Steel and Composite Structures, *Vol. 26, No. 5 (2018) 635-647*
DOI: https://doi.org/10.12989/scs.2018.26.5.635

Cap truss and steel strut to resist progressive collapse in RC frame structures

Seyed Mehdi Zahrai [*1] and Alireza Ezoddin [2a]

[1] *Center of Excellence for Engineering and Management of Infrastructures,*
School of Civil Engineering, College of Engineering, The University of Tehran, Tehran, Iran
[2] *Faculty of Civil Engineering, Semnan University, Semnan, Iran and lecturer,*
Department of Civil Engineering, Semnan Branch, Technical and Vocational University (TVU), Semnan, Iran

(*Received May 20, 2017, Revised December 12, 2017, Accepted December 25, 2017*)

Abstract. In order to improve the efficiency of the Reinforced Concrete, RC, structures against progressive collapse, this paper proposes a procedure using alternate path and specific local resistance method to resist progressive collapse in intermediate RC frame structures. Cap truss consists of multiple trusses above a suddenly removed structural element to restrain excessive collapse and provide an alternate path. Steel strut is used as a brace to resist compressive axial forces. It is similar to knee braces in the geometry, responsible for enhancing ductility and preventing shear force localization around the column. In this paper, column removals in the critical position at the first story of two 5 and 10-story regular buildings strengthened using steel strut or cap truss are studied. Based on nonlinear dynamic analysis results, steel strut can only decrease vertical displacement due to sudden removal of the column at the first story about 23%. Cap truss can reduce the average vertical displacement and column axial force transferred to adjacent columns for the studied buildings about 56% and 61%, respectively due to sudden removal of the column. In other words, using cap truss, the axial force in the removed column transfers through an alternate path to adjacent columns to prevent local or general failure or to delay the progressive collapse occurrence.

Keywords: progressive collapse; reinforced concrete (RC) frame; alternate load path method; steel strut; cap truss; non-linear dynamic analysis

1. Introduction

Progressive collapse is a dynamic and nonlinear event, as it happens in a very short period and imposes large nonlinear deformation to elements (beam, column and wall) before failure. Initial damage can occur due to various events such as vehicle impact, explosion, and fire. In all cases, behavior of structure is investigated under the gravity load effect after removing the damaged element. Disregarding the potential for progressive collapse in structural design is associated with such catastrophic events as the collapse of the Ronan Point (London city 1968) a gas explosion occurred in an apartment on the 18th floor of a 22-story precast concrete building at Ronan Point in England. At least three people were killed as a result. On April 19, 1995, when a truck loaded with explosives parked outside the Alfred P. Murrah federal building in Oklahoma City exploded, causing collapse of a large portion of the 9-story building, as well as damage to adjacent buildings in the complex, resulting in 168 casualties. On September 11, 2001, as a part of a larger terrorist plan, two planes crashed into the World Trade Center towers (Longinow and Mniszewski 1996, US Army Corps of Engineers 1999, Corley *et al*. 1998, NIST 2007). The attacks on the World Trade Center killed 2753 people. This disaster raised public awareness of progressive collapse issues, and the extent to which non-governmental buildings must be designed to resist accidental loading is being debated in the structural engineering profession.

Izzuddin *et al*. (2008) and Vlassis *et al*. (2008) proposed a novel simplified framework for progressive collapse assessment of buildings, considering a sudden column loss. This approach analyses the nonlinear static response with dynamic effects evaluated in a simple method, suggesting a practical method for assessing the structural robustness at various levels of structural idealization. Sasani and Kropelnicki (2008) studied the response of a 7-story Reinforced Concrete, RC structure following the loss of load-bearing elements, using the alternative path method. The results showed that in spite of tensile reinforcement fracture of beam bottom reinforcement, the beam has significant remaining strength and deformation capacity. Kim and Kim (2009) investigated the reinforcing effect of the panel zone on the progressive collapse studying the capacity of moment resisting steel frames with the aid of nonlinear dynamic analysis. The analysis results showed that the panel zone deformation is highly dependent on the location of removed columns.

Vlassis *et al*. (2009) offered a design oriented methodology for progressive collapse assessment of multi-story buildings subjected to impact from an above failed floor. The proposed assessment framework comprises of three main stages, including: determination of the nonlinear static response due to impacted floor system, dynamic assessment using a novel simplified approach, and ductility

*Corresponding author, Professor,
E-mail: mzahrai@ut.ac.ir
[a] Ph.D. Candidate

http://www.techno-press.org/?journal=scs&subpage=6
ISSN: 1229-9367 (Print), 1598-6233 (Online)

assessment at the maximum level of dynamic deformation attained upon impact. The offered method was applied to analyze the progressive collapse of a typical 7-storey steel-framed composite office building with the impact of a floor plate. Shi *et al*. (2010) proposed a new method for progressive collapse analysis of RC frames by considering initial given conditions and initial damage to adjacent structural members under blast loading. For evaluation, the proposed method was used for a 3-story two-span RC frame. The results showed that the proposed method gives better predictions of the structural progressive collapse with minimum additional effort in determining the initial conditions and damage in structural members at the end of the blast loading stage when progressive collapse in the building starts. Kim and Park (2010) evaluated the progressive collapse resisting capacity of building structures with outrigger trusses using nonlinear static and dynamic analyses. For evaluating, two types of 36-story structures composed of RC core walls and perimeter frames connected by outrigger trusses at the top were prepared. The results showed that the dynamic amplification factor of 2.0 provided reasonably conservative results. Wang *et al*. (2011) studied design methods to resist progressive collapse in civil engineering for the RC frame structure. Li *et al*. (2011) proposed an improved tie force method for the progressive collapse resistance design of reinforced concrete frame structures. Their results showed that the current TF method is inadequate in increasing the progressive collapse resistance.

Malla *et al*. (2011) demonstrated a dynamic analysis methodology to incorporate the complete inelastic cyclic member force–deformation behavior of truss structures and at the same time, the possible dynamic effects arising from a sudden change in the load carrying capacity of a member due to failure or buckling. The evaluated methodology was applied to a two-dimensional, three-member toggle redundant truss under external static, quasi-static, and dynamic (sinusoidal and ramp) loads. The member force–deformation behavior shows that a compression member with its actual post-buckling behavior, while having some reserve load carrying capacity, is in more critical condition than when the same member suddenly loses its full load carrying capacity at buckling or when the member is linear elastic. Sasani *et al*. (2011) experimentally and analytically studied the progressive collapse resistance of an actual 11-story structure subjected to a severe initial damage with discontinued columns on the first story similar to those of the Murrah Federal building. Hadi and Alrudaini (2012) proposed a scheme for retrofitting RC buildings to resist progressive collapse that may result from a first floor column failure. They used finite-element modelling and a nonlinear dynamic analysis following the Alternate Path Method (APM), as recommended by General Services Administration (GSA) guidelines, to assess the viability of the proposed scheme on a 10-story RC building. The proposed scheme is comprised of placing vertical cables connected at the ends of beams and hung on a hat steel braced frame seated on top of the building. In case of a column collapse, the cables transfer the residual loads above the failed column to the hat- braced frame, which, in turn, redistributes these loads to the adjacent columns. They concluded that based on numerical results, their model is efficient in resisting the potential progressive collapse of the sample building used in their study. However, they indicate that before applying the proposed scheme in actual structures, experimental investigations are recommended for future studies to demonstrate the applicability of the proposed scheme in the actual structures. Izadi and Ranjbaran (2012) conducted an analytical investigation of the approach presented by Hadi and Alrudaini (2012) to provide an alternate load path to redistribute residual loads and prevent potential progressive collapse of RC buildings. They adopted nine analytical independent failure scenarios of a 10-story regular structural building in their investigation, including six external removal cases in different floors and three removal cases in the first floor. A new detail was proposed using barrel and wedge to improve residual forces transfer to the cables after removal of the columns. Simulation results showed that progressive collapse due to failure of the columns located on floors could be efficiently resisted by using this method.

