THE STRUCTURAL DESIGN OF TALL AND SPECIAL BUILDINGS
Struct. Design Tall Spec. Build. **22**, 1359–1375 (2013)
Published online 24 February 2012 in Wiley Online Library (wileyonlinelibrary.com/journal/tal). DOI: 10.1002/tal.1010

Progressive collapse resisting capacity of tilted building structures

Jinkoo Kim[1*,†] and Min-Kang Jung[2]

[1]*Sungkyunkwan University, Suwon, Korea*
[2]*Samsung Engineering and Construction, Ltd., Seoul, Korea*

SUMMARY

In this study, the progressive collapse resisting capacities of tilted buildings are evaluated on the basis of arbitrary column removal scenario. As analysis model structures both regular and tilted moment-resisting frames, structures with outrigger trusses, and tubular/diagrid structures are designed, their progressive collapse resisting capacities are evaluated by nonlinear static and dynamic analyses. It turns out that the tilting of the structures requires increased steel tonnage due to the increased *p*-delta effect. In addition in the tilted structures the plastic hinges are more widely distributed throughout the bays and stories when a column is removed from a side or a corner of the structures. With the analysis results, it is concluded that the tilted building structures, once they are properly designed to satisfy a given design code, may have at least an equivalent resisting capacity for progressive collapse caused by sudden loss of a column. Copyright © 2012 John Wiley & Sons, Ltd.

Received 16 May 2011; Revised 10 December 2011; Accepted 15 January 2012

KEY WORDS: progressive collapse; tilted structures; outrigger trusses; tubular structures; diagrid structures

1. INTRODUCTION

Recently the geometric complexity and irregularity of building structures have been rapidly increasing. Al-Ali and Krawinkler (1998) investigated the seismic behavior of building structures with vertical irregularities and found that the seismic response of building structures is more sensitive to stiffness and strength irregularities than to mass irregularities. Scott *et al*. (2007) explored the structural challenges that are created by buildings with unique geometries or articulated forms and discussed some economic design and construction techniques. Sarkar *et al*. (2010) proposed a new method of quantifying irregularity in building frames with vertical geometric irregularity accounting for dynamic characteristics and provided a modified empirical formula for estimating fundamental period. Kim and Hong (2011) estimated the progressive collapse potential of tilted/twisted irregular buildings, where it was observed that the performance of irregular buildings subjected to sudden loss of a column depends significantly on the location of the removed column. Vollers (2008) proposed a morphological scheme that enables data to be retrieved on sustainable performance of building shapes. He categorized the geometry of high-rise buildings into *Extruders*, *Rotors*, *Twisters*, *Tordos*, *Transformers* and *Free Shapers* depending on their form-generation method. *Extruders* are buildings with basically the same floor plan over the entire height. Among the *Extruders* category, an *Ortho* and a *Cylinder* are regular extruders, having an orthogonal and circular plan, respectively. *Anglers* are buildings with a repetition of floors, piled on top of each other under a fixed inclination. The floors can have straight or curving contours. When identical floors are stacked under a varying angle, the buildings are called *sliders*.

A progressive collapse involves a series of failures that lead to partial or total collapse of a structure. The progressive collapse resisting capacity of a building depends on the capability of the force redistribution including various factors such as redundancy, ductility and configuration. For structural design of structures against progressive collapse, Starossek (2006) indicated the shortcomings of the

*Correspondence to: Jinkoo Kim, Sungkyunkwan University, Suwon, Republic of Korea.
†E-mail: jkim12@skku.edu

current design process and mentioned that the shortcomings can be overcome within the framework of reliability theory. Izzuddin *et al*. (2008) proposed a simplified framework for progressive collapse assessment of multi-story buildings. Alashker and El-Tawil (2011) proposed a design-oriented model for computing the load-resisting capacity of composite floors subjected to column loss. Recently, a series of research was conducted to investigate the performance of building structures designed with various structure systems. Kim and Lee (2010) investigated the progressive collapse potential of tube-type structures, and Almusallam *et al*. (2010) evaluated the progressive collapse potential of a framed concrete buildings subjected to blast loads. Kim *et al*. (2011) evaluated the progressive collapse resisting capacity of braced frames subjected to sudden loss of a column, and Kim and Hong (2011) evaluated the progressive collapse performance of irregular buildings based on the arbitrary column-loss scenario.

In this paper, the progressive collapse resisting capacity of the *Ortho* and *Anglers* type-tilted buildings in the *Extruders* category was evaluated. To this end, buildings with different design parameters, such as tilting angles, number of story and structure systems, were designed and were analyzed by nonlinear static and dynamic analyses. The analysis results of the tilted structures were compared with those of the regular vertical structures, and the performances of the tilted structures designed with different structure systems were also compared.

2. DESIGN OF ANALYSIS MODEL STRUCTURES

To evaluate the progressive collapse resisting capacity of tilted structures, the following analysis model structures were prepared: 7-story and 14-story steel moment-resisting frames, 36-story steel structures with outrigger trusses, and 36-story steel tubular and diagrid structures. The model structures were designed per the Korea Building Code (KBC-2009). Girders were designed with wide flange sections with yield stress of 235 MPa, braces were designed with hollow steel section with yield stress of 235 MPa, and columns were designed with hollow steel section with yield stress of 325 MPa. The design dead and live loads are 5.0 and 2.5 kN/m^2, respectively. The design wind load is computed on the basis of the basic wind speed of 30 m/s in the exposure *A* area. The design seismic load is obtained using the seismic coefficients S_{DS} and S_{D1} equal to 0.44 and 0.23, respectively, in the International Building Code (IBC 2009) format. The moment-resisting frames were designed as steel intermediate moment frames with response modification factor of 4.5, and the structures with outrigger trusses and the tubular/diagrid structures were designed with response modification factor of 3.0.

