

Research Article

Assessment the Behavior of Seismic Designed Steel Moment Frames Subjected to Progressive Collapse

Azlan Bin Adnan, Iman Faridmehr, Babak Faramanbordar, Reza Hodjati and

Mohammad Gharehzadeh Shirazi, A.B.A. Rahman

Department of Civil Engineering, Universiti Teknologi Malaysia, 81310,

UTM Skudai, Johor, Malaysia

Abstract: Recent investigations reveal that progressive collapse phenomenon is dominant behavior in the majority of steel structures. Although the design of buildings is based on the fact that they need to withstand all the loads exerted on the structure, failure occurs as a result of inadequate design and modeling techniques, particularly for abnormal and extreme loading conditions. Once one or more load bearing member is eliminated from the structure, progressive collapse, mainly in columns, will commence. By the time a column is eliminated from the structure as a result of a sudden motor vehicle strike or earthquake or fire or any other internal or external factor that could take one column out of the system, the weight of the building (gravity load) will be distributed among other columns within the structure. Failure commences in the part of the structure that has lost a column unless other columns are designed appropriately against gravity loads and are capable of redistribution of additional loads imposed on them. Failure of vertical load bearing elements will continue until the stabilization of extra loading. Hence, this could lead to serious damage and collapse of the building which will lead to higher damage to the building than the primary damage. This research is based on the regulations conforming to the specifications of UFC guidelines and the structures have been modeled using SAP2000 (2012). In order to study the effects of the progressive collapse on the seismic design of special steel moment frames, SMRF, two 5-story and 15-story structures are modeled in SAP2000 (2012). In order to have a better understanding of progressive collapse and obtain reliable results, Linear Static (LS), Nonlinear Static (NLS) and Nonlinear Dynamic analyses (NLD) procedure for single and 2 adjacent columns removal have been implemented in this study. Having a good perception of the possibility of progressive collapse involves incorporation of demand capacity ratio, plastic hinges formation and vertical displacements of removed column's location plus axial force in columns adjacent to the removed column. Other factors such as number of stories and the amount of local damage resulted from the removal of 2 adjacent columns could also lead to a better understanding of the structural behavior.

Keywords: Column removal location, progressive collapse, steel moment frame, vertical pushover analysis

INTRODUCTION

After the 9/11 attacks and collapse of the World Trade Center towers in 2001, developers and governmental organizations have shown great interest in finding ways to evaluate the potential of progressive collapse of structures and also design new structures to withstand this kind of threat. Although some literature is available regarding this issue in previous decades, the extent of work that has been done in this area is negligible. For instance, in 1968, a study was performed after the collapse of Ronan Point building in Britain; however, this investigation did not last long. Lately, updated guidelines for the purpose of evaluating the progressive collapse potential of structures have been published by the General Services Administration and Department of Defense (GSA, 2000; (UFC)-DoD,

2010). In order to localize the primary damage and avoid its spread to other parts of the structure, a series of direct and indirect design approaches are introduced by UFC documents. Alternate Path method (AP) which is a direct design approach includes the ability of a structure to bridge over the failed column and its ability to localize the amount of damage regardless of the cause of the failure of the column. This study is based on the 2010 edition of the UFC 4-023-03 ((UFC)-DoD, 2010) which describes the design of structures subjected to potential terrorist attacks. Since in the majority of building design codes, the progressive collapse issue has been addressed implicitly, an attempt has been made in this study to understand whether the structures that have been designed for common load combinations under gravitational and seismic loading are prone to progressive collapse. Jinkoo and Taewan (2009)

Corresponding Author: Iman Faridmehr, Department of Civil Engineering, Universiti Teknologi Malaysia, 81310, UTM Skudai, Johor, Malaysia

This work is licensed under a Creative Commons Attribution 4.0 International License (URL: <http://creativecommons.org/licenses/by/4.0/>).

compared linear static and nonlinear dynamic analysis methods in their work. Their investigation resulted in larger structural responses for nonlinear dynamic analysis than the linear analysis results. Their results were based on variables such as applied load, location of removed column or number of stories in a structure. A new empirical method based on Dynamic Increase Factor (DIF) which is hired for amplification of gravity loads on the affected bays of a structure was done by Liu (2013).

Once the nonlinear static alternate path analysis is performed, this method is used to predict the peak dynamic responses as a result of sudden removal of columns. In order to design economical seismic steel moment frames with an increased resistance to progressive collapse, Liu (2011) attempted to study optimization techniques for structures. This trend was accelerated by sudden elimination of critical columns. Retrofitting buildings against progressive collapse was also done by Hadi and Saeed (2012). In their study, the column of the first floor of a building failed and hence removed. A progressive collapse analysis on steel structures was performed by Khalil (2012) conforming to the specifications of the U.S. Department of Defense ((UFC)-DoD, 2010) and the General Services Administration (GSA, 2000). The Applied Element Method was suggested by him as an alternative beside

the progressive collapse analysis. Department of Civil and Environmental Engineering, Imperial College London (Department of Civil and Environmental Engineering, 2011) attempted to investigate and compare some design approaches including the relationship between “increases in tying capacity and actual resistance against progressive collapse”. Also, balancing strength, stiffness and ductility in the connections was in their interest. The behavior of various reinforced-concrete and steel moment-frame structures was studied to understand the amount and variation of the dynamic and nonlinear load increase factors. It was observed that over conservative designs will result from the current guidelines and retrofits. In this study, new load increase factors were introduced and an alternative method to incorporate these factors when Alternative Path Analysis for progressive collapse is recommended. The objective of this study is to assess the progressive collapse potential of steel moment frames designed according to AISC (2003) Load and Resistance Factor Design. The results of the linear static and dynamic step-by-step analysis procedure recommended by the DoD 2005 ((UFC)-DoD, 2010) guidelines were compared with those of nonlinear dynamic analysis. The effect of the parameters such as the location of column removal and the number of stories were also investigated.