Kim *et al*. (2014) evaluated the progressive collapse resisting capacity of steel moment frames with MR dampers and suggested a preliminary design procedure for the dampers to prevent progressive collapse. In addition, they investigated the progressive collapse potentials of 15-story steel moment frames installed with MR dampers by nonlinear dynamic analysis. The analysis results of the model structures showed that the MR dampers are effective in preventing progressive collapse of framed structures subjected to sudden loss of a first story column. Zahrai and Ezoddin (2014) compared four methods for progressive collapse analysis by studying 5- and 10-story intermediate moment-resistant reinforced concrete frame buildings. These analyses for three column-removal conditions were performed to evaluate the behavior of RC buildings under progressive collapse. Their results showed that dynamic analysis procedures -easy to perform for progressive collapse determination- are more reliable. Staylianidis and Nethercot (2015) investigated the connection behavior for progressive collapse and derived a complete model for describing the behavior of bare steel and composite connections during the progressive collapse. The model comprises of a set of explicit formulae for prediction of the connection deformations under the possible combinations of bending moments and axial loads observed at the various stages of progressive collapse response. Their results showed that the proposed model can describe connection performance in progressive collapse with a similar degree of accuracy to advanced numerical models. Xiao *et al*. (2015) studied a 3D half-scale RC frame structure subjected to sequential removal of its four columns. The efficiency of the applied numerical framework was verified for the global as well as for the local response quantities considering the entire range of the structural behavior from the elastic to the extreme plastic state, where the structure was identified as prone to collapse. Abbasnia *et al*. (2016) proposed a new method for progressive collapse analysis of RC frames. They studied a simplified theoretical method developed in order to predict general behavior of RC frames under the

Table 1 Geometric parameters and material properties of sample structures

Rectangular plan with dimensions	18 m × 22 m
Story height	3.4 m
Floor type (roof)	Hollow-tile
Compressive strength (f_c)	25 MPa
Yield strength of longitudinal reinforcement	400 MPa
Yield strength of transverse reinforcement	300 MPa
Modulus of elasticity of concrete	21000 MPa
Modulus of elasticity of steel	210000 MPa

Table 2 Value of the service loads in concrete 5- and 10-story structure models

Loads in 5- and 10-story structures	Story	Roof
Dead load (kN/m^2)	65	60
Live load (kN/m^2)	20	20
Surrounding walls (kN/m)	70	25

column removal scenario. The evaluation study indicated that the proposed theoretical procedure can establish a reliable basis for progressive collapse assessment of RC frame structures. Jalali Larijani *et al*. (2017) investigated progressive collapse analysis of buildings with concentrically and eccentrically braced frames. They considered the susceptibility of ten-story symmetric steel dual frame systems with different type of braced frames to progressive collapse. Numbers and locations of braced bays were investigated (two and three braced bays in exterior frames) to quantitatively find out its effect on progressive collapse resistance with a linear static analysis that carried out based on General Services Administration (GSA 2003) guidelines. The analysis results of the model structures showed that the three braced bays with concentrically braced frames especially X-braced and inverted V-braced frame systems had a lower susceptibility and greater resistance to progressive collapse.

This paper has two main objectives. The first objective is to determine the maximum axial force and critical position of the removed column in abnormal loading. The second objective is to introduce a new structural system to prevent progressive collapse, called cap or hat truss and then to compare it with primary structure as a building design improvement to make it safer and less vulnerable against terrorist attacks leading to progressive collapse.

2. Description of building geometry and loss scenario

In this study, 5- and 10-story concrete structures in Tehran with intermediate reinforced concrete moment-resisting frames in their both directions are investigated. Table 1 presents geometric parameters and material properties for numerical models.

Lateral load bearing systems in all structures are considered based on minimum design loads for buildings and other structures (ASCE 7-16). The service loads are summarized in Table 2. Fig. 1 shows the structure plan and numbering of beams and columns. The buildings have fully fixed at the base with a damping ratio of 5% in all modes, assuming the floors to behave as rigid diaphragms.

Since the structural sections are effective against progressive collapse, their determination is essential before selecting the structural analysis for progressive collapse. After creating the model and applying loads in the SAP 2000 program (2009) in accordance with Table 1, the structural design of models was performed based on the

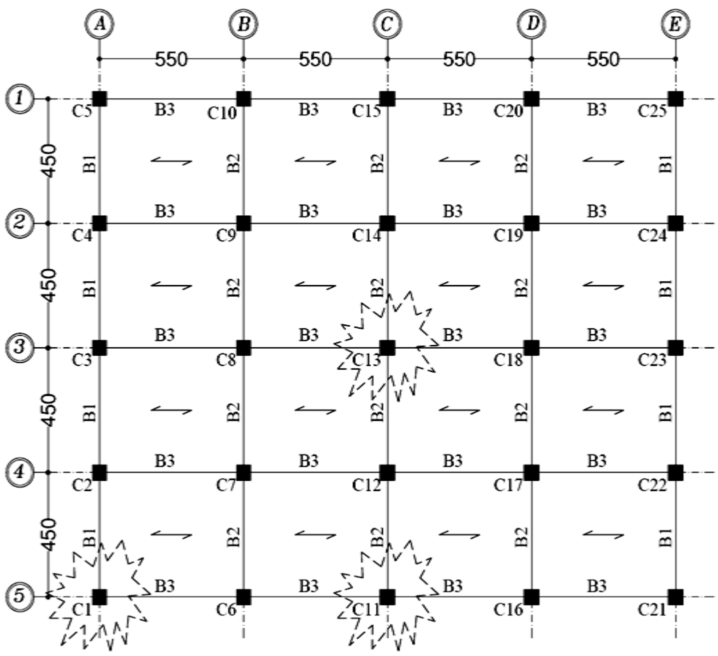


Fig. 1 Structural configuration and numbering of beams and columns for sample buildings (length in cm)

Table 3 Dimensions and amount of reinforcement of beams in 5-story building

Story	Dimension (cm) $w \times h$	Beams of external frames		Beams of internal frames		Non-load bearing beams	
		Type 1 (B1)		Type 2 (B2)		Type 3 (B3)	
		A (mm^2)		A (mm^2)		A (mm^2)	
		Top	Bottom	Top	Bottom	Top	Bottom
1	50 × 40	1470	1252	1524	1298	728	546
2	50 × 40	1517	1376	1587	1389	768	596
3	45 × 40	1278	1033	1338	1128	528	389
4	45 × 40	1036	933	1128	1003	503	317
5	40 × 35	748	431	978	560	378	293

Table 4 Dimensions and amount of reinforcement of beams in 10-story building

Story	Dimension (cm) $w \times h$	Beams of external frames		Beams of internal frames		Non-load bearing beams	
		Type 1 (B1)		Type 2 (B2)		Type 3 (B3)	
		A (mm^2)		A (mm^2)		A (mm^2)	
		Top	Bottom	Top	Bottom	Top	Bottom
1	75 × 50	1723	1378	1776	1426	1281	1089
2	75 × 50	1983	1642	2104	1787	1284	1098
3	60 × 45	2089	1655	2167	1792	1006	978
4	60 × 45	2043	1519	2189	1686	1012	931
5	50 × 40	1904	1427	2013	1547	820	768
6	50 × 40	1816	1331	1915	1478	740	542
7	45 × 35	1561	1204	1810	1324	638	487
8	45 × 35	1129	1120	1623	1268	541	403
9	35 × 30	1045	1009	1201	1107	328	293
10	35 × 30	624	539	842	560	321	249

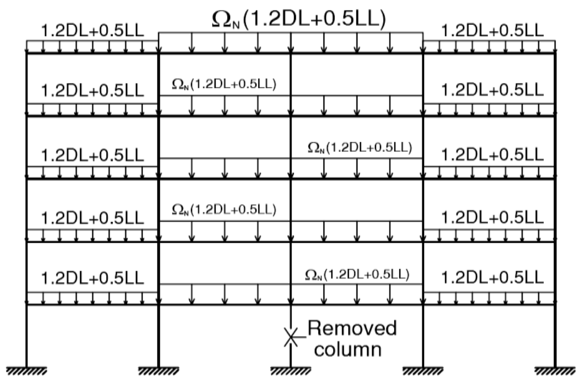
Table 5 Dimensions and amount of reinforcement of columns in the 5-story building

Story	Dimension (cm) $w \times h$	Corner column		Perimeter column		Internal column	
		Type 1		Type 2		Type 3	
		A (mm^2)		A (mm^2)		A (mm^2)	
1	50 × 50	3254	3878	4478	50 × 50	3254	3878
2	50 × 50	3264	4167	4559	50 × 50	3264	4167
3	45 × 45	2483	2978	3703	45 × 45	2483	2978
4	45 × 45	1871	2056	2488	45 × 45	1871	2056
5	40 × 40	1689	1943	2081	40 × 40	1689	1943

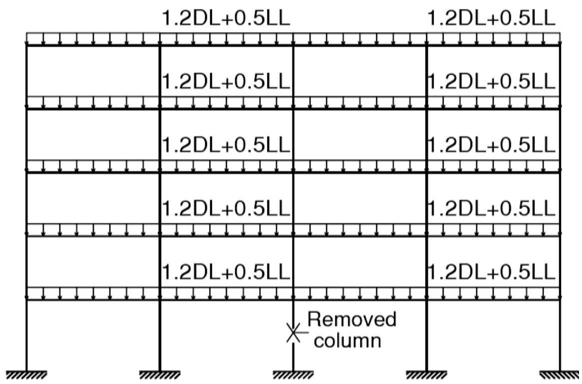
ACI 318-14 (2014) concrete building code and the sections and reinforcements of the beams and columns were obtained using linear static analysis, as presented in Tables 3, 4, 5 and 6.