Figure 1 shows the structural plan of the moment-resisting framed buildings, and the two dimensional frame enclosed within the dotted rectangle was separated as an analysis model. Figure 2 shows the structural elevations of the 14-story regular ($\theta = 0°$) and tilted ($\theta = 13.4°$) moment frames. The tilted structures were designed into two different types depending on the inclination of the interior columns; tilted structures with tilted interior columns (Figure 2(b)) and vertical interior columns (Figure 2(c)).

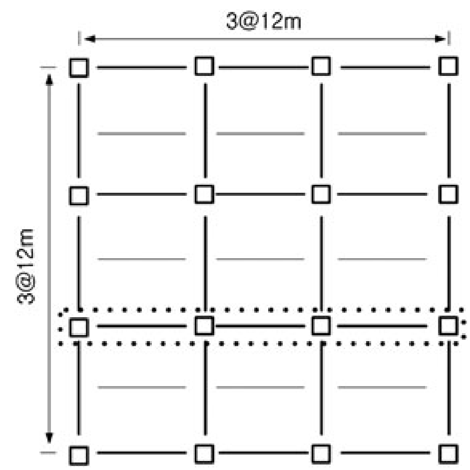
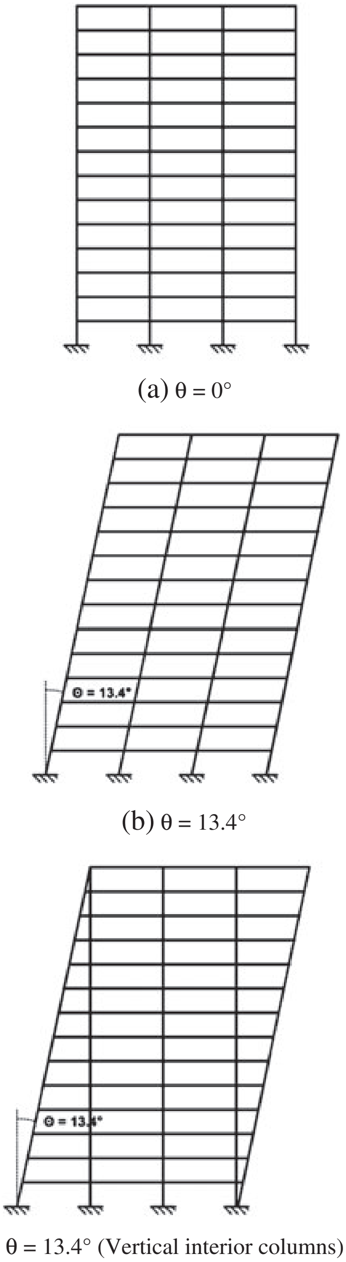


Figure 1. Structural plan of model structures.

DOI: 10.1002/tal



(a) θ = 0°

(b) θ = 13.4°

(c) θ = 13.4° (Vertical interior columns)

Figure 2. Structural elevation of 14-story moment frames.

The steel tonnages of the designed structures are shown in Table 1. In the tilted structures, the weights of the structures with tilted interior columns were presented in the table. It can be observed that as the tilting angle increases the steel tonnage required to satisfy the design code increases significantly. The increase in steel tonnage is more noticeable when the tilting angle increases from 5° to 13.4° than from 0° to 5°. Table 2 shows the steel weight of the structures with tilting angle of 13.4°. It can be noticed that the steel tonnages of the structures with inclined interior columns is much larger than those of the structures with vertical interior columns.

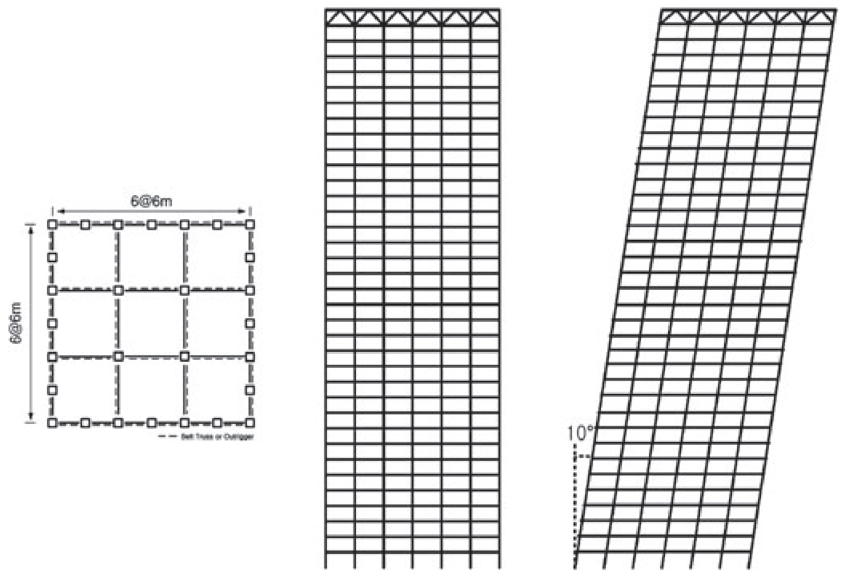
Figure 3 shows the plan shape and elevation of the 36-story steel buildings with outrigger and belt trusses at the top stories. Both regular ($\theta = 0$) and tilted ($\theta = 13.4°$) structures were designed for comparison. The structures were designed in such a way that all lateral loads were resisted by exterior moment frames combined with outrigger/belt trusses. The story height is 3.6 m, and the exterior columns are spaced in the interval of 6 m. Both regular and 10° tilted structures were prepared for comparison.

