the flexural stiffness; however this contribution is neglected for steel design calculations as a conservative approach. The height of each floor is 3.6m (12ft) and the size of the columns is decreased at every 10th floor. Figure 1 and Figure 2 show the extruded view of the steel frame and the column orientations, respectively. The beam sizes of structural elements are shown in Table 1. The connections between the primary and secondary beams are idealized as pinned connections and the perimeter (primary)

beams are connected with moment connections as part of the moment frame design. The bracings are modeled as truss elements. The prototype composite floor is replicated for the upper floors.
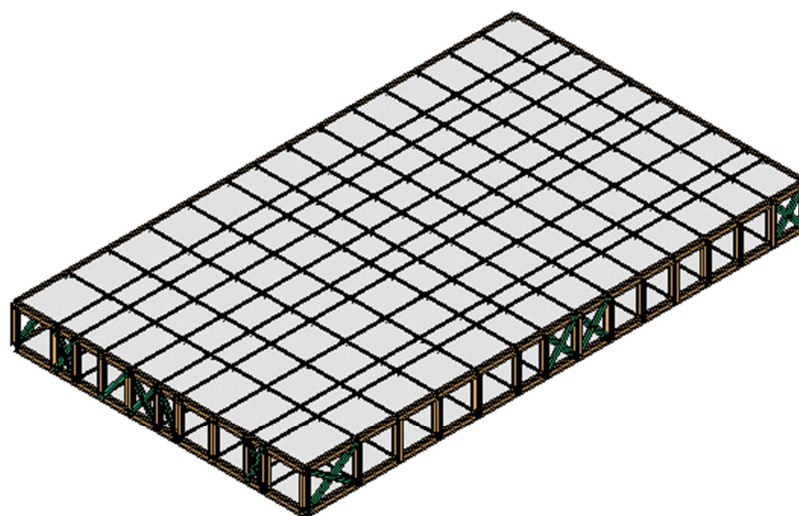

Figure 1. The isometric view of the floor layout of the multi-story building with beam extrusions. The bracings are represented with green lines.
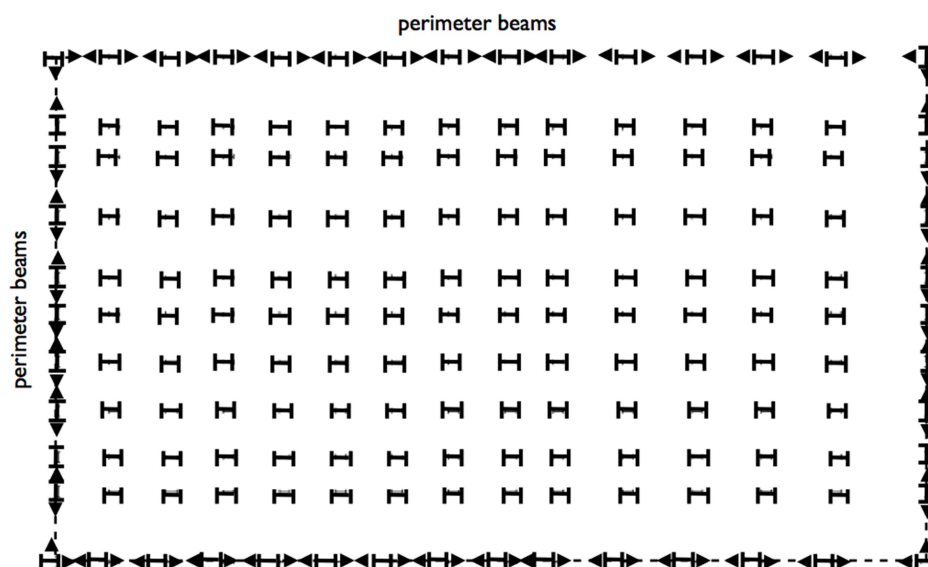

Figure 2. The column orientations for moment frame design.

Gravity and Earthquake Loads for Design

The dead load (DL) is assumed to be the self-weight of the steel members and the superimposed dead load (SDL), which is estimated to be 3.6 kN/m^2 (75 psf). SDL includes the weight of the light-weight concrete and other structural elements. The live load (LL) is assumed to be 2.4 kN/m^2 (50 psf), which is typical for an office building according to ASCE 7-10. The steel members are assumed to carry the gravity load depending on their tributary areas. The first natural period of the multi-

story building is calculated as T_d = 2.6174 sec. The modal response spectrum analysis and the equivalent lateral force analysis methods are used for the earthquake load. Los Angeles CA region is taken as basis for both methods. The response modification R and the occupancy importance I factors are taken as 8 and 3, respectively. The 0.2 sec (S_s) and 1 sec ($S1$) spectral accelerations are taken as 2.1226 sec and 0.7837 sec, respectively. The base shear coefficient is automatically calculated by ETABS as C_w = 0.177.