Table 6 Dimensions and amount of reinforcement of columns in the 10-story building

Story	Dimension (cm) $w \times h$	Corner column	Perimeter column	Internal column
		Type 1	Type 2	Type 3
		A (mm^2)	A (mm^2)	A (mm^2)
1	75 × 75	4376	5262	5824
2	75 × 75	4456	5258	5976
3	60 × 60	3689	4041	4591
4	60 × 60	3483	3876	4277
5	50 × 50	2689	3174	3533
6	50 × 50	2446	2987	3689
7	45 × 45	2161	2749	3142
8	45 × 45	2081	2693	3081
9	35 × 35	1803	2369	2478
10	35 × 35	1691	1830	1893



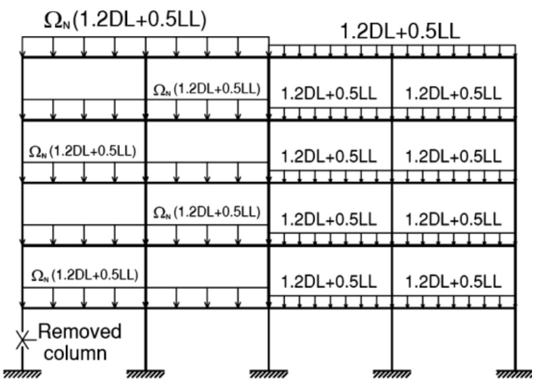
(a) Load combinations of nonlinear static analysis



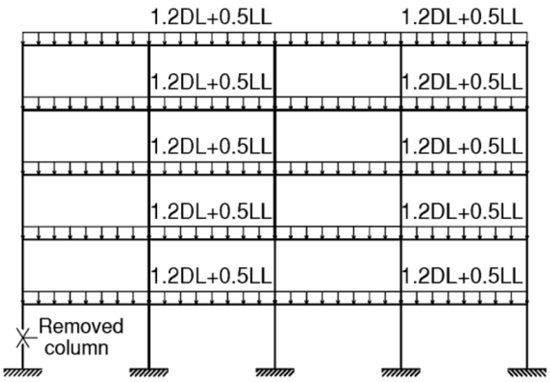
(b) Load combinations of nonlinear dynamic analysis

Fig. 2 Removal of the internal column

Gerasimidis (2014) has shown that when the column removal happens at lower floors, the collapse mechanism is governed by the failure (buckling) of a column element and when the column removal happens at higher floors, the collapse mode is governed by the flexural failure of the beams above the column removal. GSA (2013) and DoD (2016) provide an independent methodology to estimate the



(a) Load combinations of nonlinear static analysis



(b) Load combinations of nonlinear dynamic analysis

Fig. 3 Removal of the corner column

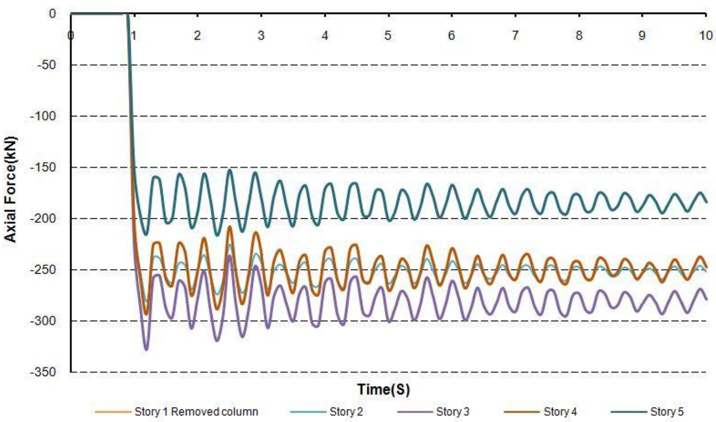
potential for progressive collapse of structures using the so called "missing column" scenarios, based on the notional removal of load-bearing elements. Therefore, using SAP2000 program (2009), load combinations recommended by the GSA (2013) are applied in accordance with Fig. 2 and 3, for those two sample structures due to a sudden removal of first story column in three locations of C1, C11 and C13. As shown in Figs. 2 and 3, dead load (DL), live load (LL) and nonlinear Static dynamic increase factors (Ω_N) are defined. Nonlinear dynamic analyses were performed to estimate the internal force formation as this method dynamically removes a member from the structure, which is then analyzed considering both the geometric and material nonlinearities. This allows larger deformations and energy dissipation through material yielding, cracking and fractures (Marjanishvili and Agnew 2006), although the nonlinear dynamic analysis method is generally more sophisticated and time consuming than linear static analysis method in characterizing the performance of a structure.

3. Progressive collapse analysis of model structures

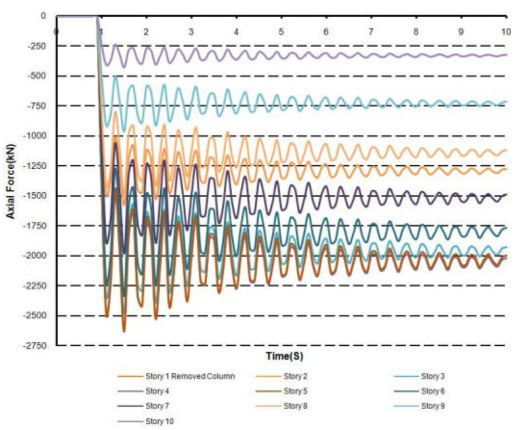
3.1 Identification of the maximum axial force due to column removal in the story

Since, 5- and 10-story structures are designed against earthquake, their beam and column sections increase on lower floors. With the sudden removal of the column in abnormal loading (explosion or collision), increase the axial force columns and shear force and bending moment, especially in the lower stories and structure has led to ultimate failure. Thus, the position of the column removed under this abnormal loading is very important. Providing sudden removal of column alternative load path can be replaced with another path. It could prevent local or general failure in the structure. For this purpose, at first, column on the first story for both structures in the positions of C1, C11 and C13 was removed separately. Using nonlinear dynamic analysis to determine the maximum axial force due to the column removal emerged as shown in Figs. 4, 5 and 6.

According to the obtained results of nonlinear dynamic analysis, remove columns for three different location C1, C11 and C13, the maximum value of the axial force is created C13 because of progressive collapse is related to gravity loading, C13 has most level loading in the story than C1 and C11. A maximum column axial force is created due to column removal at the first story in the third and fifth stories (middle story) for 5- and 10-story structures, which were respectively about 27% and 83% higher than that of the second story. Minimum column axial forces at three different positions in both structures were obtained in the 5th and 10th stories (the upper stories).

