Design of steel moment frames considering progressive collapse

Jinkoo Kim† and Junhee Park

Department of Architectural Engineering, Sungkyunkwan University, Suwon, Korea

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Abstract. In this study the progressive collapse potential of three- and nine-story special steel moment frames designed in accordance with current design code was evaluated by nonlinear static and dynamic analyses. It was observed that the model structures had high potential for progressive collapse when a first story column was suddenly removed. Then the size of beams required to satisfy the failure criteria for progressive collapse was obtained by the virtual work method; i.e., using the equilibrium of the external work done by gravity load due to loss of a column and the internal work done by plastic rotation of beams. According to the nonlinear dynamic analysis results, the model structures designed only for normal load turned out to have strong potential for progressive collapse whereas the structures designed by plastic design concept for progressive collapse satisfied the failure criterion recommended by the GSA guideline.

Keywords : steel moment frames; progressive collapse; plastic design; virtual work method.

1. 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. An abnormal load includes any loading condition that is not considered in normal design process but may cause significant damage to structures. The potential abnormal loads 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 (NIST 2006). To prevent the progressive collapse caused by abnormal loads, the National Building Code of Canada (1996) specified requirements for design of major elements, establishment of connection elements, and ways of providing load transfer paths. The Eurocode 1 (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 (ICC 2006); however the American Concrete Institute (ACI 318, 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 (2005) recommended design method and load combination as well as structural integrity as ACI 318 does. The General Service Administration (GSA) presented a practical guideline for design to reduce collapse potential of federal buildings (GSA 2003), and the Department of Defence (DoD) also presented a guideline for the new and existing DoD buildings (DoD 2005). The analysis method recommended in these guidelines is the alternative pass (AP) method. In this approach, the structure is designed such that if any one component fails, alternate paths are

†Professor, Corresponding Author, E-mail: jkim12@skku.edu

available and a general collapse does not occur. In most cases 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 load path method are linear static, linear dynamic, nonlinear static, and nonlinear dynamic methods, which were also recommended for seismic analysis and design for structures in the FEMA 274 (1997). In linear analysis modeling is simple and analysis is convenient compared to the nonlinear analysis. Kaewkulchai and Williamson (2003) investigated the analysis procedures using a two-dimensional frame analysis, and found that linear static analysis might result in non-conservative results since it cannot reflect the dynamic effect caused by sudden exclusion of columns. 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, Marjanishvili (2004) indicated that the nonlinear static procedure may result in larger ductility because the load path moves not to surroundings but to vertical direction.

To resist progressive collapse of structures Crawford (2002) proposed the use of connection details developed for earthquake such as Side Plate^{TM}, the use of cables imbedded in reinforced concrete beams to activate the catenary action, and the use of mega-truss in high-rise buildings. Suzuki *et al.* (2003) showed that the use of hat-bracing at the top of structures increases the resistance to progressive collapse. Hayes, Jr. *et al.* (2005) investigated the relationship between the seismic design and the blast or progressive collapse-resisting capacity and mentioned that the special moment frame detailing provisions required in areas of high seismicity would provide better resistance to external explosion or impact effects than the less-rigorous detailing required for OMF.

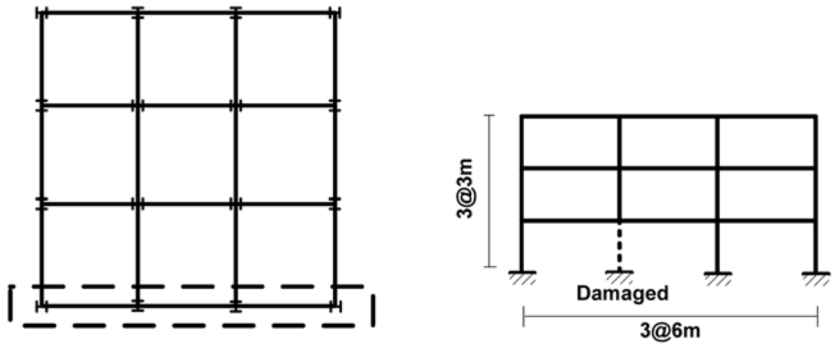
Recently Dusenberry and Hamberger (2006) explained the phenomenon of progressive collapse based on energy balance concept and mentioned that the energy-based method have strong potential to be developed into simplified procedures for collapse potential assessment. Wada *et al.* (2004) estimated the magnitude of gravity load that causes progressive collapse based on plastic analysis method. Murakami *et al.* (2004) applied the virtual work method to assess frame collapse based on thermal deformation analysis results. These studies showed that the energy-based approach or plastic analysis/ design method could be conveniently used to assess structural performance for progressive collapse.

The objective of this study is to check the applicability of the virtual work method to estimate the size of girders required to prevent progressive collapse of a steel special moment resisting frames designed per current design code. The plastic design is a procedure to find out the required plastic moment capacity of members to prevent formation of collapse mechanism of structures when they are subjected to seismic load. In this study the same concept was applied in the progressive collapse which occurs as a result of formation of plastic hinges and collapse mechanism, and its applicability for progressive collapse was evaluated. To this end the progressive collapse potential of steel moment frames designed in accordance with current design code was evaluated first, and the effect of increasing the size of beams on the prevention of progressive collapse was observed. Then the size of beams required to prevent progressive collapse was obtained using the equilibrium of the external work done by gravity load due to loss of a column and the internal work done by plastic rotation of beams. The responses of the plastic-designed structures subjected to sudden loss of a column were obtained by nonlinear dynamic analysis, and the results were compared with the failure criterion provided by the design guideline.

