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Assessment of progressive collapse-resisting capacity of steel moment frames

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ABSTRACT

In this study the progressive collapse-resisting capacity of steel moment resisting frames was investigated using alternate path methods recommended in the GSA and DoD guidelines. The linear static and nonlinear dynamic analysis procedures were carried out for comparison. It was observed that, compared with the linear analysis results, the nonlinear dynamic analysis provided larger structural responses and the results varied more significantly depending on the variables such as applied load, location of column removal, or number of building story. However the linear procedure provided more conservative decision for progressive collapse potential of model structures. As the nonlinear dynamic analysis for progressive collapse analysis does not require modeling of complicated hysteretic behavior, it may be used as more precise and practical tool for evaluation of progressive collapse potential of building structures.

1. Introduction

A progressive collapse involves a series of failures that lead to partial or total collapse of a structure. In the 'Best practice for reducing the potential for progressive collapse in buildings' published by NIST [1] 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 either not considered in structural design or addressed indirectly by passive protective measures. Most of them have characteristics of acting over a relatively short period of time and result in dynamic responses.

In the United States the General Services Administration (GSA) [2] and the Department of Defense (DoD) [3] provide detailed information and guidelines regarding methodologies to resist progressive collapse of building structures. Among many different approaches to designing structures against progressive collapse, the guidelines generally recommend the alternate path method. In this approach, the structure is designed such that if one component fails, alternate paths are available for the load and a general collapse does not occur. This approach has the benefit of simplicity and directness. In its most common application, design for redundancy requires that a building structure be able to tolerate loss of any one column without collapse.

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 [4]. Kaewkulchai and Williamson [5] investigated the analysis procedures using a two-dimensional frame analysis. They found that linear static analysis might result in non-conservative results since it cannot reflect the dynamic effect by sudden exclusion of columns. Marjanishvili [6] studied the advantage and disadvantage of each analysis procedure for progressive collapse analysis. Powell [7] compared the LS, NS, and 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. Ruth et al. [8] found that a factor of 1.5 better represents the dynamic effect especially for steel moment frames. Marjanishvili and Agnew [9] compared the four procedures using an example building, and indicated that as the four procedures had their own merits the static and the dynamic analyses need to be incorporated properly to get the best results for progressive analysis. The results of previous research mentioned above showed that the analysis procedures presented in the guidelines possess both advantage and disadvantage.

The objective of this study is to assess the progressive collapse potential of steel moment frames designed per Korean Building Code [10] and the AISC Load and Resistance Factor Design [11]. The results of the linear step-by-step analysis procedure recommended by the GSA 2003 and the DoD 2005 guidelines were compared with

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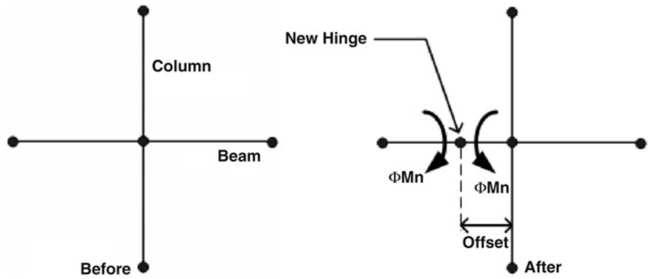


Fig. 1. Modeling of hinges (DoD 2005).

Table 1
Acceptance criteria for progressive collapse (GSA 2003)

Component	Ductility	Rotation (rad)
Steel beams	20	0.21
Steel Columns (tension controls)	20	0.21
Steel Columns (compression controls)	1	–

those of nonlinear dynamic analysis. The effect of the parameters such as the location of column removal and the number of story were also investigated.

2. Analysis procedure

2.1. Acceptance criterion for progressive collapse

The GSA 2003 proposed the use of the Demand–Capacity Ratio (DCR), the ratio of the member force and the member strength, as a criterion to determine the failure of main structural members by the linear analysis procedure:

$$\mathrm{DCR} = \frac{Q_{UD}}{Q_{CE}}, \tag{1}$$

where Q_{UD} is the acting force (demand) determined in component (moment, axial force, and shear etc.); and Q_{CE} is the expected ultimate, unfactored capacity of the component (moment, axial force, shear etc.).

In the GSA 2003 the inherent strength is obtained by multiplying the nominal strength with the overstrength factor of 1.1, and the strength reduction factor is not applied. The acceptance criteria for DCR vary from 1.25 to 3.0 depending on the width/thickness ratio of the member. In the DoD 2005 the DCR is not evaluated; instead the design strength (the nominal strength times the strength reduction factor) multiplied by the overstrength factor of 1.1 is compared with the member force to determine the failure of members.

For the nonlinear analysis procedures, the guidelines specify maximum plastic hinge rotation and ductility as acceptance criteria for progressive collapse. Table 1 shows the acceptance criteria for progressive collapse recommended in the GSA 2003, where the maximum ductility and rotation angle of steel beams and columns are presented. The ductility ratio is the ratio of the ultimate deflection at a reference point (e.g., location where a column is removed) to the yield deflection at that point determined from the nonlinear analysis procedures, and the rotation angle is obtained by dividing the maximum deflection with the length of the beam (Figure 2.2 and 2.3 in the GSA 2003).

2.2. Step-by-step procedure for linear static analysis

The step-by-step procedure to conduct the LS analysis method recommended in the GSA 2003 is as follows:

Step 1.

Remove a column from the location being considered and carry out linear static analysis with the following gravity load imposed on the bay in which the column is removed:

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

where DL and LL represent dead load and live load, respectively.

Step 2.

Check DCR in each structural member. If the DCR of a member exceeds the acceptance criteria in shear, the member is considered as failed. If the DCR of a member end exceeds the acceptance criteria in bending, a hinge is placed at the member end as shown in Fig. 1. A rigid offset (a half of the beam depth in this study) can be applied to model a hinge in proper location. If hinge formation leads to failure mechanism of a member, it is removed from the model with its load redistributed to adjacent members.

Step 3.

At each inserted hinge, equal-but-opposite bending moments are applied corresponding to the expected flexural strength of the member (nominal strength multiplied by the overstrength factor of 1.1) as shown in Fig. 1.

Step 4.

The Steps 1–4 are repeated until DCR of any member does not exceed the limit state. If moments have been redistributed throughout the entire building and DCR values are still exceeded in areas outside of the allowable collapse region defined in the guidelines, the structure will be considered to have a high potential for progressive collapse. DoD 2005 recommends similar approach for the alternate path method except the increase in applied load Eq. (3), acceptance criteria, and allowable collapse region:

$$2(1.2DL + 0.5LL) + 0.2WL, \tag{3}$$

where WL represents wind load.