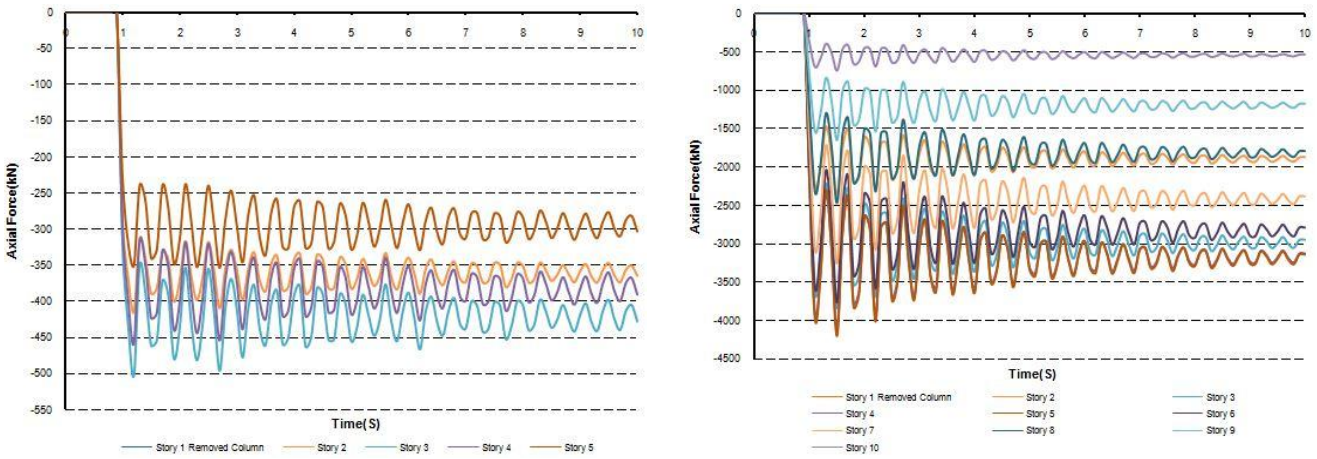


(a) 5-story



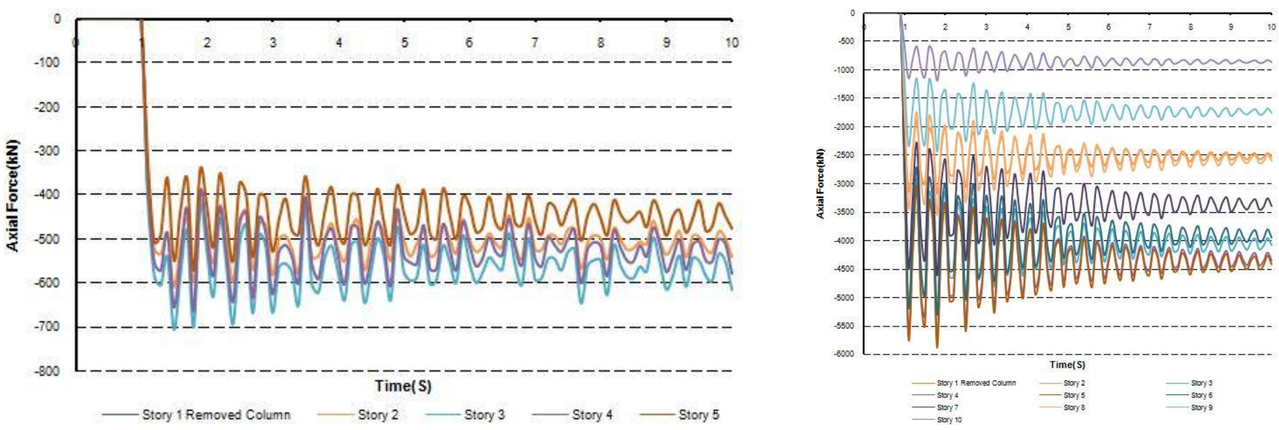
(b) 10-story

Fig. 4 Column C1 axial force due to first story column removal using nonlinear dynamic analysis for structures



(a) 5-story (b) 10-story

Fig. 5 Column C11 axial force due to first story column removal using nonlinear dynamic analysis for structures



(a) 5-story (b) 10-story

Fig. 6 Column C13 axial force due to first story column removal using nonlinear dynamic analysis for structures

3.2 Critical position of the removed column in abnormal loading

The critical position of the damaged column in 5- and 10-story buildings is studied using nonlinear static analysis. Since the area under the curve of force-displacement (pushdown) shows the dissipated energy by structure, the larger area under the pushdown curve means that the structure has more ability for energy absorption and force redistribution. Nonlinear plastic hinge properties are defined and assigned based on ASCE 41-13 as shown in Fig. 7. Point B, C, D and E based on ASCE 41-13.

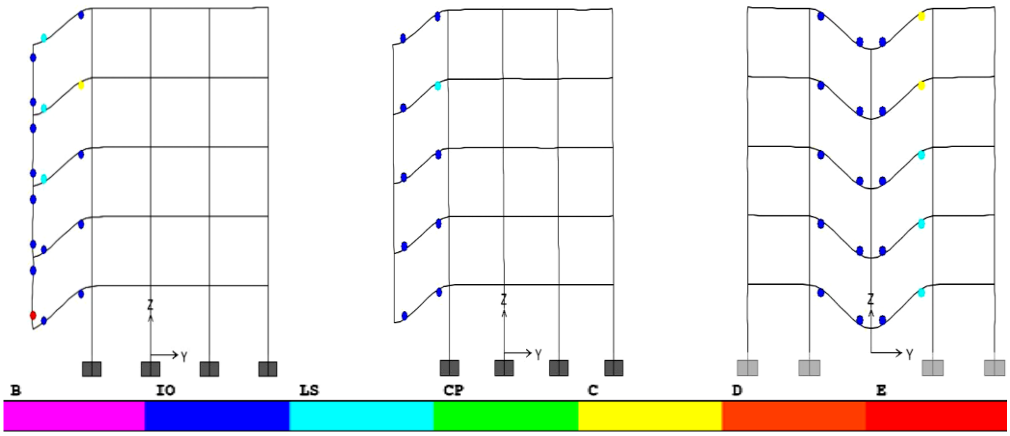
There are multiple possibilities to model the plastic hinge when this concept is used in structural analysis. In this paper, two possibilities are considered: plastic hinge of M3 type and plastic hinge of fiber type (P-M2-M3 type). The plastic hinge of M3 type is defined according to ASCE 41-13. Its behavior is showed in Fig. 7.

Columns C1, C11 and C13 were separately removed in 5- and 10-story buildings to identify the critical position of the column using nonlinear static analysis. Fig. 8 shows the location of plastic hinges and their performance level, when corner (C1), edge (C11) and internal (C13) columns are removed.

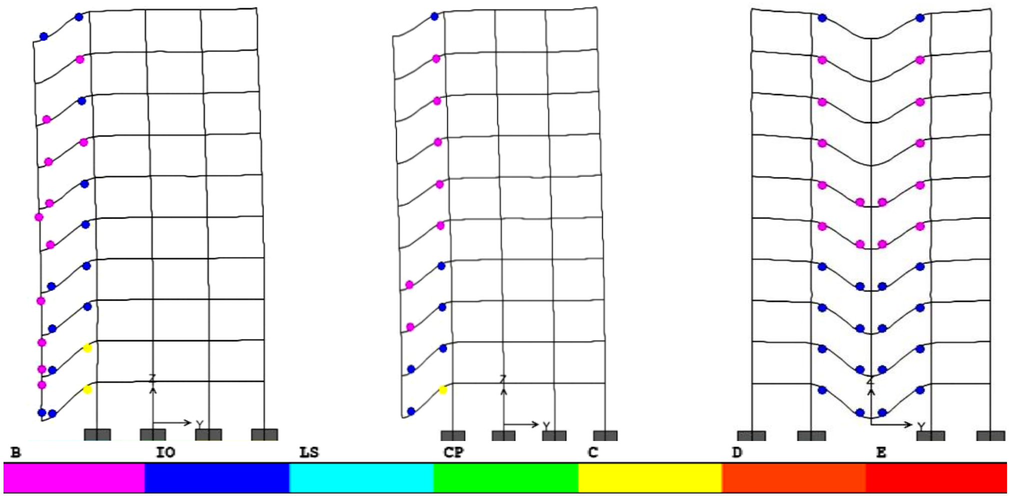
The results of Fig. 9 shows that removal of the column in the internal part (C13) in concrete structures with intermediate moment frame systems is more critical than removal of the corner columns because the progressive collapse refers to loss of gravity load capacity in structure. The interior column of the building has most level loading; sudden removal of the interior column can cause general destruction of the building. Therefore, reinforcement measures should be considered to prevent failure spread to other parts of the structure so that preserve immediate occupancy. The results of this study approximate those of a previous study by Marjanishvili and Agnew (2006).

