International Journal of High-Rise Buildings
September 2012, Vol 1, No 3, 229-235

International Journal of
High-Rise Buildings
www.ctbuh.org

Evaluation of Progressive Collapse Resisting Capacity of Tall Buildings

Kwangho Kwon, Seromi Park, and Jinkoo Kim†

Department of Architectural Engineering, Sungkyunkwan University, Suwon 440-746, Korea

Abstract

In this paper the progressive collapse potential of building structures designed for real construction projects were evaluated based on arbitrary column removal scenario using various alternate path methods specified in the GSA guidelines. The analysis model structures are a 22-story reinforced concrete moment frames with core wall building and a 44-story interior concrete core and exterior steel diagrid structure. The progressive collapse resisting capacities of the model structures were evaluated using the linear static, nonlinear static, and nonlinear dynamic analyses. The linear static analysis results showed that progressive collapse occurred in the 22-story model structure when an interior column was removed. However the structure turned out to be safe according to the nonlinear static and dynamic analyses. Similar results were observed in the 44-story diagrid structure. Based on the analysis results, it was concluded that, compared with nonlinear analysis procedures, the linear static method is conservative in the prediction of progressive collapse resisting capacity of building structure based on arbitrary column removal scenario.

Keywords: Tall buildings, Progressive collapse, Alternate load path method, Moment frames, Diagrid structures

1. Introduction

The progressive collapse refers to the phenomenon that local damage of structural elements results in global collapse of the structure. The load which may trigger progressive collapse includes any loading condition that is not considered in normal course of design but may cause significant damage to structures such as gas explosion, bomb blast, car impact, etc. To prevent the progressive collapse caused by abnormal loads, 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, 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 path method, in which the structure is designed in such a way that alternate paths are 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.

Research effort on progressive collapse of structures has been carried out on steel structures (Khandelwal and El-Tawil, 2008; Kim and Park, 2008) and on reinforced concrete structures (Sasani and Kropelnicki, 2008; Tsai and Lin, 2008; Kim and Choi, 2011). However most researches on progressive collapse have been conducted on simplified model structures such as two dimensional moment frames or beam-column subassemblages. In this paper the progressive collapse potential of building structures designed for real construction projects were evaluated using various alternate path methods specified in the GSA guidelines. The analysis model structures are a 24-story reinforced concrete building and a 44-story interior concrete core and exterior steel diagrid structure.

2. Analysis Procedure for Progressive Collapse

The GSA guidelines recommend both static and dynamic analyses for evaluation of progressive collapse resisting capacity of building structures. For static analysis the GSA guidelines use dynamic amplification factor of 2.0 in load combination as shown in Fig. 1. For linear static analysis the GSA guidelines utilize 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:

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

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

†Corresponding author: Jinkoo Kim
Tel: +82-31-290-7563; Fax: +82-31-290-7570
E-mail: jkim12@skku.edu

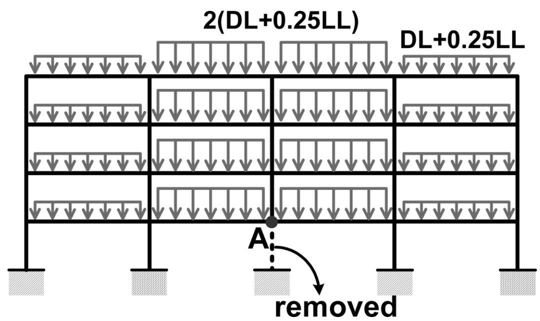


Figure 1. Applied gravity load for static analysis.

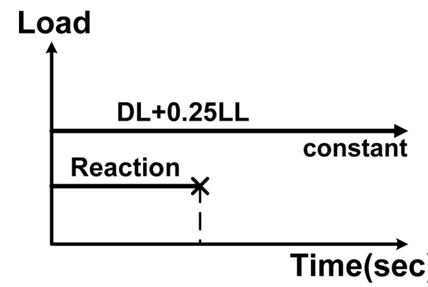


Figure 2. Time history of applied loads for dynamic analysis.

is the expected ultimate, un-factored capacity of the component (moment, axial force, shear etc.). In the GSA guidelines 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.

The step-by-step procedure to conduct the linear static 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. 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).

Step 4: The Steps 1~4 are repeated until DCR of any member does not exceed the limit state. If moments have been re-distributed 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.