Table 1. Weight of the moment frame model structures (kN).

Slope	Total structure		2D frame	
	7 stories	14 stories	7 stories	14 stories
0°	3960	9372	802	1915
5°	4889	12 850	1134	3473
13.4°	6978	19 170	2032	6554

Table 2. Weight of the structures with slope of 13.4° (kN).

Structures	Members	7 stories	14 stories
Structure with tilted int. columns	Girders	4107	11 532
	Columns	2871	7638
	Total	6978	19 170
Structure with vertical int. columns	Girders	2650	6208
	Columns	1275	4012
	Total	3925	10 220



(a) Plan (b) Elevation (θ = 0) (c) Elevation (θ = 10)

Figure 3. Thirty-six-story analysis model structures with outrigger and belt trusses.

Table 3 shows the member sizes of the structures with outrigger trusses. The exterior columns and girders were designed with steel box columns. It was observed that the steel tonnage of the tilted structure was 67% higher than that of the regular structure.

Figure 4 depicts the 36-story regular and tilted framed tube and diagrid structures. Diagrid structure system is a particular form of space truss mixed with tubular system, and the diagonal grid makes the structure stable even without any vertical column in the perimeter of the building. It has been shown that, if properly designed, diagrid systems perform better than framed tube structures in shear lag and lateral deflection (Lonard, 2007). The plan of the model structures is 36 m × 36 m square shape, and the exterior tube or diagrid structures were designed to resist all the lateral loads. The exterior column spacing of the tubular structure is 3 m, and the diagrid bracing is spaced at 6 m. According to

DOI: 10.1002/tal

Table 3. Member sizes of the structures with outrigger trusses (mm).

Slope	Stories	Exterior Columns	Exterior Girders
0°	Upper stories	560 × 560 × 20	434 × 299 × 10 × 15
	Mid stories	980 × 980 × 29	950 × 340 × 19 × 35
	Lower stories	1150 × 1150 × 35	890 × 299 × 15 × 23
10°	Upper stories	380 × 380 × 13	500 × 200 × 10 × 16
	Mid stories	1200 × 1200 × 31	1400 × 450 × 29 × 48
	Lower stories	1650 × 1650 × 42	1200 × 400 × 28 × 43

previous research (Moon 2007), diagrid structures are most effective when the diagrid members are sloped 65°–75°. In this study, the slope of the diagrid bracing was determined to be 67.4°. To evaluate the performance of tilted structures, 5° and 10° tilted tube-type structures were designed in addition to the regular structures. Table 4 shows the steel tonnage of the model structures, where it can be observed that the weight of the structural steel increases as the inclination of the tube-type structures increases. It turns out that the increase in steel tonnage is more significant in the framed tube structures.