2. Design of example structures and analysis method for progressive collapse

In this section the detailed information for example structures and analysis methods for progressive collapse were provided. To apply the plastic design procedure, special steel moment resisting frames were designed per current design codes. The example structures are 3-bay 3- and 9-story structures with square plan, and the span length are varied as 6 m, 9 m, and 12 m. Fig. 1 shows the structural plan and elevation of the 3-story structure with 6 m span length. The exterior frame enclosed in the dotted rectangle was separated and analyzed for progressive collapse. The design dead and live loads are 5.0 kN/cm^2 and 3.0 kN/cm,^{2} respectively. The design seismic load was determined based on the IBC-2006 with the seismic coefficients S_{DS} and S_{D1} equal to 0.62 and 0.18, respectively. The response modification factor corresponding to a steel special moment frame, which is 8.0, was applied to obtain design base shear. The columns and beams were designed with SM490 steel (F_y = 32.4 kN/cm^2) and SS400 (F_y = 23.5 kN/cm^2), respectively. Table 1 shows the member size of the analysis model structures. In the 3-story structures the same members were used throughout the story. In the 9-story structures member sizes were varied in every three stories.

Fig. 2(a) shows the gravity load (dead load + 0.25 × live load) applied on the model structure with a first story column removed as indicated in the GSA guideline for simulation of progressive collapse. The point load imposed on a column is from the internal beam connected to the column. The load was suddenly applied for seven seconds on the model structures with a first story column removed as shown in Fig. 2(b) to activate vertical vibration. The beams are considered to be failed when their rotations exceed the limit state of 0.035 rad as specified in the guidelines, which is much larger than the 0.02 radian recommended by the FEMA-356 (2006) to satisfy the collapse prevention design objective for seismic load.

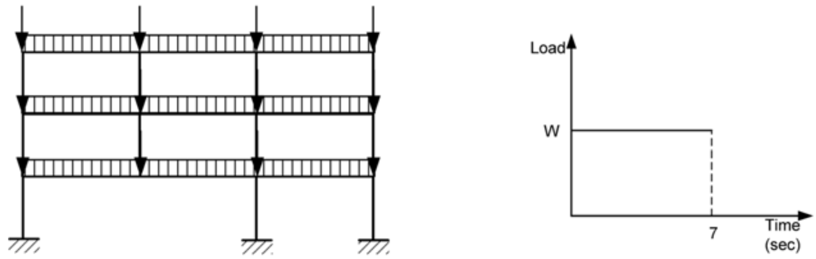


(a) Plan (b) Elevation

Fig. 1 Analysis model structures

Table 1 Member size of the analysis model structures (unit : mm)

(a) 3-story structure			
Story	Ext. columns	Int. columns	Beams
1-3	H 250 × 255 × 14 × 14	H 304 × 301 × 11 × 17	H 300 × 150 × 6.5 × 9.0
(b) 9-story structure			
Story	Ext. columns	Int. columns	Beams
1-3	H 280 × 280 × 13 × 13	H 350 × 350 × 12 × 19	H 346 × 174 × 6.0 × 9.0
4-6	H 270 × 270 × 12 × 12	H 330 × 330 × 12 × 17	H 330 × 160 × 6.0 × 9.0
7-9	H 250 × 250 × 11 × 11	H 310 × 310 × 12 × 15	H 300 × 150 × 7.0 × 9.0



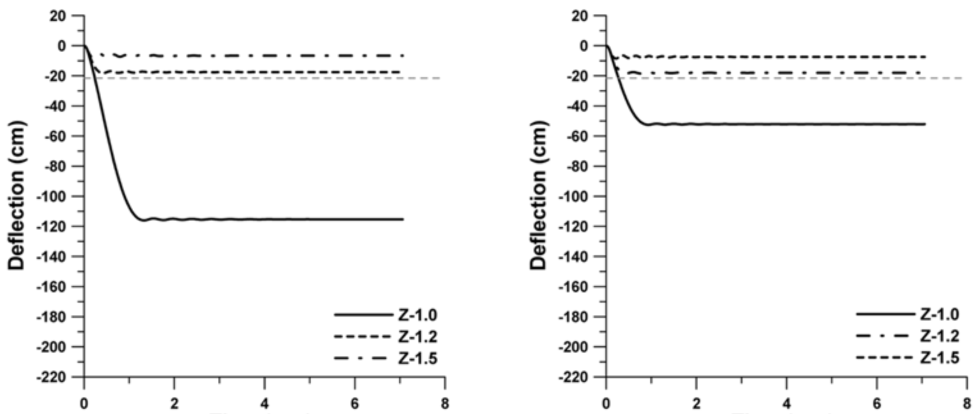
(a) Vertical load (b) Time history of vertical load

Fig. 2 Configuration of vertical load applied on a model structure with a column lost

In this study the performance of structures subjected to sudden removal of a column was investigated by nonlinear dynamic analysis using the program code OpenSees (Mazzoni *et al*. 2006). Nonlinear static (pushover) analyses were also carried out both horizontal and vertical directions to observe load-displacement relationship and plastic hinge formation of model structures. The columns and girders were modeled using the Nonlinear-Beam-Column element with five integration points (Neuenhofer and Filippou 1997). The bi-linear material model with the post-yield stiffness of 2% of the initial stiffness was used, and the damping ratio was assumed to be 5% of the critical damping. Dynamic analysis was carried out using the Newmark time integration method and the Newton-Raphson solution algorithm. Time step of 0.02 second was used for time-history analysis.