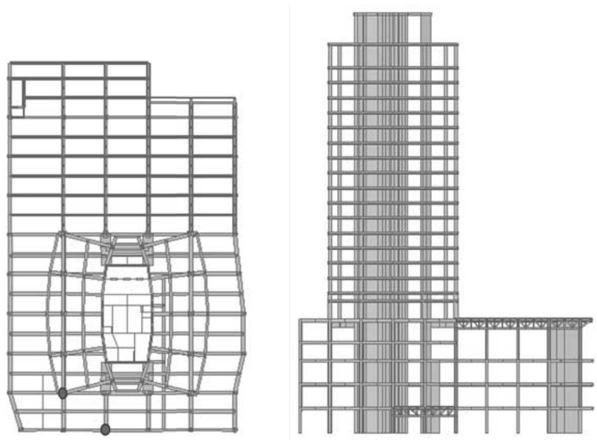
The nonlinear analysis procedures are generally more sophisticated than linear procedures in characterizing the performance of a structure. When such procedures are used, the guidelines generally permit less restrictive acceptance criteria recognizing the improved results that can be obtained from such procedures. The GSA guidelines specify maximum plastic hinge rotation and ductility as acceptance criteria for progressive collapse. 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.

The nonlinear static analysis procedure can 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 carried out by gradually increasing the vertical displacement in the location of the removed column to investigate the resistance of the structure against such deformation.

To evaluate the progressive collapse potential of structures using nonlinear dynamic analysis, all member forces are first obtained from the full structural model subjected to the applied load ($DL+0.25LL$). The structure is then re-modeled with a column removed and its member forces applied to the structure as point reaction forces. The reaction forces are suddenly removed after a few seconds to initiate progressive collapse as shown in Fig. 2. If the vertical displacement keeps increasing unbounded, then the structure is considered to have high potential for progressive collapse.

3. Design of Model Structures

Two building structures, designed for real projects, were analyzed to evaluate progressive collapse potential. The first model structure is a 95.2 m-high 22-story reinforced concrete structure composed of central core walls which were designed to resist all lateral loads and moment frames designed for gravity loads. The 22-story central tower structure is surrounded by five-story podium structure. The span length is 8.4 m in the transverse direction,



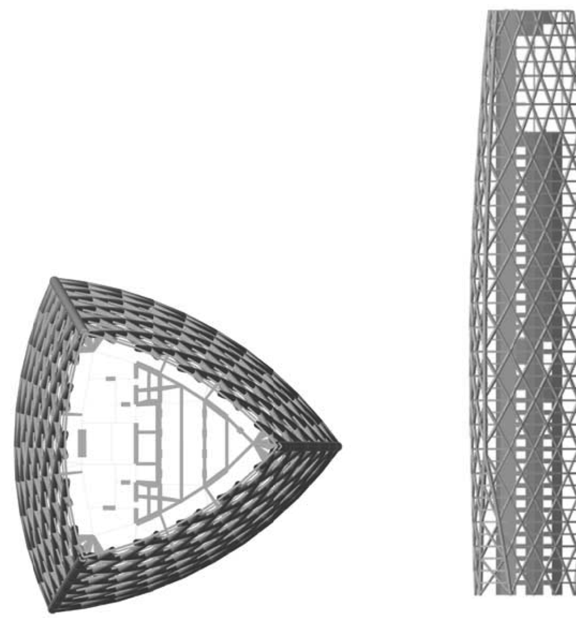
(a) Structural plan (b) Elevation

Figure 3. Twenty two-story reinforced concrete analysis model structure.

and is 7.8 m in the longitudinal direction. The concrete strength varies from 24 MPa to 30 MPa depending on the height, and two types of re-bars with ultimate strength of 400 MPa and 500 MPa were used. The wind load was obtained using the basic wind speed of 40 m/sec. and the seismic load was computed using the design spectral acceleration parameters S_{DS} and S_{D1} equal to 0.28 and 0.15, respectively, in the IBC format. The response modification factor of 5.0 was used in the calculation of the design base shear. Figure 3 shows the structural plan and side view of the 22-story structure. The locations of column removal are marked on the structural plan.

