Structural Engineering and Mechanics, *Vol. 59, No. 1 (2016) 59-82*
DOI: http://dx.doi.org/10.12989/sem.2016.59.1.059

Progressive collapse analysis of steel building considering effects of infill panels

Mohammad Abbasi Zoghi* and Masoud Mirtaheri[a]

Department of Civil Engineering, K.N. Toosi University of Technology, Tehran, Iran

(Received December 10, 2015, Revised May 8, 2016, Accepted May 11, 2016)

Abstract. Simplifier assumptions which are used in numerical studies of progressive collapse phenomenon in structures indicate inconsistency between the numerical and experimental full-scale results. Neglecting the effects of infill panels and two-dimensional simulation are some of these assumptions. In this study, an existing seismically code-designed steel building is analyzed with alternate path method (AP) to assess its resistance against progressive collapse. In the AP method, the critical columns be removed immediately and stability of the remaining structure is investigated. Analytical macro-model based on the equivalent strut approach is used to simulate the effective infill panels. The 3-dimentional nonlinear dynamic analysis results show that modeling the slabs and infill panels can increase catenary actions and stability of the structure to resist progressive collapse even if more than one column removed. Finally, a formula is proposed to determine potential of collapse of the structure based on the quantity and quality of the produced plastic hinges in the connections.

Keywords: progressive collapse; steel frame structures; column loss; alternate path method; infill panels; nonlinear dynamic analysis

1. Introduction

Local collapse in a structure can spread vertically or horizontally to the other areas of the building if no alternate path exists to redistribute the loads. Therefore, limiting the local collapse in the damaged area is major idea to mitigate progressive collapse in the buildings. Increasing ductility, connectivity and indeterminacy degrees of the structure are some solutions. Codes such as the GSA (2003), UFC (2010), ASCE 7 (2005) and the NISTIR 7396 (2007), introduce Alternate Path method (AP) as a direct design method to investigate the structure response after the local failure. In this method some critical vertical elements are removed suddenly and the rest of the structure shall be capable to resist generated dynamic loads. Slabs and infill panels generate new paths in the structure to redistribute the loads over the missed elements due to catenary actions (Mostafaei *et al*. 2004). Therefore, considering effects of the slabs and infill panels in the analysis cause more indeterminacy and connectivity in the structure to resist against global collapse.

Numerical studies on progressive collapse with simplifier assumptions based on the analysis

*Corresponding author, Ph.D. Student, E-mail: mabbasi@dena.kntu.ac.ir
[a]Associate Professor, E-mail: mmirtaheri@kntu.ac.ir

Copyright © 2016 Techno-Press, Ltd.
http://www.techno-press.org/?journal=sem&subpage=8 ISSN: 1225-4568 (Print), 1598-6217 (Online)

purposes are exist. Some of these assumptions are neglecting effects of slabs, infill panels, dynamic effects of column removal and two-dimensional simulation.

The first study involving progressive collapse analysis of steel frames was presented by Gross and McGuire in 1983. In this study, the behavior of 2-D moment resisting steel frames with the loss of one of the columns or increased load on the beams representing fallen debris was examined numerically. Both material and geometric nonlinear effects were taken into account. Shear infill panels were modeled as springs with bilinear shear stress-rotational strain relationship. Results have shown that the remaining damaged structure cannot sustain the applied loads statically. But this static analysis for load redistribution is not accurate since the actual load redistribution process is dynamic in nature. Another study on the progressive collapse in steel buildings was done by Kaewkulchai and Williamson (2004, 2006). They investigated the analysis procedures using a two-dimensional frame analysis. They found that linear static analysis might result in non-conservative results since it cannot reflect the dynamic effect caused by sudden removal of columns. Marjanishvili and Agnew (2006) presented and compared four methods (linear static, nonlinear static, linear dynamic and nonlinear dynamic) for progressive collapse analysis by analyzing steel moment-resistant frame buildings. These authors and Powell (2005) stated that using nonlinear dynamic analysis is not only more accurate but also easy to perform by using modern FEM software. Also, Marjanishvili (2004) indicated that the nonlinear static procedure may result in larger ductility because the load path moves not to surroundings but to vertical direction.

Hayes *et al*. (2005), Tsitos *et al*. (2008, 2010) presented studies on how the current seismic design provisions can improve resistance to progressive collapse. Hayes studied progressive collapse analysis on Alfred P. Murrah Federal Building (severely damaged in 1995). The key finding of the study was that strengthening the perimeter element using current seismic detailing techniques improved the survivability of the building.

Another category of experimental studies are the in-situ full scale tests that were performed on existing concrete buildings Sasani *et al*. (2007), Sasani and Sagiroglu (2008). All of the above followed the standard approach of the sudden loss of one or more exterior columns at the first floor level. In all cases the structures were able to redistribute the loads without the propagation of failure to additional members.

Astaneh-asl *et al*. (2002) investigated the strength of a typical steel structure with steel deck and concrete slab floor system to resist progressive. It was observed that after removal of the middle perimeter column, the catenary action of the steel deck and girders was able to redistribute the load of removed column to other columns without collapse.

Sasani and Sagiroglu (2008) presented experimental study of progressive collapse on the 6-stories Hotel San Diego, San Diego with RC structure. They removed two columns of the first floor to evaluate progressive collapse resistance of the building. Contrary to the numerical studies the actual sudden removal of the two columns didn’t cause any collapse of the frame.

The most recent in-situ case study on a steel building with moment resisting frames was presented by Song and Sezen (2009). The Ohio Student Union Building was tested by removing four first floor columns from one of the long perimeter. The structural system of the building consisted of in both principal directions. The building did not collapse and satisfied the GSA criteria, even after the removal of the fourth column.

Sattar (2013) studied Influence of masonry infill walls and other building characteristics on seismic collapse of concrete frame buildings. He proposed multi-scale modeling approach to simulate the response of masonry infilled frames up to the point of collapse. Results of this study captured the influence the infill panel has on the collapse performance of the frame and can be used to prioritize mitigation of the most vulnerable RC frames.

Hariri-Ardebili *et al*. (2014) investigated the effects of masonry panels on the vibration response of an infilled steel-frame building using numerical, analytical, and experimental methods. The results show that neglecting the effect of infill panels leads to considerable error. Moreover, it is shown that there is good agreement among the results obtained by the three methods considering the effect of infill panels.

The aforementioned studies show that the accuracy of the analysis strictly depends on considering complicated essence of the progressive collapse phenomenon. In this numerical study, nonlinear static and dynamic analyses based on the AP method are applied to the existing bolted steel structure to investigate progressive collapse resistant of it. This building was designed based on the IBC (2006) recommendations to resist against earthquake lateral loading. Dynamic effects of suddenly failure of column(s), material nonlinearity, large displacements equations and effect of the slabs and infill panels are considered in the analyses.

2. Analysis methods

Accuracy of progressive collapse investigating depends on analysis method. To this end, two methods named nonlinear static (NLS) and nonlinear dynamic (NLD) analyses are used in this research.

2.1 Nonlinear static method

In the NLS analysis, structural elements experience inelastic behavior under simultaneously applied load combination (1) which is recommended by United States General Services Administration (GSA) standards.

$$2.0\ [(0.9 \text{ or } 1.2)\ D + (0.5\ L \text{ or } 0.2\ S)] + 0.2\ W \tag{1}$$

In this combination, factor of 2 is dynamic amplification factor to simulate dynamic effect of loads. Combination (1) should apply to those bays immediately adjacent to the removed element(s) and at all floors above it. Factor of 2 will be eliminated for the rest of the structure. In this method, such as “pushover analysis”, applied load on the structure increase gradually considering second order or P-Δ effects until collapse of structural elements occurs. To this end, dead and live load applied to the entire structure statically. Then resultant initial forces and displacements applied as initial conditions for next step. In this step considered column(s) remove from structure and load combination (1) applied to the damaged spans. Because of vertical direction of the applying loads, this method called “push-down analysis” (Tsitos *et al*. 2008).

2.2 Nonlinear Time-History analysis (dynamic method)

As mentioned before, time-history analysis should be applied toseek dynamical response of the structure. Following load combination has recommended by GSA for the nonlinear dynamic analysis (NLD).

$$(0.9 \text{ or } 1.2)\ D + (0.5\ L \text{ or } 0.2\ S) + 0.2\ W \tag{2}$$

In this method, the stiffness, damping, and loads may depend on the displacements, velocities, and time. This requires an iterative solution to the equations of motion. In the NLD modeling, the column(s) is deleted in the structural model and the internal forces (F_{eq}) determined from undamaged equilibrium model are applied to the structure as a load case to the joint of column's end. Then static nonlinear analysis results are used as the initial conditions for the column removal. Applied equivalent loads (F_{eq}) decrease under duration of Ts equal to (1/10)T, where, T is the natural period of first mode of undamaged structure (Marjanishvili *et al*. 2004, 2006).

