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Numerical Study of Progressive Collapse in Intermediate Moment Resisting Reinforced Concrete Frame Due to Column Removal

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ABSTRACT: Progressive collapse is a chain reaction of failures propagating throughout a portion of the structure disproportionate to the original local failure occurring when a sudden loss of a critical load-bearing element initiates a structural element failure, eventually resulting in partial or full collapse of the structure. Both General Services Administration (GSA) and United States Department of Defense (DoD) guidelines incorporate a threat-independent approach to progressive collapse analysis. Therefore, there is an international trend for updating structural design requirements to explicitly design structures to resist progressive collapse. This paper presents simple analytical approach for evaluating progressive collapse potential of typical concrete buildings, comparing four methods for progressive collapse analysis by studying 5 and 10-story intermediate moment-resistant reinforced concrete frame buildings, employing increasingly more complex analytical procedures: linear-elastic static, nonlinear static, linear-elastic dynamic, and nonlinear dynamic methodologies. Each procedure is thoroughly investigated and its common shortcomings are identified. The evaluation uses current GSA progressive collapse guidelines and can be used in routine design by practicing engineers. These analyses for three column-removal conditions are performed to evaluate the behavior of RC buildings under progressive collapse. Based on obtained findings, dynamic analysis procedures -easy to perform for progressive collapse determination- yielded more accurate results.

Keywords: Intermediate Moment Resisting Frame, Linear Dynamic Analysis, Linear Static Analysis, Nonlinear Dynamic Analysis, Nonlinear Static Analysis, Progressive Collapse, Reinforced Concrete.

INTRODUCTION

In recent years with increasing terrorist activities in important buildings, it is necessary to protect the lives thus many studies have been done to design structures against progressive collapse in order to achieve appropriate adjustment in the design standards and codes. Progressive collapse is defined as the spread of an initial local failure from element to element resulting eventually, in the collapse of an entire

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structure or a large part of it (ASCE 7-05). In the best practice for reducing the potential for progressive collapse in buildings published by National Institute of Standard and Technology (NIST, 2007). The potential abnormal load hazards that can trigger progressive collapse are categorized as: aircraft impact, design/construction error, fire, gas explosions, accidental overload, hazardous materials, vehicular collision, bomb explosions, etc. As these hazards have low probability of occurrence, they are neither considered in structural design nor addressed directly by passive protective measures. Most of them have characteristics of acting over a relatively short period of time and result in dynamic responses. When local failure of primary structural members propagates to failure of adjoining members, progressive collapse will ensue unless adjoining structural members arrest further progression of failure (Kim et al., 2009). For example, if a column in a multi-story building fails due to abnormal loading failure (explosion or collision) of structural members above the column, then large sudden displacements might occur and a major part or the whole structure might be destroyed unless the beams framed to the column prevent progress of the chain response.

Progressive collapse of the buildings begins with a local damage of the structural system that cannot be absorbed or prevented. Then it spreads throughout the structural system or a part of it and eventually structural system will reach ultimate deflection. The first event of progressive collapse, a gas explosion occurred on May 16, 1968 in an apartment on the 18th floor of a 23-story precast concrete building at Ronan Point in England. The explosion resulted in a loss of support for the five stories above, and the weight of the fallen top floors caused the subsequent collapse of the floors below leading to three casualties. The second important event on April 19, 1995, a truck loaded with explosives was parked outside the Alfred P. Murrah federal building in Oklahoma City. The truck exploded, causing the collapse of a large portion of the nine-story building, as well as damage to adjacent buildings in the complex, resulting in 168 casualties. At the third event of progressive collapse on September 11, 2001, as part of a larger terrorist plan, two planes were flown into the World Trade Center towers. The initial impact and ensuing fires caused immense damage on several floors at the impact locations. Eventually, the structural systems of the two towers were overwhelmed by the damage they had sustained, and both buildings collapsed. A total of 2726 people were killed as a result of these events. After this event, several more researchers have started to refocus on the causes of progressive collapse in building structures, seeking ultimately the establishment of rational methods for the assessment and enhancement of structural robustness under extreme accidental events. Therefore for building design international codes, it is necessary to design structures against progressive collapse.

Among the codes that were recently updated to include specific clauses to require structural integrity of the structure to rule out the possibility of progressive collapse are the following codes: American Society of Civil Engineers (ASCE/SEI 7-05), American Concrete Institute (ACI 318-05), National Building Code of Canada (NBC 2005), International Building Code (IBC 2009), Eurocode 1, British Standard Institute (BS 5950-2000), and Saudi Building Code (SBC 301-2007). The analysis procedures recommended by the guidelines for alternate path method are linear elastic static (LS), linear dynamic (LD), nonlinear static (NS), and nonlinear dynamic (ND) methods, which were also recommended for seismic analysis and design for structures in FEMA 274 (1997). Kaewkulchai and Williamson (2003) investigated the analysis procedures using a two-dimensional frame analysis. They observed that linear static analysis might result in non-conservative results since it cannot reflect the dynamic effect by sudden exclusion of columns. Also the authors demonstrated that when dynamic analysis is used to assess the potential for progressive collapse of frames, the use of either the initial configuration or the deformed configuration does not significantly affect the structural response. Mohamed (2009) considered a case study for the progressive collapse analysis of a reinforced concrete building using the alternate path (AP) method with implementation of UFC 4-023-23 (DOD, 2009) to protect against the progressive collapse of corner floor panels when their dimensions exceed the damage limits through analyzing the progressive collapse potential of a reinforced concrete building. Presented numerical case studies based on the linear static analysis showed the importance of incorporating 3-dimensional effects, especially at the part of the structure where a column is notionally removed.

Kim and Kim (2009) studied the progressive collapse-resisting capacity of steel moment resisting frames by using alternate path (AP) methods recommended in the General Services Administration (GSA, 2003) and United States Department of Defense (DoD) guidelines and compared them with the linear static and nonlinear dynamic analysis procedures. The results showed, the nonlinear dynamic analysis provided larger structural responses and the results varied more significantly. However the linear procedure provided a more conservative decision for progressive collapse potential of model structures.

Powell (2005) compared the linear static (LS), nonlinear static (NS), and nonlinear dynamic (ND) analyses and found that the impact factor of 2 regulated in the LS analysis can display very conservative result, and insisted that basically the nonlinear analysis should be used. Pretlove et al. (1991) carried out experimental and numerical investigations with a tension spoke structure to examine the nature of progressive failure and dynamic effects associated with the loss of one or more spokes. They demonstrated that a static analysis for progressive failure may not be conservative if inertial effects are taken into consideration. Although several researchers presented the importance of considering inertial effects for progressive collapse analysis, dynamic load redistribution in the progressive collapse analysis of frame structures is hardly considered in practicing engineering because most of commercial software do not support progressive collapse analysis with dynamic effects.

Hansen et al. (2005) studied the performance of three-dimensional models of external columns in reinforced concrete buildings and produced response histories for edge beams using nonlinear dynamic analysis to simulate the loss of exterior columns. He also demonstrated that nonlinear dynamic analysis is important for progressive collapse investigations to capture a realistic structural response. Grierson et al. (2005) presented a method for conducting linear static progressive collapse analysis based on the provisions of the United States General Services Administration (GSA). They modeled the reduced stiffness during progressive collapse using an equivalent-spring method.

This study compares different methods of progressive collapse analysis for 5 and 10-story reinforced concrete building case studies with intermediate moment resisting frame system and subsequently investigates their advantages and disadvantages.

DESCRIPTION OF ASSUMED RC BUILDING

Concrete 5 and 10-story buildings in Tehran and on soil type III with same plans are investigated. Structure plans with dimensions of 18 m × 22 m, with 4 spans of 5.5 meters in direction X and with 4 spans of 4.5 meters in direction Y were considered. To have a practical design, real dimensions, real spans, symmetrical structure and typical story height is 3.2m and the floors of hollow-tile type are considered. A compressive strength (fc) of 25 MPa and Poisson ratio υ = 0.2 are used for the concrete. The design yield strength is 400 MPa for longitudinal reinforcement and 300 MPa for transverse reinforcement. Concrete and steel values for modulus of elasticity are 21000 and 210000 MPa, respectively. These structures utilize intermediate RC moment-resisting frame in both directions in accordance with regulations ASCE 7-05 (2005) which is summarized in Table 1. The assumed building plan is shown in Figure 1, and the sections of beams and columns for 5, 10-story building are classified in Table 3.