Table 1. The structural steel members used in the study.

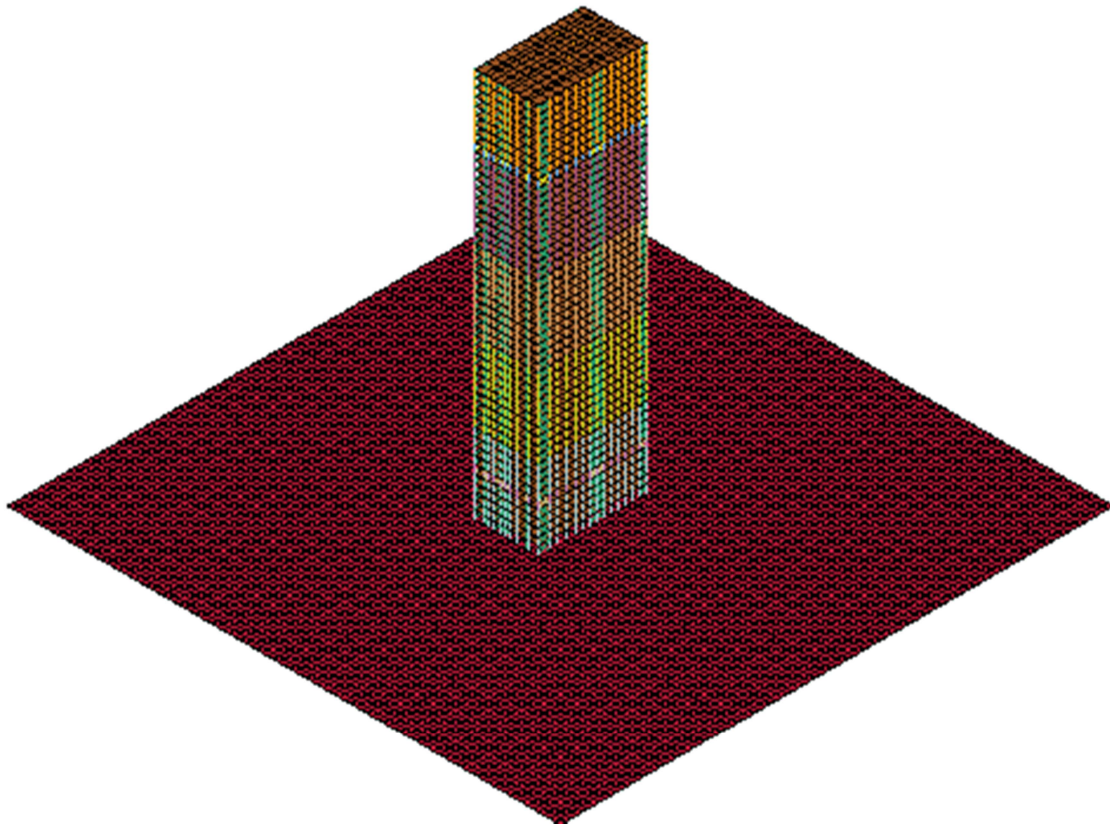| Structural Member	Steel Section
Primary Beam	W24x146
Secondary Beam	W14x34
Lateral Bracing	W14x120
Column (1-10th)	W24x335
Column (11-20th)	W24x306
Column (21-30th)	W24x207
Column (31-40th)	W24x146
Column (41-49th)	W24x103

Figure 3: The multi-story steel building model with the rigid base.

Finite Element Model in LS-Dyna
ETABS building model is imported to LS-Dyna for further analysis of fire scenarios, which requires an explicit dynamic scheme with highly nonlinear material behavior as well as large displacements. The Hughes-Liu beam elements and Belytschko-Tsay shell elements are employed in the model. The shell elements only provide additional stiffness to the building as a diaphragm. A rigid body is placed on the base to provide contact surface for the collapsed floors. The gravity loading defined in ETABS is applied with *LOAD_BODY_Z command for 6 seconds to ensure that no significant inertial forces are created before the fire starts. The reactions due to dead and live loads at the base are validated with ETABS results. In order to simulate progressive collapse, the contact interactions are introduced between the shells, beams and the rigid surface with *CONTACT_GENERAL command. The temperature dependent materials are generated with *MAT_ELASTIC_PLASTIC_THERMAL command. This material type does not have a failure limit state for the effective plastic strain.

