Investigation of Progressive Collapse-Resisting Capability of Steel Moment Frames Using Push-Down Analysis

Taewan Kim[1]; Jinkoo Kim[2]; and Junhee Park[3]

Abstract: In this study the vertical push-down analysis was conducted to investigate the resistance of steel moment frames for progressive collapse. The analysis was carried out by gradually increasing the vertical displacement in the location of the removed column and the vertical load in all spans corresponding to the increase of vertical displacement. The analysis results showed that the load resisting capacity increased as the number of stories and the number of spans increased. However, as the length of a span increased, the load resisting capacity against progressive collapse decreased. The load-displacement relationships obtained from push-down analyses were compared with those obtained by incremental nonlinear dynamic analyses, and the results showed that the maximum load factors resulted from the dynamic analyses were a little less than those from the push-down analyses. This implies that the push-down analysis might overestimate the inherent capacity of structures against progressive collapse.

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CE Database subject headings: Progressive collapse; Steel frames; Dynamic analysis; Maximum loads.

Introduction

The progressive collapse refers to the phenomenon that local damage of structural elements caused by abnormal loads results in global collapse of the structure. From a series of accidents it was observed that, to prevent the progressive collapse, a structure should have continuity for offering an alternate path and stability of the structure when an element of vertical load-resisting systems is removed. To prevent the progressive collapse, the National Building Code of Canada (National Research Council of Canada 1996) specified requirements for design of major elements, establishment of connection elements, and ways of providing load transfer paths. The Eurocode 1 (European Committee for Standardization 2002) presented a design standard for selecting plan types for preventing progressive collapse, and recommended that buildings should be integrated. In the United States, specific provisions related to the progressive collapse are not yet provided in design codes such as the International Building Code [International Code Council (ICC) 2006]. However, the ACI 318 (American Concrete Institute 2002) requires structural integrity (for example, continuity insurance of reinforcing bars) so that partial damage by abnormal load does not result in total collapse. The ASCE 7–05 (ASCE 2005) recommended design method and load combination as well as structural integrity as ACI 318 does.

Recently, the General Service Administration (GSA) presented a practical guideline for design to reduce collapse potential of federal buildings (U.S. General Services Administration 2003), and the Department of Defense (DoD) also presented a guideline for the new and existing DoD buildings [Unified Facilities Criteria-Department of Defense (UFC-DoD) 2005]. The analysis method recommended in these guidelines is the alternate path (AP) method, which is also called the alternate load path method. The AP method is executed in the following manner: (1) remove a column from a designated plan location and conduct an analysis; (2) check the limit state of elements; (3) if the limit state is exceeded, remove the element and redistribute the loads to adjacent elements; and (4) repeat analysis until no element exceeds the limit state or collapse is predicted. Currently the following four procedures are generally recommended to operate the AP method for progressive collapse analysis: linear elastic static (LS) analysis, linear elastic dynamic analysis, nonlinear static (NS) analysis, and nonlinear dynamic (ND) analysis. This categorization followed the FEMA 273 (Federal Emergency Management Agency 1997) procedures for seismic analysis.

Kaewkulchai and Williamson (2003) investigated the analysis procedures using a two-dimensional frame analysis. They showed that the LS analysis may result in nonconservative results since it cannot reflect the dynamic effect by sudden exclusion of columns. Marjanishvili (2004) studied the advantage and disadvantage of the above procedures when applying them in the progressive collapse analysis. Powell (2005) compared the LS, NS, and ND analyses and found that a factor of 2 regulated in the guidelines for static analyses can display very conservative result, and insisted that basically the nonlinear analysis should be used. Ruth et al. (2006) found that a factor of 1.5 better represents the dynamic effect especially for steel moment frames. Marjanishvili and Agnew (2006) 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. Kim and Kim (2009) investigated the progressive collapse-resisting capacity of steel moment resisting frames using the linear static method recommended in the GSA and DoD guidelines. It was observed that, compared with the ND analysis results, the

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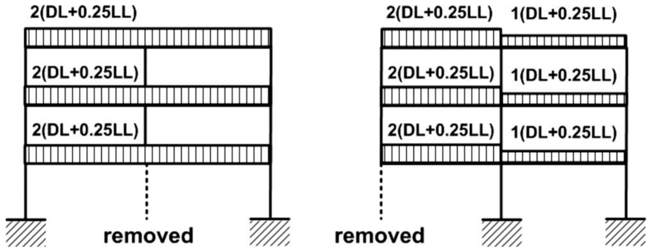


Fig. 1. Applied load for static analysis (U.S. General Services Administration 2003)

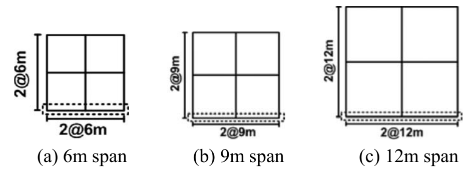


Fig. 2. Plan shape of the analysis model structures

linear static analysis provided smaller structural responses and the results varied more significantly depending on the variables such as applied load, location of column removal, or number of building stories. However the linear procedure provided more conservative decision for progressive collapse potential of model structures.