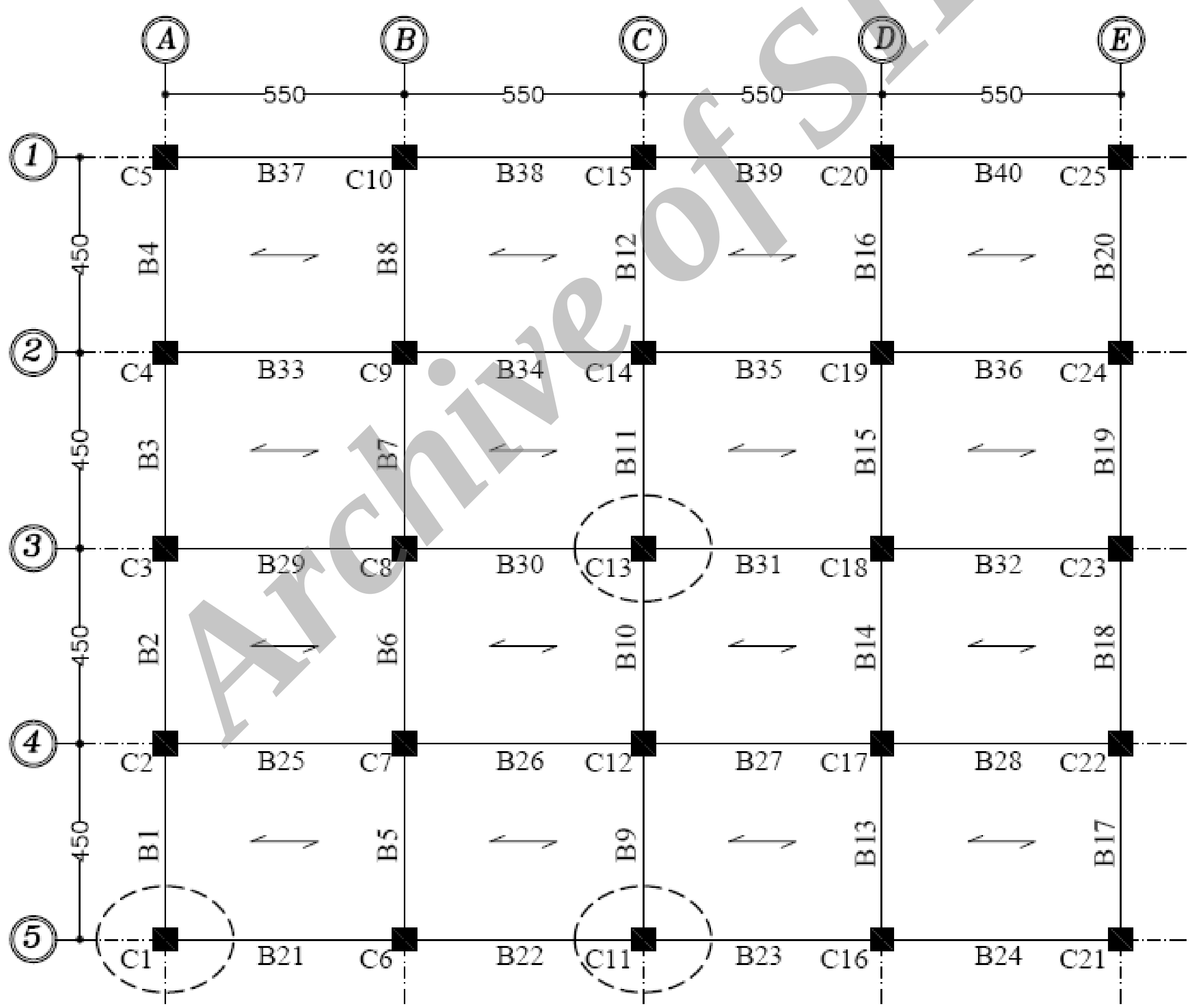


Fig. 1. The studied structural plan and numbering of beams and columns (cm).

Table 1. Value of the service loads in concrete 5 and 10-story structure models.

Loads on Structures 5-10 Story	Floors	Roof
Dead load (kN/m^2)	65	60
Live load (kN/m^2)	20	20
Surrounding walls (kN /m)	70	25

Table 2. Summary of the seismic structural parameters based on BHRC (2007).

	T_0	T_s	S	H (m)	$T = 0.07 H^{0.75}$	B	I	R	C_x , C_y
Five-story concrete structure	0.15	0.7	1.75	17.5	0.59	2.75	1	7	0.1375
Ten-story concrete structure	0.15	0.7	1.75	35	1.01	2.15	1	7	0.108

In Table 2, T_0, T_s and S are parameters determined from the soil profile type and level of seismicity. The parameter H is the height of the building in meters, measured from the base level; T is the fundamental period of vibration of the structure in the direction under consideration. B is the building response factor determined from the design response spectrum; I is the importance factor; A is the design base acceleration ratio (ratio of seismic acceleration to gravity acceleration, g) that is considered 0.35 here for very high level of relative seismic hazard zone; R is the building behavior factor for intermediate reinforced concrete moment-resisting frame and finally C is the seismic coefficient, that is determined from the following formula:

$$C = \frac{ABI}{R} \tag{1}$$

After creating the model and applying loads in SAP2000 software (CSI, 2009), the structural design of 5 and 10-story case studies is performed based on ACI318 (2008). Then sections of beams and columns for two structures are obtained using linear static analysis summarized in Table 3. For modeling and analyzing the progressive collapse in concrete structures, four methods of linear static analysis, nonlinear static, linear dynamic and nonlinear dynamic are compared. To simulate the abruptly removal of the first story column under abnormal loading (explosion or impact) in three different positions, columns C1, C11 and C13 are studied as shown in Figure 1.

Table 3. Sectional dimensions for 5, 10-story concrete structure.

Case studies	Story numbers	Beam size			Column size		
		Width (cm)	Height (cm)	Deformed bar size (top & bottom)	Width (cm)	Height (cm)	Deformed bar size
5	1 and 2	50	40	8∅20	50	50	12∅20
	3 and 4	45	40	8∅18	45	45	8∅20
	5	40	35	6∅16	40	40	8∅16
10	1 and 2	75	50	8∅22	75	75	16∅22
	3 and 4	60	45	8∅20	60	60	12∅20
	5 and 6	50	40	8∅18	50	50	8∅20
	7 and 8	45	35	6∅18	45	45	8∅18
	9 and 10	35	30	6∅16	35	35	8∅16

Since the loading condition after sudden removal of a column is completely dynamic, using amplification factor in the linear and nonlinear static analysis method, its dynamic nature is approximated with combination load that is proposed in the GSA, DoD, and ASCE. In the studied models, the dynamic amplification factor of 2 in load combination proposed by GSA guideline (2003) is used as shown in Figure 2. Loads are applied to the beams adjacent to removed columns in linear and nonlinear static analysis method as illustrated in the following equation:

$$\text{Load} = 2\,(DL + 0.25LL) \tag{2}$$

For dynamic analysis, neither the GSA nor the DoD guidelines recommend using the dynamic amplification factor. According to GSA guideline, in linear and nonlinear dynamic analysis methods, the load applied to the beams connected to the removed column is illustrated in the following equation:

$$\text{Load} = DL + 0.25LL \tag{3}$$

where DL and LL are floor dead load and Live load, respectively.

INVESTIGATION OF A COLUMN REMOVAL IN 5 AND 10 - STORY BUILDINGS

Linear Static Analysis

In the linear static analysis, the column is removed from its location and linear static analysis with the gravity load given by Eq.1 imposed on the structure is carried out. In accordance with the following equation, the linear static analysis was evaluated by the Demand to Capacity Ratio (DCR), for all beams in shear or moment.

$$DCR = \frac{Q_{UD}}{Q_{CE}} \tag{4}$$

$$DCR_M = \frac{M_{max}}{M_p} \tag{5}$$

$$DCR_V = \frac{V_{max}}{V_u} \tag{6}$$

The demand to capacity ratio calculated from linear static procedure helps to determine the potential for progressive collapse of building. This static linear procedure introduces the notion of demand to capacity DCR ratios and specifies the DCR limit values to be used, depending on the cross-sectional dimensions and on the construction materials reinforced concrete or steel (Menchel et al., 2009). In Eq. (4), Q_{UD} and Q_{CE} are the force (bending moment, axial force and shear force) determined in the analysis and the expected capacity in a component or the connection (bending moment, axial force and shear force), respectively. Q_{UD} is defined as maximum shear force or bending moment in the concrete beam due to using the proposed loading composition in the GSA or DOD guidelines for progressive collapse in buildings. In Eqs. (5) and (6), where M_{max} and V_{max} are equal to the bending moment and shear force demand calculated using linear elastic static analysis from SAP2000 and M_p and V_u are equal to the ultimate bending moment (plastic moment) and ultimate shear strength, respectively. They can be calculated for each structural member. Using these two values, the DCR value for each structural member of the building was calculated. Using the DCR criteria, structural members and connections that have DCR_V (shear demand-capacity ratio) values greater than 2.0 are considered to be severely damaged or collapsed. It is, therefore, unlikely that the structural member or connection will have adequate reserve ductility for effectively redistributing loads. Since the building has a typical structural configuration, the acceptance criterion for the primary structural components is DCR ≤ 2.0. When the DCR_V value of an end section is larger than 2.0, a hinge has to be inserted at the member end for releasing the moment. If in the linear analysis the rate of DCR_V according to GSA guideline exceeds 2, a hinge is placed at both ends of the beam, so components will be severely damaged that will lead to collapse, as shown in Figure 3. Therefore in the linear static analysis method it is necessary to control the ratio of DCR.