3. Progressive collapse potential of code-designed framed structures

To evaluate the progressive collapse potential of the model structures, which were designed per conventional designed code without considering progressive collapse, nonlinear dynamic analyses were carried out without a first story column as shown in Fig. 1 with gravity load suddenly applied on them as shown in Fig. 2. Fig. 3 shows the time history of vertical deflection, where the limit state specified in



(a) 3-story structure (b) 9-story structure

Fig. 3 Time history of vertical deflection at the lost column-beam connection of model structures with various beam plastic moment of inertias the GSA guidelines is plotted as horizontal dotted line. It can be observed that in both the 6- and the 9-story structures with 6 m span length the vertical deflections of the model structures (named Z-1.0) exceeded the given limit states. To enhance the progressive collapse resisting capacity of the model structures, the plastic section moduli of the girders located in the bay where the column was lost were increased by 1.2 and 1.5 times (named as Z-1.2 and Z-1.5, respectively) and the same dynamic analyses were carried out. It can be noticed that the vertical deflection increases as the bay width increases and the girder size decreases. Figs. 3(a) to 3(c) show that the vertical deflection in the 3-story structure with 6 m span decreased more than the limit state when the plastic section moduli of beams were increased by 20%, whereas the section moduli of the structures with 9m and 12m span length needed to be increased by 50% in order to reduce the maximum deflections less than the given limit state. It also can be noticed that the maximum deflection of the 9-story structure was smaller than that of the 3-story structure with the same bay width because the girder sizes were increased due to larger seismic load and more structural members participated in resisting progressive collapse.

The increase in beam size to enhance the resistance to progressive collapse may lead to strong beam-weak column combination and result in formation of weak story when the structure is subjected to earthquake load. The formation of weak story can be identified by large inter-story drift or formation of column plastic hinges. According to the AISC Seismic Provision for Structural Steel Buildings (2005) the weak story can be prevented if a structure is designed in such a way that the summation of plastic moment capacity of columns is larger than that of beams as shown in Eq. (1):

$$\frac{\sum M_{pc}}{\sum M_{pb}} > 1.0 \tag{1}$$

$$\sum M_{pc} = \sum Z_c(F_{yc} - P_{uc}/A_g) \tag{2}$$

$$\sum M_{pb} = \sum(1.1 R_y F_{yb} Z_b + M_{uv}) \tag{3}$$

where M_{pc} and M_{pb} are the plastic moment capacity of columns and beams, respectively; Z_c and Z_b are the plastic section modulus of columns and beams, respectively; F_{yc} and F_{yb} are the yield strengths of columns and beams, respectively; P_{uc} is the required axial strength of columns; A_g is the cross-sectional area of columns; R_y is the ratio of the expected to nominal yield strength; and M_{uv} is the additional moment caused by shear force.

Table 2 presents the ratio of the plastic section moduli of the members in the interior bay of the 3-story 6 m span structure after the plastic section moduli of girders are increased by 20%. It can be seen that the strong column-weak beam condition of Eq. (1) is not satisfied in every story of the model structure. The same phenomenon was also observed in the other model structures when the girder size was increased by the same proportion. Therefore whenever girder size is increased to prevent progressive collapse, it would be necessary to increase the column size in accordance with Eq. (1) to

Table 2 Ratio of plastic moment after the plastic section moduli of girders in the 3-story 6m span structure are increased by 20%

Story	M_{pc}	M_{pb}-$Z1.2$	M_{pc} / M_{pb}
3rd	52566.62	65942.18	0.797
2nd	50935.53	65942.18	0.772
1st	49264.08	65942.18	0.747

prevent unfavorable failure mode when the structure is subjected to seismic load.

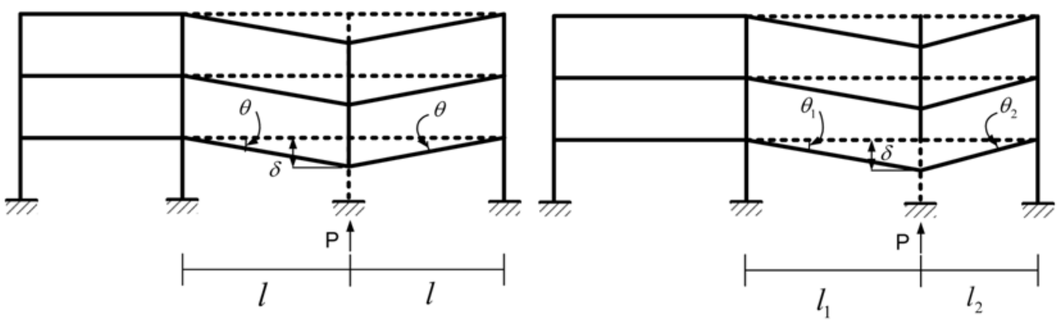
4. Plastic design to prevent progressive collapse

4.1 Determination of beam plastic moment

Progressive collapse occurs due to formation of collapse mechanism in all girders located in the bays in which a column is removed. The formation of collapse mechanism may not lead to total collapse if catenary action is fully activated in girders. However the activation of full catenary action depends on connection details and cannot be guaranteed in all steel structures. Moreover the effect of catenary action is not considered in the limit states for steel structure given in the current GSA guidelines yet. Therefore in this paper the limit state for plastic rotation of girders given in the guideline is used to define progressive collapse neglecting the contribution of catenary action. This would lead to more conservative design. In this case the plastic moment of beams required to prevent formation of collapse mechanism can easily be obtained from the virtual work method in plastic design procedure; i.e. from equilibrium of the internal and the external works triggered by sudden removal of a column. The external work is done by the gravity load imposed on the removed column and the vertical deflection. The internal work is computed by the plastic moments of the beams, which are initially unknown, multiplied by the rotation of the beams. The unknown beam plastic moment required to stabilize the structure can be obtained by equating the internal and the external works.