In case of the linear analysis, modeling is simple and analysis is convenient compared to the nonlinear analysis, and the AP method is easily executed by examining demand/capacity ratio. However as the phenomenon of progressive collapse is nonlinear in nature, it is more reasonable to carry out nonlinear analyses with nonlinear modeling of each element. Among the nonlinear analysis procedures, the NS analysis has several shortcomings compared to the ND analysis. Marjanishvili (2004) indicated that the NS analysis might result in prediction of larger ductility demand, which means that it would produce conservative results because the load path moves not to surroundings but to vertical direction. Marjanishvili and Agnew (2006) also indicated that in the NS analysis, verification of the results may be complicated and repeated run of the analysis may be involved. Tsai et al. (2007) carried out NS and dynamic analyses of an 11-story reinforced concrete structure and suggested that different acceptance criteria need to be used for NS and dynamic methods. The results of previous research mentioned above showed that the analysis procedures presented in the guidelines possess both advantage and disadvantage and more research is still required for accurate application of each analysis procedure.

In this background, this study investigated the applicability of push-down analysis to assessing the progressive collapse resisting capacity of buildings. To confirm its applicability, example buildings were designed with various design variables and the effect of the design variables on the performance of the buildings was investigated. Finally, the load-displacement relationships obtained from the push-down analyses were compared with those obtained by incremental ND analyses and ND time history analyses. It is noted that even though the collapse behavior is basically dynamic in nature, the static push-down analysis is still relevant to investigate the collapse behavior of a structure. As the lateral push-over analysis is widely used to evaluate structural properties such as yield stress, lateral stiffness, maximum lateral load resistance, and ultimate lateral displacement, it is expected that similar useful information may be obtained by the push-down analysis for progressive collapse.

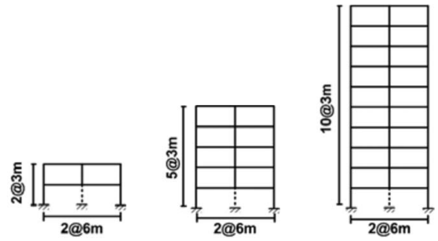
Nonlinear Push-Down Analysis of Model Structures

Application of Push-Down Analysis

The NS push-over analysis method, which has been widely used in earthquake engineering field, was adopted to investigate the structural performance of buildings against progressive collapse. The advantage of this procedure is its ability to account for nonlinear effects without sophisticated hysteretic material modeling and time-consuming time-history analysis. The disadvantages are the inability to consider dynamic effects caused by sudden removal of columns. However this procedure is useful in determining elastic and failure limits of the structure. In this study, the vertical push-over analysis or push-down analysis was applied by increasing the vertical displacement in the location of the removed column to investigate the resistance of the structure against such deformation. Since this nonlinear push-down method is displacement controlled, the analysis was carried out by increasing displacements to an arbitrary level. On the contrary, the nonlinear analysis procedure proposed in the GSA guidelines is a load control method that increases the load incrementally to a specified level. Therefore the method used in this research has difference in the control parameters with the guideline method.

As mentioned by Marjanishvili and Agnew (2006), the load controlled push-down analysis generally involves several reruns and highly depends on load step or tolerance. On the other hand, the displacement controlled push-down analysis is easy to run and there is little chance to diverge. In this study the displacement-controlled push-down analyses were carried out using the program code OpenSees (Mazzoni et al. 2006). In the analysis, gravity load was imposed on all beams with its original loading pattern unchanged. At every step during the push-down analysis, i.e., at each level of the vertical displacement, the amount of equivalent load corresponding to the displacement level was determined. The amount of the load is referred to as the "load factor," which is the ratio of the equivalent load to the full gravity load. The original loading pattern remained unchanged at every step. In this way the results of the displacement-controlled push-down analysis were maintained to be the same with those obtained from load controlled push-down analysis until the ultimate loads were reached. In this study, the load controlled push-down analyses were also conducted for comparison and the results showed that load factor versus vertical displacement relationships resulted from both methods were identical until the load factor dropped rapidly. The load controlled push-down analysis cannot find solutions when the load factor starts to drop. In the OpenSees, the size and pattern of the applied vertical loads are identical at the same displacement level for both load and displacement controlled methods, which results in the same load-displacement relationships until maximum load is reached.

The U.S. General Services Administration (2003) and the UFC-DoD (2005) proposed the amplification factor of 2 for the static analysis to account for dynamic redistribution of forces. The load combination of the U.S. General Services Administration (2003) for the analysis is 2(Dead Load$+0.25\times$Live Load) and that of the UFC-DoD (2005) is 2($1.2\times$Dead Load$+0.5\times$Live Load)$+0.2\times$Wind Load. In this study, the load combination of the U.S. General Services Administration (2003) was selected for push-down analysis. This amplified load was applied



(a) 2-story (b) 5-story (c) 10-story

Fig. 3. Elevation view of the analysis model structures

only in the spans in which a column was removed while unamplified load was applied in the other spans (Fig. 1).