Linear static analysis method in 5 and 10-story structure at three different positions was studied by sudden removal of a column of the first floor that possibly leads to progressive collapse in the structure. The Demand to Capacity Ratio (DCR) value based on the Eq. (3) due to the column removal at three different positions C1, C11 and C13 is calculated for the 5 and 10-story buildings as illustrated in Figure 4.

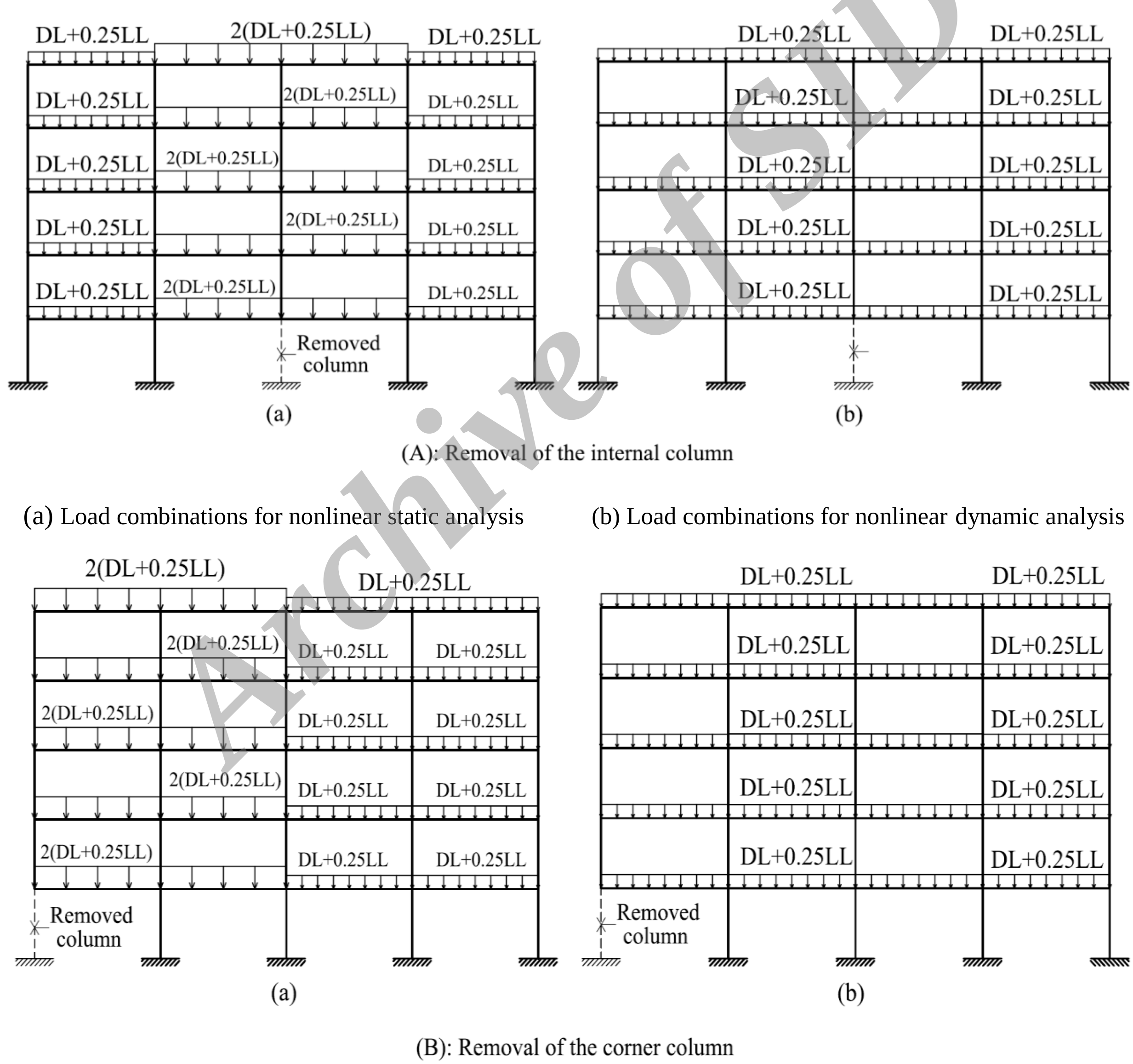


Fig. 2. The amplification factor of 2 according to GSA (2005) guideline is applied to account for dynamic effects.

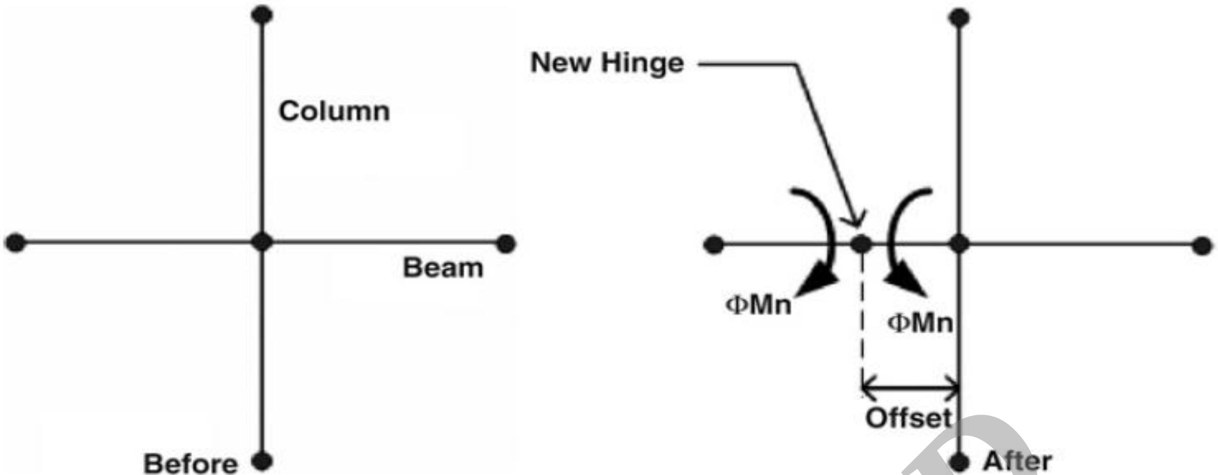
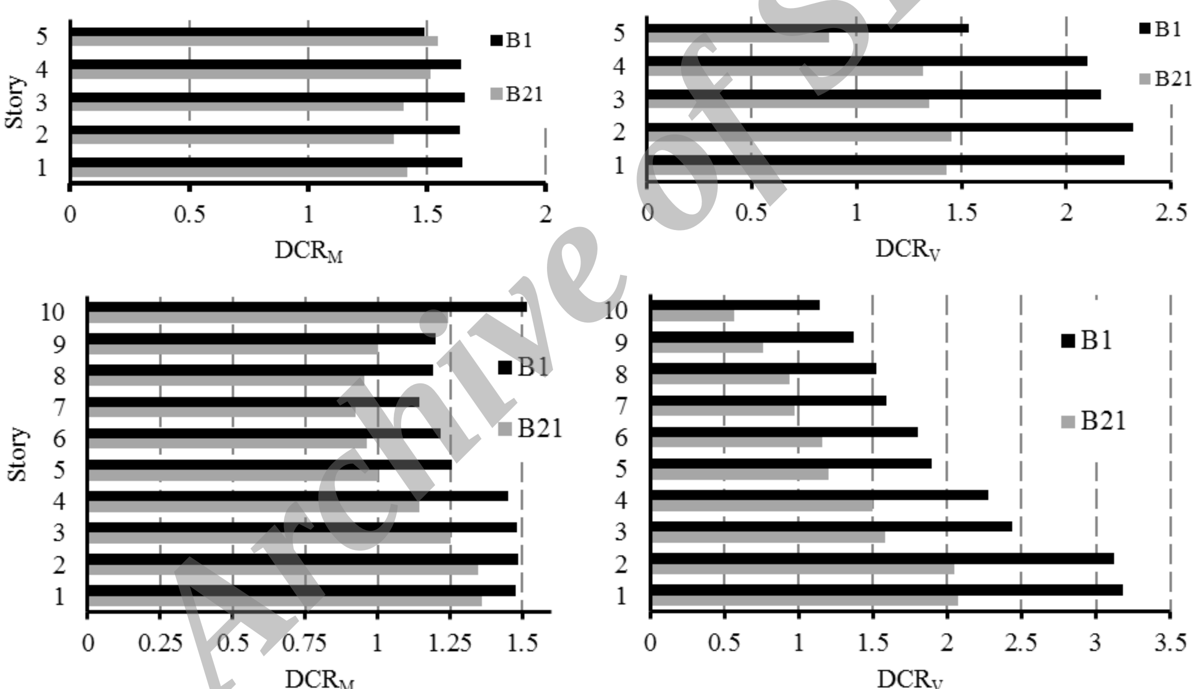
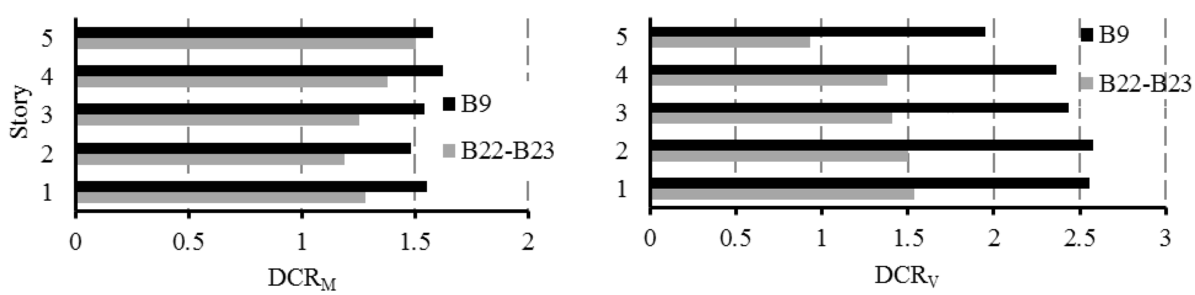
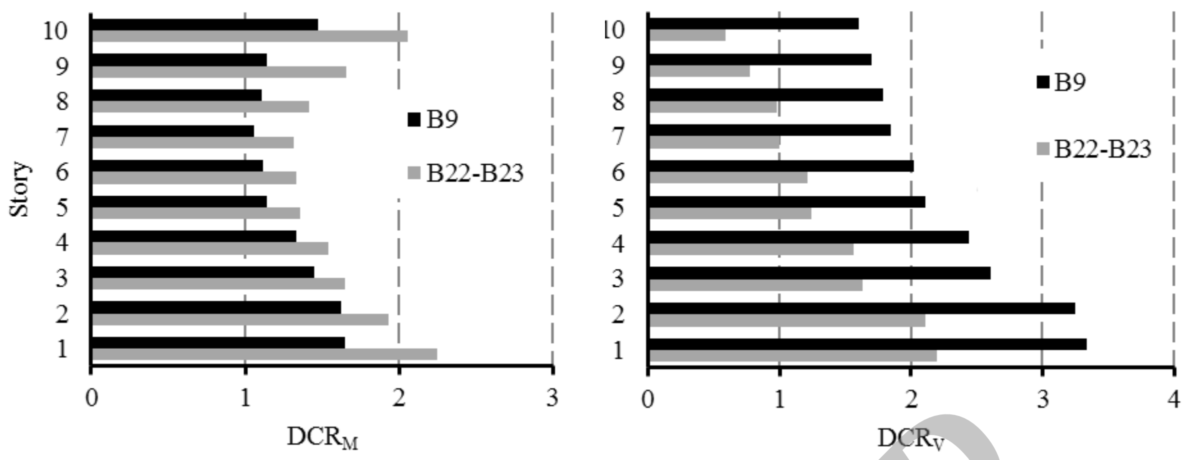