Fig. 4 shows the deformed configuration of structures with an internal column removed. The loss of a column results in the vertical deflection δ and the beam rotation θ in the structure with identical span length and θ_1 (long span) and δ_2 (short span) in the structure with different span length. After the column is removed progressive collapse will not occur if the external work done by gravity load is in equilibrium with the internal work done by plastic rotation of beams at the vertical deflection less than the limit state. Eq. (4) shows the equilibrium of the external and internal works, from which the unknown plastic moment demand of the *i*th beam M_{pi} can be obtained using the vertical load P acting on the removed column and the virtual vertical deflection δ and the beam rotation θ_1(Moy 1981):

$$P \cdot \delta = \sum_{i=1}^{N} M_{pi} \cdot \theta_i \tag{4}$$



(a) Structure with the same span length (b) Structure with different span length

Fig. 4 Beam rotation angles and joint deflection caused by loss of a column where N is the number of plastic hinges. In case all spans have the same length and the same size of beams are used in all stories, the deflection δ can be expressed in terms of the rotation θ multiplied by the span length l, and the required beam plastic moment M_p is obtained as

$$M_p = \frac{Pl}{N} \tag{5}$$

The principle of virtual work for the structure with different span lengths, shown in Fig. 4(b), can be expressed as Eq. (6) and be simplified as Eq. (7):

$$P \cdot \delta = \sum_{i=1}^{N_1} M_{p1i} \cdot \theta_{1i} + \sum_{j=1}^{N_2} M_{p2j} \cdot \theta_{2j} \tag{6}$$

$$P = \sum_{i=1}^{N_1} \left(\frac{M_{p1}}{l_1}\right)_i + \sum_{j=1}^{N_2} \left(\frac{M_{p2}}{l_2}\right)_j \tag{7}$$

where N_1 and N_2 are the number of plastic hinges in the bays located in the left- and the right-hand sides of the removed column, respectively. As the bending moment of a beam with fixed boundary condition subjected to support settlement is proportional to I / l^2 (Weaver and Gear 1990), the following relationship holds between the bending moments of the beams located in both sides of the removed column:

$$M_{p1} = M_{p2} \cdot \left(\frac{I_1}{I_2}\right)\left(\frac{l_2}{l_1}\right)^2 \tag{8}$$

where the moments of inertia of the beams located in the long and the short spans, I_1 and I_2 respectively, can be obtained from the members selected based on design codes using conventional design loads. If the sizes of beams located in a bay are all the same and if the number of plastic hinges is the same in both bays ($N_1 = N_2 = N / 2$), the required plastic moment of beams located in the span 2 can be obtained as:

$$M_{p2} = \frac{Pl_2}{\frac{N}{2} \cdot \left(1 + \left(\frac{I_1}{I_2}\right)\left(\frac{l_2}{l_1}\right)^3\right)} \tag{9}$$

The size of a beam determined in this procedure is compared with the original size obtained by conventional design procedure, and the larger one is selected for final design.

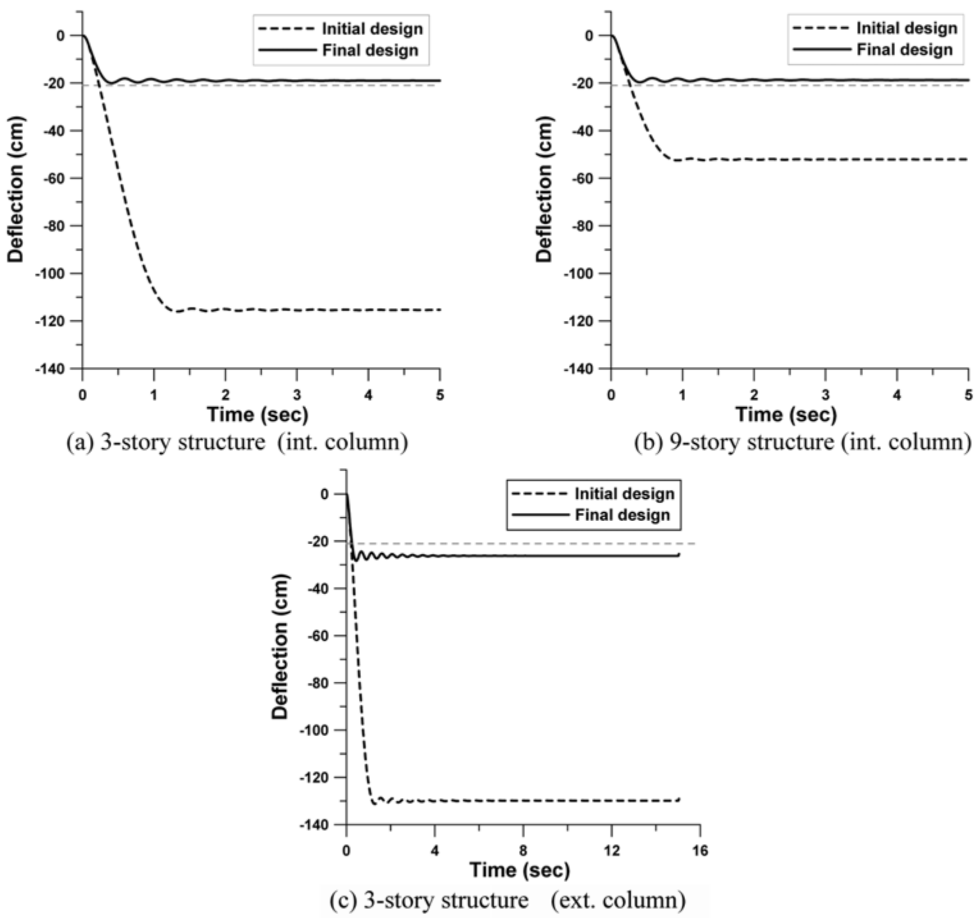
4.2 Performance of re-designed structures

The 3- and 9-story structures with 6 m span length were re-designed using the plastic design procedure, and the member sizes of the re-designed structures were presented in Table 3. Compared with the member sizes determined by considering only conventional design loads (Table 1), the sizes of beams obtained by the principle of virtual work are slightly larger than those by initial design. The column size is also increased proportional to the beam size to meet the strong column-weak beam requirement.