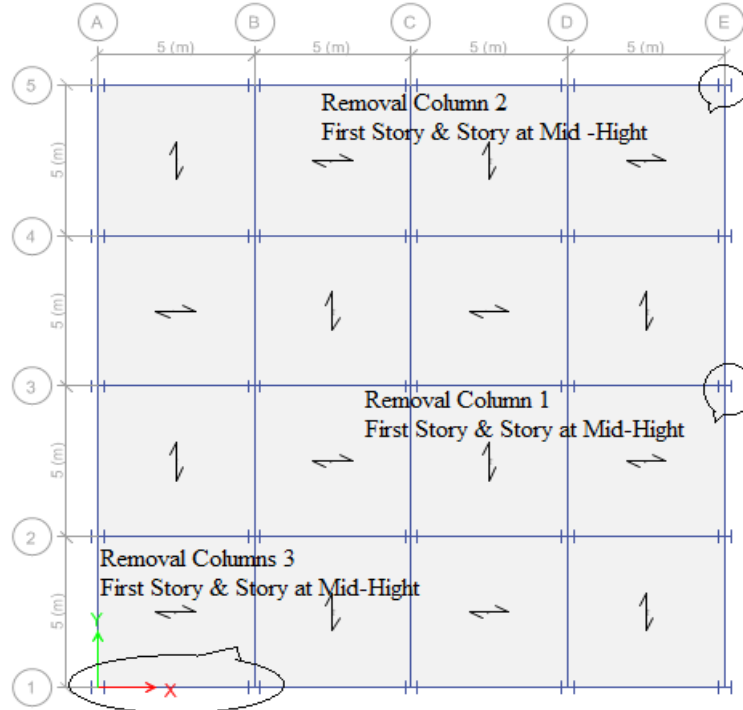


Fig. 1: Plan view of the structures and column removal locations

Table 1: Material properties

	ρ (kg/m ³)	γ (kgf/m ³)	ν	E (kgf/cm ²)	F_y (kgf/cm ²)	F_u (kgf/cm ²)	
Steel material property	800	7850	0.3	$E = 2.1 \times 10^6$	2400	3700	-
Concrete material property	245	2400	0.2	$E_c = 2.19 \times 10^4$	$f_y = 4000$	$f_{ys} = 3000$	$F'_c = 210$ (kgf/cm ²)

METHODOLOGY

Configuration and analytical modeling of preferred structures: Two 5 and 15-story three-dimensional models using Special Moment Resistant Frame (SMRF) were used in this study. Each plan is composed of 5 spans with the length of 5 m. The story height is 3 m. No underground parking was included in the model. The Dead Loads (DL) which are applied on the roof and other floors are 550 and 650 (kg/m²), respectively. The service Live Loads (LL) which are applied on the roof and other floors are 150 and 200 (kg/m²), respectively. The distributed loads on perimeter walls are 240 and 900 (kg/m), respectively. The same model is used for linear static, nonlinear static and nonlinear dynamic analysis using UFC 04-23-03 ((UFC)-DoD, 2010). 50 or more resident patients occupy the structure which places the structure in Occupancy category III and zone 4 for seismic consideration. Alternate Path Method is required for this occupancy category. It chooses elements to check their capacity for the purposes of progressive collapse resistance. Material properties for beams and columns are demonstrated in Table 1.

In this study, the selected columns that are removed were the corner and middle perimeter columns for the first and mid-height story of the 5 and 15-story structure, respectively. One column is removed in each analysis only. Hence, 4 different analyses were performed on each structure. In a similar trend, the amount of initial local damage was studied by removing two adjacent perimeter columns for the first and 8th floor of the 15-story structure. Figure 1 demonstrates the plan view for the removal location of columns in the two structures.

Table 2 and 3 demonstrate the preliminary design of columns conforming to the specifications of AISC-ASD89 seismic loading. Compact cross sections are used for all steel shapes in the structure. Improved WUF with Bolted Web were used for connections.

Analysis procedure: The ability to bridge over a structural element that has been removed is an important feature of the Alternate Load Path method which must be considered by the designer. Consequent damages should not surpass the damage limits, though. Three possible procedures are introduced in the updated UFC 4 023-03 ((UFC)-DoD, 2010) for the Alternate Load Path method: Nonlinear Dynamic (ND), Nonlinear Static (NS), or Linear Static (LS).

Linear static: Generally, linear static is the least complicated analysis. In this method, the structure is modeled in a linear static manner and two load cases are used. The deformation-controlled (or ductile) actions are calculated through load case I. Force-controlled (or brittle) actions are calculated through load case II. For ductile actions' analysis, the applied load is increased through the use of an LIF that considers dynamic and nonlinear effects both. The

Table 2: Dimensions of beams and columns for the 5-storey structure

Column section properties		Beam section properties	
1 st -2 nd floor	BOX – 25X25X2	1 st -2 nd floor	IPE30
3 rd -4 th floor	BOX – 20X20X2	3 rd -4 th floor	IPE27
5 th floor	BOX – 20X20X1	5 th floor	IPE24

Table 3: Dimensions of beams and columns for the 15-storey structure

Column section properties		Beam section properties	
1 st -3 rd floor	BOX – 45X45X3	1 st -3 rd floor	IPE45
4 th -6 th floor	BOX – 40X40X3	4 th -6 th floor	IPE40
7 th -9 th floor	BOX – 35X35X3	7 th -9 th floor	IPE36
10 th -12 th floor	BOX – 30X30X3	10 th -12 th floor	IPE33
13 th -15 th floor	BOX – 25X25X1.8	13 th -15 th floor	IPE27

linear static model, which has one removed column, wall section or another load-bearing member, will carry the increased load. The member capacity is compared with the actions (the calculated internal member forces) that contribute to the increased loads. For ductile actions, a Capacity Increase Factor (CIF) is incorporated to enhance the expected member capacity that has the same function as m-factor in ASCE 41 which is used to calculate the expected ductility. The consequent values derived from the m-factor are compared with deformation-controlled actions. A different LIF is incorporated in force-controlled actions that take into account the inertial effects only and the unmodified member capacity and the calculated demand are compared directly.