4. Retrofitting methods to resist progressive collapse

Increasing set of factors as continuity, ductility and strength of the building framing can reduce progressive collapse potential. According to recent studies, two direct design approaches for reducing the possibility of progressive collapse have been used, including: 1) Alternate Path (AP) method, 2) Specific Local Resistance (SLR) method.



(a) 5-story



(b) 10-story

Fig. 8 Formation of the hinges when completely removing C1, C11 and C13 column from left to right, respectively

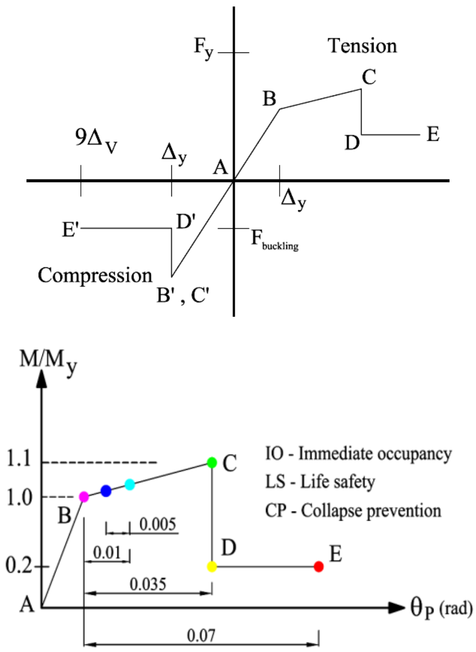


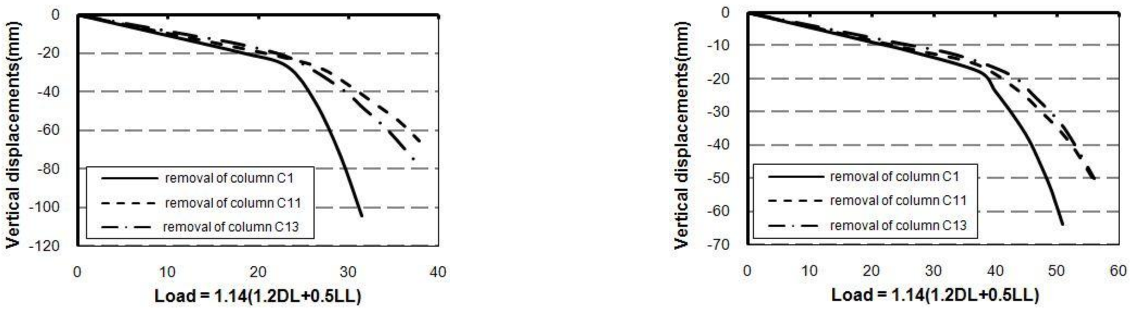
Fig. 7 Nonlinear hinges properties for concrete axial hinge elements (ASCE 41-13 2013)

4.1 Alternative load path

One of the techniques used for the design against progressive collapse is known as the alternate path (AP) method. That requires the proof to be provided by analysis and physical simulation. Alternate path method is a direct design method to guarantee that the structure will be able to bridge over a removed structural element to prevent the damage caused by exceeding the limits. If there is a structural element that cannot be bridged over, this element must be designed as a key element, which should have enough strength to resist possible extreme loads. Alternate path method is the most precise design method for progressive collapse resistance design.

Fig. 10 shows the gravitational force distribution in the structure before removal of vertical members under abnormal loading (explosion or collision). When a load-bearing member is not able to sustain gravity loads, adjacent members will distribute loads as shown in Fig. 11.

Alternate path (AP) method is the most widely used methods for the progressive collapse analysis, and its application in frame structures has been well proven (Cai *et al*. 2012). The following method can create alternative load paths in the concrete or steel buildings to prevent large



(a) 5-story (b) 10-story

Fig. 9 Force-vertical displacement the 2nd floor beams due to a column removal in three different positions

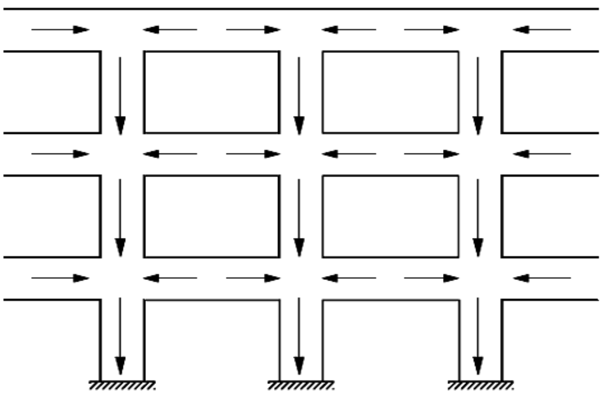


Fig. 10 Transfer of the load to the ground before the removal of vertical members (columns)

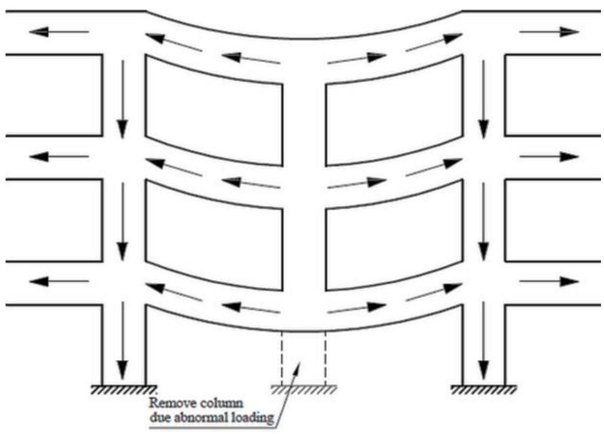


Fig. 11 Load path-transferring loads to ground after removal of the vertical member

deformations in structural.

4.1.1 Alternative load path using cap truss or hat truss

Cap truss system, consisting of a truss at upper or intermediate floors can establish an alternative load path in the structure against a sudden column removal, as shown in Fig. 12. It can transfer the gravity load to adjacent members to prevent general or local collapse of structures due to progressive collapse. Hat or cap truss is designed to help resisting loads due to the removed column.

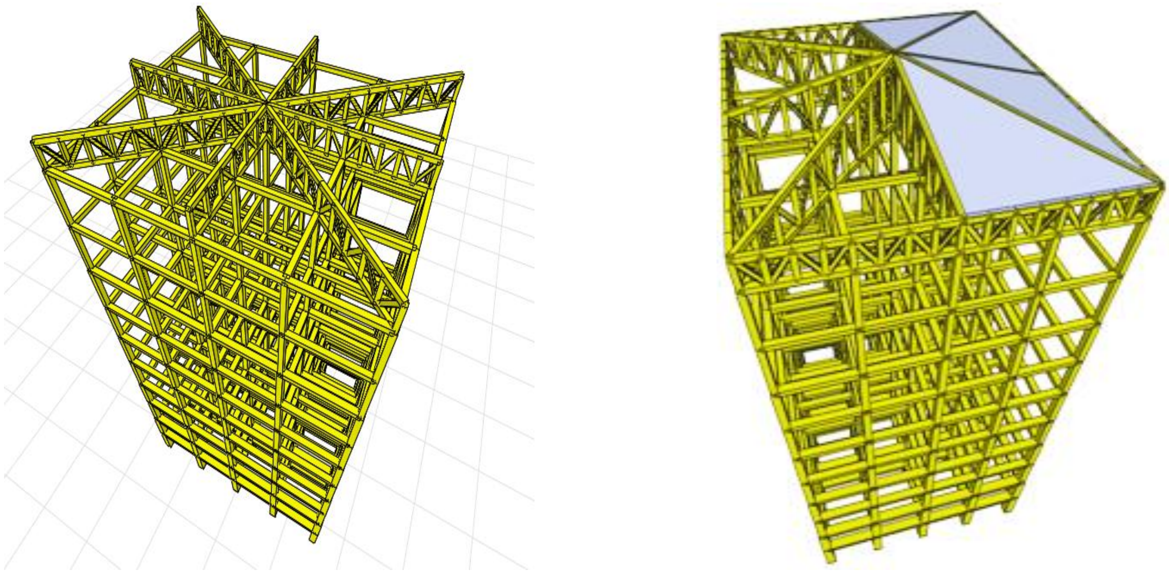
The advantage of the cap truss is that providing sudden removal of the column in any position due to the abnormal loading, the column load transfers to adjacent columns to prevent partial or total collapse and preserve serviceability in the structure. In addition, the cap truss system compared with outrigger-braced consists of a reinforced concrete or braced steel frame main core connected to the exterior columns by flexural stiff horizontal cantilevers, is very effective in increasing flexural and shear stiffness of the structure. The use of cap truss system holds the initial failure of the damaged elements and redistributes the loads supported by the failed elements with the least increase in the weight of the structure. However, outrigger-braced does not increase its resistance to shear, which has to be carried mainly by the core.