2.3. Applied loads for static and dynamic analyses

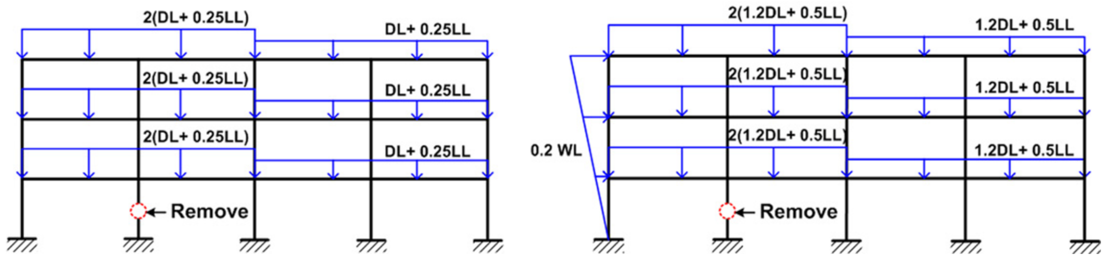
For static analysis both the GSA 2003 and the DoD 2005 use dynamic amplification factor of 2.0 in load combination as shown in Fig. 2(a) and (b). The DoD guideline recommends to use larger gravity load than the GSA guideline. The wind load is included in the DoD load combination.

For dynamic analysis both guidelines do not recommend to use the dynamic amplification factor. To carry out dynamic analysis, the axial force acting on a column is computed before it is removed. Then the column is replaced by point loads equivalent of its member forces as shown in Fig. 2(c) and (d). To simulate the phenomenon that the column is abruptly removed, the member forces are removed after a certain time is elapsed as shown in Fig. 3, where the variables P, V, and M denote the axial force, shear force, and bending moment, and W is the vertical load. In this study the forces were increased linearly for five seconds until they reached their full amounts, kept unchanged for two seconds until the system reached stable condition, and the upward force was suddenly removed at seven seconds to simulate the dynamic effect caused by sudden removal of the column.

3. Configuration and analytical modeling of model structures

3.1. Model structures

Two types of analysis model structures were prepared to assess potential for progressive collapse: the gravity load resisting system (GLRS) in which gravity load is resisted by steel moment resisting frames while lateral load is resisted by shear walls as shown in Fig. 4(a); and the lateral load resisting system (LLRS) in which



(a) Static procedure (GSA 2003).
(b) Static procedure (DoD 2005).
(c) Dynamic procedure (GSA 2003).
(d) Dynamic procedure (DoD 2005).

Fig. 2. Applied load for analysis of progressive collapse.

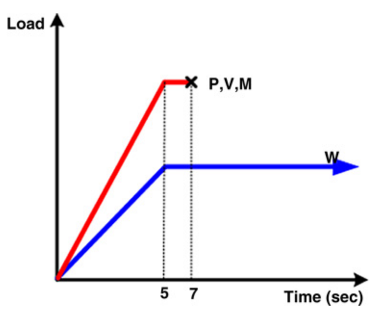


Fig. 3. Application of loads for dynamic analysis procedure.

the steel moment frames are designed to resist both gravity and lateral loads as shown in Fig. 4(b). 3-, 6-, and 15-story structures were designed in accordance with the KBC 2005 and the AISC LRFD using the program code Midas GenW [12]. The structures were assumed to be located at Seoul, Korea, which is considered to be a low seismic zone. The seismic load used is equivalent of S_{DS} and S_{D1} equal to 0.33 g and 0.18 g, respectively in the IBC 2006 [13] format. The structures were designed as ordinary steel moment frames and the R-factor of 3.5 was used, which were adopted from the IBC 2006. All the frames in the longitudinal direction were designed to share the seismic load in the LLRS. The structural design of the 15-story structure was governed by the wind load with basic wind speed of 30 m/s. The columns and girders were designed with SM490 (F_y = 310 MPa) and SS400 (F_y = 240 MPa) steel, respectively. In accordance with the GSA 2003 guideline the limit values for DCR were 3.0 for girders and 2.0 for columns based on the width/thickness ratio.

3.2. Analytical modeling

For numerical analysis the exterior frames of the model structures enclosed in the dotted rectangle in Fig. 4, were analyzed using the program code OpenSees [14]. In the material model, the rate of loading was not considered in this study since the behavior after sudden column removal is not fast enough to include the rate effect. For nonlinear analysis, a simple bilinear material model was used with the post-yield stiffness of the structural members assumed to be 2% of the initial stiffness. As the dynamic behavior caused by sudden column removal is not involved with load reversal as in structures subjected to earthquake load, to use complicated hysteretic model is not necessary. Damping ratio was assumed to be 5% of the critical damping, which is usually adopted for analysis of structures undergoing large deformation. The progressive collapse analyses were carried out by removing a column in various locations in accordance with the GSA 2003 and DoD 2005 guidelines.

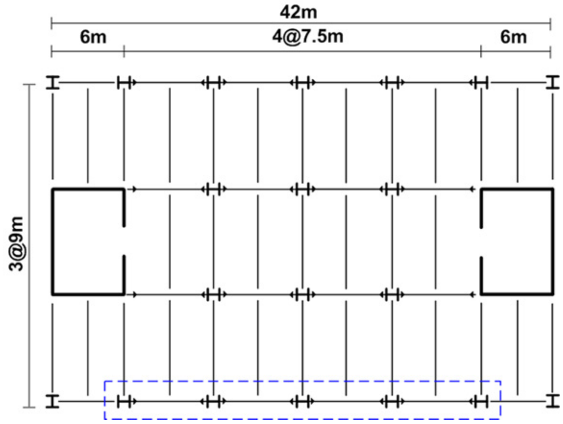
4. Analysis of model structures for progressive collapse

4.1. Linear static analysis

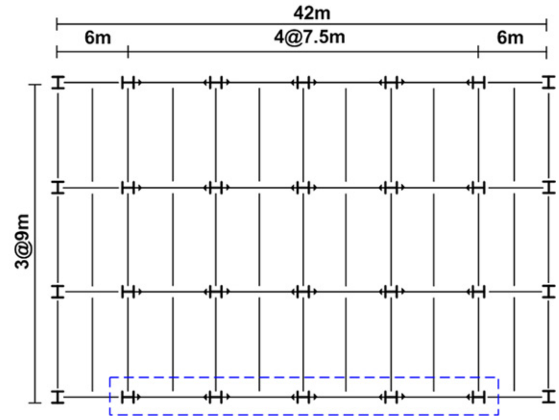
A series of step-by-step procedure recommended in the GSA guideline was applied to model structures. The corner column was removed first, and the hinge locations of the GLRS model structures were plotted in Fig. 5 after the first analysis step was over. It was observed that DCR for axial and shear forces did not exceed the limit states. The numbers above the filled circles represent the computed DCR values. It can be noticed that most of the hinges formed at the right-ends of girders in the bay that the column was removed. At the left-ends of girders hinges formed only at the lower stories. Next, the analysis steps 2 to 4 specified in the guidelines were followed; i.e. hinges were placed in the member ends where DCR exceeded 3.0 in girders and 2.0 in columns and the structures were reanalyzed. At these analysis steps it was observed that DCR in all girder ends in the left-hand-side bay exceeded 3.0, which implies that there is strong possibility of progressive collapse in the GLRS model structures when the corner column is removed.