Fig. 3. Modeling of hinges (DoD, 2005).

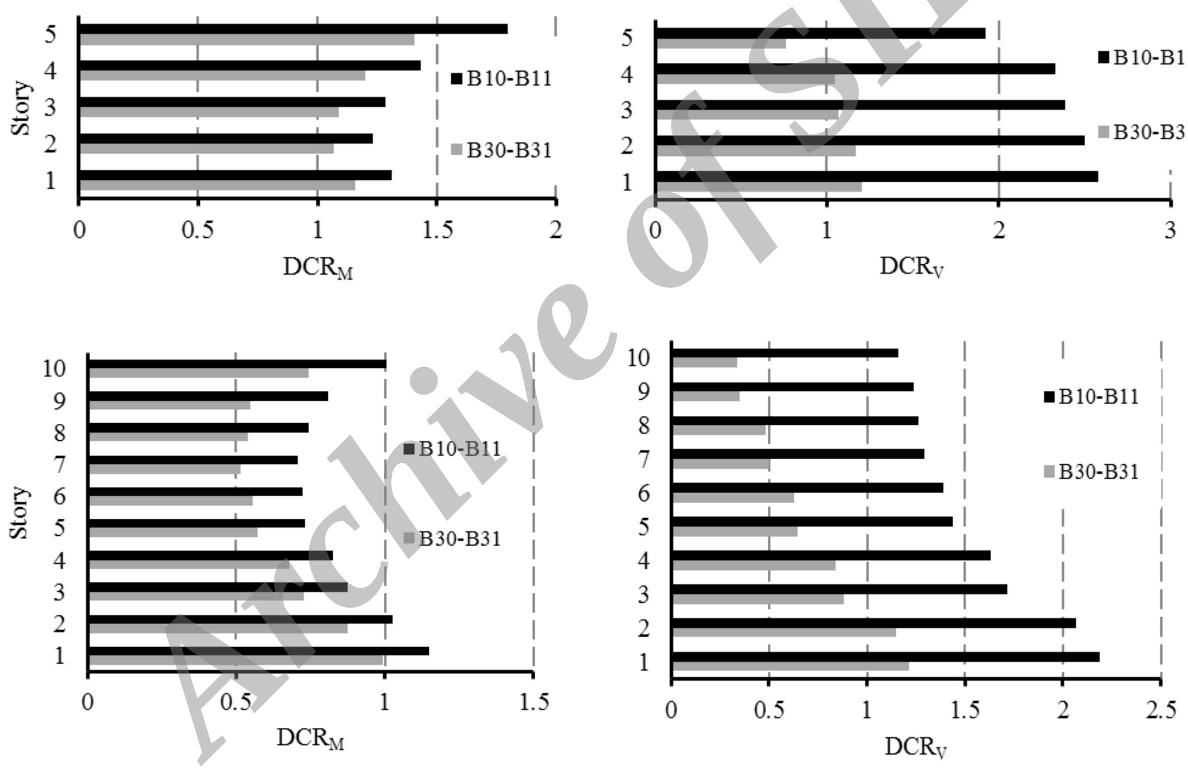


(a) Sudden removal of the column C1.





(b) Sudden removal of the column C11.



(c) Sudden removal of the column C13.

Fig. 4. DCR compared in the bending moment and shear force due to column removal in the position C1, C11 and C13 for 5 and 10-story concrete buildings.

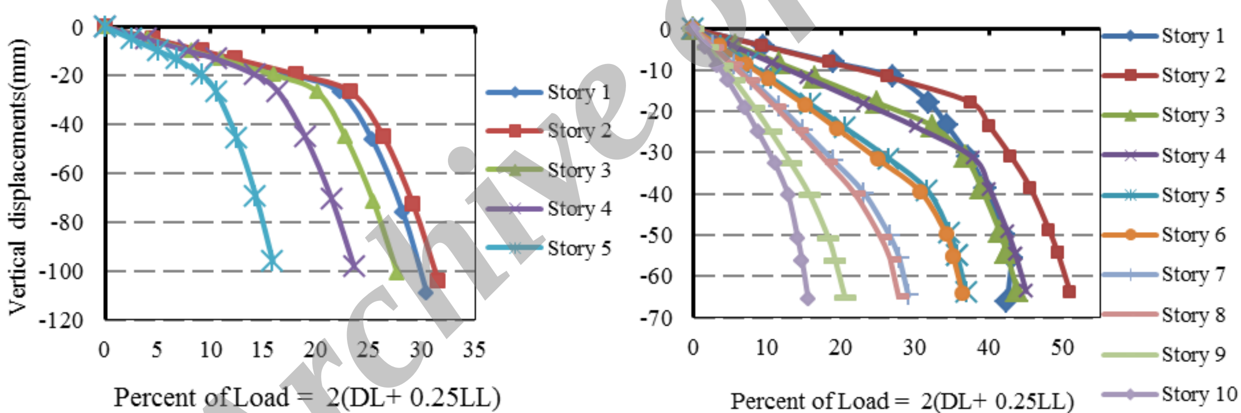
Figure 4 shows that the DCR values in shear due to column removal in lower stories are more than the case for upper stories. In accordance with GSA guideline, if the rate of DCR according to GSA guideline exceeds 2, a hinge is placed at both ends of the beam, so the component will be severely damaged and thereby leading to collapse (GSA, 2003).

Nonlinear Static Analysis

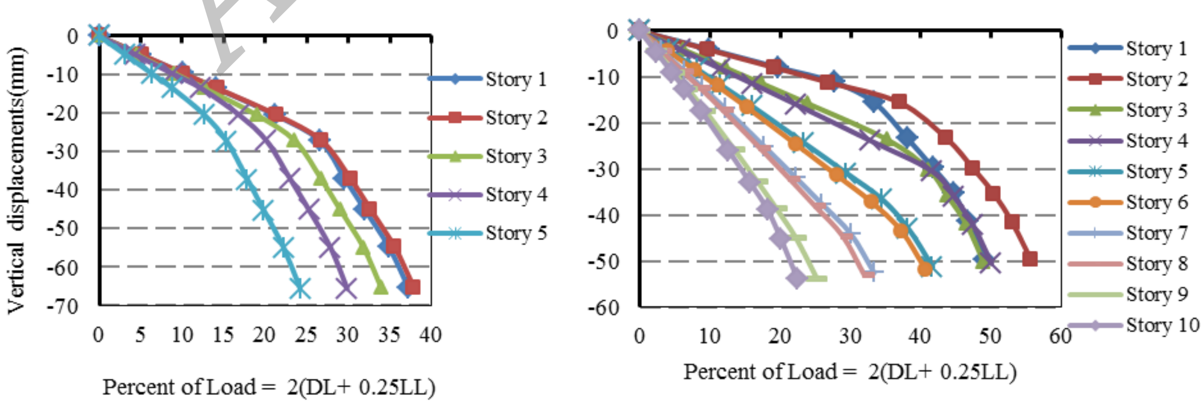
Since in the model, shear force before forming plastic hinges is not significant, so formation of the hinge is not considered as combination of bending and shear force and bending and shear hinges are separately allocated to the beams. Since the axial force and bending moment in columns are combined in plastic hinge, plastic hinge interaction effect of bending moment and axial force should be used. Therefore, the P-MM and P plastic hinges are allocated to columns and M3 and V2 hinges to beams separately and it is not required for additional calculations for each component because the program SAP2000 uses the specifications plastic hinge based on FEMA-356 (2000) guidelines.