Design of Analysis Models

As model structures for progressive collapse analysis, two-, five-, and ten-story steel moment frames with two bays in both directions were selected. Figs. 2 and 3 depict the plan shape and elevation view of the model structures. Only the exterior frames marked on the plans were analyzed. The span length was varied by 6, 9, and 12 m for investigating the effect of span length on the resistance against progressive collapse. The columns and beams were designed using the SM490 (F_y=324 MPa) and SS400 (F_y =235 MPa) steel, respectively, based on the load and resistance factor design specification of the American Institute of Steel Construction (AISC) (2000). The earthquake loads with S_{DS} and S_{D1} equal to 0.44 and 0.23 g for a 2,500-year return period, respectively, and 0.33 and 0.18 g for a 500-year return period, respectively, were used to design the model structures [International Code Council (ICC) 2006]. To investigate the effect of the number of bays on the progressive collapse potential, the number of bays of the model structures was increased to 4 and 6, and the results were compared with those of the two-bay structures. The structural members of the two- and five-story model structures with two bays are presented in Tables 1–4. To initiate progressive collapse the corner and the center columns in the first story were removed one at a time.

Analytical Modeling of Structures

For NS push-down analysis the column and beam elements were modeled using the "beamWithHinges" element in the OpenSees, and their material properties were represented by the "Hysteretic" element as shown in Fig. 4. The strain hardening ratio was assumed to be 2% of the initial stiffness. In the Hysteretic element, the strength was modeled to drop rapidly when rotation of the members reaches the maximum value of 0.035, which is specified as limit state for connections in steel frames in U.S. General Services Administration (2003). Even though the connection details generally affect the behavior of the structure, in this study the connection was assumed to be rigid for simplicity.

Table 1. Dimension of Structural Members in a Two-Story Structure (Earthquake Load with a 2,500-Year Return Period) (mm)

Members	Story		6 m		9 m		12 m
Columns	1–2	Ext.	H 125×125×6.5×9	Ext.	H 175×175×7.5×11	Ext.	H 250×250×9×14
		Int.	H 200×200×8×12	Int.	H 250×250×9×14	Int.	H 300×300×10×15
Beams			H 354×176×8×13		H 482×300×11×15		H 594×302×14×23

Table 2. Dimension of Structural Members in a Five-Story Structure (Earthquake Load with a 2,500-Year Return Period) (mm)

Members	Story		6 m		9 m		12 m
Columns	1–3	E	H 200×200×9×12	E	H 250×250×9×14	E	H 300×300×10×15
		I	H 350×350×12×19	I	H 370×370×12×19	I	H 400×400×13×21
	4–5	E	H 175×175×7.5×11	E	H 200×200×8×12	E	H 250×250×9×14
		I	H 250×250×9×14	I	H 250×250×9×14	I	H 350×350×12×19
Beams	1–3		H 380×200×8×11		H 580×200×12×20		H 594×302×14×23
	4–5		H 350×200×8×11		H 520×200×12×20		H 580×300×14×23

Table 3. Dimension of Structural Members in a Two-Story Structure (Earthquake Load with a 500-Year Return Period) (mm)

Member	Story		6 m		9 m		12 m
Column	1–2	E	H 120×120×6.5×9	E	H 150×150×7×10	E	H 240×240×9×12
		I	H 190×190×8×12	I	H 250×250×9×14	I	H 295×295×10×15
Beam			H 350×170×8×13		H 450×300×10×15		H 620×300×15×20

Table 4. Dimension of Structural Members in a Five-Story Structure (Earthquake Load with a 500-Year Return Period) (mm)

Member	Story		6 m		9 m		12 m
Column	1–3	E	H 190×190×8×12	E	H 200×200×8×12	E	H 290×290×10×15
		I	H 300×300×10×15	I	H 350×350×12×19	I	H 380×380×13×21
	4–5	E	H 150×150×7×10	E	H 175×175×7.5×11	E	H 230×230×9×14
		I	H 250×250×9×14	I	H 250×250×9×14	I	H 300×300×10×15
Beam	1–3		H 354×176×8×13		H 500×300×10×15		H 630×300×15×20
	4–5		H 330×170×8×13		H 430×300×10×15		H 580×300×14×20

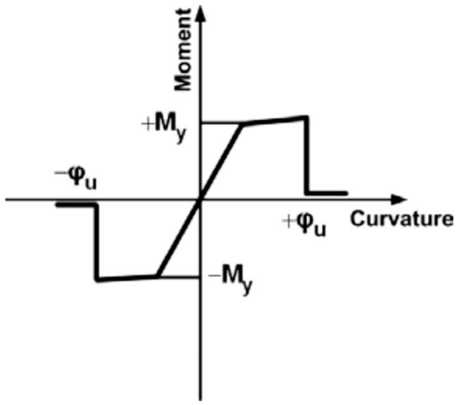
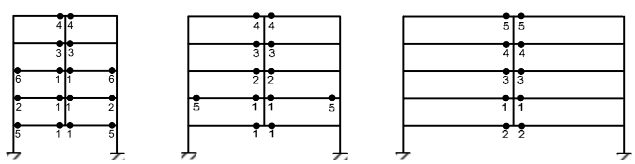
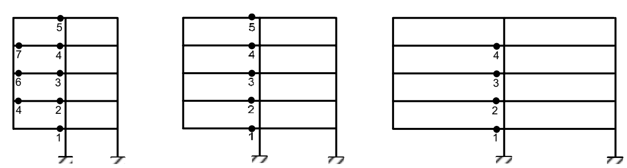