3. PERFORMANCE EVALUATION OF MODEL STRUCTURES

3.1. *Analysis method for progressive collapse*

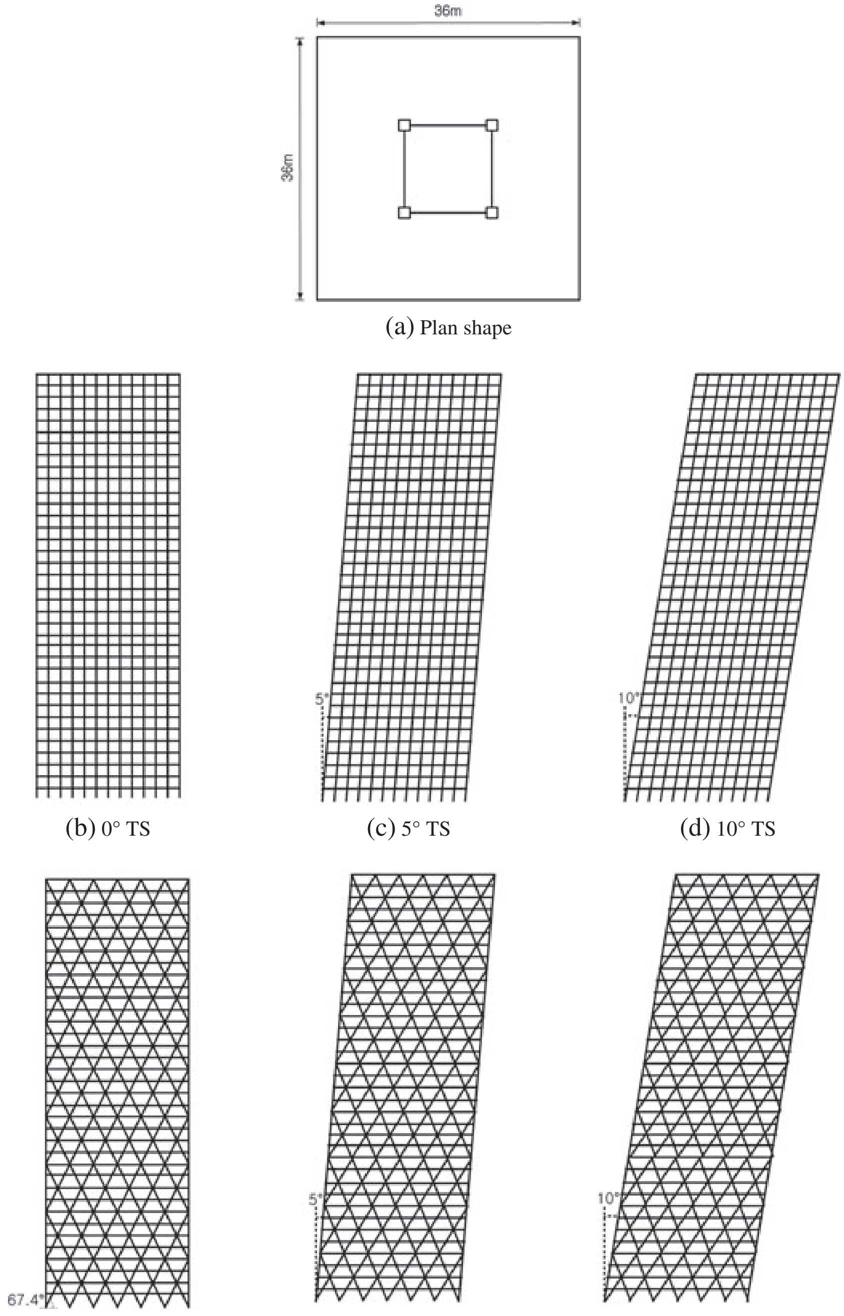
The progressive collapse performance of the analysis model structures was investigated on the basis of the arbitrary column-loss scenario. The finite element program code SAP-2000 (2004) was used for nonlinear static pushdown analysis and dynamic analysis. The pushdown analysis is generally applied not to determine whether the structure will fail or not but to evaluate the residual strength of the structure after a column is removed. For static analysis both the GSA 2003 and the DoD 2005 recommend the dynamic amplification factor of 2.0 in the applied load to account for dynamic redistribution of forces as shown in Figure 5(a). The load combination of the GSA 2003 for static analysis is 2(dead load + 0.25 × live load). In the dynamic analysis, no amplification factor is applied as shown in Figure 5(b). In order to carry out dynamic analysis, the member forces of a column, which is to be removed to initiate progressive collapse, are computed before it is removed. Then, the column is replaced by the point loads equivalent of its member forces as shown in Figure 5(b). To simulate the phenomenon that the column is removed by impact or blast, the column member forces are suddenly removed after elapse of a certain time while the gravity load remains unchanged as shown in Figure 6. In this study, the member reaction forces are increased linearly for 10 s until they reach the specified level, are kept unchanged for five seconds until the system reaches stable condition and are suddenly removed at 15 s to initiate progressive collapse.

For nonlinear analysis of bending members, the skeleton curve provided in FEMA-356 (2000) and shown in Figure 7(a) was used. The parameters a, b and c vary depending on the width–thickness ratio of the structural members and were determined based on the guidelines provided in Tables 5-6 and 5-7 of FEMA-356. The post-yield stiffness of 3% was generally used for modeling of bending members. For nonlinear analysis of truss and bracing members, the generalized load–deformation curves recommended in the FEMA-274 (1997) and shown in Figure 7(b) was used, which is based on the phenomenological model proposed by Jain and Goel (1978).

3.2. *Moment frames*

The progressive collapse resisting capacities of the moment-resisting frames were evaluated by removing one of the first-story columns. Nonlinear static and dynamic analyses were carried out using the program code SAP 2000 (2004). Figure 8 shows the nonlinear static pushdown curves of the seven-story moment frame structures with their interior columns having the same slope with the exterior columns (I-type). The displacement-controlled pushdown analyses were carried out with the right-hand side corner column (fourth column) removed. The load factor of 1.0 corresponds to the loading state

DOI: 10.1002/tal



(a) Plan shape

(b) 0° TS (c) 5° TS (d) 10° TS

(e) 0° DS (f) 5° DS (g) 10° DS

Figure 4. Structural shapes of tubular (TS) and diagrid (DS) structures.

Table 4. Weight of the exterior frames of the diagrid and the tubular model structures.