The temperature dependent material strength and stiffness of the steel sections and the material properties of the concrete slab are given in Table 2. A simple bilinear stress-strain relationship is assumed, which is based on Eurocode 3 [CEN 2001]. Since the concrete is not heated in the model, only ambient temperature material properties are used as seen in Table 2. For simplicity, no reinforcement bars are modeled for concrete material. The temperature dependent thermal expansion coefficient α of steel is taken from Eurocode 3.

**Table 2. The material properties of the structural members used in the study.
The dimensions are in kN, m and °C.**

STEEL MEMBERS								
T (°C)	20	100	300	500	600	700	900	1100
ρ (kg/m^3)	7850	7850	7850	7850	7850	7850	7850	7850
E (GPa)	200	200	160	120	62	26	13.5	4.5
σ_y (MPa)	345.0	345.0	211.3	124.1	62.0	25.8	12.9	4.3
ν	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
E_t (GPa)	2E-2	2E-2	7.1	7.6	5.3	2.8	4.1E-1	1.38E-1
α (1/°C)	1.23E-5	1.25E-5	1.33E-5	1.41E-5	1.45E-5	1.49E-5	1.34E-5	1.46E-5

Structures Congress 2013 © ASCE 2013   **2550**

Table 3. The material properties of the lightweight concrete slab used in the study. The dimensions are in kN, m and °C.

CONCRETE SLAB						
	ρ (kg/m^3)	E (GPa)	σ_y (MPa)	v	E_t (GPa)	failure strain
T (°C)	1750	24.8	27.6	0.2	24.8E-2	0.05

GRAVITY LOADING AND FIRE SCENARIOS

A total of 6 fire scenarios are investigated, which are illustrated in Figure 4 and defined in Table 4. For all cases, the steel building is gradually loaded with the dead load and the live load for 6 seconds using *BODY_LOAD_Z command in LS-Dyna. The reactions at the building base are validated with the results in ETABS model. No oscillations are observed during the gravity loading, which validates that the loading remained static. The column members are heated with ISO834 fire curve [CEN 2001]. The fire curve is applied at 6 seconds soon after the gravity load is applied. For computational efficiency, the fire curve is scaled to range only to 36 seconds as shown in Figure 4. Within the period, the kinetic energy of the system is carefully monitored in order to get a quasi-static behavior of the building during the thermal loading. The steel temperatures are calculated using lumped mass method with four-sided heating with convective heat transfer coefficient $h = 25$ W/m^2°C and emissivity $\varepsilon = 0.5$. No fire protection is applied to the columns. ISO834 fire curve and the lumped column temperature are shown in Figure 4. The temperatures are directly applied to the nodes without a heat transfer analysis in LS-Dyna. For Cases 1 through 3, the columns at 5th, 25th and 40th floors are heated uniformly considering a symmetrical collapse, respectively. For Cases 4 through 6, only half of the regions at 5th, 25th and 40th floors are heated, respectively. Hence, the asymmetrical collapse is expected. After the plastic deformations in the fire-applied floor, the gravity loading is scaled 5 times until 40 seconds to initiate collapse.

Table 4. The description of fire scenarios.