Fig. 4. Hysteretic model for beam members



(a) 6m span structure (b) 9m span structure (c) 12m span structure

Fig. 5. Sequence of hinge formation in five-story structures designed for seismic load of a 2,500-year return period when the center column is removed



(a) 6m span structure (b) 9m span structure (c) 12m span structure

Fig. 6. Sequence of hinge formation in five-story structures designed for seismic load of a 500-year return period when the corner column is removed

Push-Down Analysis Results

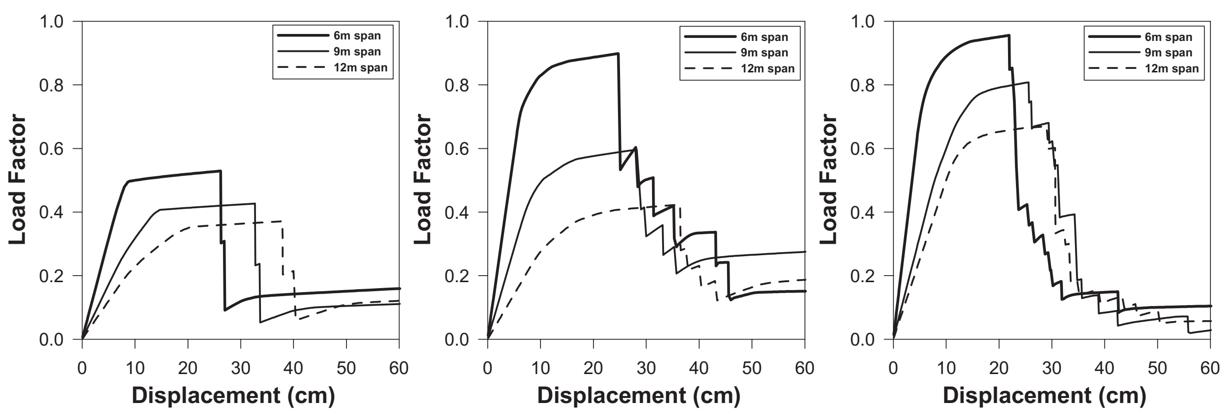
Sequence of Hinge Development

Figs. 5 and 6 show the locations and sequence of hinges in the five-story structures designed for seismic load with a 2,500-year return period when the center and the corner columns were removed, respectively. As recommended in the guideline, each member end was assumed to become the "hinge" when the maximum rotation reaches 0.035 rad. In the figures, the hinges formed right before total collapse was depicted. When the center column was removed, hinges formed first at the beam end adjacent to the joint where column was removed, and spread to upper stories. When the corner column was removed, all the hinges formed at the bay in which the removed column was located. In all structures the hinges started at the second floor beams near the center column and spread to higher stories. In the 9- and 12-m-span structures, hinges formed only at the far end of the beams located in the bay from which the column was removed.

Effect of Number of Stories and Span Length

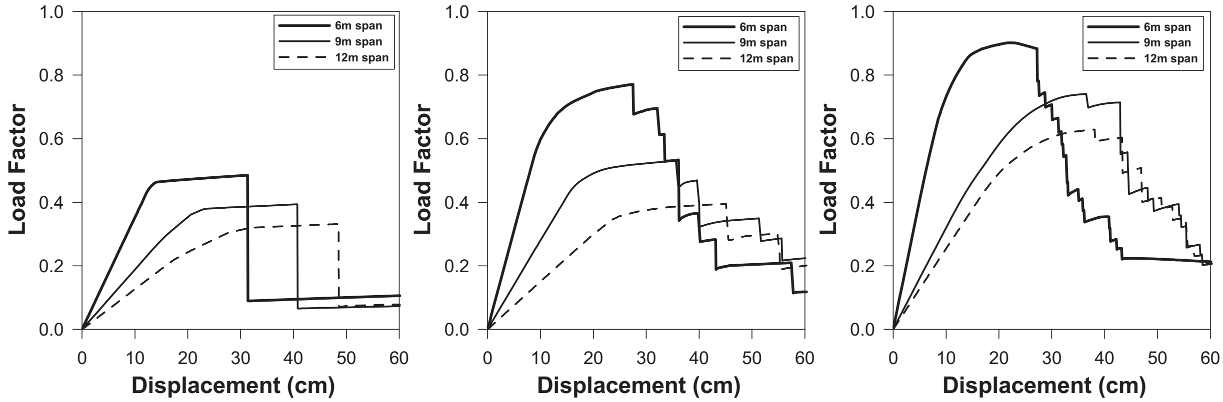
In seismic pushover analysis the relationship between the base shear and the story drift is observed to estimate the seismic load-resisting capacity. In this study of vertical push-down analysis, the vertical displacement and the sum of the vertical loads corresponding to the vertical displacement were plotted in Figs. 7–11 for various parameters such as span length, number of stories, earthquake hazard level, number of bays, and the location of column removal. Since the vertical load varies with the dimension of the plan and the elevation, the ratio of the vertical load at each step to the specified total vertical load shown in Fig. 1 was plotted in the figures as dimensionless loading parameter.