Structures	Slope	Weight (MN)
Diagrid structures	0°	33.8
	5°	42.7
	10°	59.6
Tubular structures	0°	33.7
	5°	56.0
	10°	84.2

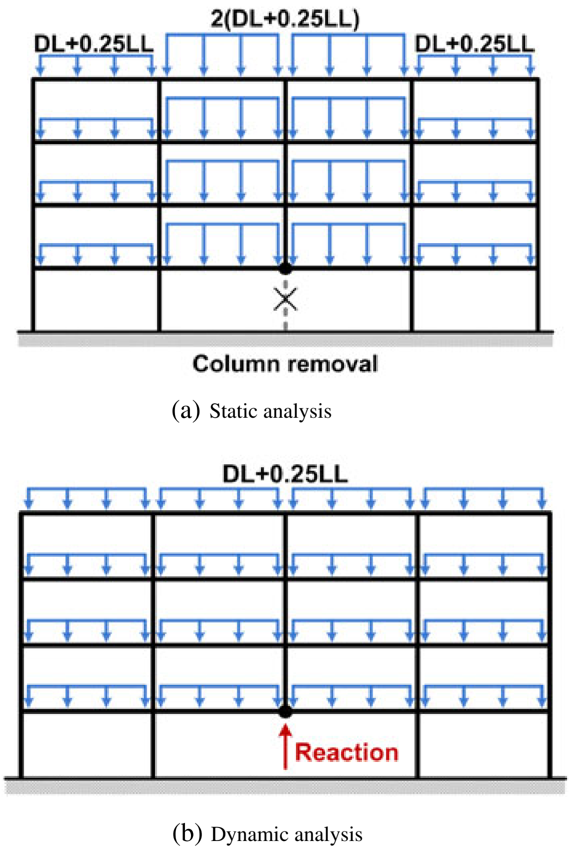


Figure 5. Applied load for dynamic analysis.

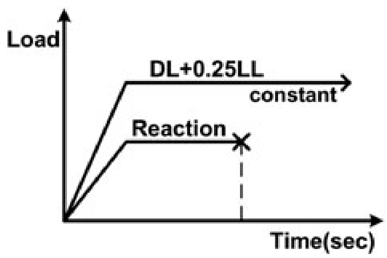


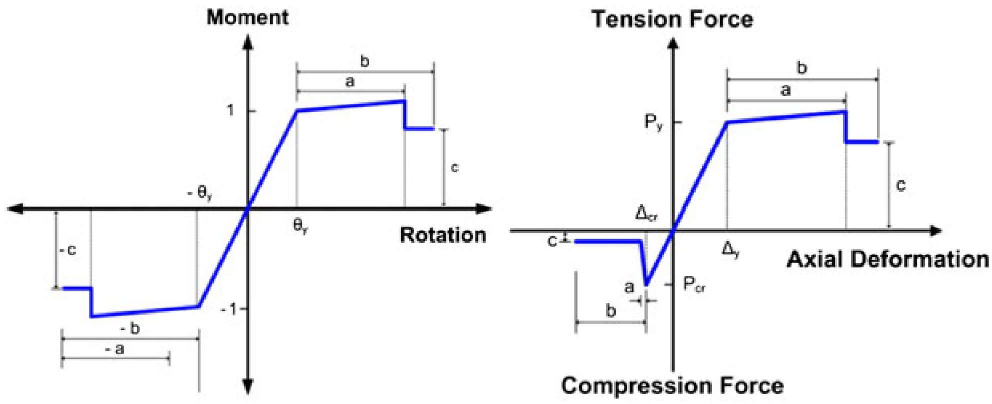
Figure 6. Time history of applied load for dynamic analysis.

specified in the GSA guidelines as shown in Figure 5(a). It can be observed that as the tilting angle of the building increases, the progressive collapse resisting capacity also increases. This can be explained by the significant increase in steel tonnage in the tilted structures as can be observed in Table 1. Even though the demand for member forces in the tilted structures is generally higher due to *p*-delta effect, the increase in strength due to increased member sizes exceeds the enhanced demand.

Figure 9 shows the pushdown analysis results of the 14-story I-type moment frames with tilted interior columns. It can be noticed that the overall maximum load factors increased compared with those of the seven-story I-type structure. The phenomenon was more noticeable as the tilting angle increased. The pushdown curves were obtained with one of the two corner columns removed. The maximum strength of the structure was larger when the right-hand side corner column (fourth column) was removed than when the first column was removed due mainly to the enhanced *p*-delta effect.

Figure 10 presents the pushdown curves of the 7-storey and 14-story 13.4° tilted moment-resisting framed structures with vertical interior columns (V-type). Figure 10(a, b) represents the analysis results obtained by removing the first and the fourth columns, respectively. It can be observed that when the first

DOI: 10.1002/tal



(a) Flexural members (b) Braces

Figure 7. Nonlinear force–displacement relationship of structural members.

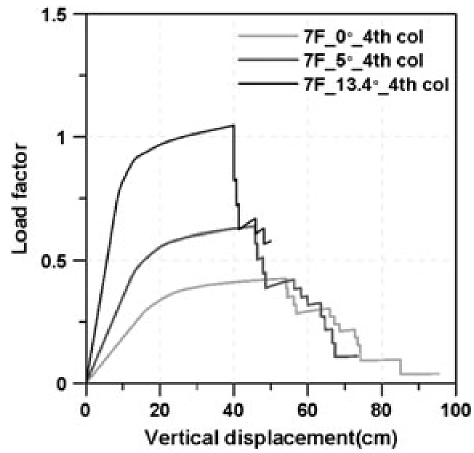
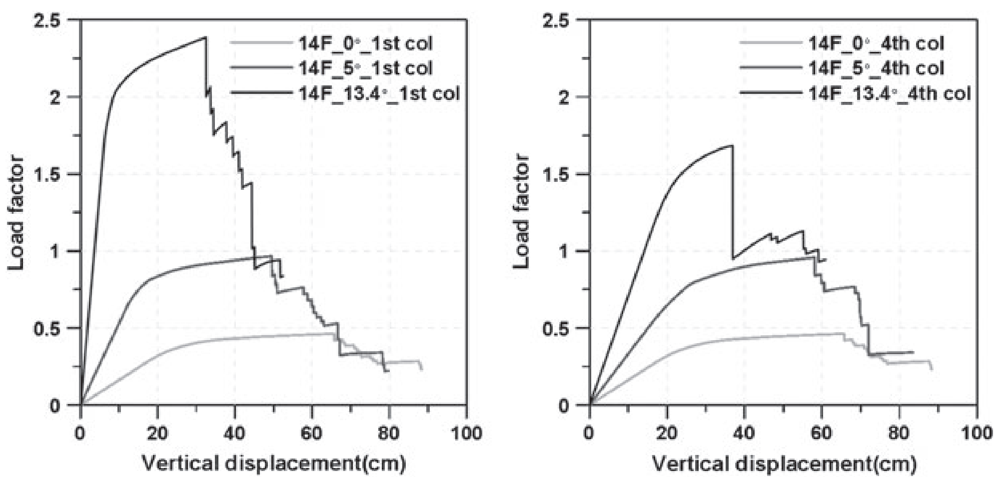