Nonlinear static: Once the nonlinear model, both materially and geometrically, is made, the loads are increased with a Dynamic Increase Factor (DIF) that takes into account the inertia effects only. The consequent load is then applied to the model when the column is eliminated. The preferred performance level indicates the deformation limits which are compared with the consequent member deformation from deformation-controlled actions. However, in the case of force controlled actions, modification of member strength does not occur. This member strength is compared with the maximum internal member forces (actions).

Nonlinear dynamic: This method involves the direct application of an unmodified-load case to a nonlinear model, both materially and geometrically, of a structure. The first phase of the dynamic analysis includes the use of an applied-load case in order for the structure to be in an equilibrium state. The second section includes the removal of a column or wall section in an instantaneous manner and calculation of the consequent motion of the structure by the Software tool. Similar to that of the nonlinear static case, a comparison is made between the force-controlled actions and the consequent maximum member deformations. Also, in the case of a force-controlled action, a comparison is made between the maximum internal member force and the member strength. Nonlinearity and inertial effect are two features of

Table 4: Comparison of progressive collapse potential of structures

Type	Location of removal column	LS	NS	ND
5 story	Col 1-1 st floor	Yes	Yes	No
	Col 1- 3 rd floor	Yes	Yes	No
	Col 2-1 st floor	Yes	Yes	No
	Col 2-3 rd floor	Yes	Yes	Yes
15 story	Col 1-1 st floor	Yes	Yes	No
	Col 1-8 th floor	Yes	Yes	No
	Col 2-1 st floor	Yes	Yes	No
	Col 2-8 th floor	Yes	Yes	Yes
	Col 3-1 st floor	Yes	Yes	Yes
	Col 3-8 th floor	Yes	Yes	Yes

Table 5: Acceptance criteria for linear static modeling of steel frame connections, m factor

Model	Removed column	Level	Fixed connection m factor
5 story	1, 2	1 st	2
		3 rd	2.1
15 story	1, 2, 3	1 st	1.9
		8 th	2

the dynamic nonlinear analysis; therefore, correction factors are not required.

ANALYSIS OF MODEL STRUCTURES FOR PROGRESSIVE COLLAPSE

The linear static analysis process: In this procedure, a column is removed and the capacity of all members and connections are checked for the purpose of sustaining the redistributed load. Since nonlinear behavior is not considered explicitly, it is incorporated in determination of a capacity factor. Modes of failure categorize members and connections into two groups: deformation controlled actions (ductile failure) and force controlled actions (brittle failure). An m factor (or demand modifier), which is calculated by means of Table 4 UFC code ((UFC)-DoD., 2010), is determined for each

component within the structure. Once the column is removed, Load Increase Factors (LIF) is assigned to the area under the removal influence. The LIF which is used for force controlled actions equals “2”. However, for the case of deformation controlled actions, the LIF for a component in the vicinity of load increase area depends on the minimum m factor. Due to the difference between the LIF for deformation controlled actions and force controlled action, the code requires two progressive collapse analyses. A summary of the m factors calculated for the removed column locations are demonstrated in Table 5. Based on the following m factors, the required LIF for the deformation controlled actions is 2.8.

Define load cases and analysis cases: Equation (1) represents the UFC code ((UFC)-DoD, 2010) recommended load combination for Alternate Path method. A combination of this vertical load plus a lateral load with a magnitude of 0.2% of the vertical load is preferred. The direction of this lateral load will be in all four main directions. In order to perform Progressive Collapse Analysis, the LIF is multiplied by the load cases required for the traditional analysis at proper locations. Each perpendicular plan direction is followed by separate load cases. Automated removal of columns for separate analysis cases is possible through the use of The “Staged Construction” option in SAP2000 (2012). Each separate group is assigned by one column that is going to be removed. Only one column is removed in each separate analysis. Design combinations must be defined for the purpose of using the design procedures of the SAP2000 (2012) software:

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

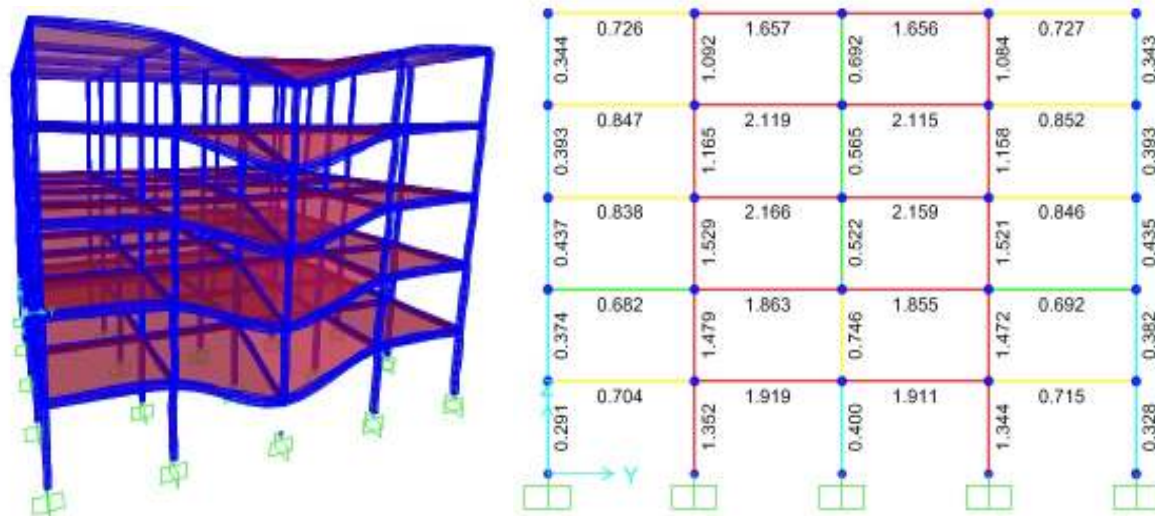


Fig. 2: Deformed shape of removal column 1

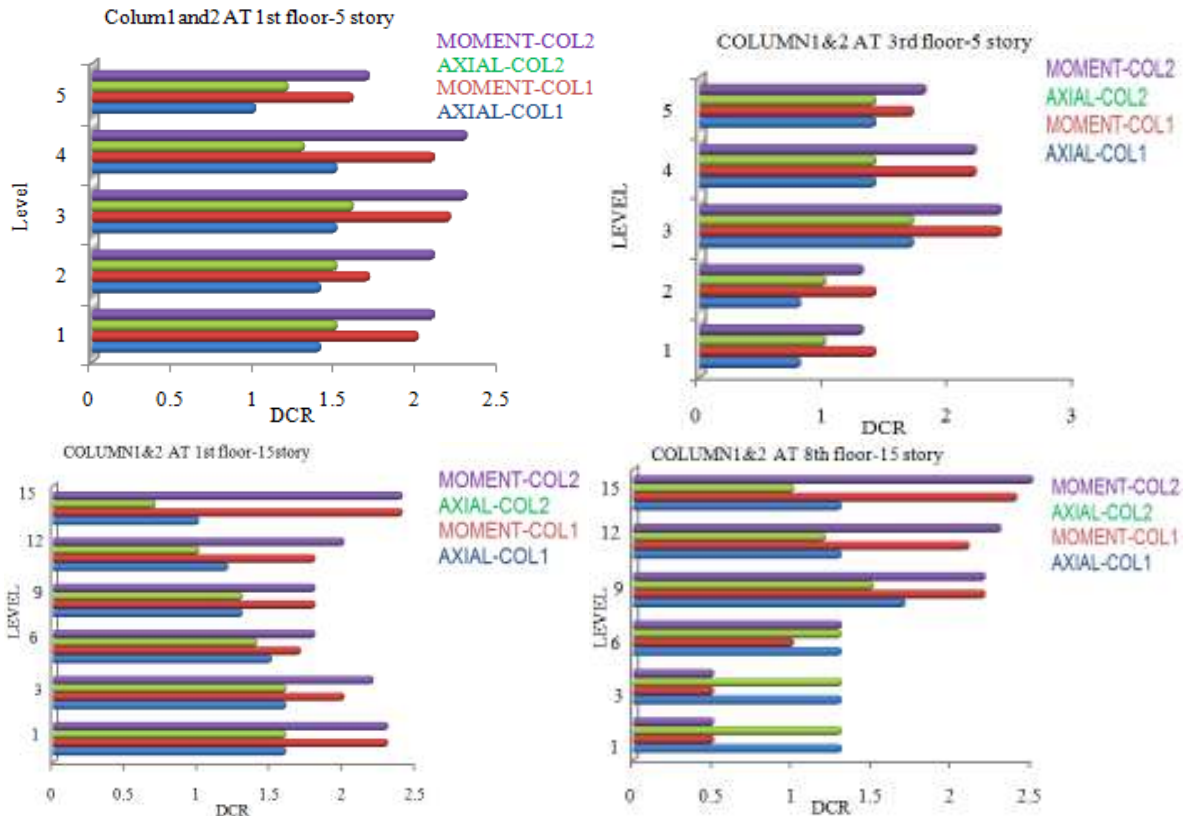


Fig. 3: DCR ratio after removal column 1

Run design and compare to acceptance criteria: The m factor can be compared to $QUD/\Phi QCE$ ratio and $QUF/\Phi QCL$ (<1) by means of design details from SAP2000 (2012). The criterion which can govern the controlled moment acceptance at beam ends is the m factor that is used for the improved WUF moment connections. A comparison is made between the moment ratio (for the defined load combinations) of the deformation controlled model which is derived from SAP2000 (2012) and the connection m factor. Once column 1 is removed, the resulting deformed shape and the DCR ratio will be similar to the ones in Fig. 2 and 3, respectively.

Once the DCR values are obtained for all cases of removal columns, the DCR vs. Stories are plotted as in Fig. 4.

The nonlinear static analysis process: Nonlinear static analysis, also known as ‘pushover analysis’, is incorporated to analyze a building against lateral loads. The process involves increasing the applied loads in a step-by-step manner to achieve either the maximum load (load controlled) or the maximum displacement (displacement controlled). Once the ductility measurement of the structure for lateral loading is preferred, this method could be useful. The ratio of maximum displacement to yield displacement is referred

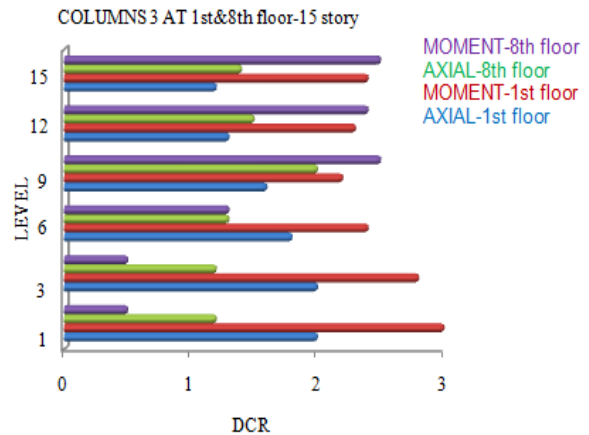
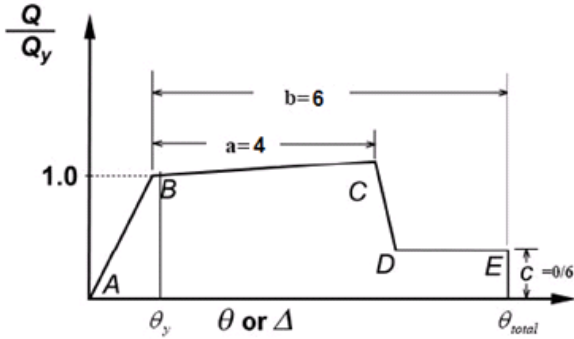