Table 3 Member size of model structures designed by virtual work method (unit: mm)

(a) 3-story structure		
Ext. columns	Int. columns	Beams
H 260 × 260 × 14 × 14	H 340 × 340 × 11 × 17	H 318 × 190 × 7.0 × 9.0
(b) 9-story structure		
Story	Ext. columns	Int. columns
1-3	H 300 × 300 × 13 × 13	H 370 × 370 × 12 × 19
4-6	H 290 × 290 × 12 × 12	H 350 × 350 × 12 × 17
7-9	H 270 × 270 × 11 × 11	H 330 × 330 × 12 × 15

Fig. 5 shows the time history of the vertical deflection triggered by sudden loss of a first story column. The post-stiffness ratio and the damping ratio were assumed to be 2% of the initial stiffness

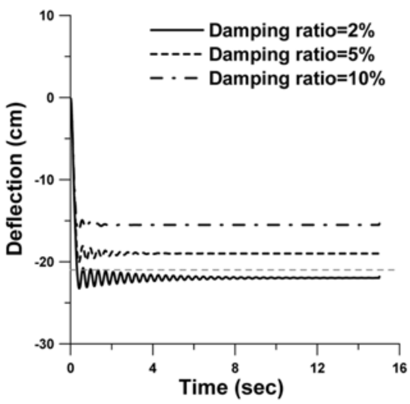


(a) 3-story structure (int. column)
(b) 9-story structure (int. column)
(c) 3-story structure (ext. column)

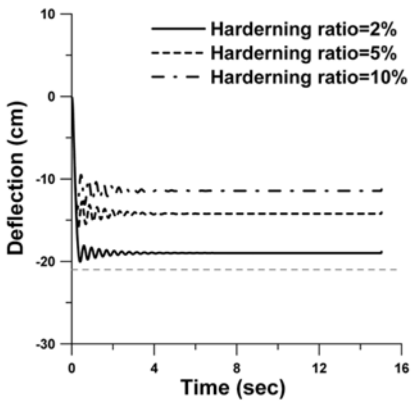
Fig. 5 Time history of vertical deflection in model structures with 6 m span length (5% damping, 2% post-yield stiffness) and 5% of the critical damping, respectively. The horizontal dotted lines represent the maximum allowed value for vertical deflection corresponding to the maximum beam rotation of 0.035rad specified in the GSA guideline. It can be observed that the model structures designed in accordance with the current design code experienced large vertical deflections far exceeding the limit state, while the maximum deflections of the 3- and 9-story structures redesigned by the virtual work method turned out to be less than the given limit state when one of their interior column was lost (Figs. 5(a) and 5(b)). When an exterior column was suddenly removed, the maximum deflection of the 3-story structure slightly exceeded the allowable value, which may be acceptable in the preliminary design stage.

Fig. 6 shows the vertical deflection of the 3-story model structures with 6m span length with various damping and post-yield stiffness ratios. It can be observed that, as expected, the vertical deflection of the structure decreased as the damping ratio and the post-yield stiffness ratio increased. The maximum deflection of the structure with 2% damping and 2% post-yield stiffness ratios slightly exceeded the allowable value. However considering the fact that the damping ratio of a structure subjected to large deformation is larger than 5% (Chopra 2001), the structure redesigned by the plastic design procedure can be considered to be safe for progressive collapse.

Fig. 7(a) shows the load-displacement relationship of the 3-story model structure with 6 m span length obtained by nonlinear static pushdown analysis, and Figs. 7(b)-(d) depict the plastic hinge locations in the redesigned structure at three different loading stages. The order of plastic hinge formation is shown in Fig. 7(e). The post-yield stiffness of the beams and columns were assumed to be 2% of the initial stiffness. Vertical point load was applied gradually at the removed interior column-beam connection and the locations of the plastic hinges were indicated at three loading stages. The dotted horizontal line indicating load factor of 1.0 denotes the loading state of the specified by the GSA guideline (Dead Load + 0.25 × Live Load). The dotted vertical line represents the limit state for progressive collapse at which the beam rotation reaches the limit state of 0.035 rad specified in the GSA guideline. It can be observed that the 3-story structure not designed for progressive collapse yielded at load factor of around 0.7 and reached the limit state at load factor of 0.8. However the structure redesigned by plastic design yielded at higher load factor of about 0.9 and reached the limit state at load factor of about 1.1. It can be observed in Figs. 7(a) and 7(b) that at the global yield points of the pushdown curves plastic hinges formed at the ends of some beams in the redesigned structure.