Fig. 6 shows the hinge locations in the GLRS structures at the first three iterative analysis stages when the second column from the left was removed. The number of hinges formed at the first iteration was smaller than that formed when the corner column was removed. However after three iterations DCR in most of the girders located in the bay in which a column was lost exceeded the limit value, and the model structures were considered to be progressively collapsed. When the center column was removed, the hinge formation was almost the same as when the second column was removed.

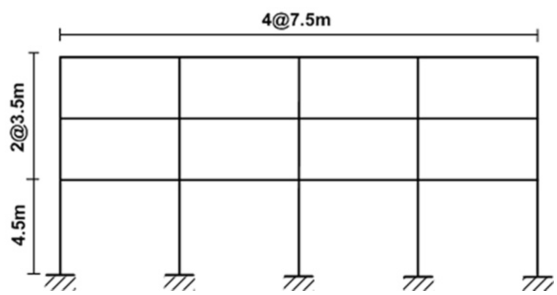
In the DoD 2005 wind load is included in the load combination and the load factor for gravity load is larger than that of the GSA 2003. Furthermore the member strength is evaluated conservatively by multiplying strength reduction factor. In the



(a) Structural plan of the GLRS structure.

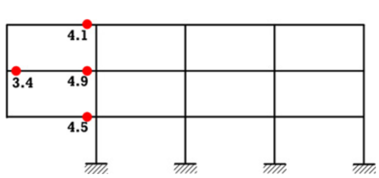


(b) Structural plan of the LLRS structure.

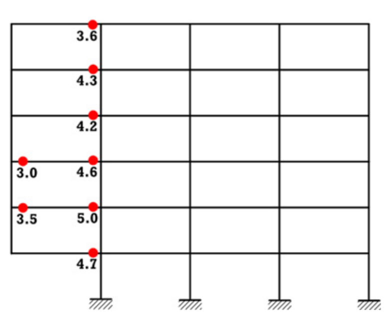


(c) Elevation of the three-story model structure.

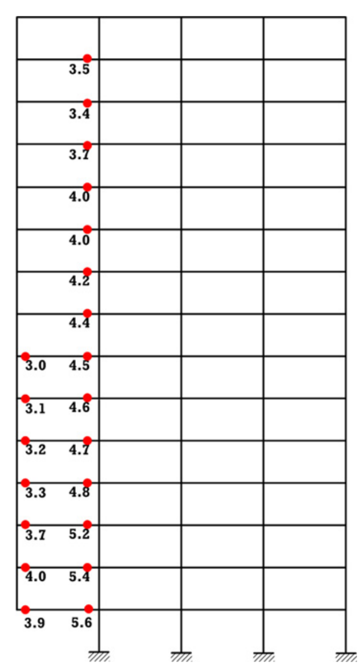
Fig. 4. Analysis model structures.



(a) 3-story structure.



(b) 6-story structure.



(c) 15-story structure.

Fig. 5. Locations of hinges when the first-story corner column of the GLRS structures was removed (GSA 2003).

DoD guideline the member strength is directly compared with the member force, which means that the limit state for DCR is equal to 1.0. Therefore the DoD 2005 recommends more rigorous criteria in the determination of progressive failure. Fig. 7 plots the locations of hinges formed by removal of a corner column, where it can be observed that in the first step of analysis the member forces exceeded the member strength in most girders and in many columns even in the bays where no column was removed. When the DoD guideline was applied to the GLRS structures, hinges formed even in columns and progressive collapse occurred in the first analysis step. It was observed that, compared with the case of the removal of the second column (not shown here), more