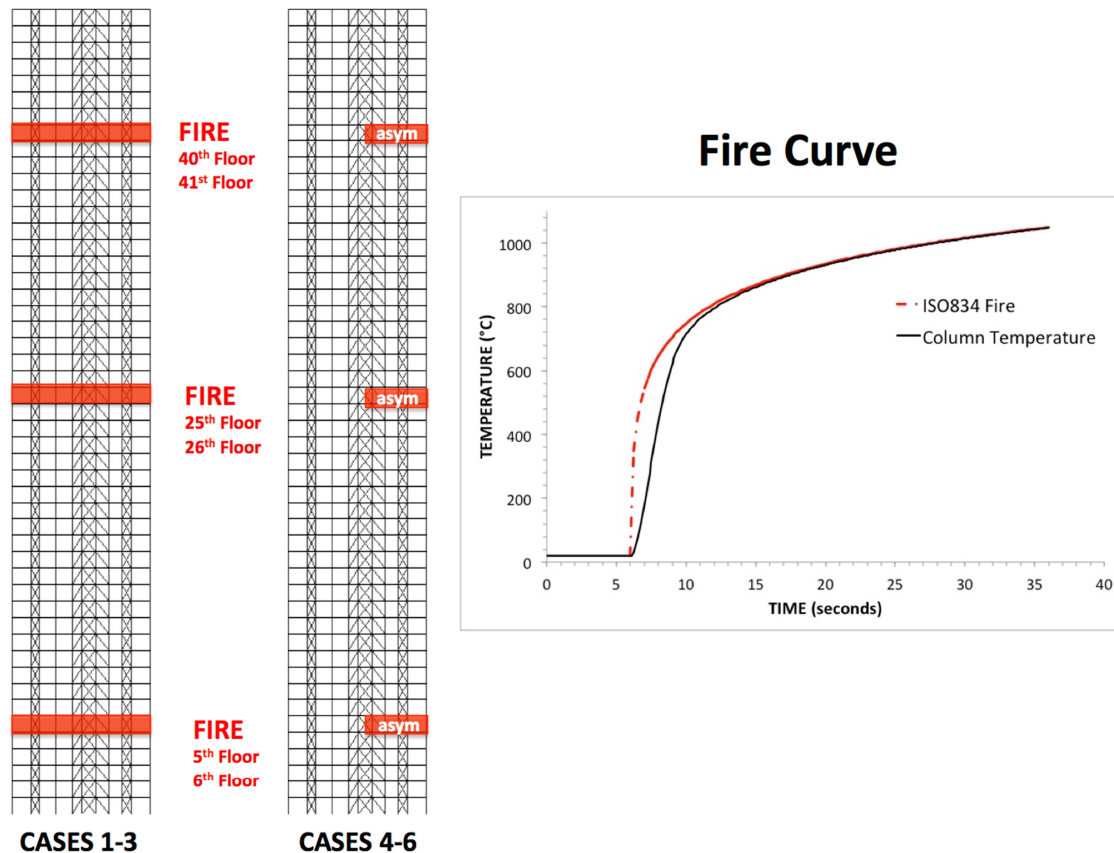| Fire Scenarios	Floor location of the Columns	Fire spread
CASE 1	5th – 6th	symmetrical
CASE 2	40th – 41th	symmetrical
CASE 3	25th – 26th	symmetrical
CASE 4	5th – 6th	asymmetrical
CASE 5	40th – 41th	asymmetrical
CASE 6	25th – 26th	asymmetrical

Figure 4. The location and the progression of fires in the multistory building.

OBSERVATION AND RESULTS

Since the steel building is designed to resist both the gravity and the lateral loading for a large earthquake, it survives the fire duration but it is weakened significantly. Once the gravity loading increases, the collapse initiates. Figure 5 clearly shows the distinct collapse characteristics of the multi-story steel building. At the onset of the collapse, the asymmetric fire causes the building to lean over one side causing more debris to spread to a larger area, which is a great concern in urban areas with high density of tall buildings.

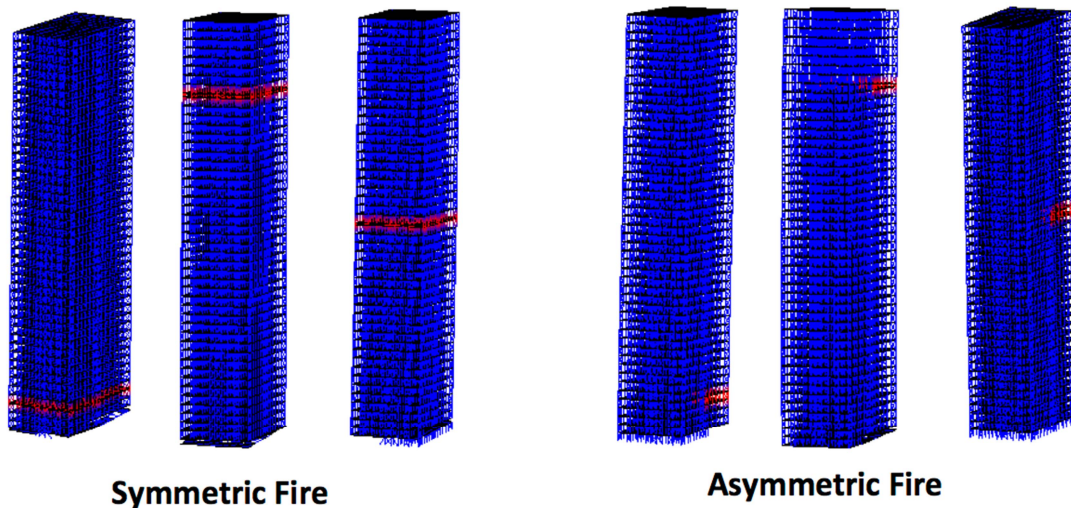**Symmetric Fire**   **Asymmetric Fire**

Figure 5. The deformation of the building at the onset of the collapse for Cases 1 through 6. Note that the heated regions are in red.