Fig. 7 shows the vertical load-displacement relationship of the two-bay model structures designed for seismic load with a return period of 2,500 years when the center column of the exterior frame was removed. It can be observed that as the span length decreases both the stiffness and the ultimate strength of the model structures increase as well. It also can be observed that as the number of stories increase, the stiffness and the ultimate strength also increase. This seems to be reasonable considering that as the number of stories increases the number of structural members involved in the load transfer also increases and so do the redun-



(d) 2-story structure (e) 5-story structure (f) 10-story structure

Fig. 7. Load-displacement relationship of the model structures when the center column is removed



(d) 2-story structures (e) 5-story structures (f) 10-story structures

Fig. 8. Load-displacement relationship of the model when the corner column is removed

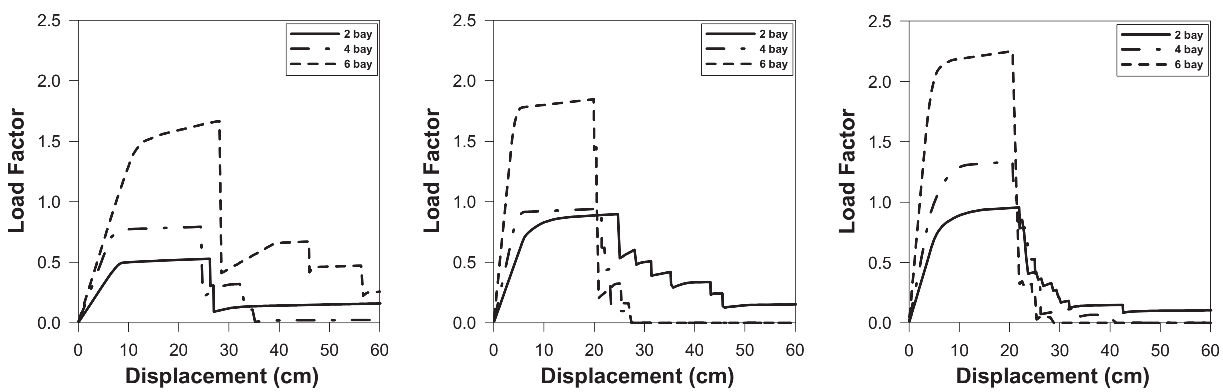
dancy and the resistance to progressive collapse. The difference in ultimate strength of structures with different number of stories decreases as the span length increases and both the stiffness and the strength decrease as the span length increases. Based only on ultimate strength it can be concluded that the progressive collapse resisting capacity increases as the number of story increases and as the span length decreases.

In a nonlinear analysis, not only the load resisting capacity but also the displacement resisting capacity is important to define the behavior of the structure. In Fig. 7, it can be observed that the maximum displacements at the limit state of failure decrease as the number of stories increases. It also can be noticed that the maximum displacements at the limit state increase with the span length. For the same displacement, the rotation in the connection decreases as the span length increases. Likewise, for given limit state for end rotation, the structure with longer span length will sustain larger displacement.

The displacement resisting capacity may also be estimated by ductility which is the ratio of the maximum deformation to the yield deformation. The ductility was determined from the ratio of the maximum displacement at the limit state described above to the yield displacement and it was observed that the ductility capacity decreased as the span length increased. This results from the increase of the yield displacement due to the increase of the span length while the maximum displacement does not change significantly due to the fixed maximum rotation at joints, which is 0.035 rad. It also was observed that the ductility is almost independent of the number of stories. According to the results of this study, it is concluded that the number of stories only affects the ultimate strength.

Removal of the Corner Column

Fig. 8 shows the vertical load-displacement relationship of the model structures designed for seismic load with a return period of 2,500 years when the corner column of the exterior frame was removed. As a result of the removal of the corner column, the structure became unsymmetrical and the overall behavior was expected to be different from that caused by the removal of the center column. Compared with the results from the center column removal, it can be observed that the maximum strengths and the ductility decrease and the maximum displacements before failure increase. The reason for the decrease in the maximum strength is that only single span resists the vertical load resulting from the corner column removal. However, the effects of the change in the span length and the number of stories on the load-displacement relationship are quite similar to the case of center column removal.