Figure 8. Pushdown curves of the seven-story moment frames (I-type).



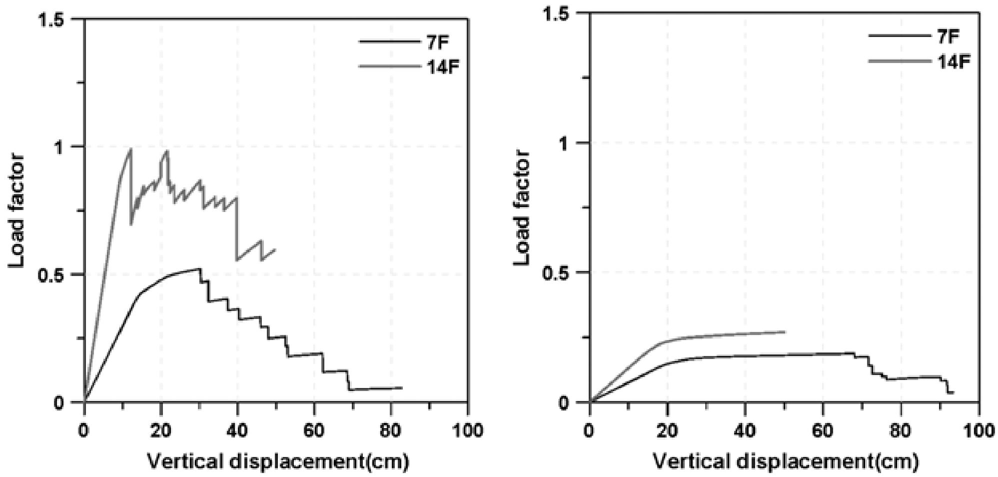
(a) Removal of the first column (b) Removal of the fourth column

Figure 9. Pushdown curves of the 14-story moment frames (I-type).

DOI: 10.1002/tal

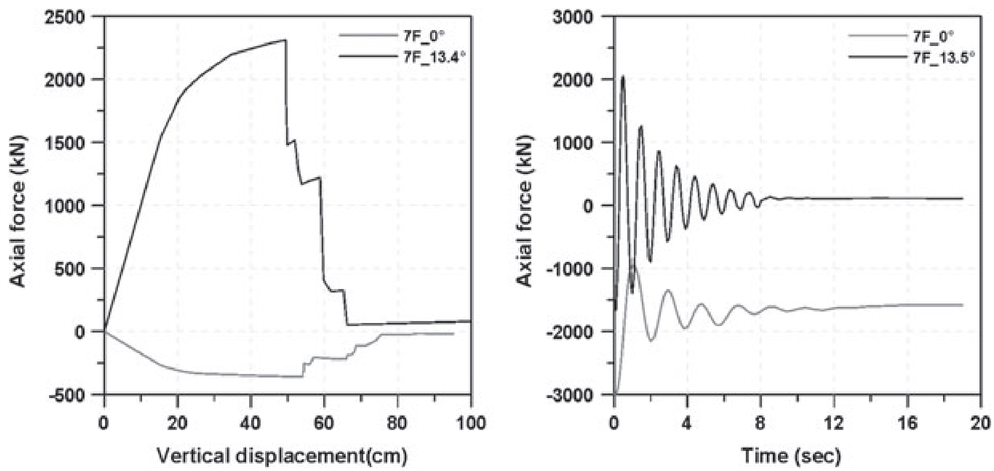
column was removed, the maximum strength of the 14-story structure reached 1.0, whereas that of the seven-story structure reached about 0.5. When the column in the opposite corner (fourth column) was removed, the progressive collapse resisting capacity was significantly reduced both in the 7-story and the 14-story structures. Comparison of Figures 8 and 9 shows that the capacities of structures with tilted interior columns are generally higher than those of the structures with vertical interior columns. This, however, does not imply that the structures with tilted interior columns have higher progressive collapse resisting capacity than that of the structures with vertical interior columns considering the steel tonnage required to satisfy the design codes. Table 2 shows that the weights of the structural steel in the structures with tilted interior columns are almost twice as high as those of the structures with vertical interior columns.