Nonlinear material specifications are applied to the analysis using plastic hinges.

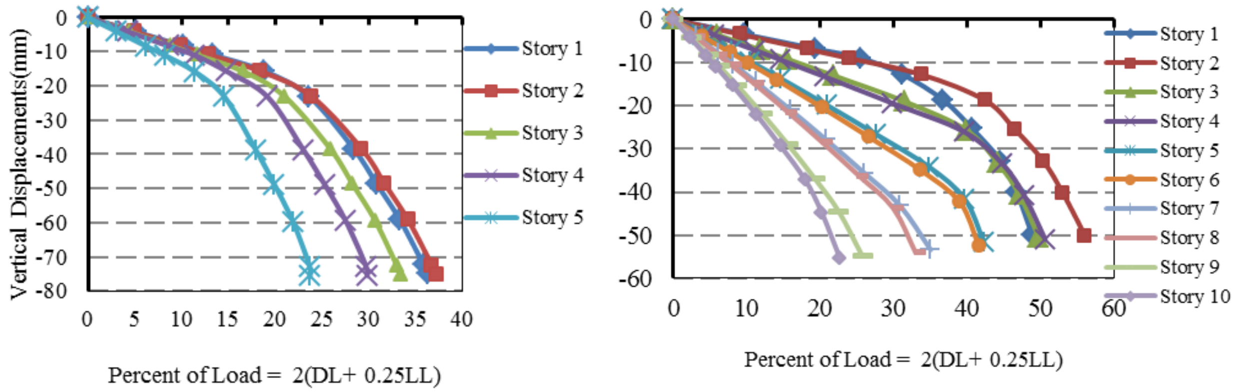
The formation of the plastic hinge is permitted if structure can maintain its stability generally or locally until it reaches the stage of plastic failure or the balance. Since abnormal loading (explosion or impact) due to sudden column removal in the structure cause the structural system leading to progressive collapse, it will create large deformation and nonlinear response of materials, hence in the nonlinear static analysis method it is needed to consider geometric and material nonlinear behavior. Therefore, nonlinear effects related to materials can be applied by activating large deformation and P-delta options. After column removal at different positions (C1, C11 and C13) of the first story and carrying out nonlinear static analysis method, the load-vertical displacement graph of beams for both 5 and 10-story structures are obtained as shown in Figure 5.



(a) Sudden removal of the column C1.



(b) Sudden removal of the column C11.



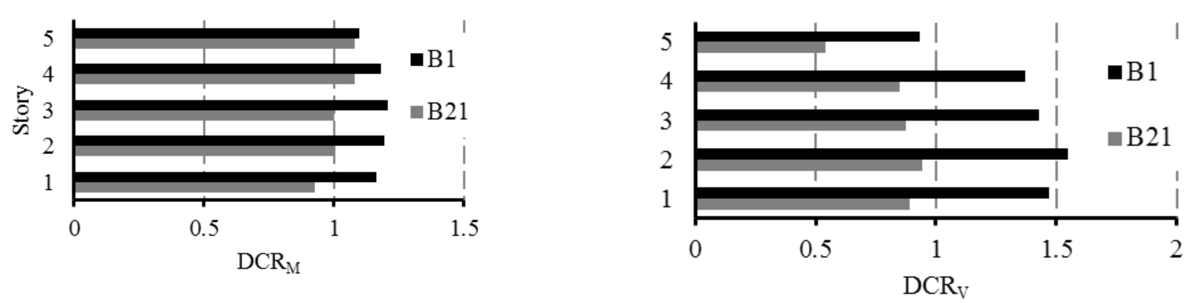
(c) Sudden removal of the column C13.

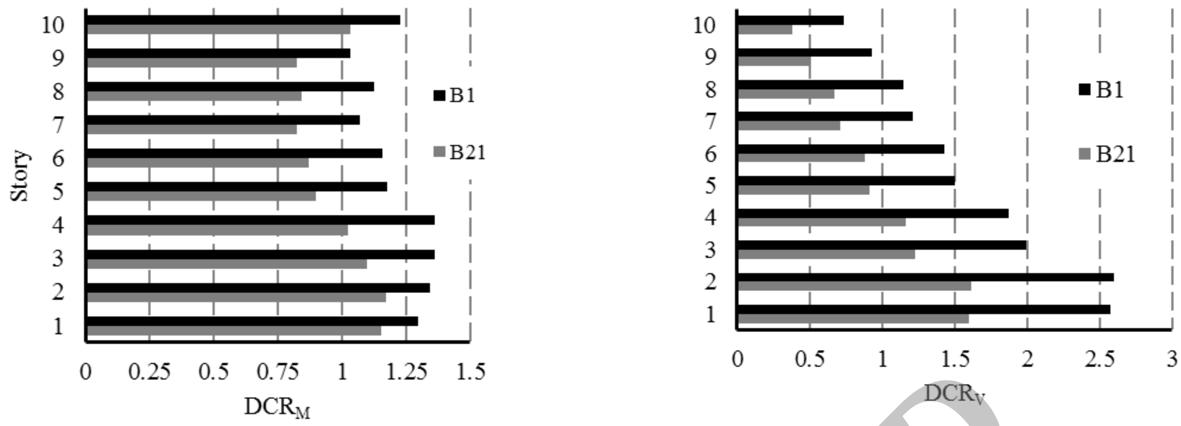
Fig. 5. Calculated load–displacement graphs.

The advantage of nonlinear static procedure is its ability to account for nonlinear effects. In the nonlinear static procedure, pushover load-displacement results can predict the mechanism of progressive collapse in the structure. The area under the curve indicates the amount of energy dissipated by the structure, the greater the area under the graph, the higher the ability of structure in absorption and dissipation of energy. The lowest area for force-displacement curves for both 5- and 10-story structures are in the fifth and tenth floors, respectively, as shown in Figure 5. Therefore in both 5- and 10-story buildings, structural failure might begin form top floor.