Fig. 4: Plotted DCR vs. stories

to as ductility. Generally, better performance in case of an earthquake could be obtained once the structure is capable of attaining larger ductility. Nonlinear static analysis is useful in separate occasions including the nonlinear analysis of a structure (material and geometric nonlinearity), considering the effects of P-delta or large displacements and execution of buckling analysis. In order to perform progressive collapse analysis, the vertical loads (including gravity) will be incremented in a step-by-step manner until maximum loads are reached

Table 6: Weight increase of the 5 and 15-story structures after the removal of column 2 using LS and ND analyses

Type (story)	Original weight (ton)	LS (ton)	ND (ton)
5	80.5	130.4	115.1
15	493.6	745.3	645.8



	M/My	e/e _y
A	0	0
B	1	1
C	1.12	5
D	0.2	5
E	0.2	7

Fig. 5: Generalized force-deformation hinge definition

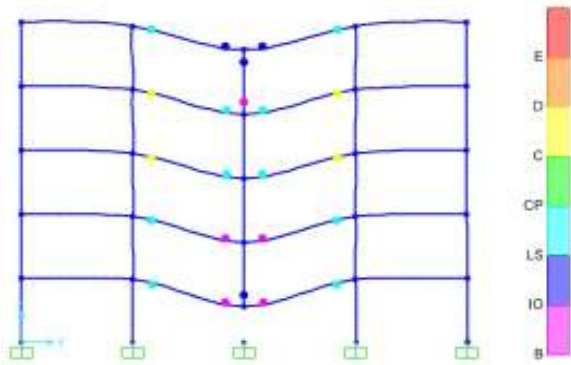


Fig. 6: Hinge formation of removal of column 1 in a 5-story model

or the structure collapses. A good example could be the “vertical pushover analysis”. This analysis would be load-controlled because in progressive collapse potential analysis, structural performance under service loads is investigated.

Hinge locations and properties: The occurrence of hinges anywhere along the beam is possible according to theory. On the contrary, occurrence of hinges is allowed at the ends of each member. For flexural members, hinges could occur at mid spans though. In this study, for beams subjected to flexure or flexure

plus axial tension, the Collapse Prevention criteria, which is defined as plastic rotation/yield rotation = 3, based on FEMA 356 (FEMA, 2000) is used for both primary and secondary elements as nonlinear acceptance criteria. Also, connections’ rotation of beams should be controlled by the limits demonstrated in table E-8 based on UFC code ((UFC)-DoD, 2010). The values in table E-8 are obtained from Eq. (2):

$$0.021 - 0.0003D \text{ rad} \tag{2}$$

Figure 5 shows the formation of the plastic hinges based on Table 4 and 6 FEMA 356 (FEMA, 2000).

Figure 6 indicates the final deformed shape of removal of column 1 in a 5-story model and hinge formation. Once convergence is achieved, the beams capacity is compared with collapse criterion according to FEMA 356 (FEMA, 2000). Also, UFC code ((UFC)-DoD, 2010) recommends to check the rotation of connections against the allowable rotation values. Acceptance criteria vs. hinge results are plotted in the Fig. 6 and 7.

The nonlinear dynamic analysis process-time history analysis: Allowable inelastic deformation limits are defined as component capacities for deformation-controlled actions by taking the effects of dynamic loading into account in nonlinear procedures. In other words, once the bearing elements are removed in a dynamic manner, plastic hinge formation will occur along the members. Although the implementation of NDP is time consuming, the results yielded from this method are more accurate than other analysis methods. Since this method is vulnerable to wrong data including inaccurate modeling and assumptions, mistaken results might be achieved from this method. The following steps need to be taken in order to perform NDP:

- The plastic hinges are assigned to structural members (beams and columns).
- A combination of gravity (without Load Increase Factor) and notional lateral loads are applied to all bays in an incremental manner until equilibrium is obtained in all bays.
- The force component of the column which is going to be removed must be determined.
- The columns member must be eliminated.
- The force component is applied to the removed column location as a joint force.

In order to perform the time history analysis, a triangular load function, shown in Fig. 8, must be defined for the purpose of demonstrating the sudden elimination of column.