According to the American Institute of Steel Construction (AISC) Seismic Provisions (2016), cap truss is required to be designed to maintain the elastic behavior of the truss members, columns, and connections, except for the members that are involved in the formation of the yield mechanism. All members of the cap truss are to be designed for calculated loads by applying the combination of gravity and lateral loads recommended by the GSA (2013) that are necessary to develop the maximum expected nominal shear strength of the cap truss. The cap truss members are subjected to a combination of removed column axial load and large inelastic rotations of the beam. Therefore, the cap truss members are designed based on the most critical gravity axial load due to sudden removal of the column in three locations of C1 (edge column), C11 (corner column) and C13 (center column) in elastic state developed using the structural analysis software SAP2000. For example, due to the removal of the middle column (C13), the axial load value of the column obtained using load combination proposed by GSA 2013 guideline using nonlinear dynamic analysis is applied at that point to the bottom chord of cap truss and then all cap truss members are designed in the elastic range using allowable stress design method.

Fig. 13 shows the view 2D of the cap truss utilized in the mitigation scheme and the designed steel sections are given in Table 7.

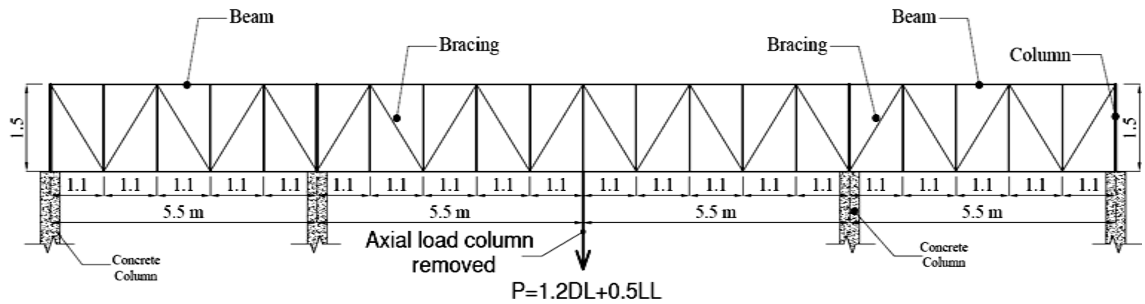
4.2 Specific local resistance

Another rehabilitation method is to directly strengthen a specific load-bearing element called Specific Local

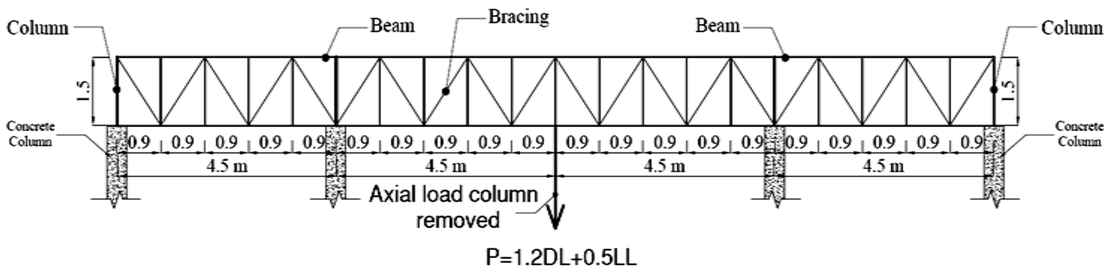


(a) With flat roof (b) With sloping roof

Fig. 12 The view 3D of cap truss utilized in the mitigation scheme



(a) X direction



(b) Y direction

Fig. 13 The 2D view of cap truss (unit: meter)

Resistance (SLR) or structural hardening. It can also be applied to newly constructed buildings in design and to existing buildings for the retrofit schemes to resist extreme events. This approach reduces the likelihood or extent of the initial damage and can be effective, for those cases where the threat can be quantified through risk analysis or specified through prescriptive design requirements. Some rehabilitation methods for specific local resistance are wrapping fiber-reinforced polymer (FRP) around the critical columns and installing fiber sheets under the beams or slabs (Crawford 2002). The following method can increase specific local resistance in the concrete or steel buildings to impede large deformations in structural.

Table 7 Cap truss sections

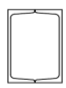
Element	Section	Shape
Column	2IPE240	
Beam	IPE270	
Bracing	2UNP300	

Table 8 Steel strut section

Element	Section	Shape
Steel strut	2IPE240	

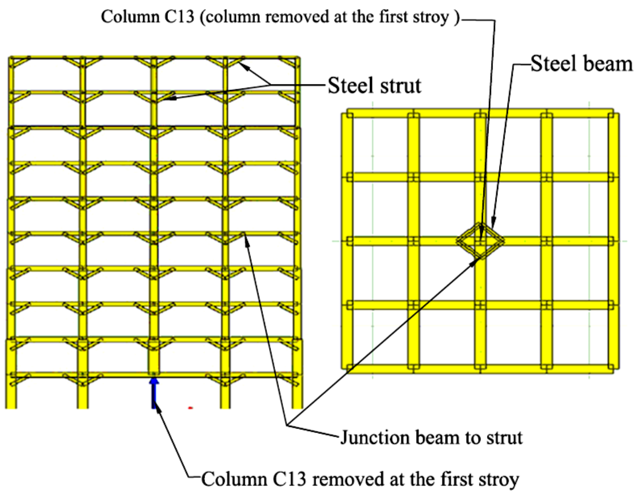


Fig. 14 Specific local resistance using steel struts

4.2.1 Specific local resistance using steel strut

Strut is a structural member of the steel used as a brace to resist compressive axial forces. It is similar to knee braces in the geometry, responsible for enhancing ductility and preventing shear force localization around the column. When a column is suddenly removed, steel strut can transfer the gravity load to adjacent members and enhance local resistance for the structure to delay a final collapse. In addition, steel struts could enhance resistance of panel zone. Strut members were designed based on the gravity axial load due to sudden removal of the column, as shown in Fig. 14. In addition, rigid connections of steel strut to RC beam and the column to steel strut were assumed. The designed steel strut sections are given in Table 8.

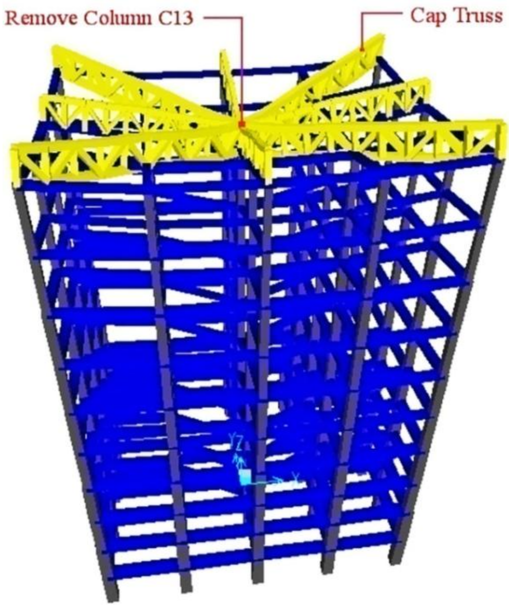


Fig. 15 Preventing sudden progressive failure of members by cap truss system

4.3 Results using cap truss and steel strut

To evaluate the progressive collapse potential of the retrofit methods proposed, nonlinear dynamic analyses are conducted with removal of column C13 (internal zone) on the first story for 5-and 10-story concrete buildings, then the results are compared with the case of the primary model (without retrofitting). Fig. 15 depicts that cap truss can restrain sudden failure of a structure due to removal of column C13 and transmit the gravity load to adjacent members. It is shown that the structure is capable of bridging over a removed column. Figs. 16 and 17, show vertical displacements of beam B10-B11 at the first story due to remove column C13 for 5- and 10-story concrete buildings.