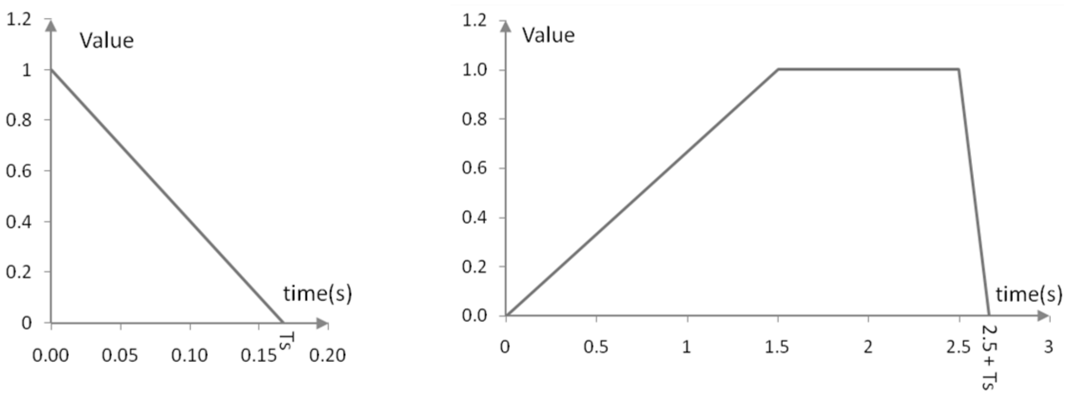
3. Design method

Alternate Path method (AP) as an applicable direct methods that recommended by Codes is applied to design the structure. It is applied to verify that the structure can bridge over the deficient or missing element(s) or not. The locations of the removable elements include, as a minimum, the center of the short side, the center of the long side, and the building corner. The AP method is threat independent and doesn't care of removal reasons. (GSA, UFC)

4. Modeling

Frame elements are used to simulate beams and columns in 3-dimensional modeling. Composite slabs are modeled by shell elements to distribute the loads and each floor is taken as a rigid diaphragm, separately. Dynamic analysis is carried out using Hilber-Hughes-Taylor method with α=0.0, β=0.25 and γ=0.5 and the Newton-Raphson solution algorithm. Damping ratio is considered equal to 5% of the critical damping. (Kim *et al*. 2009). Time step of 0.001 second is used for time-history analysis.

As described in section (2), for the NLD analysis F_{eq} decreases under duration of Ts. To this end, two following functions are used by researchers; a) initial zero condition and b) static equilibrium (see Fig. 1). The values of these diagrams are applied to the F_{eq} in each time step to simulate removal of the desired column(s). For example, in the function (1), 1×F_{eq} is applied to the



(a) Function 1: initial zero condition (b) Function 2: static equilibrium

Fig. 1 time-history functions applied to the F_{eq}

model at the beginning of the analysis and then its value decreases to zero in 0.15 (s). This means that the column is removed at the time of 0.15 (s).

As we know, each structure experiences initial elastic deformations under dead and live loads before missing of the column. This fact will not be considered in the function (1) procedure. Therefore, function (2) proposed to simulate column loss after static equilibrium. In this method, considered column(s) deleted from the structure and dead and live load applied statically. Then ascending equivalent load applied to the structure gradually to avoid dynamic effects (duration of ascending equal to 1.5 s is selected in this study). Then the structure remains in this position for a while to obviate probable dynamic response. Third and final stage of this method is suddenly removal of F_{eq} under duration of Ts.

Large displacement and plastic hinges might be occurred in some structures in stage (1) in this method. Thus applying function (2) for analyzing of these structures is not applicable, otherwise it causes imprecise results. In other word, plastic hinges which are produced in stage (1), change the stiffness matrix and initial conditions of stage (3). It decreases the structure stiffness so deformations are obtained larger than reality. Therefore one could use another procedure such as assignment of compression limit for considered column(s).

To evaluate resistant of the structure against progressive collapse the AP method is applied. According to the code guidelines, there is no Demand-Capacity-Ratio or geometric irregularity limitations, so nonlinear static and nonlinear dynamic time-history analysis methods considering P-Δ effects are performed to demonstrate progressive collapse potential of the building.

4.1 Infill panels modeling

Unreinforced masonry (URM) infill panels in full contact with the frame elements on all four sides shall be considered as primary elements of a lateral force-resisting system. In-plane lateral stiffness of an infilled frame system is not equal to sum of the frame and infill stiffnesses because of interaction of the infills with the surrounding frames. Experiments have shown that under lateral forces, the frame tends to separate from the infill near windward lower and leeward upper corners of the panels, causing compressive contact stresses to develop between the frame and the infill at the other diagonally opposite corners. Recognizing this behavior, the stiffness contribution of the infill is represented with an equivalent compression strut connecting windward upper and leeward lower corners of the infilled frame (see Fig. 2). In such an analytical model, if the thickness and modulus of elasticity of the strut are assumed to be the same as those of the infill, the problem is reduced to determining the effective width of the compression strut. Solidly infilled frames may be modeled with a single compression strut in this fashion (Madan *et al*. 1997).

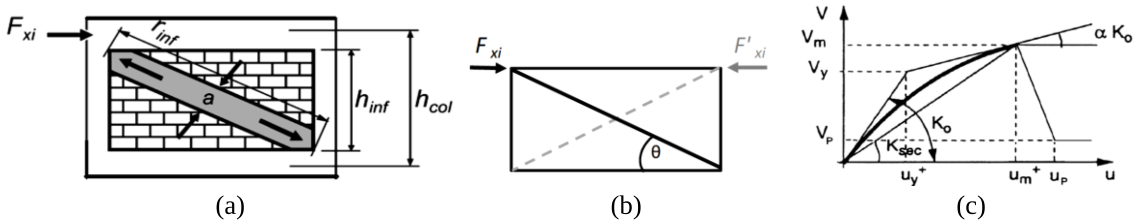


Fig. 2 (a) equivalent compression-concentric strut model, (b) modeling of strut based on lateral load direction, (c) strength envelope for masonry infill panels based on constitutive model for masonry

The stress-strain relationship for masonry in compression which is used to determine the strength envelope of the equivalent strut is idealized by the polynomial function. Since the tensile strength of masonry in negligible, the individual masonry struts are considered to be ineffective in tension. Therefore the monotonic lateral force-deformation relationship for the structural URM infill panel in assumed to be a smooth curve bounded by a bilinear strength envelope (see Fig. 2).

The equivalent strut is represented by the actual infill thickness that is in contact with the frame (t_{inf}) and the diagonal length (r_{inf}) and an equivalent width given by

$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{\inf} \tag{3}$$

$$\lambda_1 = \left[\frac{E_{me} t_{\inf} \sin 2\theta}{4E_{fe} I_{col} h_{\inf}}\right]^{0.25} \tag{4}$$

Where E_{fe} and E_{me} are the expected modulus of elasticity of frame and infill material respectively and I_{col} is the moment of inertial of column. As recommended by FEMA 356, E_{me}=550f'_m, where f'_m is the compressive strength of the infill.

The strength capacity of an infill panel is a complex phenomenon. Four failure modes are recognized in URM infill panels; Sliding-shear, Compression, Diagonal Tension and General shear Failures. (Madan *et al*. 1997) Here the compression failure mode dominates for response of the panels of the considered building in progressive collapse phenomenon (because of larger displacement of structure rather than earthquake loading). Based on indicated diagram in Fig. 2(c), V_y is the yield strength of URM infill, V_m is the maximum strength, V_p is the residual strength and u_y, u_m and u_p is the related displacement, respectively. α is the ratio of the post yield infill stiffness to the initial stiffness.

In the Compression Failure, the shear force (horizontal component of the diagonal strut capacity) is calculated as

$$V_m = a t_{\inf} f'_{me90} \cos\theta \tag{5}$$

Where f'_{m90} is expected strength of masonry in the horizontal direction, which may be set at 50% of the expected stacked prism strength f'_{me} which is estimated as 1.3f'_m.

Other parameters are calculated based on following relations.

$$u_m = \frac{\varepsilon_m r_{\inf}}{\cos\theta} \tag{6}$$

In which ε_m is the corresponding strain of the infill.

$$K_0 = 2(\frac{V_m}{u_m}) \tag{7}$$

Thus

$$V_y = \frac{V_m - \alpha K_0 u_m}{1-\alpha}, u_y = \frac{V_y}{K_0} \tag{8}$$

V_p and u_p derivate from experimental test and hysteretic curves. In this study, residual strength is neglected due to weakness of infill panels. (Madan *et al*. 1997)

4.2 Frame hinge properties

Yielding and post-yielding behavior of frame elements in connections are modeled using discrete plastic hinges. Hinges only affect the behavior of the structure in nonlinear static and nonlinear direct-integration time-history analyses.

Plastic force-displacement curve and plastic moment-rotation behavior are specified for axial force and bending potential, respectively. The axial force and the two bending moments may be coupled through an interaction surface. In this type of hinges, the plastic rotations in both directions measured after yielding, then resultant moment and the projected plastic rotation calculated by equations related to moment angle θ; (FEMA-356)

$$\mathrm{M} = \mathrm{M}_{2-2} \times \mathrm{Cos}\theta + \mathrm{M}_{3-3} \times \mathrm{Sin}\theta \tag{9}$$

$$\mathrm{R_p} = \mathrm{R}_{\mathrm{p}_{2-2}} \times \mathrm{Cos}\theta + \mathrm{R}_{\mathrm{p}_{3-3}} \times \mathrm{Sin}\theta \tag{10}$$

State of the structure at end of analyzing are determined by total numbers of hinges that experience points IO (immediate occupancy), LS (life safety), or CP (collapse prevention). (FEMA-356).

Two hinges properties are assumed for these plastic hinges based on FEMA-356: Axial Hinge (just for columns) and Moment and Coupled Hinge.

FEMA-356, table 5-6 has defined acceptance criteria for rotation of elements based on yield rotation, θ_y which is calculated by following relations (FEMA-356, Eqs. (5-1) and (5-2)).

$$\theta_y = \frac{ZF_{ye} l_b}{6EI_b} \tag{11}$$

$$\theta_y = \frac{ZF_{ye} l_c}{6EI_c}\left(1 - \frac{P}{P_{ye}}\right), P_{ye} = A_g F_{ye} \tag{12}$$

If any elements violate criteria mentioned in Tables 3-1 in the UFC 4-023-03 after analyzing, it will be removed and re-analysis and re-design will be done until no more violation predicted. Beside, the GSA acceptance criteria recommend deformation limits for the performance of structural members.