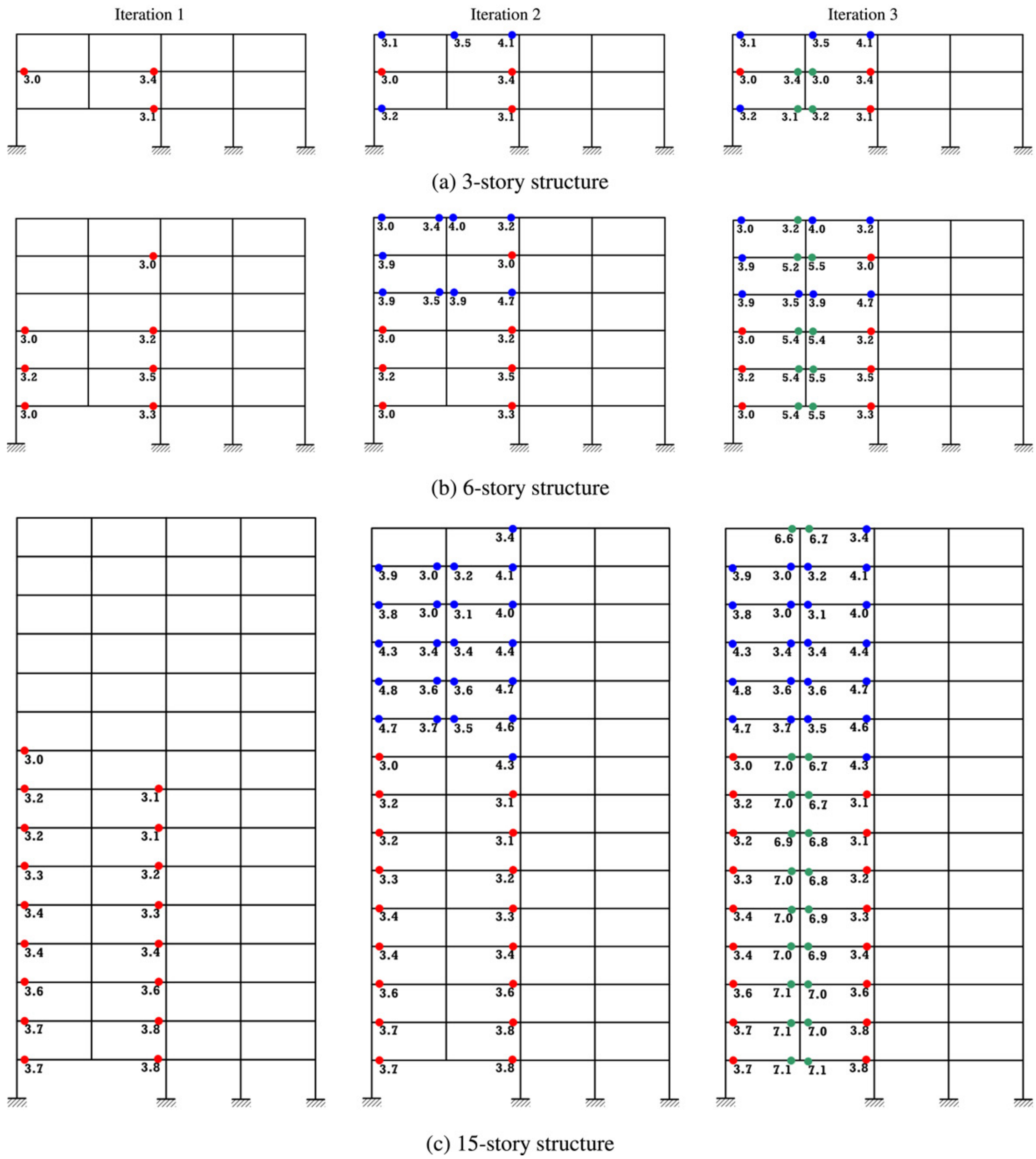


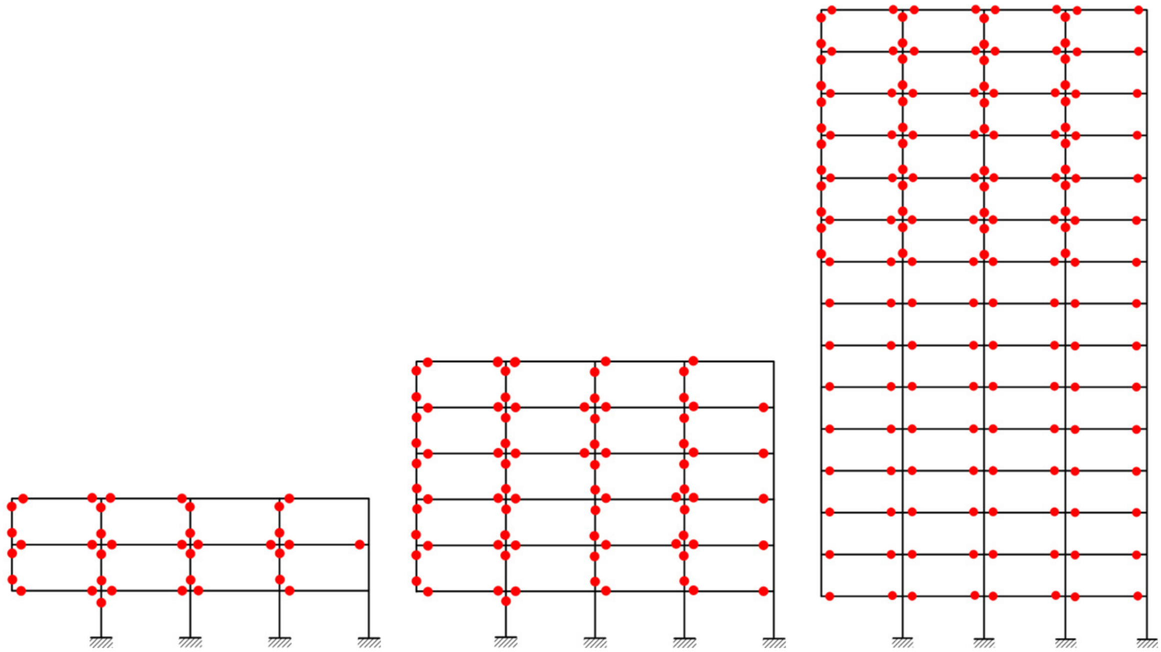
Fig. 6. Locations of hinges when the first-story 2nd column of the GLRS structures was removed (GSA 2003).

hinges formed when the corner column was removed, i.e. when the structure deformed unsymmetrically.

The LLRS structures were designed with larger member size compared with the GLRS structures; therefore it was expected that the progressive collapse-resisting capacity is higher than that of the GLRS structures. Fig. 8 shows the locations of hinges in the LLRS structures estimated in accordance with the GSA guideline when the corner column was removed. It was observed that in the first step of analysis the number of hinges was less than that of the GLRS structures; in the second iteration the first bay of the 3- and the 6-story structures turned out to be collapsed progressively. The first bay of the 15-story structure failed in the third iteration. However when the second or center column was removed no hinges formed in any model structures, which implies that in this case progressive collapse is not expected based on the GSA guideline. The hinge locations determined based on the DoD 2005 in the LLRS structures were almost identical to those in the GLRS structures regardless of the location of the column removal. Therefore, the LLRS model structures turned out to be vulnerable to progressive collapse when the DoD guideline was applied.

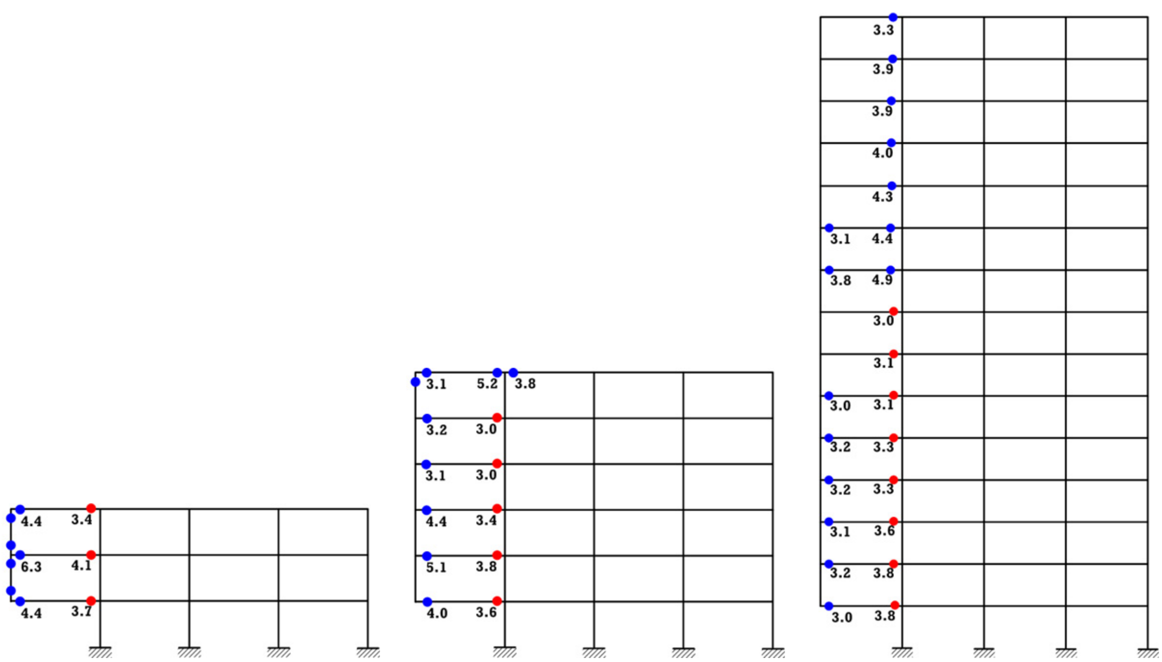
4.2. Linear dynamic analysis

In the LD analysis the dynamic amplification factor of 2.0 used in the static analysis is not applied. Fig. 9 compares the formation of hinges in the 3-story GLRS structure resulted from the first step LS and LD analyses. It can be observed that less hinges formed as a result of the dynamic analysis, and the DCR values obtained from dynamic analysis were also less than those computed by static analysis. Fig. 10 presents the time history of the vertical displacement at the girder-removed column joint. The displacements obtained by the LS analysis with and without using the amplification factor are also shown in the figure. It can be seen that the maximum displacements obtained from dynamic analysis are smaller than those by static analysis using the dynamic amplification factor. It also can be noticed that



(a) 3-story structure. (b) 6-story structure. (c) 15-story structure.