Figures 6a through 6c show the rigid body accelerations of the buildings in X-, Y- and Z-directions with respect to the explicit time scale. The accelerations reverse their signs, which indicate that the building collapses part by part, not entirely as intuitively expected. As seen in Figure 5, the columns at the base reach their failure limit state and the floors above start to fall down accumulating on top of each other, hence slowing down the rigid body accelerations to 0.5g on average in the vertical direction (Z-axis).
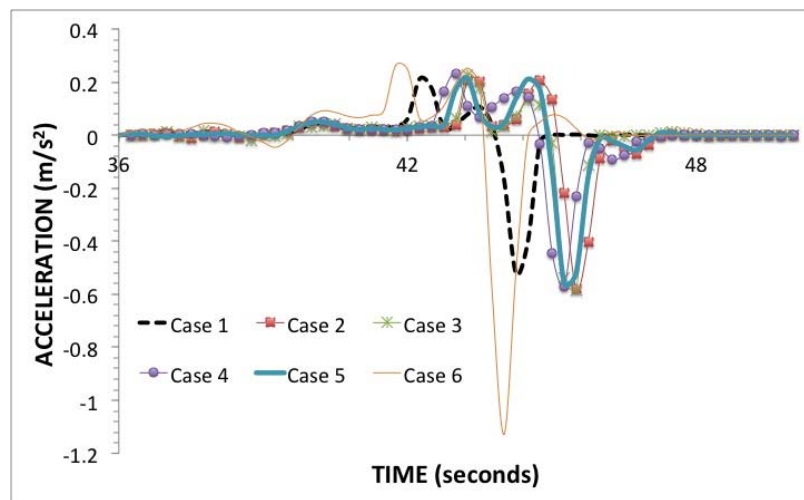
(a)
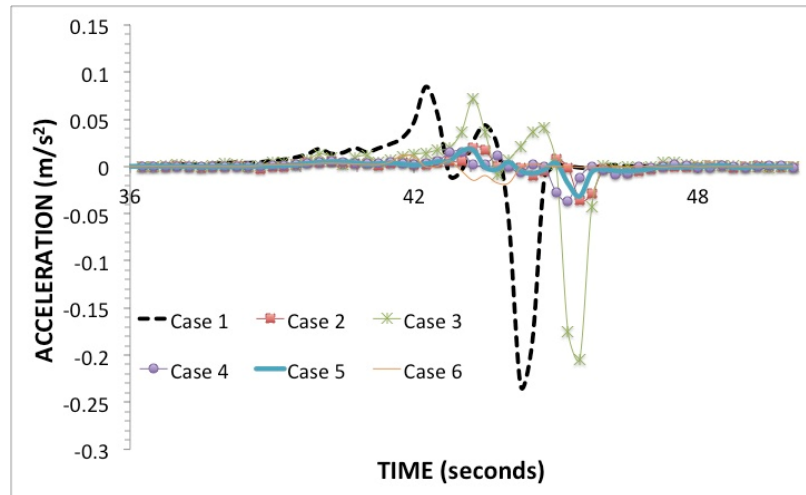
(b)
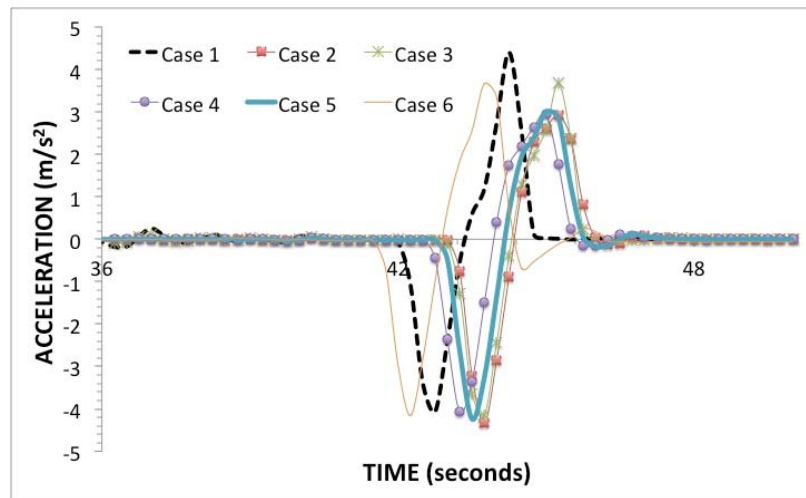
(c)

Figure 6. The rigid body accelerations of the multi-story building in (a) X-, (b) Y- and (c) Z- directions during collapse.