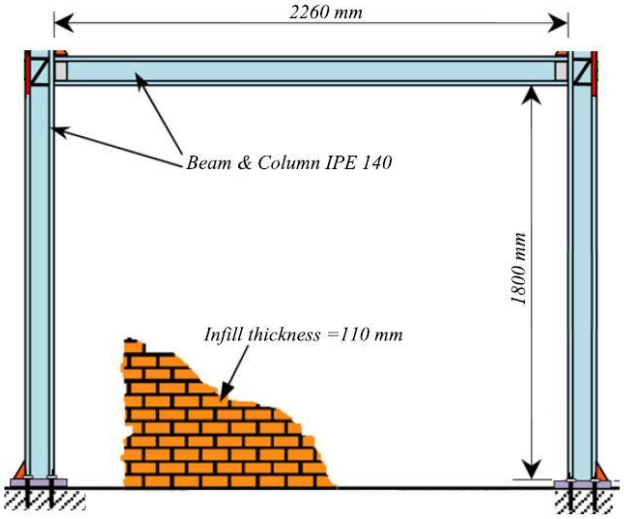
Observations of the authors' analyses showed that just the vicinity removed column(s) elements experience plastic deformations. Thus plastic hinge properties just assigned to the main columns and girders immediately adjacent to the removed element(s). Furthermore, no plastic hinges occur in pinned joist beams because they restrained in slabs.

5. Model validation

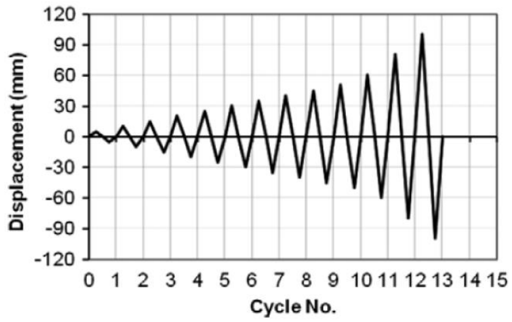
To evaluate modeling of the URM infill panels, experimental study done by Tasnimi and Mohebkhah (2011) is utilized. They studied behavior of the brick-infilled steel frames experimentally and analytically under cyclic quasi-static loading (see Fig. 3(a), (b)). Table 1 indicates material properties of specimen. To this end, main steel frame is modeled by beam elements using OPENSEES software. Equivalent compression struts are modeled with beam elements with no tension capacity. Strain hardening for steel is taken into account about 2% of Es

Table 1 Materials properties of the specimen (kg/cm^2)

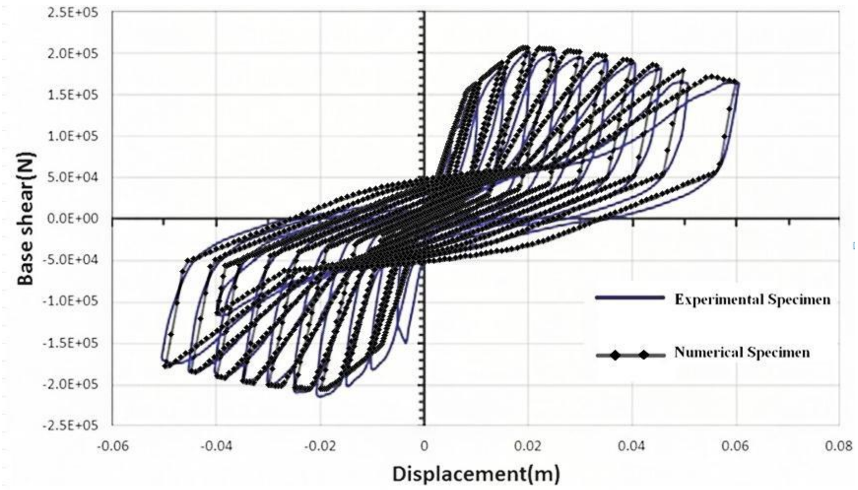
Steel Frame- IPE140		URM infill panel	
Fy	*Es*	average prism compressive strength	*Eme*
3210	2.039e+6	77.8	700×77.8=54462



(a) Brick-infilled steel frames specimen (Tasnimi and Mohebkhah 2011)



(b) Applied displacement history



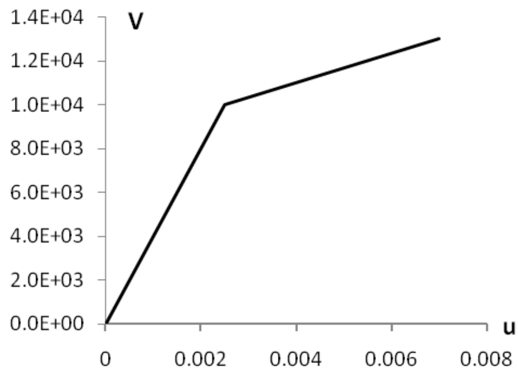
(c) Load-displacement relation for the specimen

Fig. 3 Evaluating modeling of the URM infill panels

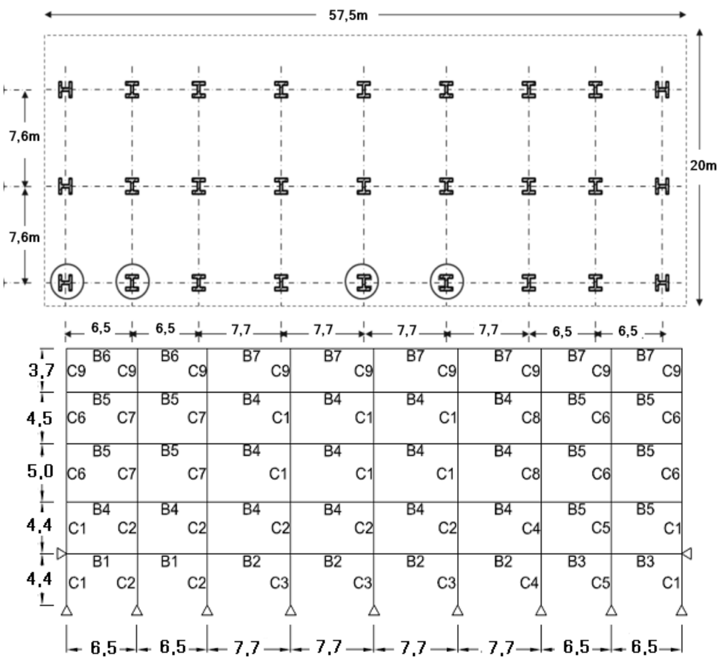
and finally, load-displacement relation was obtained and compared with the experimental test curve (see Fig. 3(c)).

Fig. 3(c) shows acceptable compliance between experimental curve and numerical model used in this research.

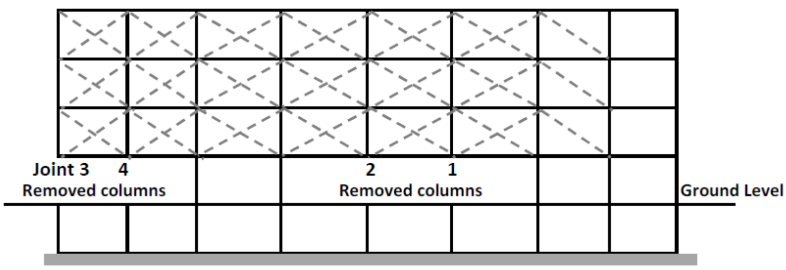
(a) The Ohio Student Union Building after columns removal



(b) monotonic lateral force-deformation bilinear curve for URM infill panels



(c) Columns and beams plan and location of removal columns



(d) Location of removal columns and equivalent struts in elevation view

Fig. 4 The Ohio Union Building test after the removal of four columns (Song and Sezen 2009)

Table 2 Column and beam sections of the Ohio Union building

Column section		Beam section	
Number	Type	Number	Type
C1	10 WF 72	B1	24 B 76
C2	12 WF 133	B2	21 B 68
C3	12 WF 120	B3	16 B 58
C4	10 WF 100	B4	21 WF 62
C5	10 WF 89	B5	18 WF 50
C6	10 WF 54	B6	14 B 17.2
C7	10 WF 112	B7	14 B 22
C8	10 WF 60	B8	24 WF 76
C9	10 WF 33	B9	18 WF 45

In-situ full-scale case study on progressive collapse in steel building which was done by Song and Sezen (2009) is selected to verify the proposed model (see Fig. 4). In that study, the five stories Ohio Student Union Building, (Columbus, Ohio), was tested by removing four columns of the ground floor from one of the long perimeter frames prior to the building's scheduled demolition (Fig. 4(c), (d)). The structural system of the building consisted of steel moment resisting frames in both principal directions. The beams and columns sections indicated in Table 2. Letters WF and B designated wide-flange shaped I-section and light I-section, respectively, based on the ASCE definitions. This building had one underground story and four stories above ground level. Dimensions of the building were 57.5×20.0 m^2 with 18.5 m height from the ground level (four stories high with a full basement).

Song and Sezen (2009) revealed that the building did not collapse and satisfied the GSA criteria, even after the removal of the fourth column.