Fig. 7. Locations of hinges when the first-story corner column of the GLRS structures was removed (DoD 2005).



(a) 3-story structure. (b) 6-story structure. (c) 15-story structure.

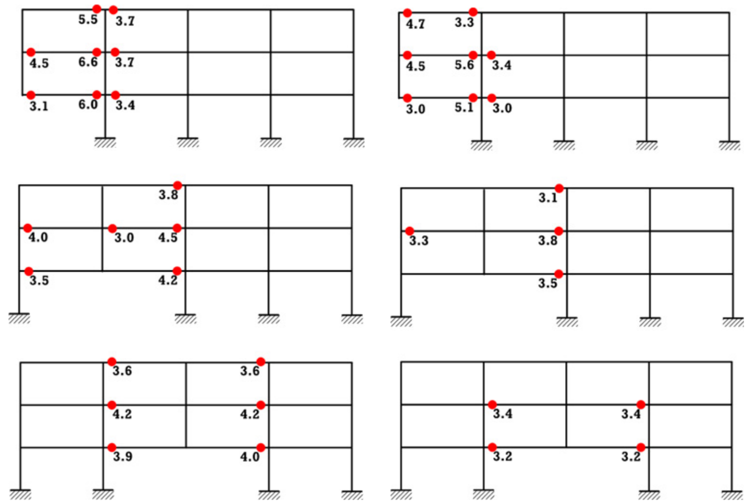
Fig. 8. Locations of hinges when the first-story corner column of the LLRS structures was removed – after second iteration (GSA 2003).

as the number of story increases the maximum displacement decreases since more structural members participate in resisting progressive collapse. The displacement computed by dynamic analysis gradually approached the static analysis result obtained without considering the amplification factor.

4.3. Nonlinear dynamic analysis

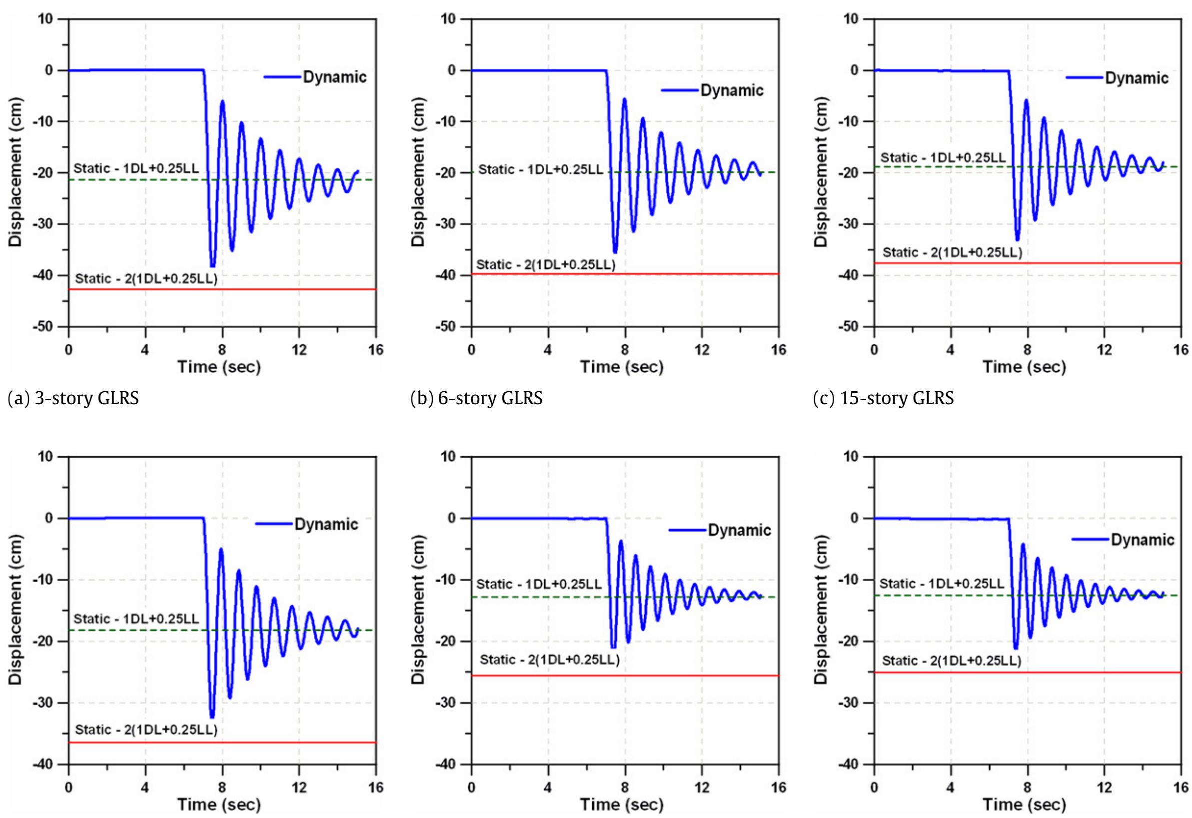
The nonlinear analysis procedures are generally more sophisticated than linear procedures in characterizing the performance of a structure. When such procedures are used, the guidelines generally permit less restrictive acceptance criteria recognizing the improved results that can be obtained from such procedures. The guidelines, however, indicate that potential numerical convergence problems may be encountered during the execution of the nonlinear analysis, along with sensitivities to assumptions for boundary conditions, geometry and material models, etc.

In this study nonlinear dynamic time-history analyses were carried out by removing each column in the first story. Figs. 11 and 12 show the vertical displacements of the model structures obtained from LD and ND time-history analyses following the GSA and DoD guidelines, respectively, when the second columns in the first story were removed. It can be observed that the results from



(a) Linear static analysis. (b) Linear dynamic analysis.

Fig. 9. Comparison of hinge locations and DCR values determined from linear static and dynamic analyses (GLRS, GSA 2003).