As illustrated in Figs. 16 and 17, cap truss can reduce the vertical displacement in the 5- and 10-story buildings

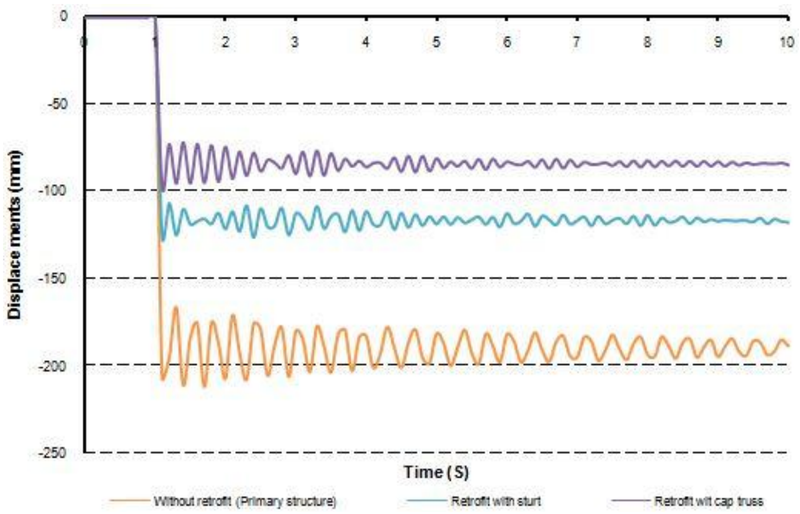


Fig. 16 Comparison of vertical displacement of beam (B10-B11) for the 5-story frame

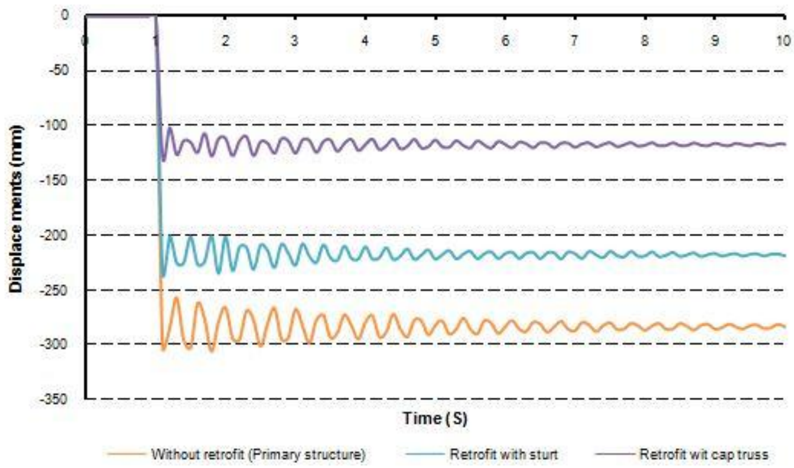


Fig. 17 Comparison of vertical displacement of beam (B10-B11) for the 10-story frame

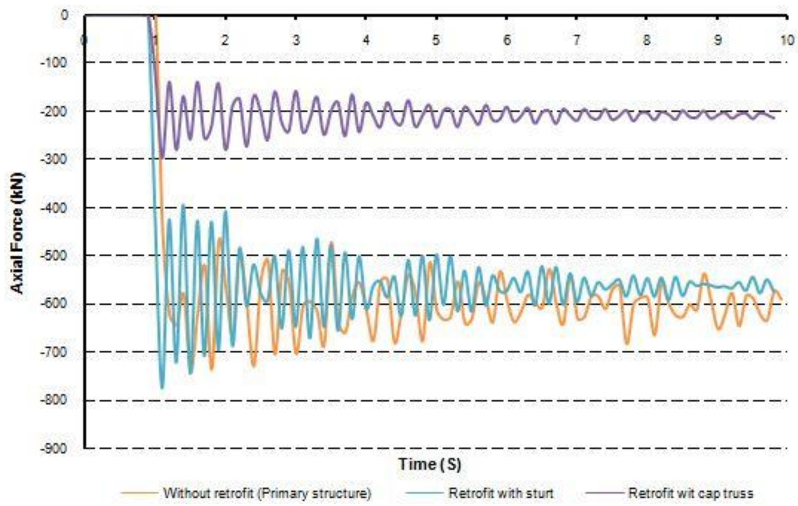


Fig. 18 Comparing axial load of column at the third story for 5-story building due to 1st story column, C13 removal

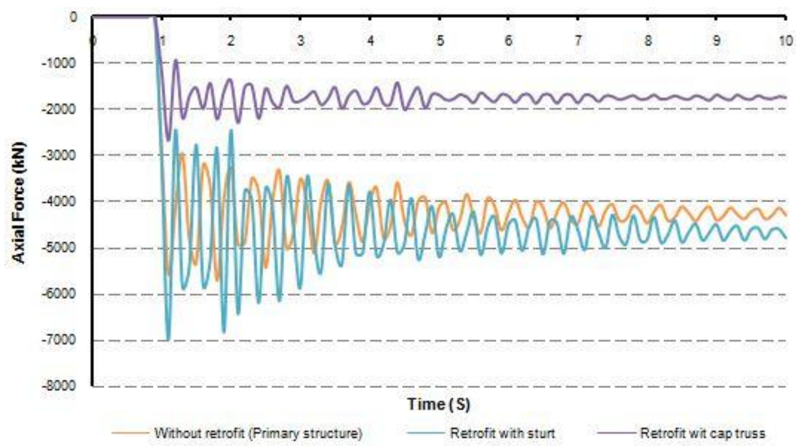


Fig. 19 Comparing axial load of column at the fifth story for 10-story building due to 1st story column, C13 removal

about 54% and 57%, respectively. This implies that the structure can be able to resist the gravity load imposed by he loss of an internal column. When an internal column is removed, the strength drops rapidly after the failure of some members in the structure consequently large deflections occur as shown in Figs. 16 and 17. Therefore, using the cap truss system can highly improve structure potential against progressive collapse. Axial loads of the column of the story having the maximum axial load, at the third and the fifth story in both 5- and 10-story is shown in Figs. 18 and 19,

Table 9 Summary of maximum analysis results due to column removal C13 on the first story

Story	Displacement beam B10-B11 (mm)					Axial force column (kN)				
	Without retrofitting	Steel strut	Percent decrease	Cap truss	Percent decrease	Without retrofitting	Steel strut	Percent decrease	Cap truss	Percent decrease
5	211	126	40	97	54	736	776	–	296	59
10	303	234	23	130	57	6836	6990	–	2589	62

respectively.

Based on the nonlinear dynamic analysis results in 5- and 10-story concrete structures, it is observed that using steel strut decreases the vertical displacement of the beam in both 5- and 10-story about 40% and 23%, respectively. It cannot transmit axial load of removed column to adjacent columns in both 5- and 10-story as shown in Figs. 18 and 19. It can reinforce connections of beam to column. Also, the results of the nonlinear dynamic analysis at 5- and 10-story concrete structures with cap truss and without retrofitting (primary structure) show that using cap truss can decrease the axial force of removing column (C13) in both 5-and 10-story about 59% and 62%, respectively, as shown in Figs. 18 and 19. It means that, using cap truss can transmit about 61% of the forces to adjacent columns in average. Therefore, cap truss can provide an alternative load path to redistribute loads, decrease excessive beam displacement and transmit axial load of removed column to adjacent columns. In addition, the results for 5- and 10-story concrete structures using steel strut and cap truss are compared to that of primary structure (without retrofit) as summarized in Table 9.

The results of this study compare well with those of a previous study by Marjanishvili and Agnew (2006). Accordingly, engineers can restrain the risk of progressive collapse in important buildings using cap truss design. In other words, using the cap truss method can improve the behavior of structure under these abnormal events (collision or impact), increase ductility and redistribute internal stresses in members.