In this paper, the Union building is modeled by frame elements. Yield strength and modulus of elasticity of the steel frame members are equal to 3500 kg/cm^2 and 2.0e6 kg/cm^2, respectively, as

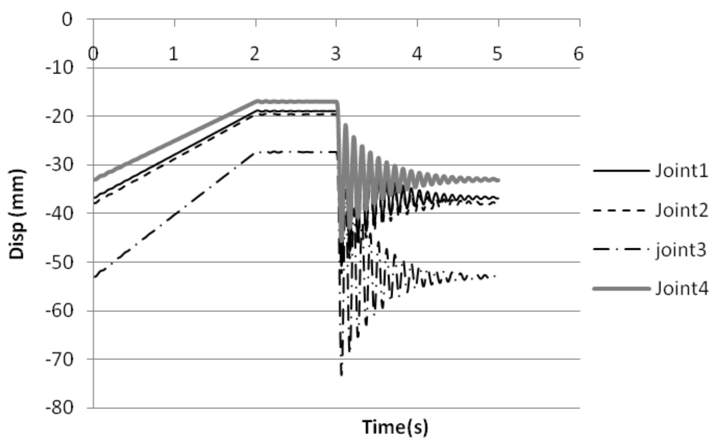


Fig. 5 Time-history of displacement of the four removed columns top joints

Table 3 comparison of the results of numerical analysis and test

Position	$Disp._{max}$ (mm)	
	Test	Analysis (Permanent)
Joint1	36.6	37.0
Joint2	37.1	38.0
Joint3	50.8	52.8
Joint4	33.8	33.3

specified in the original design. According to reports, URM infill panels and composite slabs are assigned to the structure. Slabs are modeled using shell elements with 10cm thickness.

Other modeling assumptions are:

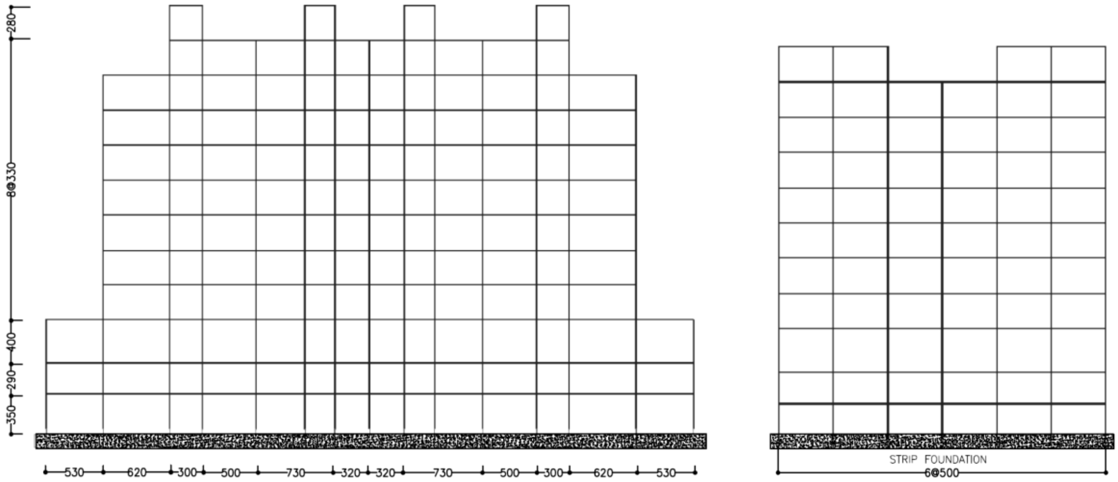
- Live load is considered equal to zero.
- Thickness of infill panels is equal to 22 cm. Monotonic lateral force-deformation bilinear curve is obtained and indicated in Fig. 4(b).
- Opening of all effective infill panels was neglected.

Initial analysis shows that stiffness of the building increases due to considering effects of slabs and URM infill panels. Thus, after static stage of the analysis, no joint experiences plastic rotation. Therefore, linear increasing of equivalent loads and suddenly removal of them is utilized. Nonlinear time-history of displacement of the four removed columns joints are indicated in Fig. 5. Also Table 3 shows final displacement of top joints of the four removed columns in both test and analysis results. Good comparison is shown between test and numerical studies. It is shows that the building does not collapse after this removal scenario and the building destroyed based on its scheduled demolition.

6. Case study

In this paper, existing 11 stories building includes two parking, one commercial, eight residential and no underground stories with bolted steel structure is studied. The structure includes 79 columns in parking stories and 77 columns in the rest floors (see Fig. 6). The building's designer has utilized H-shape irregular plan to provide more lighting area, better rooms' arrangement and view of it (Fig. 7). These kinds of architectural plans can limit distribution of any local failure in damaged area to other parts of structure due to its irregularity. Therefore, they might be mitigating progressive collapse. The structure utilizes special moment resistant frames in both directions to resist earthquake lateral loading based on the IBC code. Design of structure is according to the AISC ASD-2001 considering P-Δ effects and Special Moment Frames (SMF) additional design requirements. Weak beam-strong column theory is used. As known, based on this theory, the sum of column flexure strengths at a joint should be more than the sum of beam flexure strengths. The column flexural strength should reflect the presence of axial force present in the column. The beam flexural strength should reflect potential increase in capacity for strain hardening to facilitate the review of the strong column-weak beam criterion.

The structure comprises prefabricated welded tubes as tree columns and I-shape plate girder beams which are connected with 8.8 graded bolts. All columns were fixed to the strip foundation. Composite 8 cm thick concrete slab with 1m spaced #16 castellated beam and UNP-profile as



(a) Front view (b) side view

Fig. 6 Front and side views of the selected structure (in model)

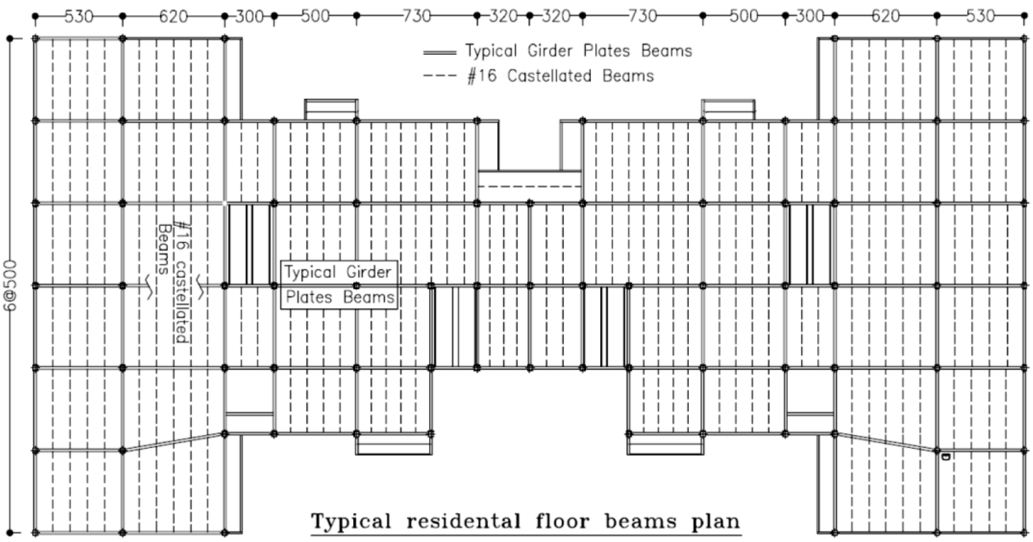


Fig. 7 Floors plan of the structure

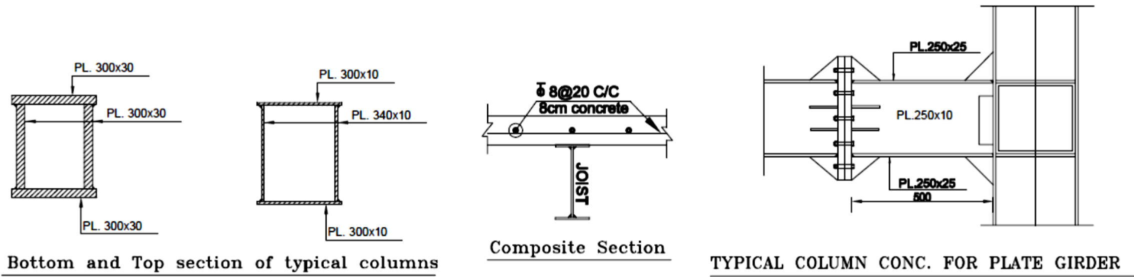


Fig. 8 Some sketch of the structural elements

Table 4 Properties of the frame sections

Position	Section
Columns(From base floor to roof elevation)	Bo×360×300×30
	Bo×360×300×25
	Bo×360×300×20
	Bo×360×300×15
	Bo×360×300×12
	Bo×360×300×10
	Bo×270×250×10

Position	Length× thickness (mm)	
	web	flange
Girder Beams (I shape weld-fabricated)	280×6	200×15
	290×8	200×20
	290×8	250×20
	300×10	250×25
	400×12	250×15
	274×6	200×12
	270×6	200×10
	400×8	120×15
	270×6	150×10
	270×6	120×10

Table 5 Measures of the applied load cases

Load case	with slab	Without slab
Dead load, partitions, finishing (kg/m^2)	450	642
Live load (kg/m^2)	200	200
Peripheral walls load considering 30% openings (kg/m)	250	250

* Snow load was neglected. Steel structure and concrete slabs are considered self-weighted in the model.

shear keys are used in the structure. It is used URM infill panels as walls and partitions and common residential finishing (see Fig. 8 and Table 4).