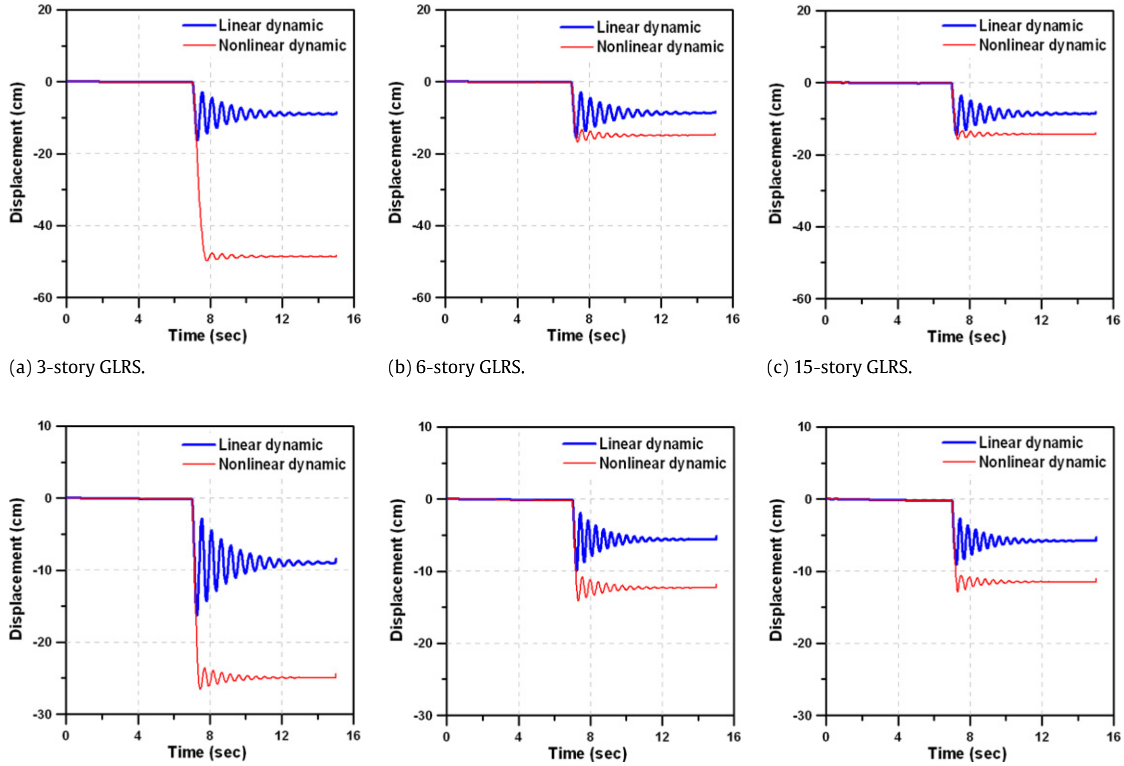
(a) 3-story GLRS (b) 6-story GLRS (c) 15-story GLRS

(d) 3-story LLRS (e) 6-story LLRS (f) 15-story LLRS

Fig. 10. Displacement time history at the joints when the corner column is removed (GLRS, GSA 2003).

linear analyses significantly underestimate those from nonlinear analyses. This is compatible with the findings of Pretlove et al. [15]. It also can be observed that, in comparison with the linear analysis results, the results of nonlinear dynamic analysis vary significantly depending on the size of applied loads and the number of stories.

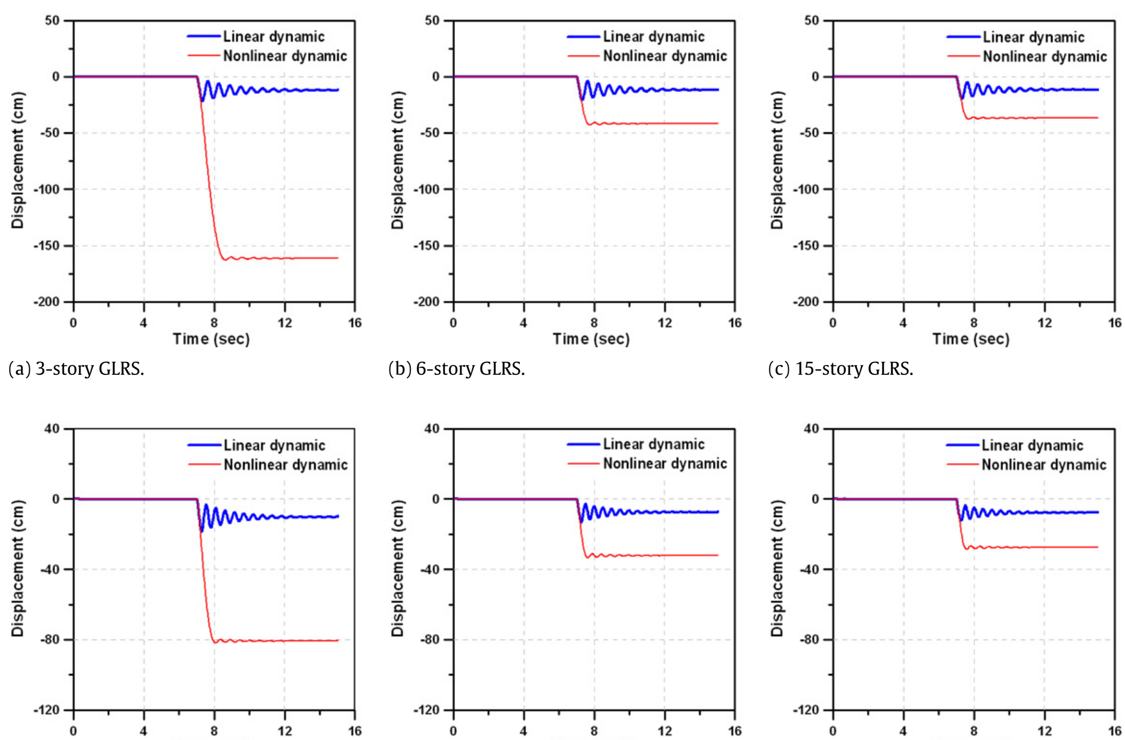
Fig. 13 shows the locations of plastic hinges and their rotation angles in radians in the 3-story GRLS structure subjected to the gravity load specified in the GSA 2003. When the corner column was removed plastic hinges formed in columns as well as in beams and the acceptance criterion 0.21 radian was exceeded in many structural members. When the second and the center columns were removed the plastic rotations were relatively small compared with those obtained when the corner column was removed. Fig. 14 shows the plastic hinges in the 3-story LLRS structure, where it can be observed that the amounts of plastic rotations were significantly reduced compared with those in the GLRS structure. No plastic rotation exceeded the given acceptance criterion. However when the same structure was subjected to the gravity load specified in the DoD guideline (Fig. 15), the criterion was exceeded in many locations when the corner column was removed. In this case the collapse mechanism is evident. Fig. 16 depicts the plastic hinge locations in the 15-story LLRS subjected to the load recommended



(a) 3-story GLRS. (b) 6-story GLRS. (c) 15-story GLRS.

(d) 3-story LLRS. (e) 6-story LLRS. (f) 15-story LLRS.

Fig. 11. Comparison of the linear and the nonlinear dynamic analyses results when the second column was removed (GSA 2003).