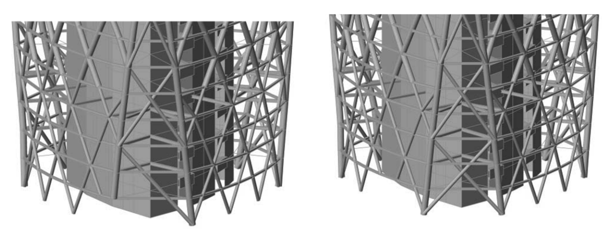
The second analysis model structure is the 201.6 m high 44-story building consisting of RC core walls and exterior steel diagrid structure. Both the core walls and diagrid structure were designed to resist the lateral loads. The span length varies from 9.3 m in the first story to 7 m at the top story. The basic design wind speed is 30 m/sec. and the design spectral acceleration parameters S_{DS} and S_{D1} are 0.37 and 0.15, respectively. As the response modification factor of diagrid structure is not specified in the design code, the seismic load was obtained using the *R* factor equal to 3.0. The diagrid members were designed to have the strength ratio of demand and capacity around 0.9. The nominal strength of concrete varies from 55 MPa in the lower stories to 30 MPa at the top, and the structural steel with ultimate strength of 490 MPa was used in the lower story diagrid structure and the 570 MPa steel was used in the top stories. The structural plan and side view of the structure is shown in Fig. 4, and the location of removed column is shown in Fig. 5.

For nonlinear analysis the beams and columns were modeled by beam elements and the braces were modeled by truss elements. The nonlinear force-deformation relationship of structural members is depicted in Fig. 6, which is suggested in the FEMA-356 (2000), where P_y is the yield strength, θ is the rotation angle, and Δ is the displacement. For braces the post-buckling strength (P'_{cr}) was determined to be 40% or 20% of the buckling strength (P_{cr}) depending on the width-thickness ratio. The parameters *a, b*, and *c* also vary depending on the width-thickness ratio of the structural members, and were determined based on the guidelines provided in the Tables 5-6 and 5-7 of the FEMA-356. The post-yield stiffness of 3% was used for modeling of bending members.



(a) Structural plan (b) Elevation

Figure 4. Forty four-story structure with RC core and steel diagrid.

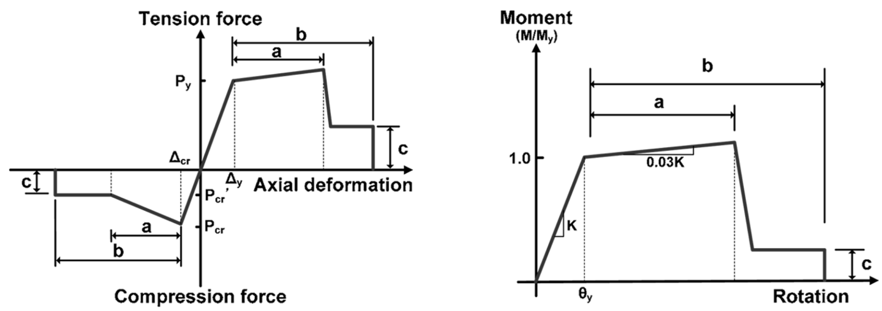


(a) Corner diagrid (b) Exterior diagrid

Figure 5. Locations of removed diagrid members in the 44-story structure.

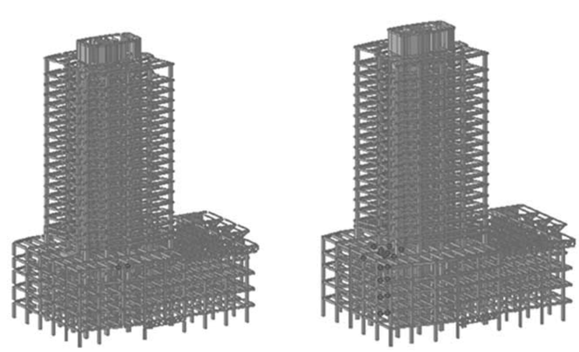
4. Analysis Results of the 22-Story Building

A series of step by step procedure recommended in the GSA guideline was applied to the 22-story model structure using the program code MIDAS (2006). Two columns were removed from the first story of the structure one at a time, and the member forces were computed to obtain the DCR values. Figure 7 depicts the locations of failed members obtained from linear static analysis, where the DCR values exceed the limit state specified in the guide-



(a) Braces (b) Bending members

Figure 6. Nonlinear force-deformation relationships of structural members used in the analysis.