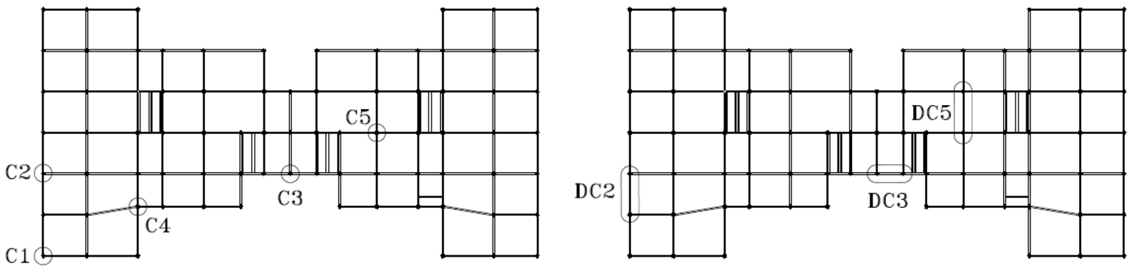
Common Steel profiles St-37 with yield strength, ultimate tension strength and modulus of elasticity equal to $F_y = 2400\ kg/cm^2$,$F_u = 3600\ kg/cm^2$ and $E_s = 2.1e6\ kg/cm^2$were used.

6.1 Loading

As mentioned before, the structure is designed under earthquake lateral loading based on the IBC code, but it is assumed that after missing element(s), wind and earthquake loads are not applied because of low probability of occurrence in removal time. Also it is logical to assume that if an element failed under earthquake loading, duration of this load is short enough to effect after failure. Therefore, according to the BHRC Codes, No. 6: loading on the building, following load cases are applied to the structure.

6.2 Removal scenarios

According to the GSA and UFC codes recommendations and as it was described in section (3), removal scenarios are applied for critical columns in each floor level. There are five critical columns locations and 11 stories, therefore 55 analyses shall be performed. If bridging over cannot be demonstrated for one of the removed load-bearing elements, the structure must be re-designed or retrofitted to increase the bridging capacity and all similar columns must be retrofitted.



(a) Single column (b) double columns

Fig. 9 Columns removal scenarios

Table 6 parameters of equivalent compression struts

f'_m	t_{inf}	r_{inf}	Θ	E_{me}	E_{fe}	h_{inf}	h_{col}	f'_{me90}	α	ε_m
4142 kPa	0.22 m	5.42 m	1.03 rad	2278 MPa	200 GPa	2.7 m	3.0 m	2692 kPa	0.2*	0.001

*Mostafaei *et al*. (2004)

In this study, removal scenarios based on the AP method are applied just for critical columns of ground floor. Initial analyses results based on the single column removal scenarios (C) show that the structure is strength enough to experience plastic hinges. Therefore, double columns removal scenarios (DC) are investigated too (see Fig. 9). Eight removal scenarios are applied to the structure analysis. Columns are removed at the corner (C1), center of small side of the structure (C2), center of large side (C3), concave corner (C4) or interior of the plan (C5) as indicated in Fig. 9(a). Scenarios of double columns removal are indexed with DCi in Fig. 8(b). Twenty four analyses are performed consist of nonlinear static and dynamic analyses with and without considering effects of slabs and URM infill panels. Finally, one NLD analysis considering both effects and based on DC2 removal scenario is applied to the structure.

6.3 URM infill panels modeling

Various analyses with different removal scenarios were done to determine efficiency of far infill panels from damaged in analysis results. It has been mentioned that just the panels adjacent to the missed column(s) are effective. Therefore these panels are modeled as equivalent struts.

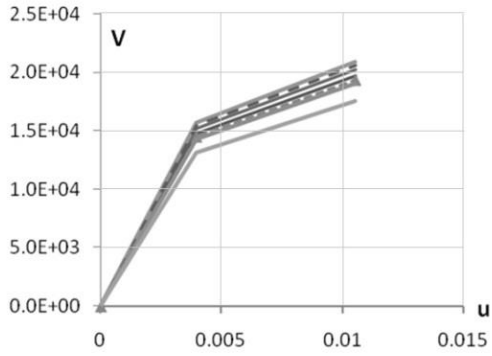
Following assumption are applied to the URM infill panels modeling:

- Effective panels have no opening (such as window and door).
- Strength of beam and column members adjacent to infill panels meet FEMA-356 acceptance criteria.
- Equivalent compression struts are modeled with beam elements with no tension capacity.

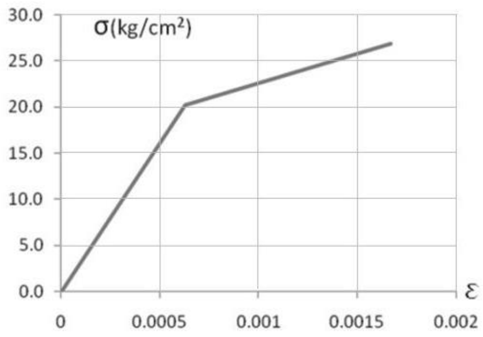
Calculated parameters in according with section (4.1) for infill panels are indicated in Table 6.

Based on the Table 6 and Eqs. (5) to (7), monotonic lateral force-deformation bilinear curve for the considered structural URM infill panels are obtained and indicated in Fig. 10(a).

It is shown that almost the curves of all infill panels are uniform because of similar geometric of them (span, height, thickness etc.). Thus normalized strain-stress curve which is shown in Fig. 10(b) are applied for all effective panels in the structural model.



(a)



(b)

Fig. 10 (a) monotonic lateral force-deformation bilinear curve for URM infill panels, (b) normalized stress-strain curve

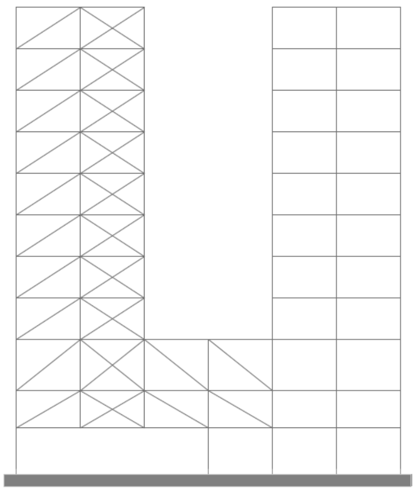


Fig. 11 modeling of the equivalent compression struts (DC2)

Fig. 11 indicates struts modeling in the DC2 scenario as an example based on initial analyses. The equivalent struts are modeled like bracing in the structure at all vicinity spans of the damaged area, unless designer predicts deformation form of the spans. Therefore, elements with tension internal forces are not modeled to optimized simulation.

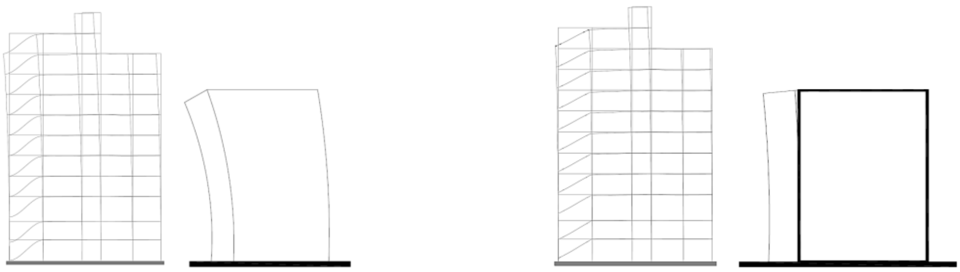
7. Results

Based on the section (2), the NLS and NLD methods are applied to evaluate progressive collapsing potential of the structure. The results obtained from 25 analyses are categorized based on the considering effects of slab stiffness, infill panels, removal scenarios and the analysis types. To this end, observations are classified in two main categories; single column and double columns removal scenarios.

Table 7 The results of single column removal scenarios

Analysis		Considering		d_s (cm)	Roof Drift		Axial force (ton)				
							Column			Beam	
Name	Type	Slab	Wall		x	y	Near	Far	Top	Small span	Large span
C1	NLS	No	No	19.00	-9.38	-6.07	-151.70	-131.00	-2.04	3.58	3.41
		Yes	No	18.22	-0.20	-1.31	-200.00	-225.00	-0.20	15.53	17.32
	NLD*	Yes	No	4.23	-0.24	-0.60	-169.20	-186.30	11.30	-0.30	-0.16
C2	NLS	No	No	22.92	-1.51	7.72	-227.50	-335.20	-4.15	11.62	12.04
		Yes	No	13.77	-0.95	2.40	-318.90	-477.60	-4.06	12.63	15.26
	NLD*	Yes	No	3.95	-0.60	1.37	-168.30	-245.80	38.40	-0.09	-1.20
C3	NLS	No	No	2.13	-	-	-135.10	-177.50	-2.45	1.55	1.94
		Yes	No	2.10	-	-	-155.03	-209.20	-2.36	1.24	0.08
	NLD*	Yes	No	1.14	-	-	-94.20	-126.20	-3.21	0.11	0.20
C4	NLS	No	No	13.84	-	-	-255.30	-491.50	-6.68	18.20	7.53
		Yes	No	8.13	-	-	-353.10	-573.90	-6.02	17.26	11.34
	NLD*	Yes	No	2.2	-	-	-180.00	-245.60	-4.80	0.51	0.21
C5	NLS	No	No	17.84	-	-	-270.00	-334.50	-0.90	22.44	8.63
		Yes	No	5.00	-	-	-289.10	-371.60	-1.33	6.13	3.93
	NLD*	Yes	No	2.83	-	-	-186.50	-225.80	-5.45	0.20	0.30

*: Final state are submitted



(a) without slab (b) with slab

Fig. 12 deformed shape of the structure in C1-NLS method (scale factor: 15)

7.1 Single column removal scenarios

Table 7 and Figs. 12 to 14 indicate required parameters to compare results of the different analyses with single column removal scenarios.

Following conclusions could be derivate.