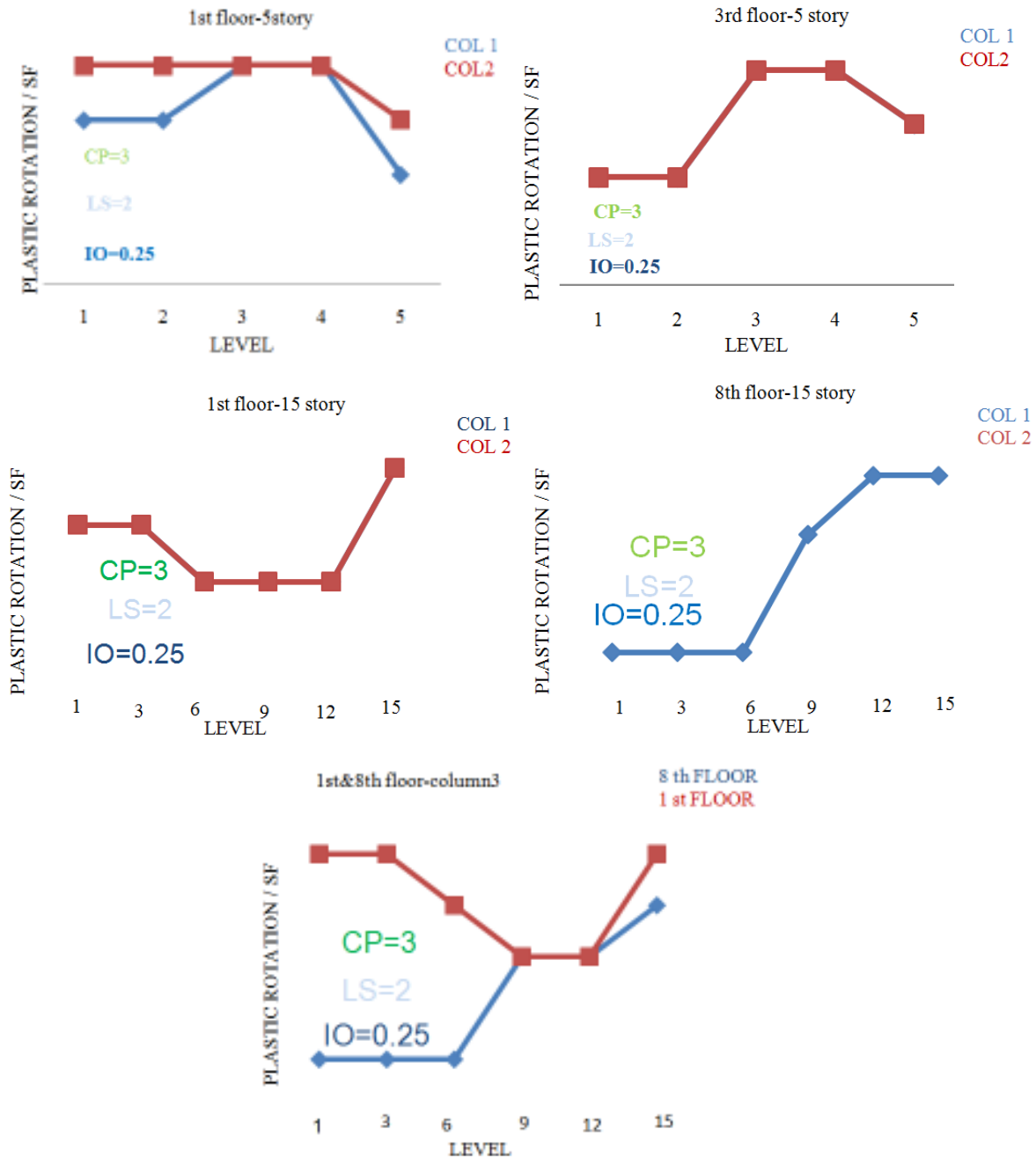


Fig. 7: Acceptance criteria vs. hinge result

Maximum vertical displacement of removal locations (U_j): Once the column is removed, the column removal vertical motion takes place and continues until the state of equilibrium is reached. Figure 9 demonstrates the vertical displacement of a single column removal for a 5-story model performed by time history and nonlinear static analysis.

In Fig.10, a comparison is made between the displacements resulted from the Nonlinear Static (NS) and Nonlinear Dynamic (ND) analyses in 5 and 15-story structures.

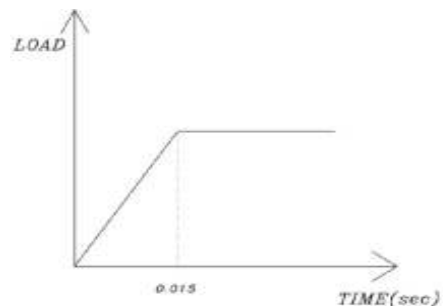
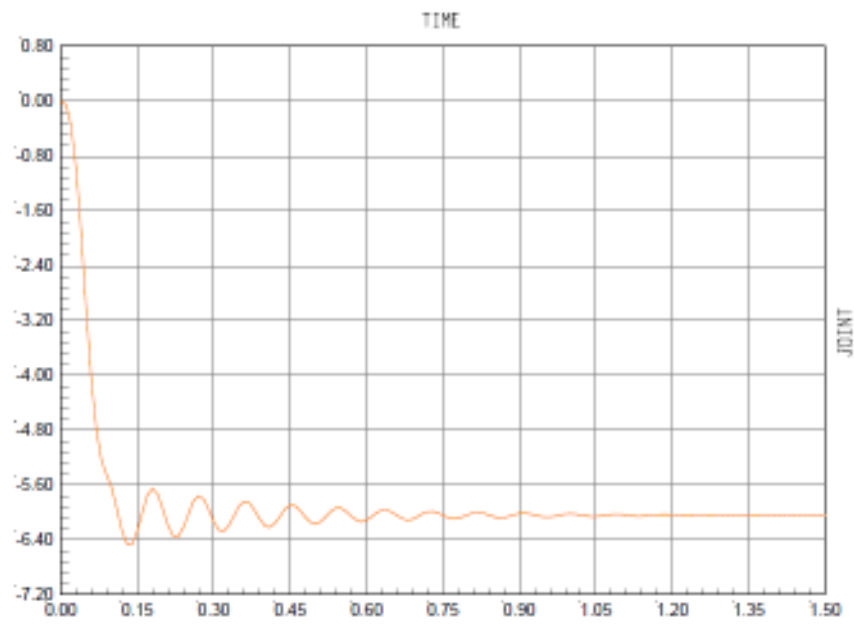
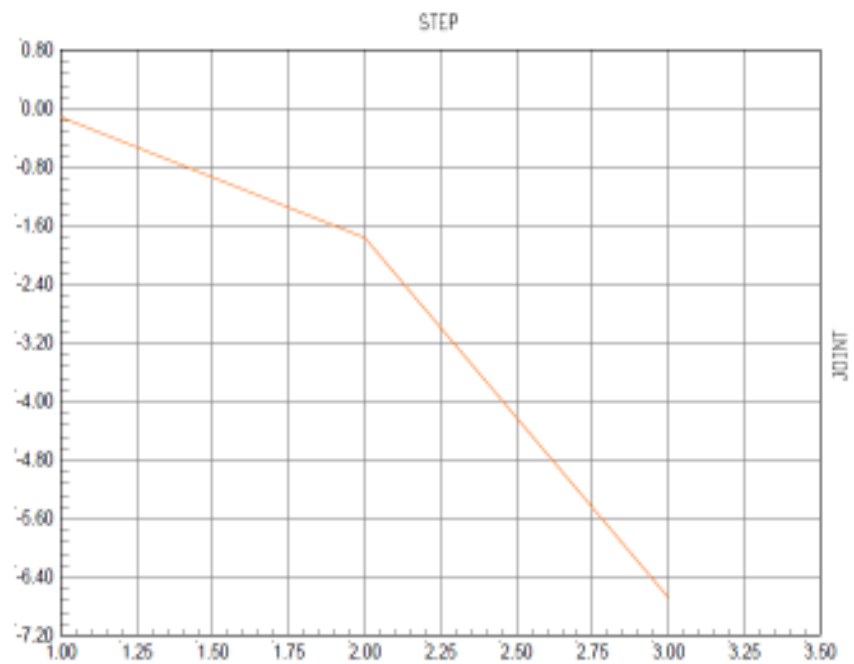