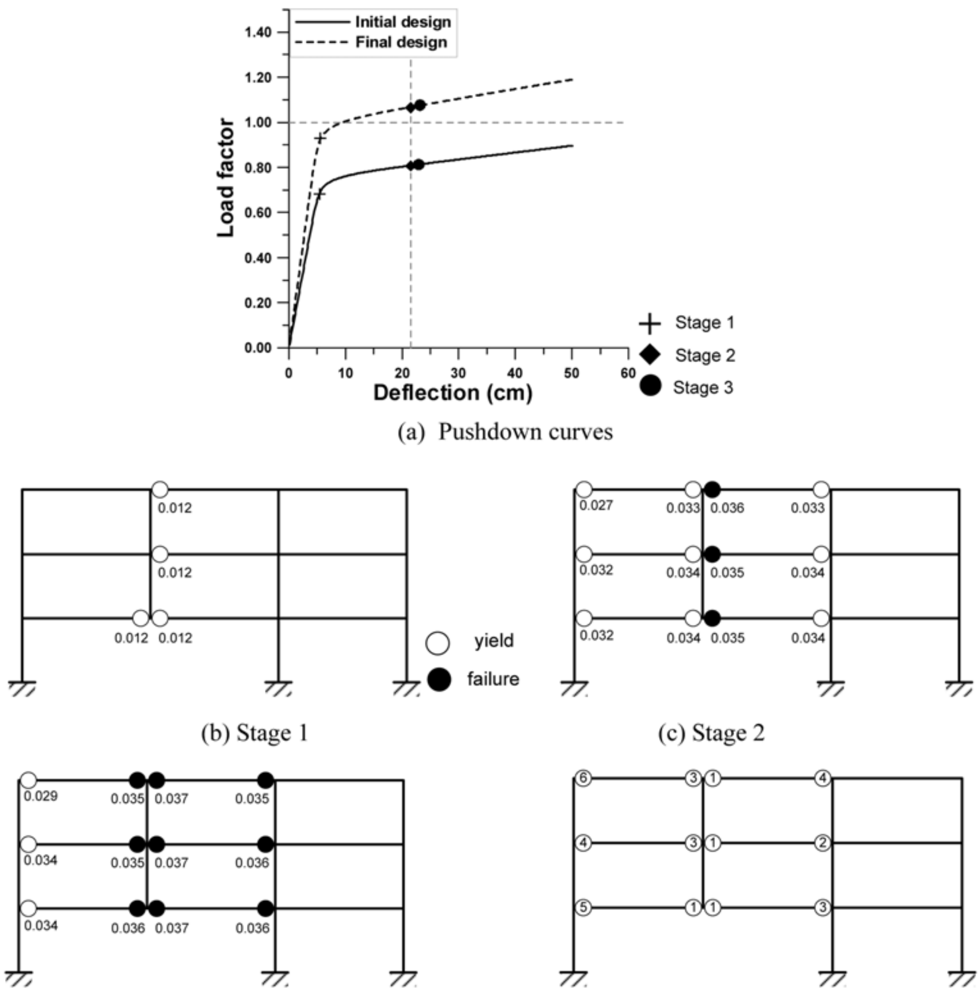


(a) Variation of damping ratios (2% post-yield stiffness)



(b) Variation of post-yield stiffness ratios (5% damping)

Fig. 6 Time history of vertical deflection in the 3-story model structures with 6 m span length for various damping ratios and post-yield stiffness ratios



(a) Pushdown curves

(b) Stage 1

(c) Stage 2

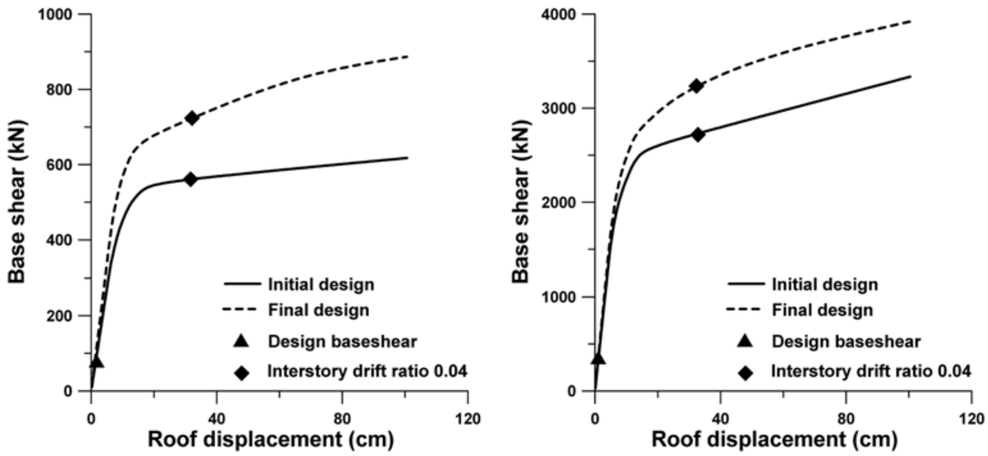
(d) Stage 3

(e) Order of plastic hinge formation

Fig. 7 Vertical pushdown curves and plastic hinge formation of the redesigned 3-story model structures with 6 m span length when an internal column was removed

When the vertical deflection reached the limit state plastic hinges formed at the ends of all beams located in the bays with a column removed and the rotation of some plastic hinges exceeded the limit state given in the guideline (Fig. 7(c). Right after that loading stage two thirds of the plastic hinges reached failure state and the structure is considered to be collapsed (Fig. 7(d)). Fig. 7(e) shows that the plastic hinges form first at the beam ends located in the removed column line and spread to the other beam ends.

Fig. 8 shows the lateral pushover curves of the model structures before and after they were redesigned to prevent progressive collapse, where it can be observed that the lateral strength of the redesigned structures increased significantly due to the increase in member sizes. It was also observed (although not shown in this paper) that no column plastic hinge formed except at the bottom of the first story columns until the structures experienced significantly large plastic deformation.