- Considering effects of slab in the NLS method decreases almost 50% of the vertical displacement of top joint of the missed column (d_s), but it depends on the location of element removal. As the number of involved slabs increase, d_s decreases. Besides, variations of d_s in one-way slab systems depend on the direction of their joists in position. Therefore, arrangement of joist directions can be a solution to decrease loading of a critical column.

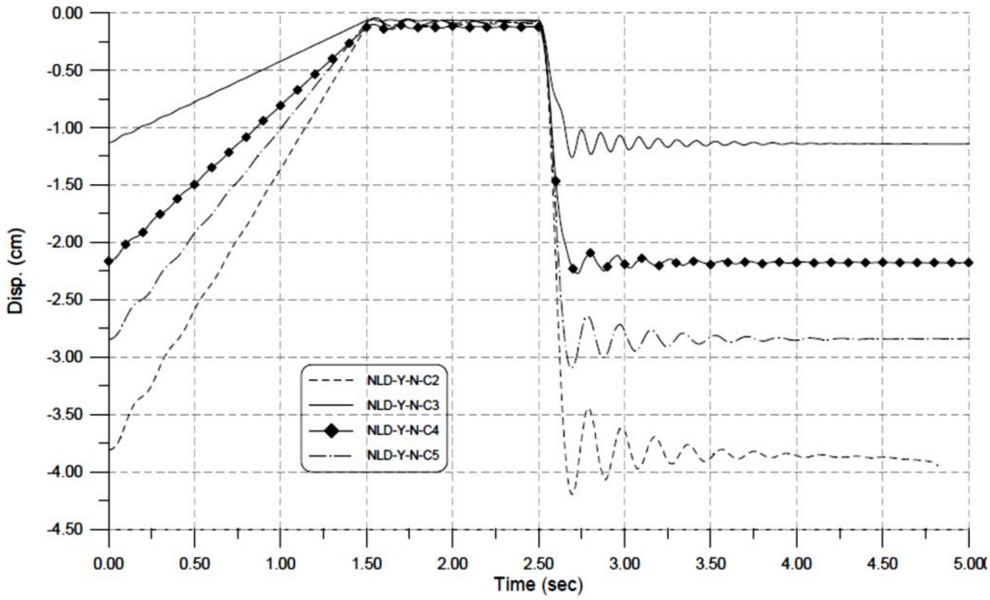


Fig. 13 time-history diagram of d_s in C scenarios

- Limitation of the roof drift based on the seismic design codes is equal to $\bar{\Delta}_M \leq 0.005H$ in operation level (IBC). It is obtained 18 cm for this structure. Comparing $\bar{\Delta}_M$ with roof drifts in Table 7 shows that based on the removal scenario and the plan's irregularity, checking the roof drift in progressive collapse analyses is necessary.
- As indicated in Fig. 12, modeling the slabs increase the structure lateral stiffness through forming the rigid diaphragms, thus the rest of undamaged structure almost behave as a rigid frame and support damaged region. Although this behavior forms more developed plastic hinges in the damaged area (see Fig. 14; compare NLS-Y-N-C1 with NLS-N-N-C1).
- As indicated in the C1-NLS cases (Table 7), the critical column position changes from near column to the far one due to load distribution path in on-way slab system. Therefore, one can predict critical column(s) just based on considering slab and joists direction in this system.
- Axial forces of beams in the NLD analyses are negligible. It is logical that small d_s causes little tension forces in beams.
- Based on removal scenario, d_s decreases 50% to 85% in the NLD analyses than the NLS analyses through considering effects of slabs.
- Maximum d_s, occurs in case C2 (see Fig. 13). Close spaces of columns in case C3 cause the minimum d_s. It is notable that evaluating the most critical scenario should be based on crosschecking of all parameters include d_s and plastic hinges.
- Peering to the Fig. 14, show that total number of the plastic hinges increase if the number of involved spans and columns space increase and it decreases when slabs are modeled.
- According to the location of the plastic hinges indicated in Fig. 14, weak beam-strong column approach was observed by designer for this structure.
- As indicated in Fig. 15, beams in the damaged area experience new boundary conditions. Before column demolition, fixed end of the one-span beam bears negative moment. Afterwards, large positive moment and deformation achieve in the middle of new two-span

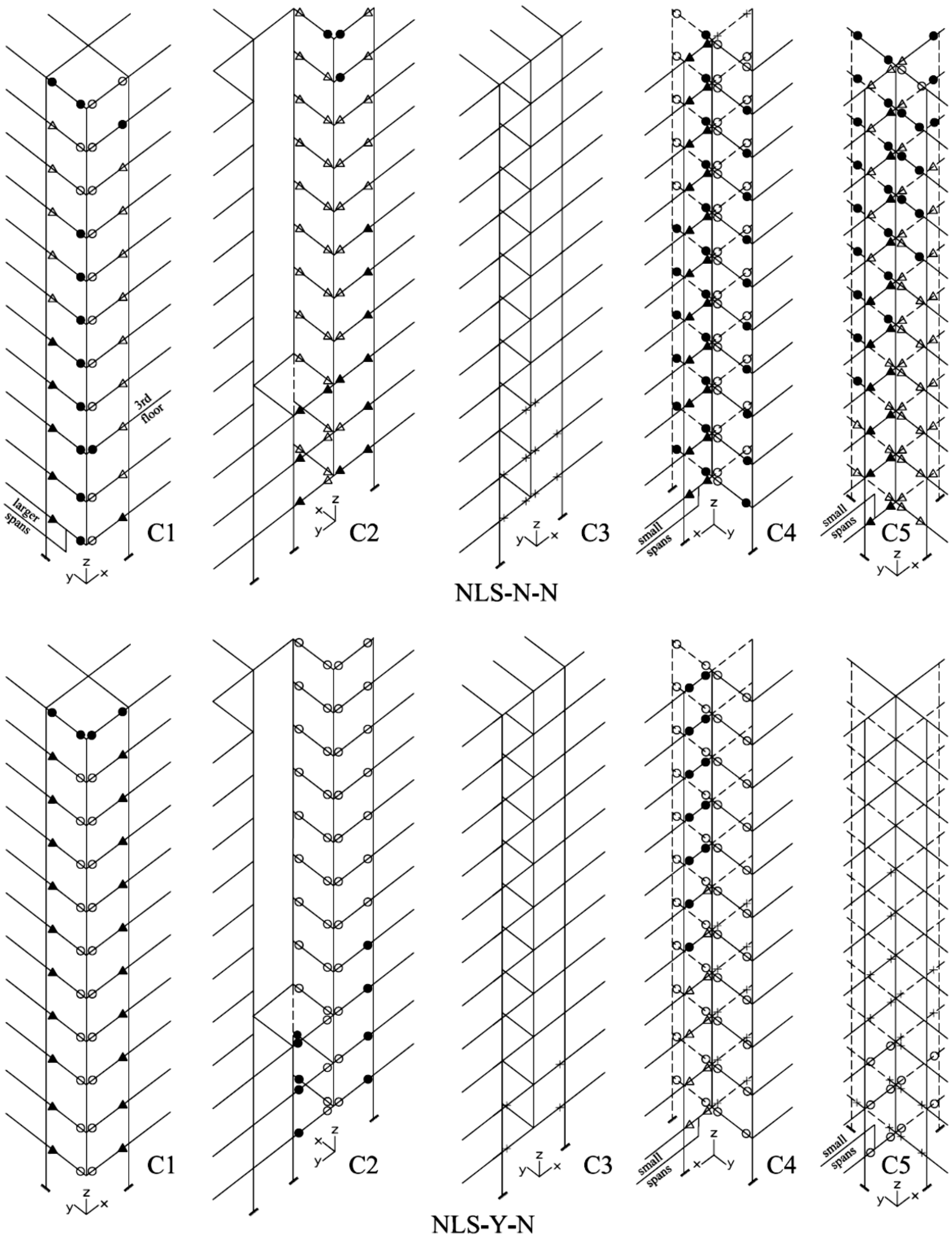


Fig. 14 hinges formation in single column removal scenarios

beam. Here main change in load redistribution path and vertical resisting system occurs in the beam to bridging over the missed column(s). Therefore, the considered beams must be designed or analyze based on laterally unsupported length equal to two spans. Aforementioned note was base to inventing Side Plate Technique.

Table 8 The results of double columns removal scenarios

Analysis		Considering		d_s (cm)		Roof Drift	
Name	Type	Slab	Wall	Joint 1	Joint 2	*x*	*y*
DC2	NLS	No	No	21.90	24.50	-6.50	13.23
		Yes	No	19.15	21.00	-4.44	4.58
	NLD*	Yes	No	8.85	10.26	-2.83	3.02
		Yes	Yes	1.26	1.46	-2.00	1.80
DC3	NLS	No	No	11.20	14.00	-8.60	-8.80
		Yes	No	4.00	4.91	-1.60	-1.60
	NLD*	Yes	No	1.67	2.07	-0.15	-0.33
DC5	NLS	No	No	17.02	17.92	-	-
		Yes	No	4.41	5.41	-	-
	NLD*	Yes	No	3.34	4.84	-	-

*: Final state are submitted

- As a result we can say pinned beams are very vulnerable in progressive collapse because there is no moment resistant facility at the end of them.

7.2 Double column removal scenarios

Table 8 and Figs. 15 and 16 indicate required parameters to compare results of the different analyses with double columns removal scenarios.

Following conclusions could be derivate.