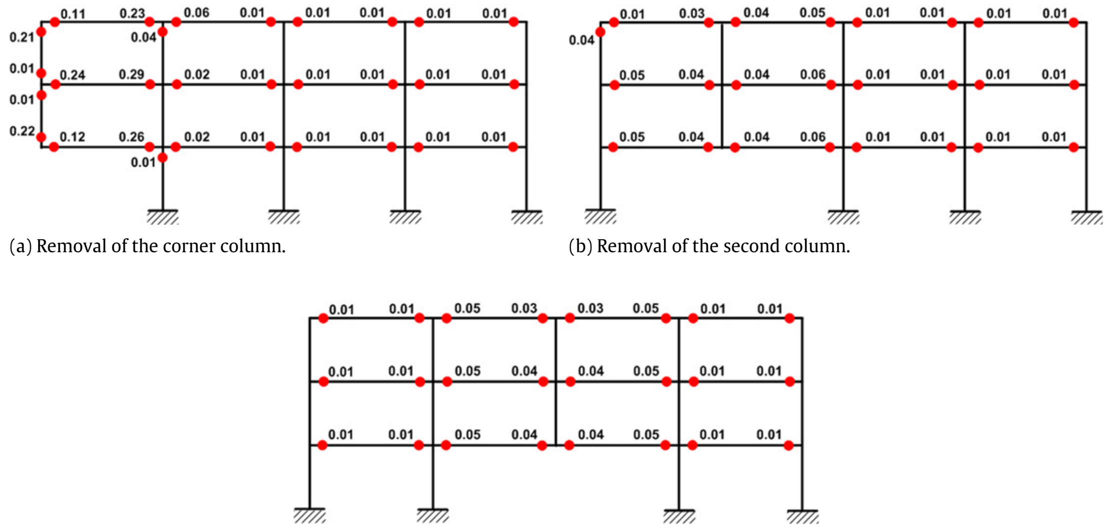


(a) 3-story GLRS. (b) 6-story GLRS. (c) 15-story GLRS.

(d) 3-story LLRS. (e) 6-story LLRS. (f) 15-story LLRS.

Fig. 12. Comparison of the linear and the nonlinear dynamic analyses results when the second column was removed (DoD 2005).

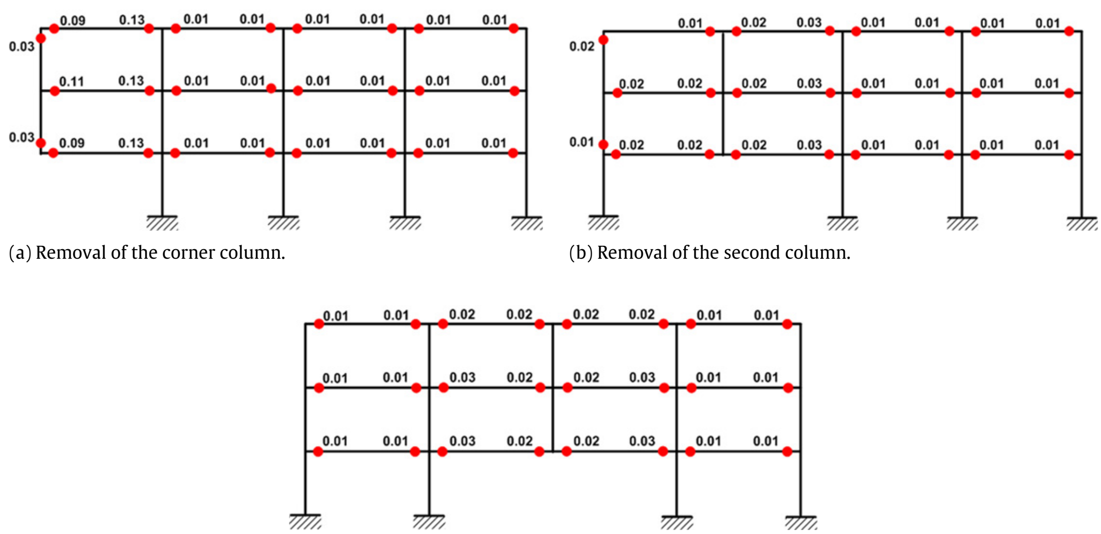
in the DoD 2005 guideline when the second column was suddenly removed. It can be observed that, even though plastic hinges formed in all beams, the plastic rotations did not exceed 3% radian, which is far less than the acceptance criterion of 21%.



(a) Removal of the corner column. (b) Removal of the second column.

(c) Removal of the center column.

Fig. 13. Rotation of members in radian in the 3-story GLRS structure (GSA 2003).



(a) Removal of the corner column. (b) Removal of the second column.

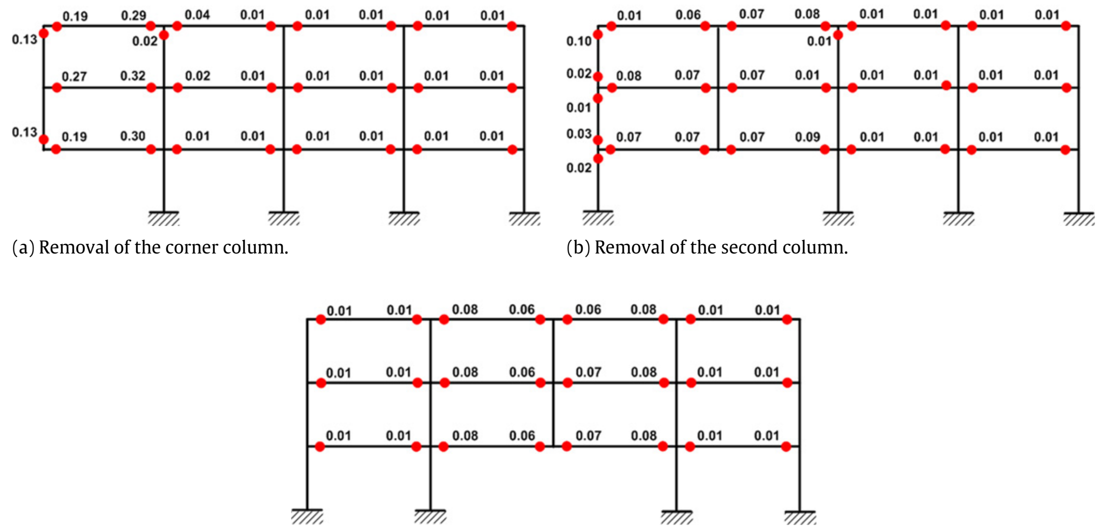
(c) Removal of the center column.

Fig. 14. Rotation of members in radian in the 3-story LLRS structure (GSA 2003).

Table 2
Ductility of 3-story structures in locations where a column is removed

(a) GSA 2003						
Removed column	GLRS			LLRS		
	Yield displacement (mm)	Max. displacement (mm)	Ductility	Yield displacement (mm)	Max. displacement (mm)	Ductility
Corner	90	2905	32.3	86	1307	15.2
Second	60	498	8.3	56	266	4.7
Center	60	438	7.3	56	245	4.4
(b) DoD 2005						
Location	GLRS			LLRS		
	Yield displacement (mm)	Max. displacement (mm)	Ductility	Yield displacement (mm)	Max. displacement (mm)	Ductility
Corner	90	–	–	86	3244	37.7
Second	60	1628	27.1	56	818	14.6
Center	60	1435	23.9	56	752	13.4

Table 2 shows the ductility ratios of the girder connected to the removed column in the 3-story structures, and Table 3 presents the ductility ratios in model structures with different number of stories when the second column was removed. The yield displacements were obtained by nonlinear static push-down analyses and the maximum displacements were computed from nonlinear dynamic analyses. The ductility ratio is the ratio of the maximum displacement and the yield displacement. The ductility



(a) Removal of the corner column.

(b) Removal of the second column.

(c) Removal of the third column.