Fig. 8: Dynamic load function



(a)



(b)

Fig. 9: Vertical displacement at the single column removal, (a) time history analysis (b) nonlinear static analysis

Axial forces in columns adjacent to the removed column: In order to study the nonlinear dynamic response of the structure once the column is removed suddenly, an assessment must be made on the redistribution of axial forces on the columns adjacent to the removal column. It is concluded that elimination of the column will create large axial forces in columns

adjacent to the removed one. However, no significant change is observed in other columns. A 32 and 53% increase in axial force of adjacent columns will occur by the time the first plastic hinge is formed. Figure 11 shows the time-dependent axial force variation diagrams of the columns on the first and last level while the column 1 of 5 story building removed.

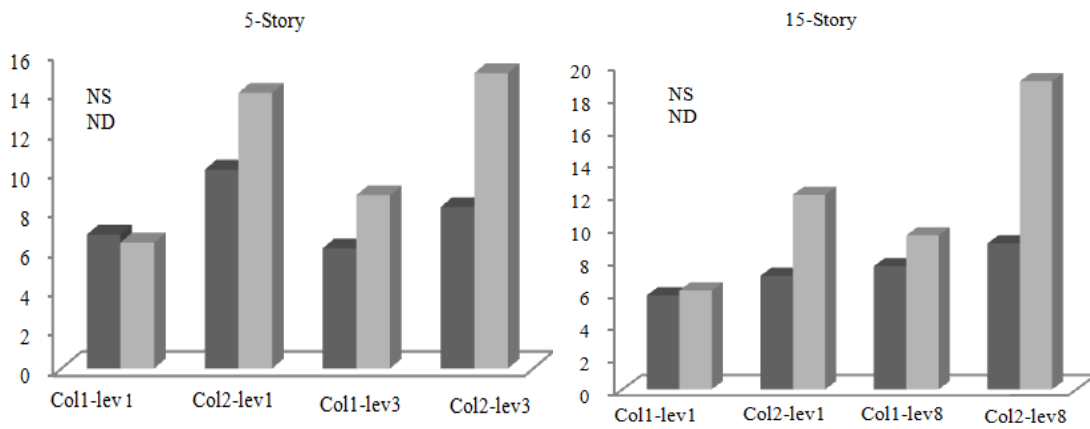


Fig. 10: Displacements of 5 and 15-story structures using NS and ND analyses

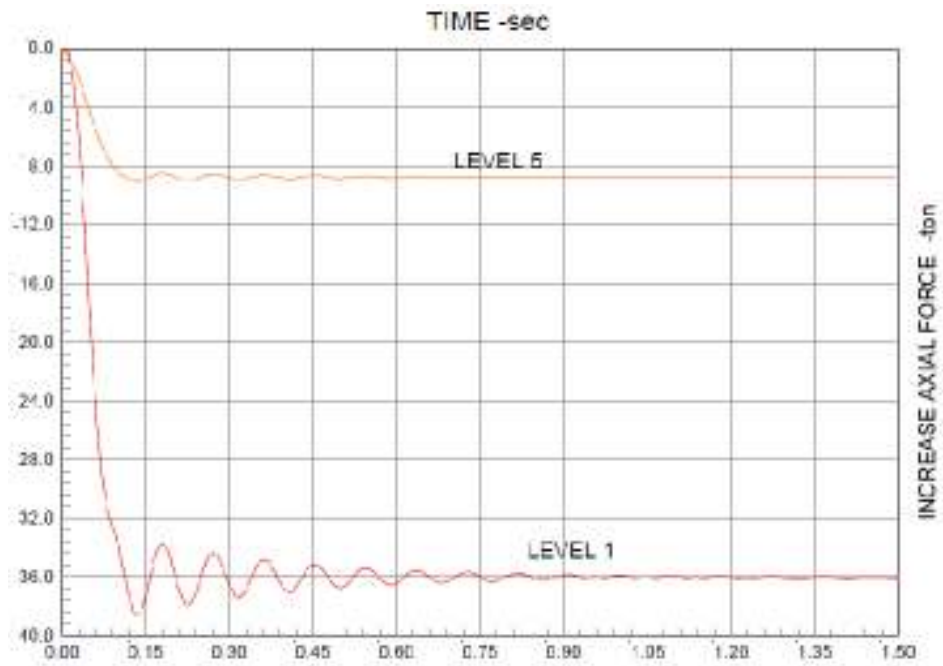


Fig. 11: Time dependent axial force variation diagram

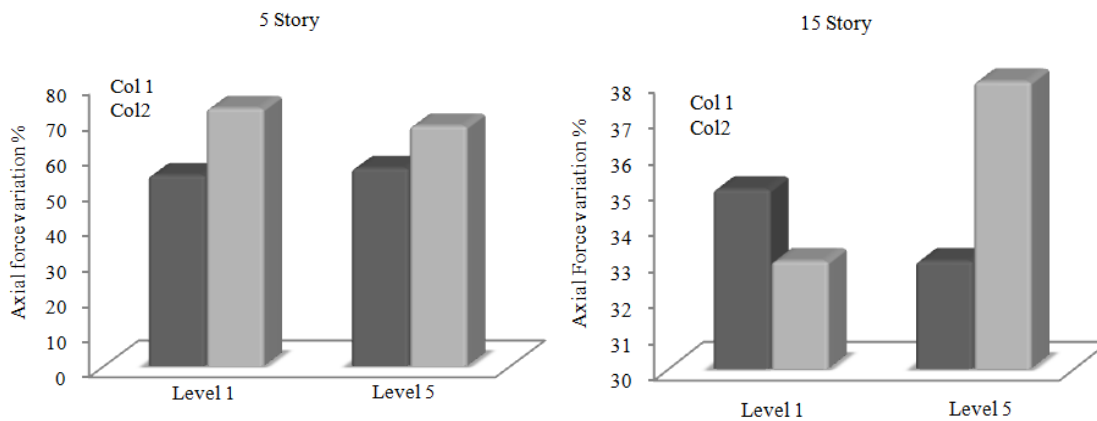


Fig. 12: Percentage of axial force variation in levels 1 and 5 under removal of columns 1 and 2

Figure 12 demonstrates the percentage of axial force variation of 5 and 15-story structures by means of removing columns 1 and 2 in separate analyses in levels 1 and 5 which resulted in 8 different analyses.

DISCUSSION AND CONCLUSION

- The progressive collapse potential of modeled structures incorporating linear static is demonstrated in Table 4, nonlinear static and nonlinear dynamic analyses.
- According to this research, discrepancies occurred in the assessment of progressive collapse potential for many cases according to the acceptance criteria mentioned in the guidelines including DCR for the LS and ductility ratio and Connection Hinge rotation for the ND and NL; it was observed that more conservative results was obtained using LS method compared to that of ND method. Also, it was observed that the progressive collapse potential was higher compared to other occasions when a corner column is removed. This potential will be the highest once two adjacent columns are removed suddenly. The variation of progressive collapse analysis results is a function of variables such as applied load, location of removed column and number of stories in the structure. Removal of column in upper floors (compared to the first floor) will lead to the spread of collapse to other parts of the structure. Also, removal of 2 adjacent columns of the mid height, considering vertical displacements and plastic hinges, will lead to the most critical case resulting in progressive collapse.
- Providing the progressive collapse resistance involves increasing the size of the structural members which will eventually increase its weight. Table 6 demonstrates the required weight for the structure once column 2 in the first story of the 5 and 15-story structures is removed using Linear Static (LS) and Nonlinear Dynamic (ND) analyses.

RECOMMENDATIONS