- Based on Table 8, displacements of the columns' top joints in DC2-NLS are almost similar to C2-NLS due to the columns position, structural system and catenary action of the beams and slabs.
- As obtained in the section 7.1 and based on Table 8, considering effects of slabs in the NLS method decrease almost 60% of the vertical displacements of top joints of the missed columns but it depends on the location of removed elements.
- Roof drift in DC2-NLS is closer to the criteria ($\bar{\Delta}_M \leq 0.005H$ =18 cm) than the other cases.
- Depends on the removal scenarios, d_s decreases 60% to 85% in the NLD analyses than the NLS analyses. Also d_s decrease 95% considering effects of the URM infill panels and slabs. This result matches by full-scale in-situ tests done by the other researchers.
- As indicated in Fig. 15, applying the NLD analyses and modeling the slabs decrease total number of the plastic hinges.
- Modeling the URM infill panels in the structure increases axial loads in vicinity columns because they act as braced frames.

As mentioned before, compression loads are applied to the URM infill panels in the damaged area due to generated vertical displacements. These large deformations are occurred when the structure tends to settle in damaged area due to columns demolitions. It is notable that generated two-span beam which bridging over the removed column, rests on the infill panels until the panels fail. But it is not consider in modeling when the infills simulate as equivalent strut, thus it is one of the drawbacks of this simulation. Though, the equivalent struts bear new distributed loads until

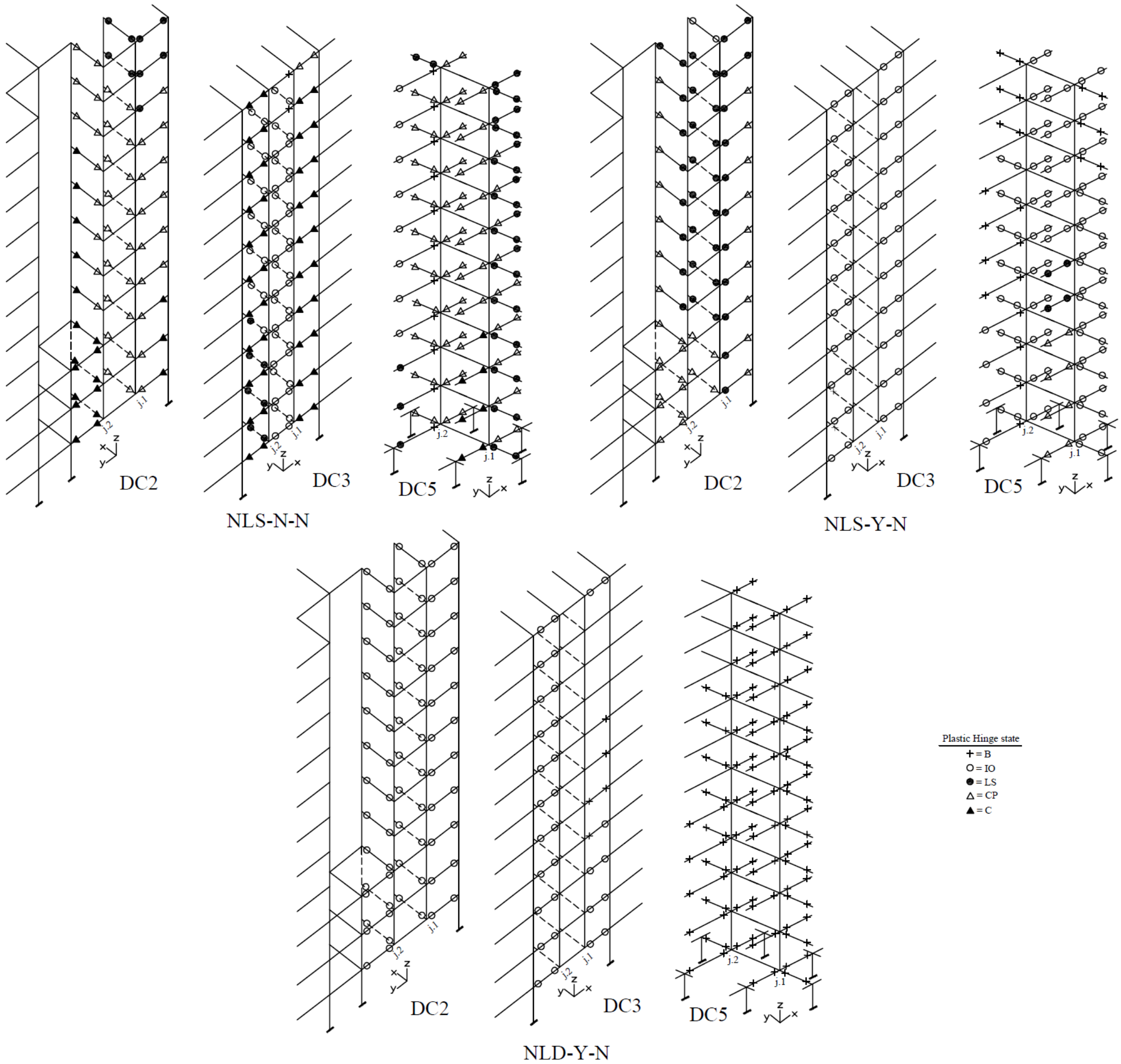


Fig. 15 hinges formation in double columns removal scenarios

their failure. Failure of any strut causes new load bearing path. These dynamic relocations of the distributed loads between the struts cause shifts in the axis of the structure oscillation until equilibrium state of it. In this study, three oscillation steps are observed due to consecutive failure of the compression struts. Time-history diagrams of top joints displacements of the missed columns (d_s), in the NLD-Y-Y-DC2 scenario are indicated in Fig. 16.

Finally, to evaluate status of the structure (STs) after the elements collapse, quantities and qualities of the hinges shall be considered to precise participation of hinges in failure rate of the structure. Obviously locations of the hinges have different effect on STs. As an example, full plastic hinge which formed in ground floor column is more serious than hinge in roof beam. Therefore, a formula named hazard factor (H_f) is proposed to evaluate vulnerability of the structure

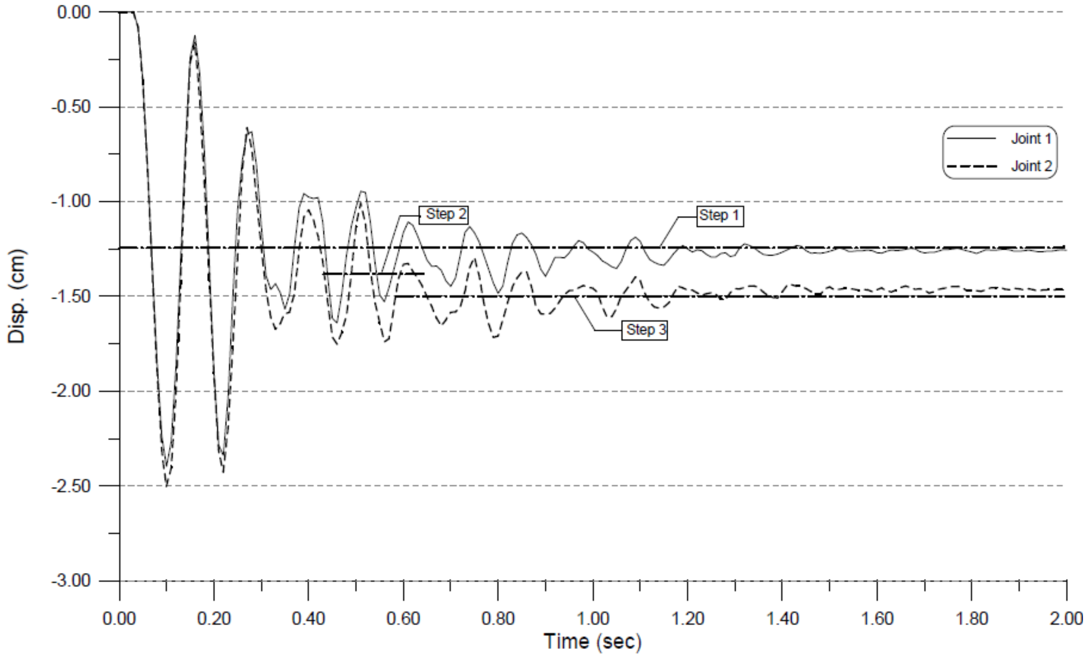


Fig. 16 time-history diagrams of d_s in NLD-Y-Y-DC2 scenario

under progressive collapse. More researches should be implemented to evaluate this proposed formula.

$$H_f = \sum(\prod_{i=1}^{3}\lambda_i)h_B + \sum(\prod_{i=1}^{3}\lambda_i)h_{IO} + \sum(\prod_{i=1}^{3}\lambda_i)h_{LS} + \sum(\prod_{i=1}^{3}\lambda_i)h_{CP} + \sum(\prod_{i=1}^{3}\lambda_i)h_C \quad (13)$$

Where h_{index} are the hinges with "index" status based on Fig. 4.

$$\lambda_1 = \text{hinge importance factor} = \begin{cases} 2 & ;\text{ if the hinge has located in column} \\ 1 & ;\text{ if the hinge has located in beam} \end{cases}$$

$$\lambda_2 = story\ factor = 1 - \frac{story\ of\ \ the\ hinge\ from\ base\ level}{total\ stories}$$

$$\lambda_3 = hinge\ status\ factor = \begin{cases} 1 & ;\text{"B" status of hinge based on Fig.4} \\ 2 & ;\text{"IO"} \\ 3 & ;\text{"LS"} \\ 4 & ;\text{"CP"} \\ 5 & ;\text{"C"} \end{cases}$$

Fig. 17 shows H_f diagram for different analyses of the structure. It is concluded that in C1-NLS, C2-NLS, C5-NLS-N-N and DC3-NLS-Y-N scenarios, total number of the plastic hinges are approximately equal but based on the H_f values, different probability of collapse are predicted for