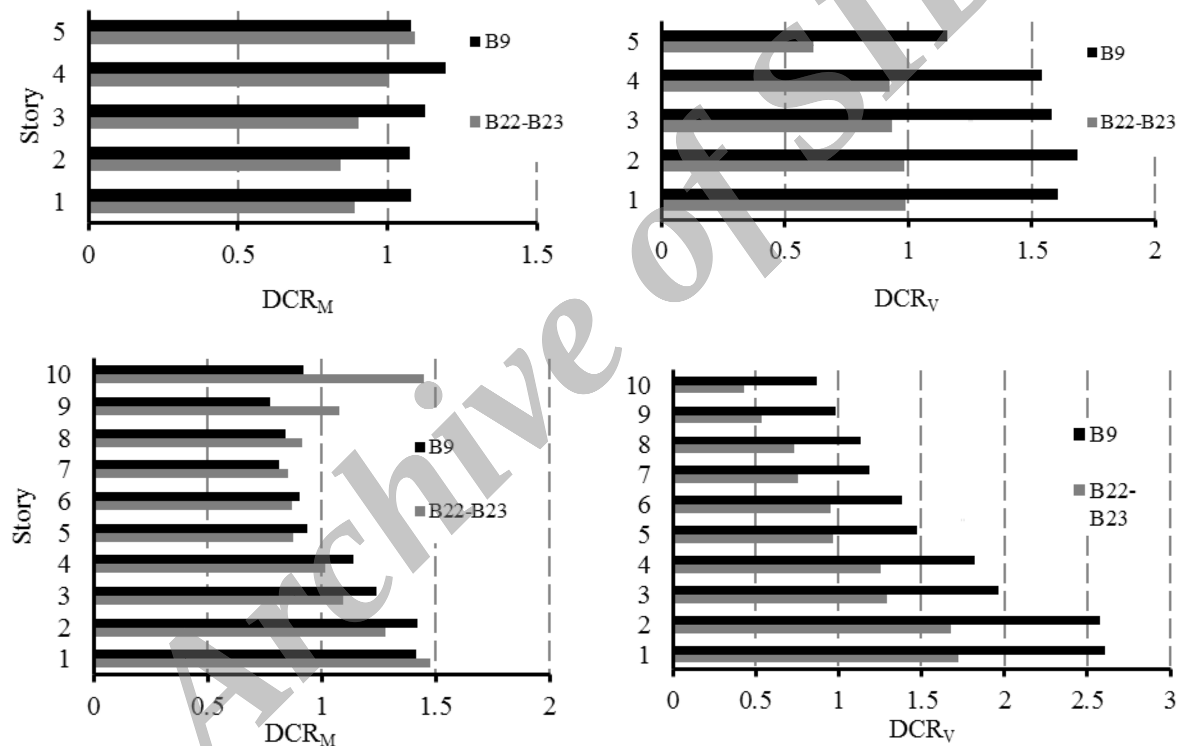
Linear Dynamic Analysis

Linear dynamic method inherently incorporates amplification factors, inertia, and damping forces (Buscemi and Marjanishvili, 2005). The main advantage of this procedure is its ability to account for dynamic amplification effects. This procedure is limited to structures that are expected to remain elastic during the event. To perform linear dynamic analysis method in progressive collapse the initial conditions method of Buscemi and Marjanishvili (2005) is used. In the linear dynamic analysis, after modeling by software SAP2000 software (CSI, 2009), time history analysis is carried out with activating the zero initial conditions option in the software SAP2000, then DCR_M and DCR_V values are calculated based on Eqs. (4) and (5), respectively. The DCR values at three different positions (C1, C11 and C13) for both 5- and 10- story buildings due to the column removal are compared as shown in Figure 6. The amount of vertical displacement of the beam due to the column removal at three different positions with respect to time is shown in Figure 7. Damping ratio was assumed to be 5% of the critical damping, which is usually adopted for analysis of structures undergoing large deformation.

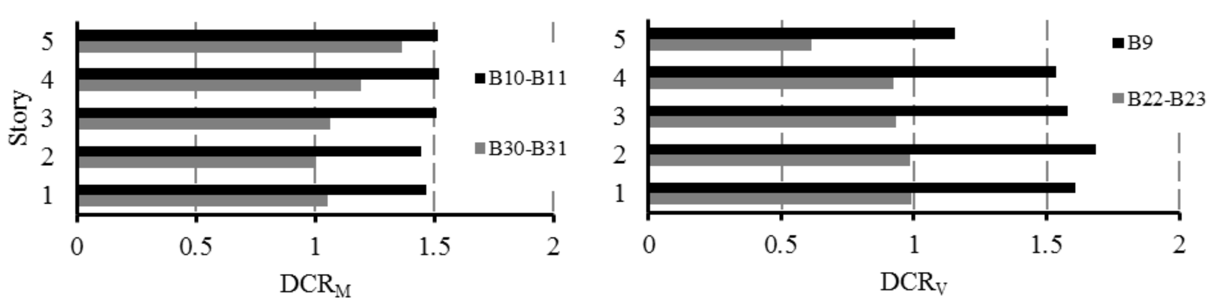


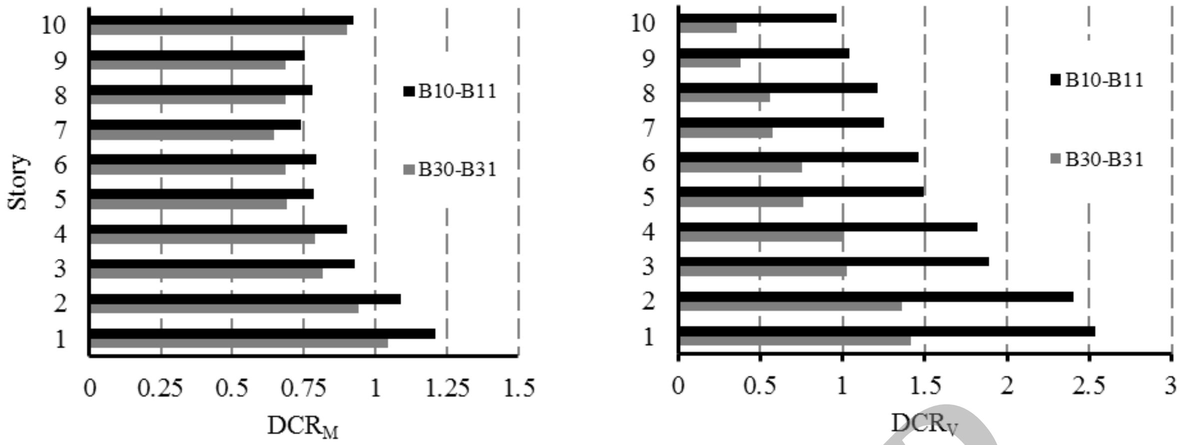


(a) Sudden removal of the column C1.



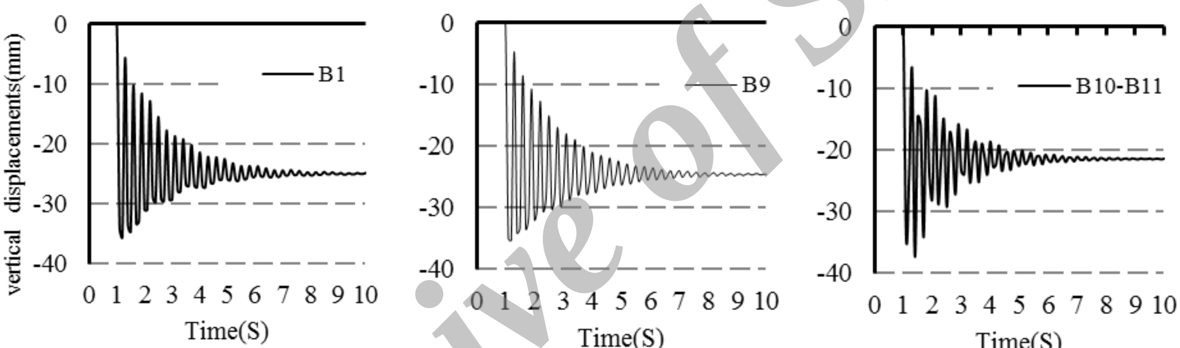
(b) Sudden removal of the column C11.



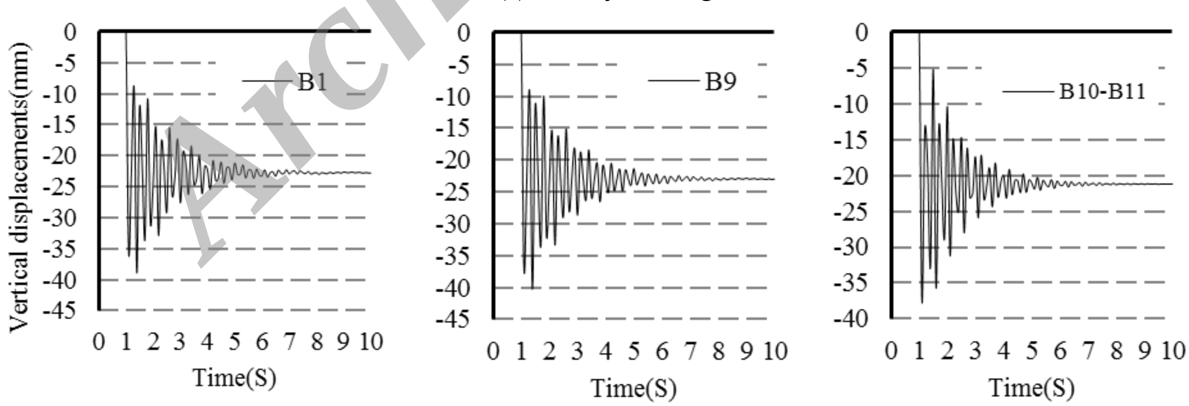


(c) Sudden removal of the column C13.

Fig. 6. DCR values compared for the bending moment and shear force due to column removal in the position C1, C11 and C13 for concrete 5 and 10-story buildings.



(a) 5 story building



(b) 10 story building

Fig. 7. Displacement time history at the joints when the C1, C2 and C3 column are removed.

Results of the linear dynamic analysis procedure are presented in Figures 6 and 7. As shown in Figure 6, the DCR values in the shear due to the column removal is more in the lower stories than in the upper stories; so 10-story structure is susceptible to progressive collapse according to GSA guidelines. If the DCR is greater than the number 2, plastic hinges will be established at the two ends of the beam and severe damage at members might lead to collapse.