(a) 6m span length b) Span length ratio 1:0.6

Fig. 8 lateral pushover analysis results of the 3-story model structures

4.3 Performance of structures with different span lengths

The column location of the model structures was varied to check whether the plastic design method is also applicable to structures with different span lengths. Fig. 9 shows the elevations of the 3-story structures with different lengths of spans adjacent to the lost column, and Table 4 compares the plastic moments of conventionally designed and re-designed beams. It can be observed that the plastic moments of the longer beams designed by conventional design load are larger than those of the shorter beams, whereas the opposite is true in the beams designed for progressive collapse by plastic design method. According to Eq. (8) the bending moment of a beam caused by vertical support settlement is inversely proportional to the square of the beam length. Therefore for the two beams to reach the collapse mechanism simultaneously, the plastic moment of the shorter beam needs to be larger than that

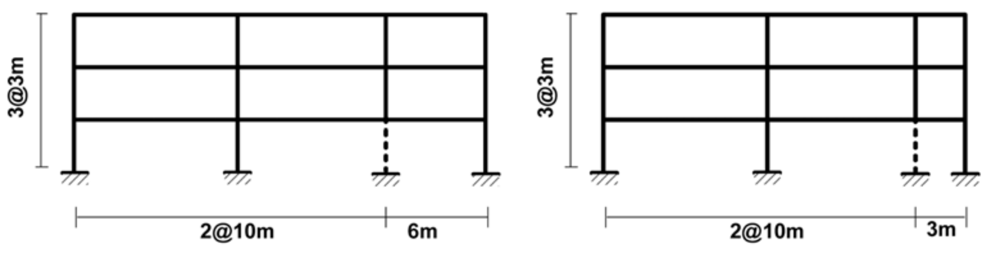


Fig. 9 Model structures with different span length

Table 4 Plastic moment of beams in the structure with span length ratio of 1 : 0.6 (unit : kN.cm)

Span	Initial design	Energy design	Ratio
Long span	59576.61	39159.39	1.52
Short span	32725.16	44698.16	0.73

of the longer beam. The selected beam sizes satisfying the required plastic moments are presented in Table 5(a), and the columns determined to satisfy the strong column-weak beam condition are shown in Table 5(b). As the model structures need to be designed both for current design code as well as for progressive collapse, the larger member is selected for final design. Therefore the consideration of the progressive collapse leads to increase in the size of shorter beams.

Fig. 10 depicts the time histories of vertical deflections in the model structures with different span lengths caused by sudden removal of a first story interior column. The dotted lines are the limit states for progressive collapse determined by the given maximum allowable rotation of short beams. The analysis results show that the maximum deflections of all the model structures designed for conventional loads exceeded the failure criterion, whereas the redesigned structures satisfied the limit states.

Table 5 Member size of structures with different span length

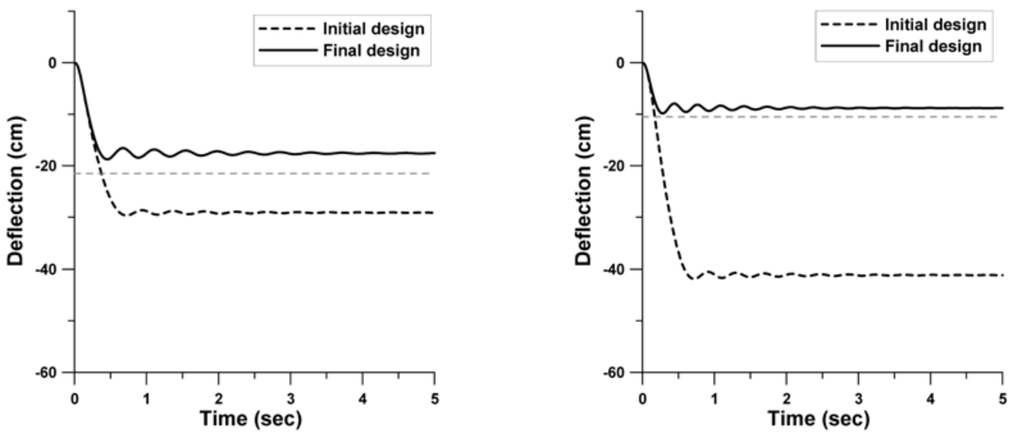
(a) Beams

Span ratio	Design	Long span	Short span	Z_f / Z_i	
1:0.6	Conventional	H 596 × 199 × 10 × 15	H 446 × 199 × 8.0 × 12	1.00	1.00
	Energy-based	H 480 × 200 × 9.0 × 13	H 530 × 200 × 9.0 × 13	0.66	1.37
1:0.3	Conventional	H 596 × 199 × 10 × 15	H 248 × 124 × 5.0 × 8.0	1.00	1.00
	Energy-based	H 229 × 100 × 5.0 × 10	H 390 × 200 × 10 × 12	0.11	4.07

(Z_i, Z_f, : Plastic section modulus determined by initial and energy-based design, respectively) (unit: mm)

(b) Columns

Span ratio	Design method	Column-1	Column-2	Column-3	Column-4
1:0.6	Conventional	H 400 × 400 × 15 × 20	H 550 × 550 × 15 × 20	H 480 × 480 × 15 × 20	H 300 × 300 × 15 × 20
	Energy-based	H 400 × 400 × 15 × 20	H 550 × 550 × 15 × 20	H 480 × 480 × 10 × 15	H 340 × 340 × 15 × 20
1:0.3	Conventional	H 400 × 400 × 15 × 20	H 550 × 550 × 15 × 20	H 450 × 450 × 10 × 15	H 220 × 220 × 5.0 × 9.0
	Energy-based	H 400 × 400 × 15 × 20	H 550 × 550 × 15 × 20	H 450 × 450 × 10 × 15	H 390 × 390 × 5.0 × 10



(a) Span ratio 1: 0.6 (b) Span ratio 1:0.3

Fig. 10 Vertical deflection in model structures with different span length before and after redesign (5% damping, 2% post-yield stiffness ratio)

5. Conclusions

In this study a simple plastic design method was applied for design of steel moment-resisting structures to prevent progressive collapse. The plastic moment of beams required to stabilize a structure subjected to sudden loss of a column was computed from the equilibrium condition of the external work done by gravity load and the internal work done by plastic rotation of beams.