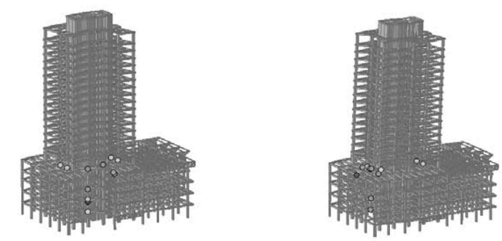


(a) Removal of an exterior column (b) Removal of an interior column

Figure 7. Locations of failed members obtained from linear static analysis.

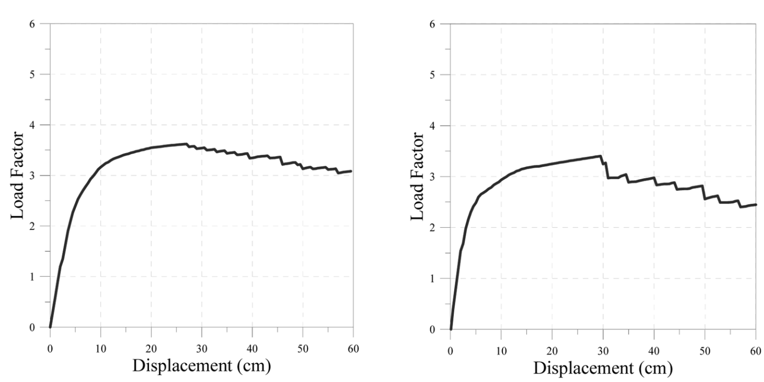
lines,. When one of the exterior columns was removed, DCR of two beams in the fifth story exceeded the limit state as shown in Fig. 7(a). This implies that the removal of the exterior column will not lead to progressive failure of the members located above the removed column. Fig. 7(b) shows the locations of the failed members when an interior column, which becomes an exterior column of the tower structure above the sixth story, was removed. It can be observed that the beams located above the removed column up to fifth story failed.

As the possibility of progressive collapse was observed by the linear analysis procedure, more rigorous method needs to be applied to confirm the progressive collapse potential of the structure. To this end nonlinear static and dynamic analyses were carried out to observe nonlinear behavior of the structure. Figure 8 shows the pushdown curves of the 22-story structure subjected to the loss of an exterior and an interior column. As in the case of linear static analysis, the load combination $2(DL + 0.25LL)$ was applied and the displacement-controlled pushdown analysis was carried out. It can be observed that the maximum strengths reached the load factor of 3.62 and 3.41, which implies that the structure has strength more than three times higher than the applied load and thus safe for progressive collapse caused by the removal of a column



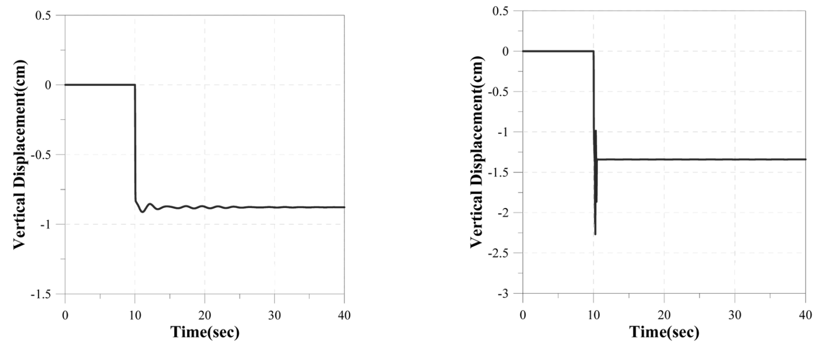
(a) Removal of an exterior column (b) Removal of an interior column

Figure 9. Plastic finge formation in the 22-story structure at failure.



(a) Removal of an exterior column (b) Removal of an interior column

Figure 8. Pushdown curves of the 22-story structure.



(a) Removal of an exterior column (b) Removal of an interior column

Figure 10. Vertical displacement time history of the 22-story structure obtained by nonlinear dynamic analysis.

in the first story. Figure 9 depicts the plastic hinge formation in the structure at the maximum strength, where it was observed that even though some members were damaged, total collapse mechanism did not occur.