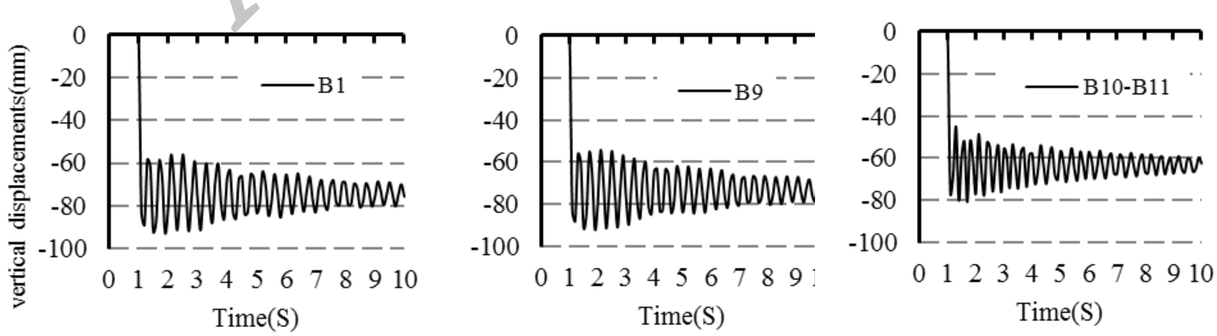
As shown in Figure 7, the maximum displacement with respect to time history in column removal in the middle part is greater and more critical. Thus removing the column in the middle part (C13) a general collapse does occur; while the created displacement due to removal of corner column (C1) will lead to local failure of structure.

Nonlinear Dynamic Analysis

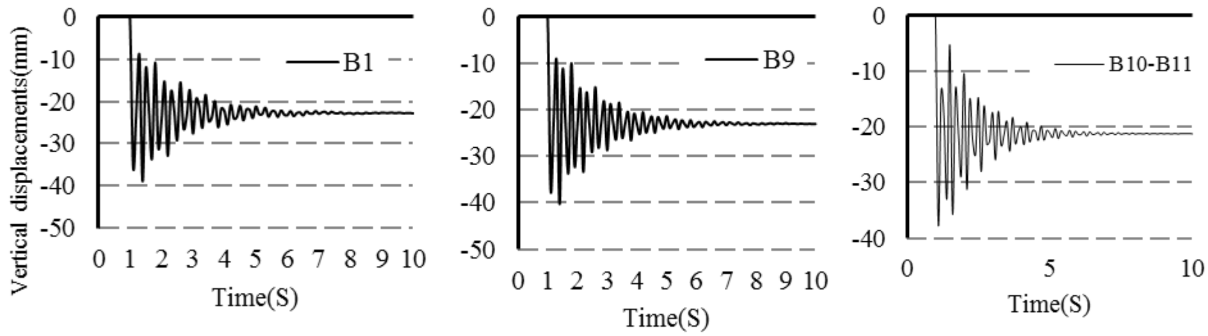
Nonlinear dynamic method is performed similar to linear dynamic procedure. Nonlinear dynamic analysis allows structural elements to enter their inelastic range creating energy dissipation and larger deformations through material yielding, cracking, and fracture. Failure mechanism of a member performed using time history analysis with direct nonlinear integration, until a stable and physically possible solution is found; also it uses the initial conditions methodology. By activating "continue from state at end of nonlinear case" option there is a possibility of movement, speed, tension, loads, energy and nonlinear deformation histories as a result of time history analysis with nonlinear direct integration. Figure 8 shows the graph of the beam vertical displacement with respect to time due to the column removal at three different positions in both 5- and 10- story building. Damping ratio was assumed to be 5% of the critical damping, which is usually adopted for analysis of structures undergoing large deformation.

COMPARING RESULTS OF DIFFERENT ANALYSIS METHODS FOR 5 AND 10-STORY CONCRETE BUILDINGS

As mentioned earlier, progressive collapse is a nonlinear and dynamic phenomenon occurred in much less time in which frame and structural members undergo nonlinear deformation before failure. To accurately analyze the progressive collapse potential of structure, nonlinear dynamic analysis should be performed to account for the energy dissipation in the whole structure. Therefore, in the GSA and DoD guidelines it is recommended to use dynamic amplification factor of 2.0 in load combination to apply the dynamic effects for static analysis method. Figure 9 shows a comparison of different analysis methods performed for 5- and 10- story building in the first story and implies that nonlinear dynamic analysis method is safe and accurate method for analysis of the building regarding the progressive collapse and this is compatible with the research results of Marjanishvili et al. (2006).

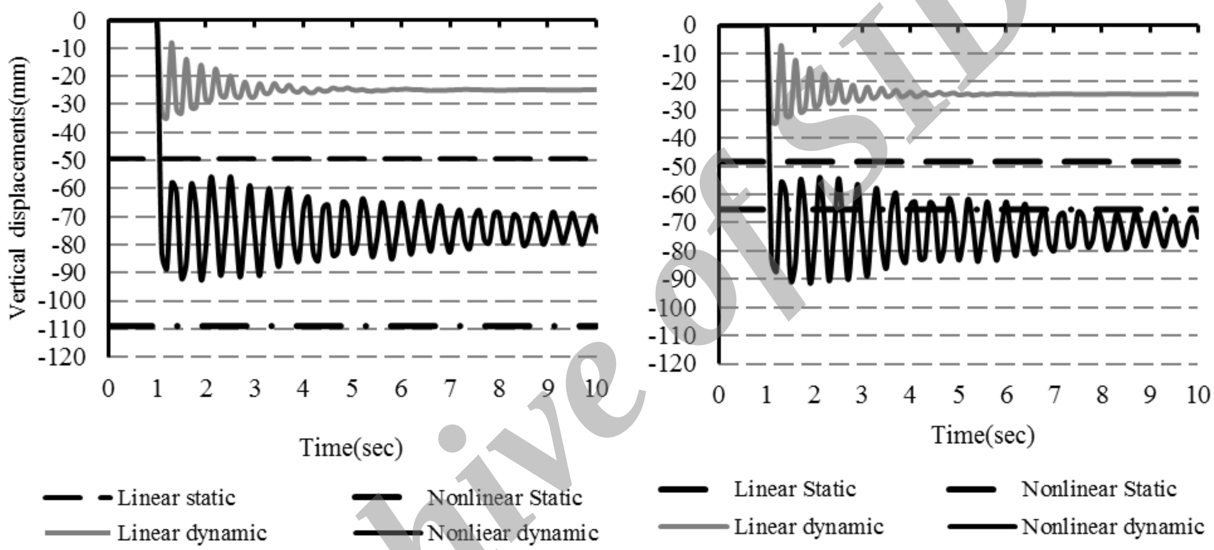


(a) 5 story building.

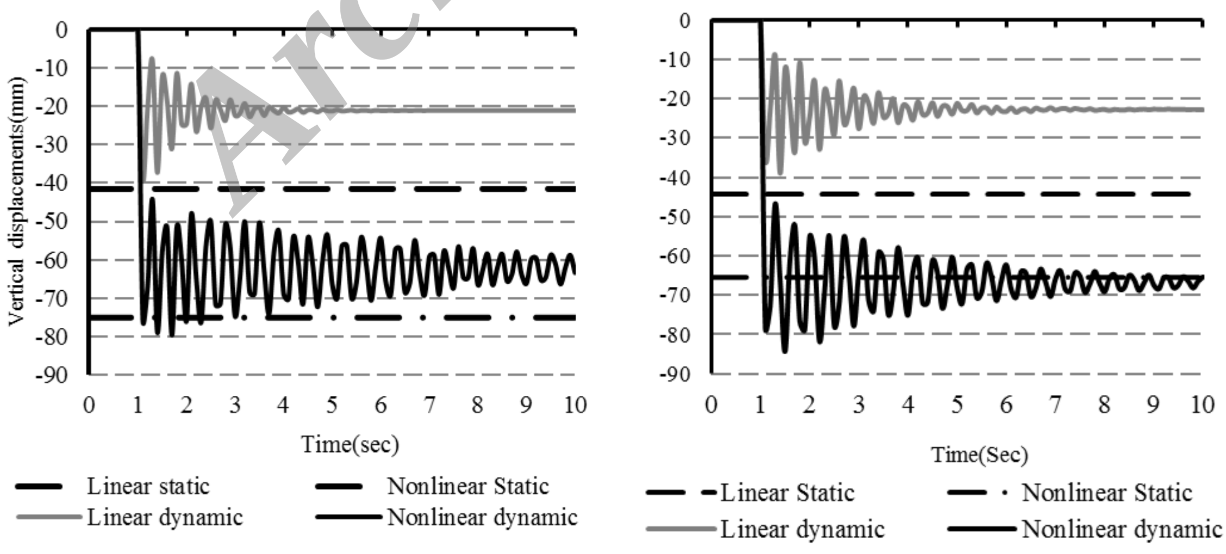


(b) 10 story building.