Figure 11 shows the variation of the axial force in the left-hand side corner column (first column) of the seven-story structures when the right-hand side corner column (fourth column) was removed. The variation of the column axial force obtained by pushdown analysis is shown in Figure 11(a). It can be observed that as the imposed vertical displacement increases, the first column of the regular structure is subjected to compression. However, the first column of the tilted structure is subjected to significant amount of tension when the corner column in the opposite side is removed. As the vertical



(a) Removal of the first column (b) Removal of the fourth column

Figure 10. Pushdown curves of the structures with vertical interior columns.



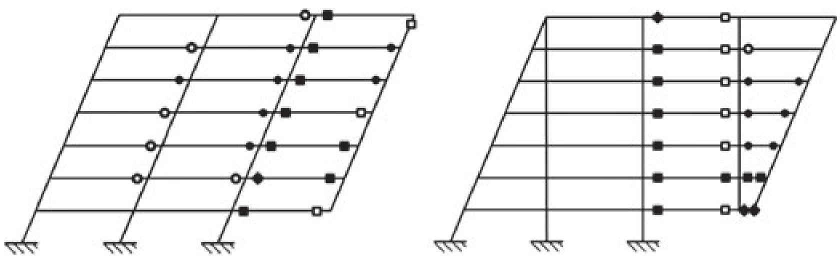
(a) Pushdown analysis (b) Dynamic analysis

Figure 11. Variation of the first column axial force when the fourth column is removed.

DOI: 10.1002/tal

displacement increases, the tensile force keeps increasing until the maximum tensile force of 2313 kN is reached. Figure 11(b) shows the variation of the axial force of the first column obtained by nonlinear dynamic analysis when the fourth column was suddenly removed. The results show that before the fourth column was removed, compression of 2993 kN was imposed on the first column of the regular structure. When the fourth column was removed, the axial force of the first column oscillated in the compression region and finally approached compression of 1586 kN. In the case of the tilted structure, the axial force of the first column was 1674 kN in compression before the fourth column was removed, and converged to 111 kN in tension after sudden removal of the fourth column.

Figure 12 illustrates the plastic hinge distribution in the seven-story tilted model structures when the first-story fourth columns are removed. The symbols representing the location of plastic hinges, such as empty circles and filled squares, also indicate the state of plastic deformation. Figure 13 shows the deformation levels corresponding to the immediate occupancy (IO), life safety (LS) and collapse prevention (CP) performance points as specified in FEMA-356. It can be observed that in the structure



(a) Tilted interior columns (b) Vertical interior columns

Figure 12. Plastic hinge distribution in the seven-story structure subjected to the loss of the fourth column.

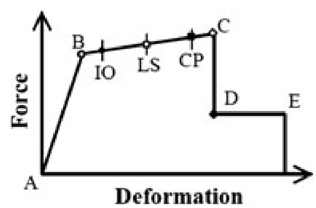
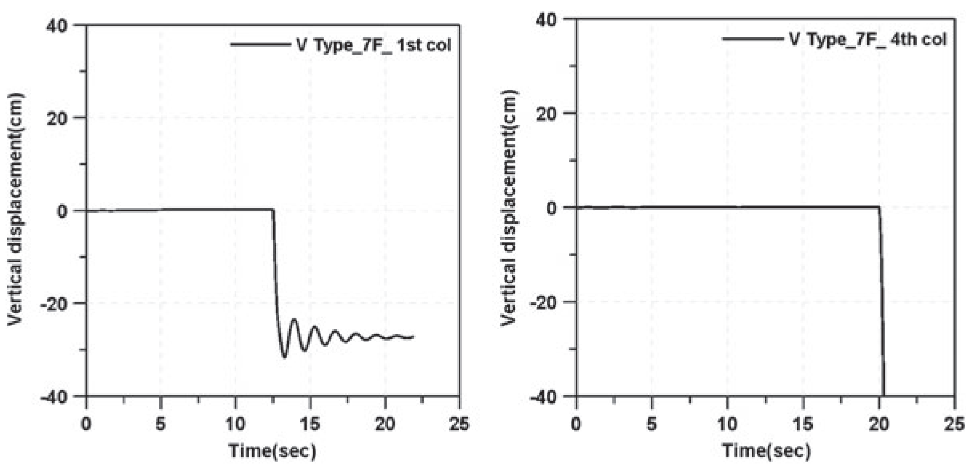


Figure 13. Deformation level for each performance point.



(a) Removal of the first column (b) Removal of the fourth column

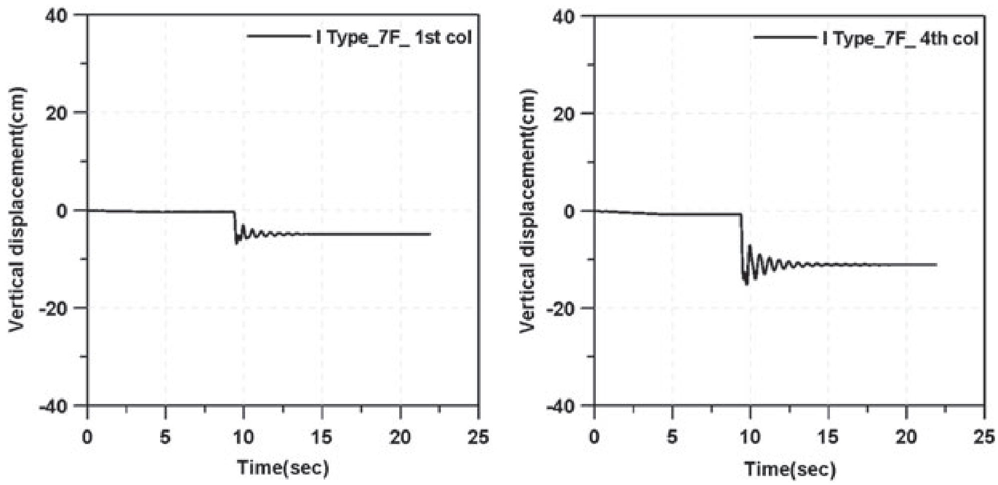
Figure 14. Vertical displacement time history of the seven-story structure with vertical interior columns.