Fig. 15. Rotation of members in radian in the 3- story LLRS structure (DoD2005).

Table 3
Ductility of model structures when the second column is removed

Story	GLRS			LLRS		
	Yield displacement (mm)	Max. displacement (mm)	Ductility	Yield displacement (mm)	Max. displacement (mm)	Ductility
(a) GSA 2003						
3		498	8.3	56	266	4.7
6	60	168	2.8	54	141	2.6
15		158	2.6	51	129	2.5
(b) DoD 2005						
3		1628	27.1	56	819	14.6
6	60	429	7.2	54	334	6.2
15		378	6.3	51	285	5.6

ratio turned out to be large when the corner column was removed and when the load specified in the DoD guideline was imposed on the structures. It also can be observed that the ductility ratio decreased as the number of story increased. In the 3-story GLRS structure under the GSA-specified vertical load, the maximum ductility ratio exceeded the acceptance criterion of 20 as shown in Table 1 when the corner column was suddenly removed. When the vertical load specified in the DoD guideline was imposed, the acceptance criterion was exceeded regardless of the location of the removed column. It can be observed in Table 2(b) that in the 3-story LLRS structure subjected to the DoD load the ductility ratio far exceeded the criterion only when the corner column was removed. When the second column was removed (Table 3) the ductility ratio exceeded the limit state only in the 3-story GLRS subjected to the load specified in the DoD guideline. These results generally coincide well with the plastic hinge rotations shown in Figs. 13–15. Based on the nonlinear dynamic analysis results it was observed that the LLRS structures are not vulnerable to progressive collapse caused by sudden removal of a column.

Tables 4 and 5 compare the progressive collapse potential of model structures determined by both linear and nonlinear processes. Based on the acceptance criteria given in the guidelines, i.e. DCR for the LS and ductility ratio for the ND, there were discrepancies in the evaluation of progressive collapse potential in many cases; the decisions based on LS method were too conservative compared with those made by ND method. This, however, is not consistent with the comparison of the maximum deflections shown in Fig. 11, where the maximum deflections obtained by nonlinear dynamic analysis were larger than those from linear analysis.

Table 4
Comparison of progressive collapse potential of model structures when the second column is removed

Type	Story	GSA 2003		DoD 2005	
		LS	ND	LS	ND
	3	Yes	No	Yes	Yes
GLRS	6	Yes	No	Yes	No
	15	Yes	No	Yes	No
	3	No	No	Yes	No
LLRS	6	No	No	Yes	No
	15	No	No	Yes	No

Table 5
Comparison of progressive collapse potential of 3-story structures with different locations for column removal

Type	Location of removed column	GSA 2003		DoD 2005	
		LS	ND	LS	ND
	Corner	Yes	Yes	Yes	Yes
GLRS	Second	Yes	No	Yes	Yes
	Center	Yes	No	Yes	Yes
	Corner	Yes	No	Yes	Yes
LLRS	Second	No	No	Yes	No
	Center	No	No	Yes	No

5. Conclusions

In this study the progressive collapse potential for steel moment resisting frames was investigated using the linear static, linear dynamic, and nonlinear dynamic analysis procedures recommended in the GSA 2003 and the DoD 2005 guidelines. It was

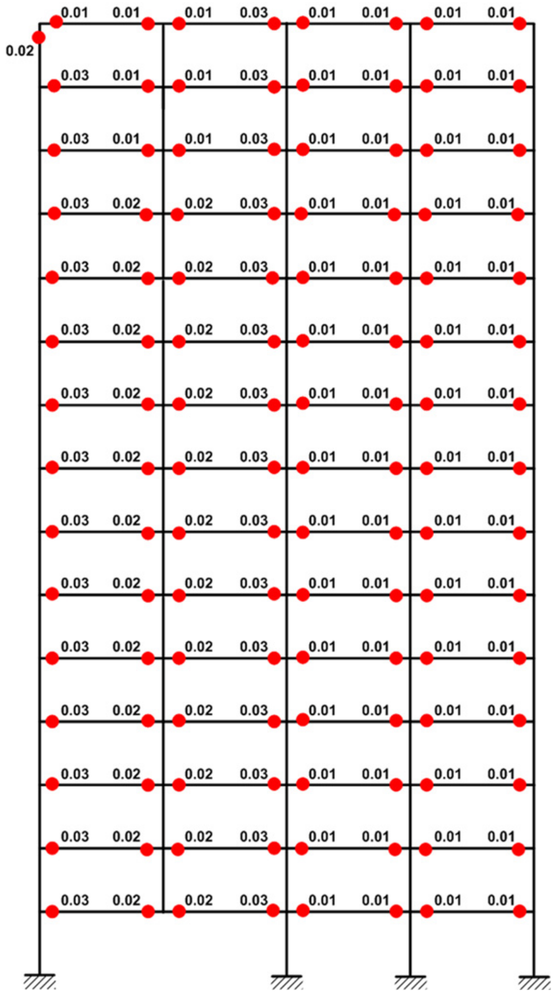


Fig. 16. Rotation of members in radian in the 15-story LLRS structure subjected to sudden removal of the second column (DoD2005).

observed that, as expected, the steel moment frames designed for lateral load as well as gravity load turned out to be less vulnerable for progressive collapse. It was also observed that the potential for progressive collapse was highest when a corner column was suddenly removed, and that the progressive collapse potential decreased as the number of story increased.

Even though the linear static step-by-step analysis procedure has advantage in that it is theoretically simple and can be conducted without sophisticated nonlinear modeling, a lot of manual works were required to evaluate DCR in each analysis step and to remodel/reanalyze the structure until DCR of any member does not exceed a given limit state. It was observed that, even though the maximum vertical deflections estimated by linear analysis were smaller than those obtained by nonlinear dynamic analysis, the linear procedure made more conservative decision for progressive collapse potential. Also the dynamic analysis results varied more significantly depending on the variables such as applied load, location of column removal, or number of building story.

The nonlinear dynamic time-history analysis of structures involves nonlinear modeling of members and connections and has been considered as complex and costly. However recent advancements in computer hardware and commercial analysis software packages have made it possible for practical engineers to employ sophisticated structural assessment techniques without much difficulty. Furthermore, the mathematical modeling of structural members for progressive collapse analysis does not require the complex hysteretic behavior with load reversal as in the structures subjected to seismic load. In this regard the nonlinear dynamic analysis may be used as more precise and practical tool for evaluation of progressive collapse potential.

In this study the panel zones in girder-column joints were assumed to be rigid and the catenary action of girders was not considered. When panel zone is not rigid, the deflection of girders caused by sudden removal of a column will be greater than that of the rigid panel zone case and the progressive collapse potential of the structure will be increased. On the other hand, when the girder–column joints are strong enough to activate full catenary action of girders, the girders can sustain larger deformation even after significant plastic rotation occurs at girder ends. Therefore for more accurate evaluation of progressive collapse potential it would be necessary to consider connection strength including panel zone effect and the development of catenary action in the analysis. Further study is still required to provide more information about the connection properties of structures and to validate the failure criteria currently recommended in the guidelines.

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