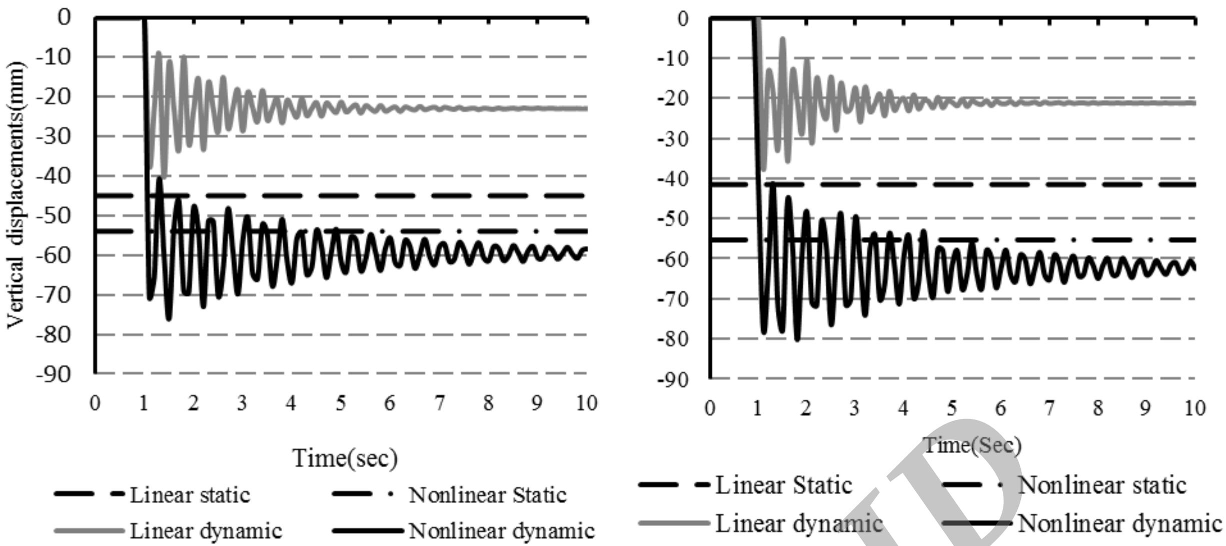
Fig. 8. Displacement time history at the joints when C1, C2 and C3 column is removed for linear dynamic analysis method.



(a) Sudden removal of the column C1.



(b) Sudden removal of the column C11.



(c) Sudden removal of the column C13.

Fig. 9. Comparison of the linear and the nonlinear static and dynamic analyses results for 5- and 10- story building.

Analysis results of the nonlinear dynamic procedure are presented in Figure 9, showing the deflection time history of all four analysis procedures. Results of the static analysis procedures are shown as constant horizontal lines. From Figure 9, the following observations can be made:

1- Maximum displacements calculated for 5- and 10-story building by linear static and linear dynamic analysis procedures regarding the removal of columns at different position as shown in Table 4. These are relatively close, which leads to the conclusion that the dynamic amplification factor of 2 used in the linear static load case is a good estimate.

2- A fiddling difference between calculated deflections based on linear static and linear dynamic analysis is influenced by an equivalent viscous damping ratio of 5% of critical, which was assumed throughout the analysis. Although theoretically it is possible to reduce static deflection to match linear dynamic deflection, it is not recommended, since the static approach is not capable of predicting dynamic response with good accuracy.

3- Maximum calculated deflections due to sudden removal of column C1 for 5- and 10-story building are -109, -91 mm, respectively, which are greater than those for abrupt removal of columns C11 and C13. Therefore, sudden removal of column C1 (corner column) and C13 (center column) leads to partial or total collapse, respectively.

Table 4. Vertical displacements (mm) obtained due to sudden column removal at different positions.

linear static and dynamic analysis of analysis 5- and 10-story building	5-story building			10-story building		
	C1	C11	C13	C1	C11	C13
Linear Static	49.61	48.74	42.15	44.82	43.01	41.92
Linear Dynamic	35.53	35.31	37.27	38.94	40.28	37.65

CONCLUSIONS

According to the performed analyses observed in this paper, following conclusions are obtained:

1- To analyze rigorously progressive collapse potential of a structure, nonlinear dynamic analysis should be performed to account for energy dissipation, large inelastic deformations, materials yielding, cracking and fracture. The amount of calculated displacement in nonlinear dynamic analysis is more accurate than other analysis methods. However, the nonlinear dynamic method is complicated and requires a long time to perform the progressive collapse analysis of the building so that for facilitation and expedition, nonlinear static analysis is used in which the proposed load combination is multiplied by the coefficient of 2 to consider the dynamic effects in the structure.

2- According to the results obtained from linear static and dynamic analysis, the column removal in the lower floors established a more critical state in the building because the DCR due to column removal is more in the lower floors and in fact according to GSA regulations, if the DCR is greater than 2, plastic hinge and severe damage will occur at two ends of the beam leading to collapse. But opposite is true for the bending moment.

3- The linear dynamic analysis procedure may be used when the nonlinear response of the structure can easily and intuitively be predicted. Results from a linear static analysis procedure can be used to validate analysis results by comparing maximum dynamic deflection and static deflection due to amplified (by a factor of 2) load combinations. Again, demand to capacity ratios (DCR) value in shear for linear analysis procedures should be limited to a maximum of 2. To avoid the progressive collapse of beams and columns, caused by damage of particular column, sufficient reinforcement is required to limit the DCR according to the acceptance criteria.

4- Comparing the results obtained from static and dynamic analysis used in this paper for two 5 and 10-story concrete buildings and applying the proposed combination of the GSA regulations show that, the vertical displacement of the beam due to column removal by this abnormal loading (explosion or collision) is similar when the static method is considered. A slight difference between calculated deflections based on linear static and linear dynamic analysis is influenced by an equivalent viscous damping ratio of 5% assumed throughout the analysis.

Finally, structures designed with an adequate level of reinforcement can develop alternative load paths following the loss of an individual member and transfer of the load to ground in case of failure or local collapse of structures to prevent progressive collapse and maintain their serviceability.

REFERENCES

American Concrete Institute (ACI), (2008). *Building code requirements for structural concrete and commentary (ACI 318m-08)*, Detroit, Michigan.

ASCE. (2005). *SEI/ASCE 7-05 minimum design loads for buildings and other structures*. Washington DC, *American Society of Civil Engineers*.

BHRC. (2007). *Iranian Code of Practice for Seismic Resistant Design of Buildings*, Standard No. 2800 (3rd Edition), Building and Housing Research Center. BHRC Publication No. S – 465, 1st print.

Buscemi, N. and Marjanishvili, S.M. (2005). "SDOF model for progressive collapse analysis", *Proceedings of SEI Structures Congress, ASCE, Reston, Va.*

CSI, SAP2000 V-14.0.0, (2009). *Integrated finite element analysis and design of structures basic analysis reference manual*, Computers and Structures Inc., Berkeley, CA, USA.

Department of Defense (DoD). (2009). *Design of buildings to resist progressive collapse*, (UFC 4-023-03). Washington D.C.

FEMA 274. (1997). *NEHRP Commentary on the guidelines for the seismic rehabilitation of buildings*, Washington D.C., Federal Emergency Management Agency.

FEMA-356, (2000). *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, Prepared by American Society of Civil Engineers, Reston, Virginia, prepared for Federal Emergency Management Agency, Washington D.C., Nov.

Grierson, D., Safi, M., Xu, L. and Liu, Y. (2005). "Simplified methods for progressive collapse analysis of buildings", *Proceedings of Metropolis and Beyond Structures Congress,* ASCE, Reston, VA.

Hansen, E., Wong, F., Lawver, D., Oneto, R., Tennant, D. and Ettouney, M. (2005). "Development of an analytical database to support a fast running progressive collapse assessment tool", *Proceedings of Metropolis and Beyond-Structures Congress,* ASCE, Reston, VA.

Iranian code of practice for seismic resistant design of buildings. (2007). *Standard 2800*, 3rd Edition, BHRC Publication No. S – 465, 1st print.

Kaewkulchai, G. and Williamson, E. (2003). "Dynamic behavior of planar frames during progressive collapse". Proceedings of the *16th ASCE Engineering Mechanics Conference*, University of Washington, Seattle.

Kim, H.S., Kim, J. and An, D.W. (2009). "Development of integrated system for progressive collapse analysis of building structures considering dynamic effects", *Advances in Engineering Software, 40, 1–8.*

Kim, J. and Kim, T. (2009). "Assessment of progressive collapse-resisting capacity of steel moment frames", *Journal of Constructional Steel Research, 65(1), 169 - 179.*

Marjanishvili, S. and Agnew, E. (2006). "Comparison of various procedures for progressive collapse analysis", *Journal of Performance of Constructed Facilities,* ASCE, 20(4), 365–374.

Menchel, K., Thierry, J., Rammer, M.Y. and Bouillard, P. (2009). "Comparison and study of different progressive collapse simulation techniques for RC structures". *Journal of Structural Engineering,* ASCE, 135(6), 685-697.

Mohamed, O.A. (2009). "Assessment of progressive collapse potential in corner floor panels of reinforced concrete buildings", *Engineering Structures*, 31(3), 749-757.

National Institute of Standard and Technology (NIST 2007). *Best practices for reducing the potential for progressive collapse in buildings*, (Draft). U.S. Department of Commerce.

Powell, G. (2005). "Progressive collapse: Case study using nonlinear analysis", *Proceedings of the 2005 Structures Congress and the 2005 Forensic Engineering Symposium*, New York, 2185–2198.

Pretlove, A.J., Ramsden, M. and Atkins, A.G. (1991).

"Dynamic effects in progressive failure of structures". *International Journal of Impact Engineering*, 11(4), 539-546.

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

Unified Facilities Criteria (UFC)-DoD, (2005). *Design of Buildings to Resist Progressive Collapse*, Department of Defense, Washington D.C.