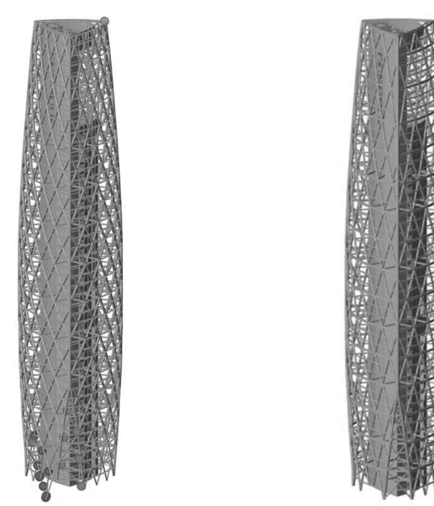
Figure 10 shows the vertical displacement of the joint from which a column was removed obtained by nonlinear time-history analysis. In the dynamic analysis the load combination ($DL + 0.25LL$) was applied without the dynamic amplification factor of 2.0. It was observed that when the exterior column was suddenly removed the structure vibrated vertically and the vertical displacement became stable at the displacement of 0.87 cm, which is significantly smaller than the limit state specified in the GSA guidelines. In case the interior column was removed, the vertical displacement reached 1.34 cm and the structure remained stable.

The comparison of linear and nonlinear analysis results indicates that the linear static analysis procedure resulted in highly conservative prediction on the progressive collapse potential of the model structure.

5. Analysis Results of the 44-Story Building

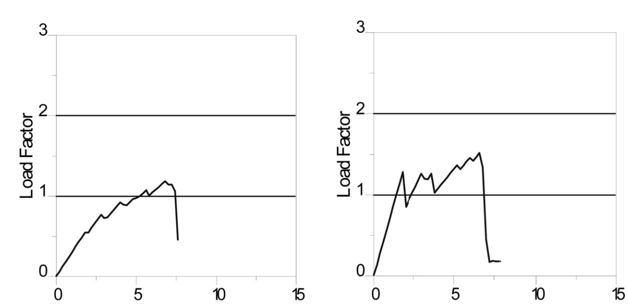
In this section the 44-story diagrid structure with RC core was analyzed to investigate its progressive collapse potential. Figure 11 shows the locations of the failed members obtained from linear static analysis with a couple of diagrid removed from the first story. When a couple of diagrid were removed from a corner, the DCR values of many structural elements exceeded the limit states as shown in Fig. 11(a). However no element turned out to be failed when a couple of exterior diagrid were removed from the first story, which implies that alternate load path is provided after the loss of the members.

Figures 12(a) and 12(b) show the pushdown curves of the 44-story diagrid structure when a couple of diagrid were removed from the corner and from an exterior face of the structure, respectively. When the diagrids were removed from the corner, the maximum load factor reached 1.4, which implies that the structure has enough strength against progressive collapse. When the diagrids were removed from the exterior face, the maximum strength and thus the progressive collapse resisting capacity increased slightly. Figure 13 depicts the locations of members damaged by the column removal at the state of maximum strength obtained by pushdown analysis. It can be observed that the damaged members are more widely distributed when diagrids are removed from the exterior



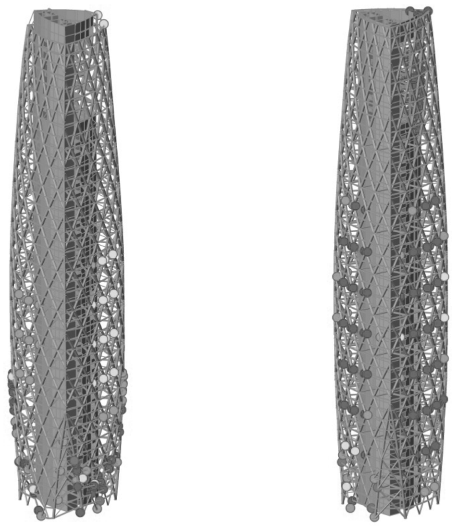
(a) Removal of corner diagrids (b) Removal of exterior diagrids

Figure 11. Failed members in the 44-story structure obtained by linear static analysis.



(a) Removal of corner diagrids (b) Removal of exterior diagrid

Figure 12. Pushdown curves of the 44-story structure.



(a) Removal of corner diagrids (b) Removal of exterior diagrids

Figure 13. Damaged members in the 44-story structure obtained from pushdown analysis.

face. It also can be noticed that as the diagrids surround the building surfaces, the damaged members propagate not only on the surface from which the diagrids are removed, but on the other surfaces around the structure. This implies that diverse alternate load paths may exist after a member is lost.