It was shown that the increase of only the girder size for the purpose of preventing progressive collapse may result in weak story when the structure is subjected to seismic load. The formation of weak story can be prevented by increasing the column size in such a way that the strong column-weak beam requirement is satisfied. The nonlinear dynamic analyses results showed that the structures designed without considering progressive collapse did not satisfy the failure criterion required by the GSA guidelines; on the other hand, the structures redesigned by plastic design method to prevent progressive collapse turned out to satisfy the given failure criterion in most of the model structures. However it should be mentioned that, as the catenary action which arises after the flexural resistance of a beam is lost is not considered in this study, the analysis and design results may be in conservative side. Further research is still required to evaluate the rotational and axial capacity of individual members and various beam-column connection types for more accurate prediction of progressive collapse potential of structures.

Acknowledgement

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References

ACI 318 (2002), “Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (ACI 318R-02)”, American Concrete Institute, Farmington Hills, Michigan

AISC (2005), “Seismic provisions for structural steel buildings”, American Institute of Steel Construction, Chicago, Illinois

ASCE7-05 (2005), “Minimum design loads for buildings and other structures”, American Society of Civil Engineers, New York

Chopra, A. K. (2001), *Dynamics of Structures*, 2nd edition, Prentice Hall.

Corley, W. G., Mlakar Sr., P. F., Sozen, M. A., and Thornton, C. H. (1998), “The Oklahoma city bombing: summary and recommendations for multihazard mitigation”, *J. Performance Constr Facilities*, **12**(3), 100-112.

Crawford, J. E. (2002), “Retrofit methods to mitigate progressive collapse, the multihazard mitigation council of the national institute of building sciences”, *Report on the National Workshop and Recommendations for Future Effort*.

Dusenberry, D. O. and Hamburger, R. O. (2006) “Practical means for energy-based analyses of disproportionate collapse potential”, *ASCE J Performance of Constr. Facilities*, **20**(4), 336-348.

Eurocode 1 (2002), “Actions on structures”, European Committee for Standardization, Brussels

FEMA (1997), “NEHRP Guidelines for the Seismic Rehabilitation of Buildings”, FEMA-273, Federal Emergency Management Agency, Washington, D.C.

FEMA (2006), “Prestandard and commentary for the seismic rehabilitation of buildings”, FEMA-356, Federal Emergency Management Agency, Washington, D.C.

GSA (2003), “Progressive collapse analysis and design guidelines for new federal office buildings and major modernization projects", The U.S. General Services Administration

Hayes Jr., J. R, Woodson, S. C., Pekelnicky, R. G., Poland, C. D., Corley, W. G., Sozen, M. (2005), "Can strengthening for earthquake improve blast and progressive collapse resistance?", *ASCE J. Struct. Eng.*, **131**(8), 1157-1177.

ICC (2006), "International Building Code," International Code Council, Falls Church, Virginia

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

Longinow, A. and Mniszewski, K. R. (1996), "Protecting buildings against vehicle bomb attacks", *Practice Periodical on Structural Design and Construction*, **1**(1), 51-54.

Marjanishvili, S. M. (2004), "Progressive analysis procedure for progressive collapse", *J. Performance Constr. Facilities*, 79-85.

Mazzoni, S., McKenna, F., Scott, M. H., and Fenves, G. L (2006), "Open system for earthquake engineering simulation", User Command-Language Manual, Pacific Earthquake Engineering Research Center, Berkeley, California.

Moy, S. S. J. (1981), *Plastic Methods for Steel and Concrete Structures*, The Macmillan Press, LTD

Murakami, Y, Fushimi, M., and Suzuki, H. (2004) "Thermal deformation analysis of high-rise steel buildings" *Proceedings of the CTBUH Seoul International Conference on Tall Buildings*, Oct. Seoul, Korea.

National Building Code of Canada (1995), National Research Council of Canada, Ottawa, Canada

National Institute of Standard and Technology (2006), *Best Practices for Reducing the Potential for Progressive Collapse in Buildings (Draft),*

Neuenhofer A. and Filippou F.C. (1997), "Evaluation of nonlinear frame finite elements", *ASCE J. Struct Eng*, **123**(7), July, 958-966.

Unified Facilities Criteria (UFC)-DoD (2005), "Design of buildings to resist progressive collapse", Department of Defense, USA

Suzuki, I., Wada, A., Ohi, K., Sakumoto, Y., Fusimi, M. and Kamura, H. (2003), "Study on high-rise steel building structure that excels in redundancy, Part II evaluation of redundancy considering heat induced by fire and loss of vertical load resistant members", *Proc. CIB-CTBUH International Conf. on Tall Buildings*, 251-259

Wada, A., Ohi, K., Suzuki, H., Sakumoto, Y., Fushimi, M., Kamura, H., Murakami, Y., and Sasaki, M. (2004) "A study on the collapse control design method for high-rise steel buildings", *Proceedings of the CTBUH Seoul International Conference on Tall Buildings*, Oct., Seoul, Korea.

Weaver, W. and Gere, J. M. (1990), *Matrix Analysis of Framed Structures*, 3rd edition, Van Nostrand Reinhold, New York, NY.

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