“Extreme Loading[®] for Structures v3.1 (ELS)” (Structures, 2009) software will help engineers to perform accurate analyses and visualize the progressive collapse of structures subjected to extreme loading conditions such as dynamic loads and impact loads, severe wind loads, earthquake loads and blast loads. Engineers could also distinguish whether a structure is vulnerable to progressive collapse or not by means of making “multiple event scenarios”. As a result, failure of different components could be simulated and determining whether the resulting collapse will be partial or complete will be possible. The only software that is capable of full analysis of progressive collapse scenarios by means of progressive collapse procedures is ELS. This is true because ELS is based on Applied

Element Method (AEM) analysis that can calculate the initiation of cracks and also, propagate and separate the elements. The authors of this study intend to use “Extreme Loading[®] for Structures v3.1 (ELS)” (Structures, 2009) software for the purpose of comparing the results of this research with those resulted from SAP 2000 (SAP2000, 2012) in the future.

REFERENCES

- AISC, 2003. Load and Resistance Factor Design Specification for Structural Steel Buildings. American Institute of Steel Construction, Chicago, Illinois.
- Department of Civil and Environmental Engineering, I.C.L., 2011. Design of building structures to improve their resistance to progressive collapse. Proc. Eng., 14: 1-13.
- FEMA, 2000. Prestandard and Commentary for the Seismic Rehabilitation of Buildings. FEMA 356. In: Agency, F.E.M. (Ed.), American Society of Civil Engineers, Washington, D.C.
- GSA, 2000. Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects. Office of Chief Architect, Washington, D.C.
- Hadi, M. and A. Saeed, 2012. New building scheme to resist progressive collapse. J. Archit. Eng., 18(4): 324-331.
- Jinkoo, K. and K. Taewan, 2009. Assessment of progressive collapse-resisting capacity of steel moment frames. J. Construct. Steel Res., 65(1): 169-179.
- Khalil, A.A., 2012. Enhanced modeling of steel structures for progressive collapse analysis using the applied element method. J. Perform. Construct. Facilit., 26(6): 766-779.
- Liu, M., 2011. Progressive collapse design of seismic steel frames using structural optimization. J. Construct. Steel Res., 67: 322-332.
- Liu, M., 2013. A new dynamic increase factor for nonlinear static alternate path analysis of building frames against progressive collapse. Eng. Struct., 48: 666-673.
- SAP2000, 2012. SAP2000 Three Dimensional Static and Dynamic Finite Element Analysis and Design of Structures. Version 15.1.0. Computer and Structures Inc., Berkeley, CA.
- Structures, E.L.F., 2009. Advantages of using ELS for Progressive Collapse Analysis. In: Applied Science International, LLC. Retrieved from: [http://www.extremeloading.com/Application.aspx/Progressive Collapse Analysis](http://www.extremeloading.com/Application.aspx/Progressive%20Collapse%20Analysis).
- (UFC)-DOD, U.F.C., 2010. Design of Buildings to Resist Progressive Collapse. In: Defense, D.O. (Ed.). Retrieved from: wbdg.org/ccb/DOD/UFC/ufc_4_023_03.pdf.