Figure 14 depicts the time histories of vertical displacements obtained by nonlinear dynamic analysis. As expected from the results of nonlinear static pushdown analysis, the vertical displacements remain less than 2 cm after a couple of diagrids were removed from the first story. It was observed that the structure behaved elastically after the diarid members were suddenly removed.

6. Conclusions

This paper investigates the progressive collapse potential of building structures designed for real construction projects based on arbitrary column removal scenario using various alternate path methods specified in the GSA guidelines. The analysis model structures are a 22-story reinforced concrete moment frames with core wall building and a 44-story interior concrete core and exterior steel diagrid structure. The progressive collapse resisting capacities of the model structures were evaluated using the linear static, nonlinear static, and nonlinear dynamic analyses.

The linear static analysis results showed that progressive collapse occurred in the 22-story model structure when an interior column was removed. However the structure turned out to have enough strength against progressive collapse according to the nonlinear static and dynamic analyses. Similar results were observed in the 44-story diagrid structure, where the damaged members were more widely distributed when diagrids were removed from the exterior face. It was also noticed that the damaged members existes not only on the surface from which the diagrids were removed, but on the other surfaces around the structure. This implies that diverse alternate load paths may exist after a member is lost. Based on the analysis results, it was concluded that, compared with nonlinear analysis procedures, the linear static method is conservative in the prediction of progressive collapse resisting capacity of building structure based on arbitrary column removal scenario.

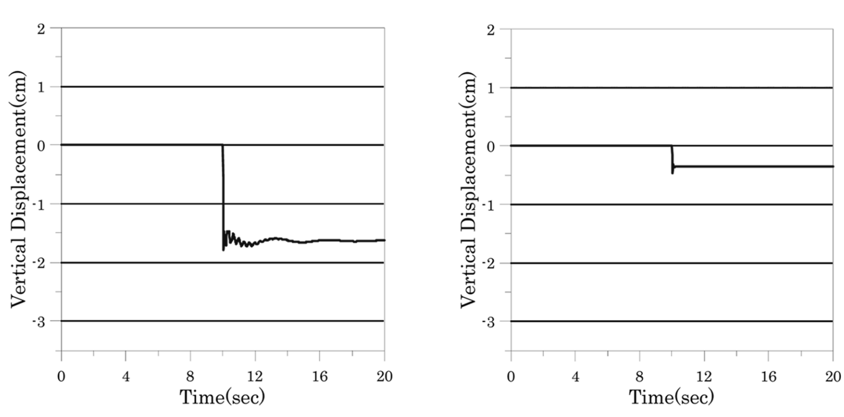
Acknowledgements

This research was supported by a grant (Code# '09 R&D A01) funded by the Ministry of Land, Transport, and Maritime Affairs of Korean government.

References

AISC. (2000) Load and resistance factor design specification for structural steel buildings, American Institute of Steel Construction, Chicago.

DoD. (2005) Unified Facilities Criteria (UFC) - Design of



(a) Removal of corner diagrids (b) Removal of exterior diagrids

Figure 14. Vertical displacement time history of the 44-story structure obtained by nonlinear dynamic analysis.

Buildings to Resist Progressive collapse, Department of Defense, Washington, D.C.

Eurocode 1. (2002) Actions on structures, European committee for standardization. Brussels, Belgium.

FEMA. (2000) 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.

Kaewkulchai, G. (2003) Dynamic progressive collapse of frame structures, Ph. D. Thesis. University of Texas at Austin.

Khandelwal, K. and El-Tawil, S. (2007) Collapse behavior of steel special moment resisting frame connections. *Journal of Structural Engineering*, 133(5), pp. 646~655.

Kim, J. and Park, J. (2008) Design of steel moment frames considering Progressive collapse, *Steel and Composite Structures*, 8(1), pp. 85~98.

Kim, J. and Choi, H. (2011) Progressive collapse-resisting capacity of RC beam-column sub-assemblage, *Magazine of Concrete Research*, 63(4), pp. 297~310.

MIDAS. (2006) General Structure Design System for Windows.

Sasani, M. and Kropelnicki, J. (2008) Progressive collapse analysis of an RC structure, *The Structural Design of Tall and Special Buildings*, 17(4), pp. 757~771.

Tsai, M. H. and Lin, B. H. (2008) Investigation of progressive collapse resistance and inelastic response for an earthquake-resistant RC building subjected to column failure, *Engineering Structures*, 30(12), pp. 3619~3628.