CONCLUSION

This study presents the ongoing research on the collapse mechanisms of multi-story steel buildings with symmetric and asymmetric fire conditions. The observations suggest that the asymmetric heating causes the steel building to collapse by leaning on the side that corresponds to the heated floor region. It also suggests that a building designed for a large earthquake is resistant to weakened floors due to a fire scenario. The location of the fire on a single floor does not significantly change the collapse mechanism of the structure. The authors intend to expand the study by applying fire conditions to multiple floors in order to initiate various fire-induced collapse mechanisms. Furthermore, material properties with multiple failure criteria and element erosion capabilities will be used in order to increase the accuracy of the simulations. The heating of the horizontal members such as the perimeter girders and the secondary beams will be investigated.

REFERENCES

Alashker, Y., El-Tawil, S., Sadek, F. (2010). "Progressive collapse resistance of steel-concrete composite floors", *Journal of Structural Engineering ASCE*, 136 (10), 1187-1196.

ASCE (2010) *ASCE/SEI 7-10 Minimum Design Loads for Buildings and Other Structures, 1st* Edition, Reston, Virginia.

CEN (2001). *Eurocode 3: Design of steel structures Part 1.2: General Rules Structural fire design ENV 1993-1-2:2001,* Brussels, Belgium.

Fu, F. (2009). "Progressive collapse analysis of high-rise building with 3-D finite element modeling method", *Journal of Constructional Steel Research*, 65, 1269-1278.

Gross, J.L.; McAllister, T.P. (2005). "Structural fire response and probable collapse sequence of the World Trade Center Towers. Federal building and fire safety investigation of the World Trade Center disaster (NIST NCSTAR 1-6)", NIST

Hoffmann, C.; Popescu, V.; Kilic, S. A.; Sozen, M. (2004). Modeling, simulation, and visualization: The Pentagon on September 11th, *Computing in Science & Engineering (CISE)*, 6(1), 52-60.

Izzuddin, B.A., Vlassis, A.G., Elghazouli, A.Y., Nethercot, D.A. (2009). "Progressive collapse of multi-storey buildings due to sudden column loss – Part I: Simplified assessment framework", *Engineering Structures*, 30, 1308-1318.

Kim, J., Park, J-H., Lee, T-H. (2011). "Sensitivity analysis of steel buildings subjected to column loss", *Engineering Structures*, 33, 421-432.

Kwasniewski, L. (2010). "Nonlinear dynamic simulations of progressive collapse for a multistory building", *Engineering Structures*, 32, 1223-1235.

Lange, D., Roeben, C., Usmani, A. (2012). "Tall building collapse mechanisms initiated by fire: Mechanisms and design methodology", *Engineering Structures*, 36, 90-103.

LSTC (2012). *LS-DYNA Keyword User's Manual Volume I & II version 971*, Livermore, CA.

Sadek, F., El-Tawil, S., Lew, H.S. (2008). "Robustness of composite floor systems with shear connections: Modeling, simulation, and evaluation", *Journal of Structural Engineering ASCE*, 134 (11), 1717-1725.

Selamet, S., and Garlock, M. (2012). "Plate buckling strength of steel wide-flange sections at elevated temperatures", *Journal of Structural Engineering ASCE*, doi: 10.1061/(ASCE)ST.1943-541X.0000769.

Sun, R., Huang, Z., Burgess I.W. (2012). "The collapse behaviour of braced steel frames exposed to fire", *Journal of Constructional Steel Research*, 72, 130-142.

Vlassis, A.G., Izzuddin, B.A., Elghazouli, A.Y., Nethercot, D.A. (2009). "Progressive collapse of multi-storey buildings due to failed floor impact", *Engineering Structures*, 31, 1552-1534.

Yu, M., Zha, X., Ye, J. (2010). "The influence of joints and composite floor slabs on effective tying of steel structures in preventing progressive collapse", *Journal of Constructional Steel Research*, 66, 442-451.