(a) 2-story structures (b) 5-story structures (c) 10-story structures

Fig. 9. Load-displacement relationship of the model structures with 6–m span length with different numbers of bays (center column removed)

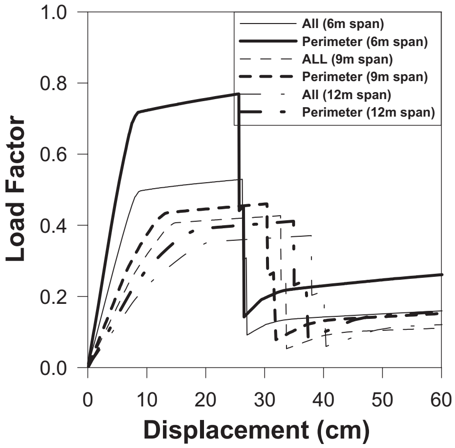


Fig. 10. Load-displacement relationship of the two-story structure when the center column is removed (All: seismic load is shared by all frames; Perimeter: seismic load is resisted only by perimeter frames)

Effect of Number of Bays

Fig. 9 shows the push-down analysis results of the model structures with two, four, and six bays. The structures were designed for seismic load of a 2,500-year return period and the center column was removed for progressive collapse analysis. It can be observed that the strength increases significantly as the number of bays increases. From Figs. 7–9, it can be noticed that the load factors in all structures with six bays and the 4-bay 10-story structure exceed 1.0. The maximum load factor of the 4-bay 10-story structure, which is about 1.3, is similar to those observed by Tsai et al. (2007), who carried out push-down analysis of the 3-bay 11-story reinforced concrete structure with span length of 5.6 and 6.55 m and found that the load factor of the external frame varies from about 1.2–1.4 depending on the location of removed column.

Effect of Seismic Load-Resisting Frames

The analysis results shown previously were obtained from model structures designed in such a way that seismic load was shared by all frames located in its direction. When seismic load is not so significant, it is common practice that only perimeter frames are designed to resist seismic load and the other frames are designed only for gravity load. Fig. 10 compares the push-down curves of the model structures with different seismic load resisting frames. In the figure legends the terms "All" and "Perimeter" represent the structure in which seismic load is shared by all frames and by perimeter frames, respectively. As expected, the maximum strengths of the perimeter frames which were designed to resist all seismic load were higher than those of the structures in which the seismic load was distributed to all frames. The increase in stiffness and strength became less significant in the structures with longer span length, in which the contribution of gravity load was more dominant in structural design.

Earthquake Hazard Level

The results previously shown in Figs. 7–10 are obtained for the model structures designed for the earthquake hazard with a 2,500-year return period. To investigate the effect of the earthquake hazard level on the load-resisting capacity, push-down analyses were carried out with structures designed for earthquake load with a 500-year return period. Fig. 11 shows the analysis results of the structures with the center column removed. The load resisting capability of the structures designed for earthquake load with a 500-year return period decreased as compared with those of structures designed for the 2,500-year return period earthquake. This is reasonable since the members were designed with smaller lateral loads. However, in the two-story buildings with 9 and 12–m span, little difference was observed. This is due to the fact that as the earthquake loading is small compared to the vertical loads in the two-story structure, the selected member sizes are similar. It also can be observed that the difference decreases as the span length increases, which is due to the fact that as the span length increases the contribution of gravity load increases relative to the seismic load. Also the difference increases as the number of stories increase.

Compared with the results of the structures with earthquake load of a 2,500-year return period, the maximum displacement and the ductility are not very different. This is resulted from the fact that the limit state for rotation of beams is given as 0.035

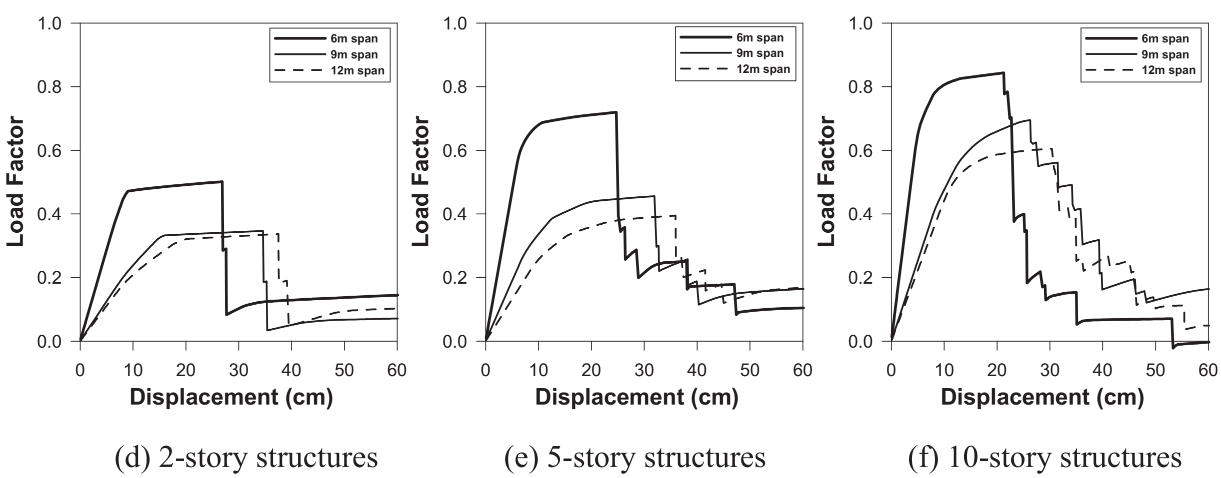


Fig. 11. Load-displacement relationship of the model structures designed for seismic load of a 500-year return period when the center column is removed