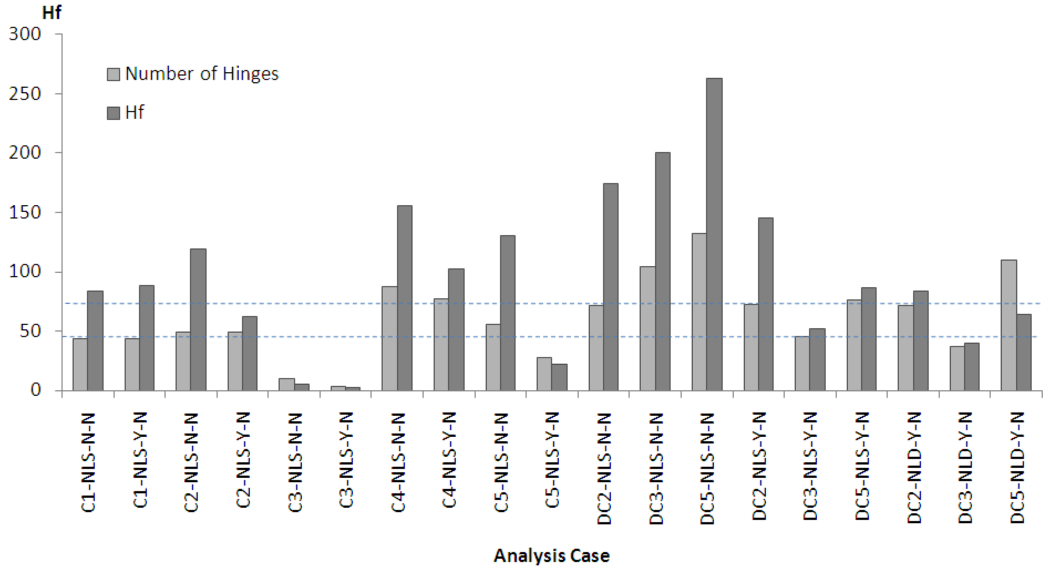


Fig. 17 hazard factor (H_f) for different analysis case

each of them and case C2-NLS-N-N is the most critical. This conclusion can be extended to C4-NLS, DC2-NLS, DC2-NLD and DC5-NLS-Y-D, scenarios in which the case DC2-NLS-N-N is the most critical. It is observed that C4-NLS-N-N (removal of column located in concave corner of the structure) and DC5-NLS-N-N (removal of two interior columns) scenarios are the most hazardous.

Comparing C1-NLS-N-N and C1-NLS-Y-N cases and other pair of analyses indicate that considering effect of slabs usually decrease total number of hinge and H_f values based on location of the elements removal and geometric of the structure. But considering effects of URM infill panels and slabs together realize progressive collapse analyzing and cause significant reduction in hinges qualities and quantities.

8. Conclusions

In this study, existing seismically code-designed steel building with bolted connections was analyzed with the alternate path method (AP) to assess the building resistant against progressive collapse. In this method time-history function named static equilibrium was used to model suddenly removal of column(s). Effects of the infill panels and the concrete slabs of the desired building were simulated by the analytical macro-model based on the equivalent strut approach and shell elements with membrane acting in rigid diaphragms, respectively. The nonlinear static and dynamic analyses were applied to the structure. Initial analyses results based on the single column removal scenarios (C) showed that the structure was strength enough to experience plastic hinges. Therefore, double columns removal scenarios (DC) were investigated too.

Results indicate that the slabs and infill panels increase the stability of the structure to resist progressive collapse even if more than one column removed. The simulation reveal that stiffness of the slabs and infill panels decrease deformations, state and number of plastic hinges, lateral drifts, internal forces in elements and increase damping ratio and resistant of the structure to mitigate progressive collapse. Finally, it is concluded that considering in-plane resistant of the walls and out-plane resistant of the slabs, realize response of the structure to progressive collapse and cause high reduction in hyper displacements which are observed in simple analyses.

Also, in this research, a formula was proposed to determine potential of global collapse of the structure named hazard factor. It reveals that locations and statuses of the plastic hinges are important to evaluate status of the structure after the elements collapse. If the hazard factor be increased then vulnerability of the structure will be increased under progressive collapse phenomenon.

References

AISC ASD (2001), “Specification for Structural Steel Buildings”, American Institute of Steel Construction.

ASCE 7-05 (2005), “Minimum design loads for buildings and other structures”, American Society of Civil Engineers.

Astaneh-Asl, E.A., Madsen, C., Noble, R., Jung, D., McCallen, M.S., Hoehler, W., Li and Hwa, R. (2002), “Use of catenary cables to prevent progressive collapse of building”, Report Number UCB/CEE-Steel-2001/02, Dept. of Civil and Env., Univ. of Calif., Berkeley.

BHRC, No6, Iranian Building and housing Research Center, Loading on the buildings.

FEMA 356 (2000), *Prestandard and commentary for the seismic rehabilitation of buildings*, Federal Emergency Management Agency,Washington, DC.

Gross, J.L. and McGuire, W. (1983), “Progressive collapse resistant design”, *J. Struct. Eng.*, **109**(1), 1-15.

GSA (2003), *Progressive collapse analysis and design guidelines for new federal office buildings and major modernization projects*, US General Services Administration, Washington, DC.

Hariri-Ardebili, M.A., Rahmani Samani, H. and Mirtaheri, M. (2014), “Free and rorced vibration analysis of an infilled steel frame: experimental, numerical, and analytical methods”, *Shock Vib*., doi:10.1155/2014/439591

Hayes, J.R. Jr., Woodson, S.C., Pekelnicky, R.G., Poland, C.D., Corley, W.G. and Sozen, M. (2005), “Can Strengthening for Earthquake Improve Blast and Progressive Collapse Resistance?”, *ASCE J. Struct. Eng.*, **131**(8), 1157-1177.

IBC (2006), *International Building Code*, International Code Council, Washington, DC.

Kaewkulchai, G. and Williamson, E.B. (2004), “Beam element formulation and solution procedure for dynamic progressive collapse analysis”, *Comput. Struct.*, **82**, 639-651.

Kaewkulchai, G. and Williamson, E.B. (2006), “Modeling the impact of failed members for progressive collapse analysis of frame structures”, *ASCE J. Perform. Constr. Facil.*, **20**(4), 375-383.

Kheyroddin, A., Gerami, M. and Mehrabi, F. (2014), “Assessment of the dynamic effect of steel frame due to sudden middle column loss”, *Struct. Des. Tall Spec. Build*., **23**, 390-402.

Kim, J.K. and An, D.W. (2009), “Evaluation of progressive collapse potential of steel moment frames considering catenary action”, *Struct. Des. Tall Spec. Build.*, **18**, 455-465.

Madan, A., Reinhorn, A.M., Mander, J.B. and Valles, R.E. (1997), “Modeling of masonry infill panels for structural analysis”, *J. Struct. Eng.*, **123**, 1295-1302

Marjanishvili, S. and Agnew, E. (2006), “Comparison of various procedures for progressive collapse analysis”, *J. Perform. Constr. Facil.*, **20**(4), 356-374.

Marjanishvili, S.M. (2004), “Progressive analysis procedure for progressive collapse”, *J. Perform. Constr. Facil.*, **18**(2), 79-85.

Mirtaheri, M. and Abbasi Zoghi, M. (2016), “Design guides to resist progressive collapse for steel structures”, *Steel Compos. Struct.*, **20**(2), 357-378.

Mostafaei, H. and Kabeyasawa, T. (2004), “Effect of infill masonry walls on the seismic response of

reinforced concrete buildings subjected to the 2003 Bam earthquake strong motion: a case study of Bam telephone center", Earthquake Research Institute, The University of Tokyo.
NISTIR 7396 (2007), *Best Practices for Reducing the Potential for Progressive Collapse in Buildings*, National Institute of Standards and Technology, U.S. Department of Commerce.
PEER (2005), Open System for Earthquake Engineering (OpenSees), Univ. of California.
Powell, G. (2005), "Progressive collapse: case studies using nonlinear analysis", *Proceedings of ASCE 2005 Structures Congress: Metropolis and Beyond*, New York.
Sasani, M. and Sagiroglu, S. (2008), "Progressive collapse resistance of Hotel San Diego", *ASCE J. Struct. Eng.*, **134**(3), 478-488.
Sasani, M., Bazan, M. and Sagiroglu, S. (2007), "Experimental and analytical progressive collapse evaluation of actual reinforced concrete structure", *ACI J. Struct. Eng.*, **104**(6), 731-739.
Sattar, S. (2013), "Influence of masonry infill walls and other building characteristics on seismic collapse of concrete frame buildings", Ph.D. Thesis, University of Colorado, Boulder, USA.
Song, B.I. and Sezen, H. (2009), "Evaluation of an existing steel frame building against progressive collapse", *ASCE Structures 2009 Congress*, Austin, Texas, U.S.A.
Tasnimi, A.A and Mohebkhah, A, (2011), "Investigation on the behavior of brick-infilled steel frames with openings, experimental and analytical approaches", *Eng. Struct.*, **33**, 968-980.
Tsitos, A. and Mosqueda, G. (2010), "Experimental investigation of progressive collapse of conventional, and post-tensioned steel frames", *14ECEE*, Ohio.
Tsitos, A., Mosqueda, G., Filiatrault, A. and Reinhorn, A.M. (2008), "Experimental investigation of progressive collapse of steel frames under Multi-Hazard Extreme loading", *14th World Conference on Earthquake Engineering*, Beijing, China.
UFC 4-023-03 (2010), *Design of buildings to resist progressive collapse, Unified Facilities Criteria*, Dept. of Defense, Washington, DC.

CC