5. Conclusions

In this paper, progressive collapse-resisting mechanisms and robustness values of two 5- and 10-story regular buildings are quantitatively investigated using steel strut or cap truss based on nonlinear dynamic analysis procedure. The main findings obtained from this research are summarized in the following:

- The maximum value of the axial force is created in C13 than C1 and C11 due to sudden removal of the column. The maximum column axial force is induced at the first story in the third and fifth stories (multi-story) about 27% and about 83% higher than the second story for 5- and 10-story structures, respectively.
- Using steel strut can decrease vertical displacement due to sudden removal of the column C13 story in both 5- and 10-story structures about 40% and 23%, respectively, but it cannot transmit axial load of removed column to adjacent columns.
- Using cap truss can reduce the average vertical displacement due to sudden removal of column C13 (internal column) at the first story in both 5- and 10-story structures about 56%. In addition, using cap truss can transfer the column axial forces about 61% in average. This means that, the axial force in the removed column transfers with an alternate path to adjacent columns to prevent local or general failure or to delay the progressive collapse occurrence. Therefore, using a hat or cap truss at upper or middle floors can transfer the axial force of removed column in every position to adjacent columns and redistribute bending moment and shear force on the structure such that the load is transmitted to the ground by the alternative paths.

References

Abbasnia, R., Mohajeri Nav, F., Usefi, N. and Rashidian, O. (2016), "A new method for progressive collapse analysis of RC frames", *Struct. Eng. Mech., Int. J.*, **60**(1), 31-50.

ACI Committee 318 (2014), Building Code Requirements for Structural Concrete and Commentary (ACI 318 R-14); American Concrete Institute: Farmington Hills, MI, USA.

American Institute of Steel Construction (AISC) (2016), Load and resistance factor design specification for structural steel buildings; Chicago, IL, USA.

American Society of Civil Engineers (ASCE) (2016), "Minimum Design Loads for Buildings and Other Structures", ASCEISEI 7-16.

ASCE 41-13, American Society of Civil Engineers (2013), "Seismic evaluation and retrofit of existing buildings", Public comment draft, Reston, VA, USA.

Cai, J.G., Xu, Y.X., Zhuang, L.P., Feng, J. and Zhang, J. (2012), "Comparison of various procedures for progressive collapse analysis of cable-stayed bridges", *J. Zhejiang Univ.-SCIENCE A* (*Appl. Phys. Eng.*), **13**(5), 323-334. DOI: 10.1631/Jesus. A1100296

Chen, Y. (2012), "Progressive collapse analysis and safety assessment method for steel truss roof", *J. Perform. Constr. Facil.*, **26**(3), 230-240. DOI: 10.1061/ (ASCE) CF.1943-5509.0000236

Corley, W.G., Mlakar, P.F., Sozen, M.A. and Thorton, C.H. (1998), "The Oklahoma City Bombing: Summary and Recommendations for Multi-hazard Mitigation", *J. Perform. Construct. Facial ASCE*, **12**(3), 100-112.

Crawford, J.E. (2002), "Retrofit methods to mitigate progressive collapse", *Multihazard Mitigation Council National Workshop on Prevention of Progressive Collapse*, Chicago, IL, USA, July.

CSI, SAP2000 (2009), "Three dimensional static and dynamic finite element analysis and design of structures", Analysis, Reference, Version 14.1, Computer and Structures, Inc., Berkeley, CA, USA.

DoD (Department of Defense) (2016), “Unified facilities criteria (UFC: Design of structures to resist progressive collapse”, UFC 4-023-03 change 3, 1 November 2016, Washington, DC, USA.
Gerasimidis S. (2014), “Analytical assessment of steel frames progressive collapse vulnerability to corner column loss”, *J. Constr. Steel Res.*, **95**, 1-9.
GSA (2013), “Alternate Path Analysis & Design Guidelines for Progressive Collapse Resistance”, General Service Administration, Washington, DC, USA.
Hadi, M. and Alrudaini, T.S. (2012), “New building scheme to resist progressive collapse”, *J. Architect. Eng.*, **18**(4), 324-331.
Izadi, I.T. and Ranjbaran, A. (2012), “Investigation on a mitigation scheme to resist the progressive collapse of reinforced concrete buildings”, *Front. Struct. Civil Eng.*, **6**(4), 421-430.
Izzuddin, B.A., Vlassis, A.G., Elghazouli, A.Y. and Nethercot, D.A. (2008), “Progressive collapse of multi-storey buildings due to sudden column loss-Part I: Simplified assessment framework”, *Eng. Struct.*, **30**(5), 1308-1318.
Jalali Larijani, R., Dashti Nasserabadi, H. and Aghayan, I. (2017), “Progressive collapse analysis of buildings with concentric and eccentric braced frames”, *Struct. Eng. Mech., Int. J.*, **61**(6), 755-763.
Kim, T. and Kim, J. (2009), “Progressive collapse-resisting capacity of steel moment frames considering panel zone deformation”, *Adv. Struct. Eng.*, **12**(2), 231-240.
Kim, J. and Park, J. (2010), “Progressive collapse resisting capacity of building structures with outrigger trusses”, *Struct. Des. Tall Special Build.*, **25**(1), 19-26. DOI: 10.1002/tal. 628
Kim, J., Lee, S. and Min, K.W. (2014), “Design of MR dampers to prevent Progressive collapse of moment frames”, *Struct. Eng. Mech., Int. J.*, **52**(2), 291-306.
Li, Y., Lu, X., Guan, H. and Ye, L. (2011), “An improved tie force method for progressive collapse resistance design of reinforced concrete frame structures”, *Eng. Struct.*, **33**(10), 2931-2942.
Longinow, A. and Mniszewski, K.R. (1996), “Protecting buildings against vehicle bomb attacks”, *Practice Periodical Structure Design Construct*, **1**(1), 51-54.
Malla, R.B., Agarwal, P. and Ahmad, R. (2011), “Dynamic analysis methodology for progressive failure of truss structures considering inelastic post buckling cyclic member behavior”, *Eng. Struct.*, **33**(5), 1503-1513.
Marjanishvili, Sh. and Agnew, E. (2006), “Comparison of various procedures for progressive collapse analysis”, *J. Perform. Constr. Facil.*, **20**(4), 365-374.
National Institute of Standards and Technology (NIST) (2007), Best Practices for Reducing the Potential for Progressive Collapse in Buildings; Technology Administration, U.S. Department of Commerce, Washington, D.C., USA.
Sasani, M., Kazemi, A., Sagiroglu, S. and Forest, S. (2011), “Progressive collapse resistance of an actual 11-story structure subjected to severe initial damage”, *J. Struct. Eng.*, **137**(9), 893-902.
Sasani, M. and Kropelnicki, J. (2008), “Progressive collapse analysis of an RC structure”, *Struct. Des. Tall Special Build.*, **17**(4), 757-771.
Shi, Y., Li, Z.X. and Hao, H. (2010), “A new method for progressive collapse analysis of RC frames under blast loading”, *Eng. Struct.*, **32**(6), 1691-1703.
Stylianidis, P.M. and Nethercot, D.A. (2015), “Modelling of connection behaviour for progressive collapse analysis”, *J. Constr. Steel Res.*, **113**, 169-184.
United States Army Corps of Engineers (1999), “Technical instructions: structural design criteria for buildings”, TI 809-02, US Army Corps of Engineers, Washington DC, USA.
Vlassis, A.G., Izzuddin, B.A., Elghazouli, A.Y. and Nethercot, D.A. (2008), “Progressive collapse of multi-storey buildings due to sudden column loss-Part II: Application”, *Eng. Struct.*, **30**(5), 1424-1438.
Vlassis, A.G., Izzuddin, B.A., Elghazouli, A.Y. and Nethercot, D.A. (2009), “Progressive collapse of multi-storey buildings due to failed floor impact”, *Eng. Struct.*, **31**(7), 1522-1534.
Wang, H., Su, Y. and Zeng, Q. (2011), “Design Methods of Reinforce-concrete Frame Structure to Resist Progressive Collapse in Civil Engineering”, *Syst. Eng. Procedia*, **1**, 48-54.
Xiao, Y., Kunnath, S., Li, F., Zhao, Y., Lew, H. and Bao, Y. (2015), “Collapse test of three-story halfscale reinforced concrete frame building”, *ACI Structure Journal*, **112**(4), 429.
Zahrai, S.M. and Ezoddin, A. (2014), “Numerical study of progressive collapse in intermediate moment resisting reinforced concrete frame due to column removal”, *Civil Eng. Infrastruct. J.*, **47**(1), 71-88.

CC