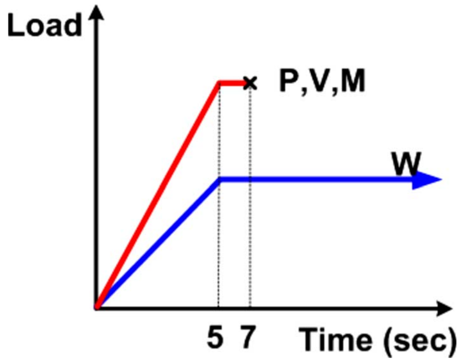
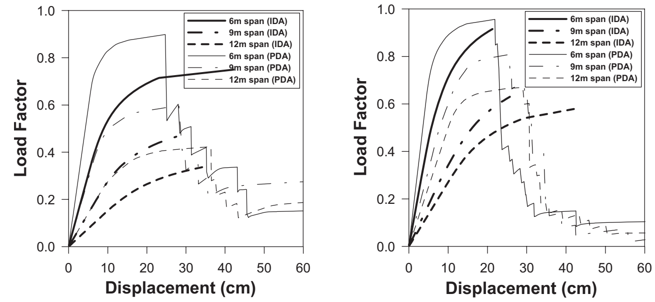


Fig. 12. Application of dynamic load for progressive collapse



(a) 5-story structures

(b) 10-story structures

Fig. 14. Load-displacement relationship of the 5- and 10-story model structures obtained by IDA when the center column is removed

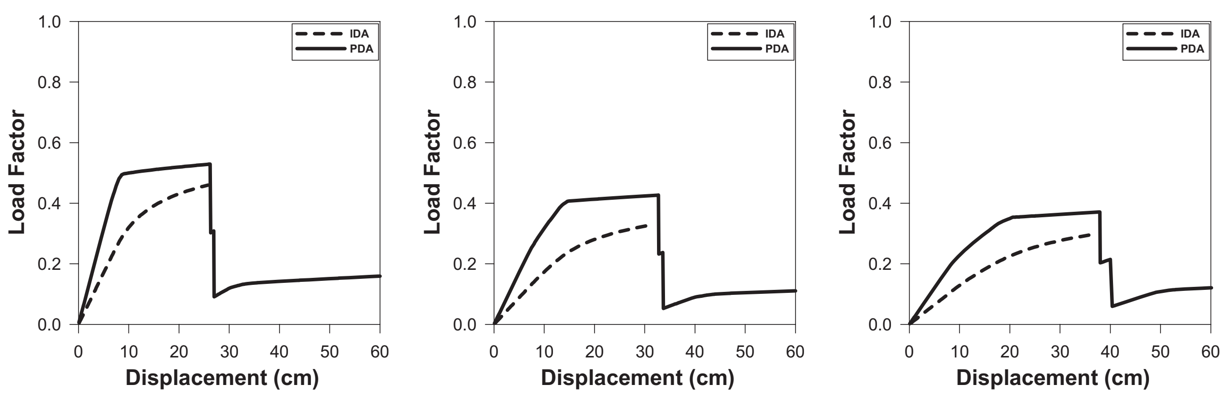
regardless of the beam size. In fact this value should be confirmed by experiments since the maximum value may depend on the dimension of the members.

Nonlinear Dynamic Analysis Results

Load-Displacement Relationship Obtained by Incremental Dynamic Analysis

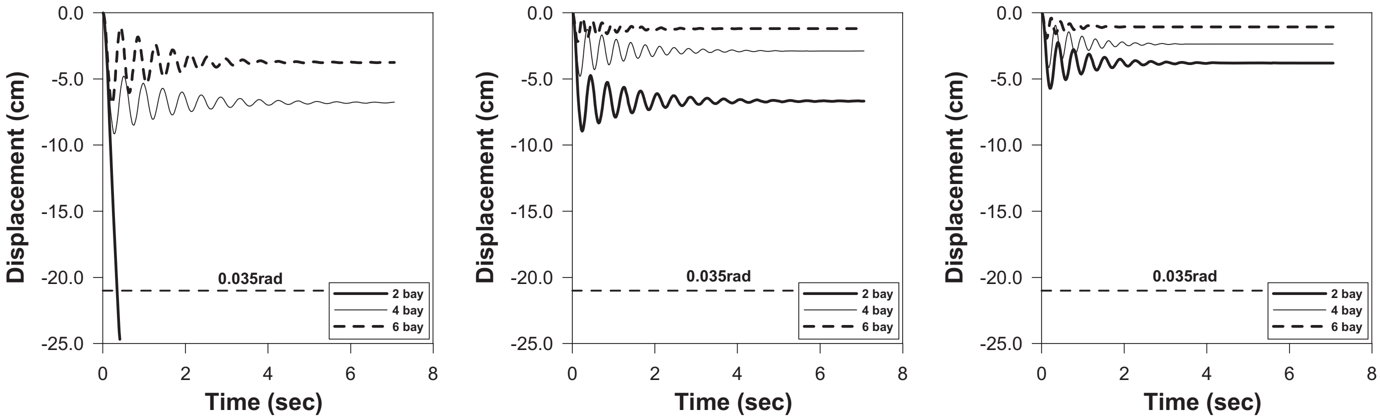
To confirm the results obtained from the push-down analyses, the incremental dynamic analyses (IDA) were conducted with the same model structures. The IDA is usually used in performance evaluation of a structure subjected to seismic load, in which seismic load is increased incrementally and the maximum story drift is obtained at each analysis step. In this study, vertical load was increased incrementally and the maximum vertical displacement was obtained by ND analysis at the location where a column was removed. To carry out dynamic analysis, the axial force acting on the column was computed first before it was removed. Then the column was replaced by point forces equivalent of its member forces. To simulate the phenomenon of abrupt column removal, the forces were increased linearly for 5 s until they reached their full amounts, kept unchanged for 2 s until the system reached stable condition, and were suddenly removed at 7 s to simulate the sudden removal of the column as shown in Fig. 12 where the variables P, V, and M denote the axial force, shear force, and bending moment acting on the removed column, respectively, and W is the vertical load.

The IDA results for the model structures designed for seismic load of a 2,500-year return period when the center column was removed were compared with the push-down curves in Figs. 13 and 14. For comparison with the push-down analysis results, the load factor for each IDA was obtained by means of dividing the applied vertical load by 2(Dead Load$+0.25\times$Live Load). Each IDA was stopped when the vertical displacement became significantly large. It can be observed that both stiffness and strength of the load-displacement curves obtained by IDA are smaller than those from the push-down analyses in all structures. Therefore, the load-resisting capacity represented by the load factor tends to be overestimated when it is predicted by NS push-down analyses. These results are compatible with those obtained by Tsai et al. (2007) in an 11-story reinforced concrete structure. However, the trend obtained from various parameters was similar in both the push-down analysis and the IDA. It also can be noticed that the maximum displacement decreases as the number of stories increase and the span length decreases. If they are compared with the displacements at which the load factors obtained from push-down analyses drop, the trend is almost identical.



(a) 6m span structures (b) 9m span structures (c) 12m span structures

Fig. 13. Load-displacement relationship of the two-story model structures obtained by IDA when the center column is removed (push-down analysis)



(a) 2-story structures (b) 5-story structures (c) 10-story structures

Fig. 15. Vertical displacement at the joint obtained by dynamic time history analysis of the model structures with 6-m span lengths

Time Histories of Vertical Deflection

Fig. 15 depicts the ND time history analysis results for vertical deflection of the joint from which a column was removed. The first story center column was removed from model structures with a 6-m-span length designed for seismic event with a return period of 2,500 years. For dynamic analysis, the load combination (Dead Load+0.25×Live Load) was applied following the GSA guidelines for dynamic analysis. The limit state for vertical deflection stipulated by the GSA guidelines was also plotted as dotted horizontal lines for comparison. It can be observed that the maximum deflections of the model structures satisfy the given limitation, except for the two story structure with two bays. By comparing these results with the push-down curves of the 6-m-span model structures shown in Fig. 9, it can be noticed that the structures with the maximum load factor larger than 0.5 satisfies the given failure criterion for vertical deflection in dynamic analysis. This observation is compatible with the Marjanishvili and Agnew (2006) who asserted that the structure having the load factor under 0.5 has a high potential for progressive collapse. Based on their opinion and on the analysis results obtained in this study, some example buildings, especially the two-story two-bay buildings with 9- and 12-m spans and the five-story two-bay building with 12-m spans, have a potential for progressive collapse and should be redesigned to be adequate for resisting progressive collapse.

Conclusions

This study investigated the progressive collapse resisting capacity of steel moment framed structures using the NS push-down analysis method. The effect of various design variables, such as the span length, number of stories, number of bays, and the level of earthquake load, were investigated. The load-displacement relationships of the model structures obtained by push-down analyses were compared with those from nonlinear IDA. The findings of this study are summarized as follows:

1. The progressive collapse potential of steel moment resisting frames increases as the number of stories decrease and the span length increases.
2. The increase in number of bays results in a significant increase in resistance to progressive collapse.
3. As the design earthquake load increases, the load resisting capacity of structures also increases. The contribution of earthquake load on the load-resisting capability of model structures decreases as the number of stories decrease and the span length increases.
4. Compared with the incremental ND analysis results, the static push-down analysis may overestimate the inherent capacity of structures against progressive collapse.
5. According to the ND analysis results, the structures with the maximum load factor larger than 0.5 satisfy the failure criterion for vertical deflection given in the GSA guidelines.

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