DOI: 10.1002/tal

with vertical interior columns, the plastic hinges formed only in the bays from which the column was removed, whereas in the structure with tilted interior columns, the plastic hinges also formed in the adjacent bays.

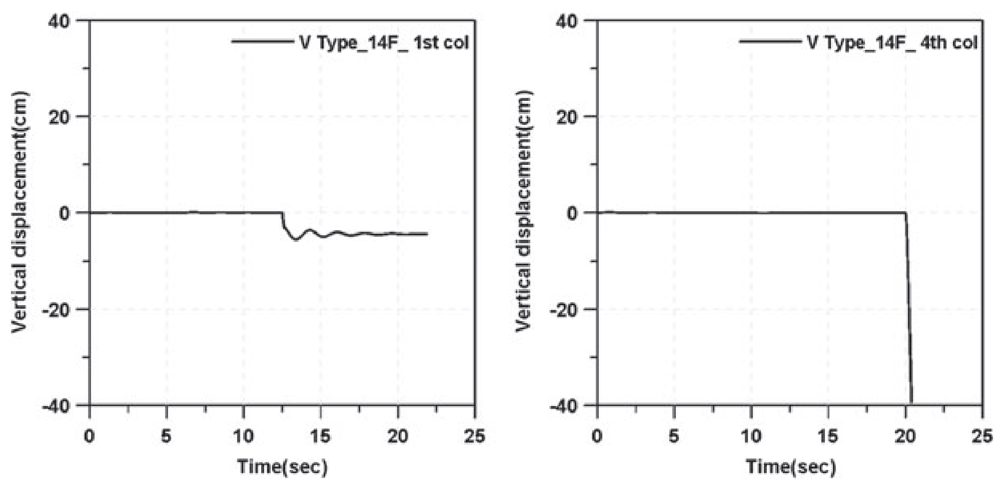
Figure 14 shows the nonlinear dynamic analysis results of the seven-story 13.4° tilted model structures with vertical interior columns subjected to sudden loss of a corner column. When the left-hand side corner column (first column) was suddenly removed, the vertical displacement at the joint remained stable after some oscillation. However, when the other corner column in the tilted side was removed, the vertical displacement became unbounded. This implies that the structure will collapse right after removal of the fourth column.

The time history analysis results of the seven-story tilted structure with tilted interior columns (I-type) and the 14-story tilted structure with vertical interior columns (V-type) subjected to sudden loss of a corner column are presented in Figures 15 and 16, respectively. It can be observed that the final vertical displacement is larger when the fourth column is removed than when the first column is removed. In both cases, the vertical displacement remained stable, and thus, the structure is safe against progressive collapse caused by sudden removal of a corner column. The 14-story tilted



(a) Removal of the first column (b) Removal of the fourth column

Figure 15. Vertical displacement time history of the seven-story structure with tilted interior columns.



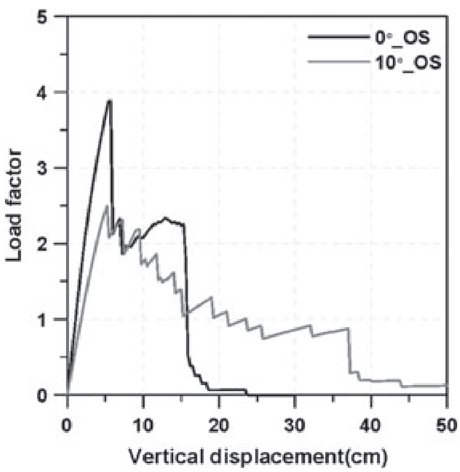
(a) Removal of the first column (b) Removal of the fourth column

Figure 16. Vertical displacement time history of the 14-story 13.4° tilted structure with vertical interior columns.

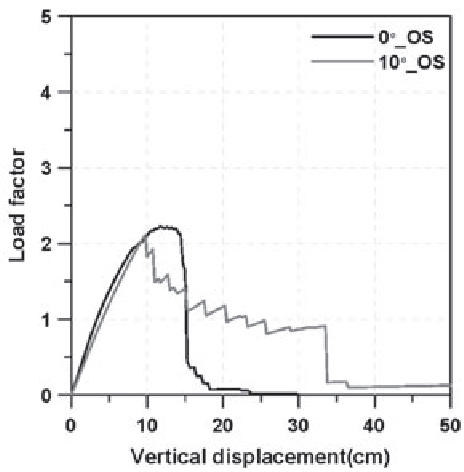
structure with vertical interior columns turned out to be vulnerable for progressive collapse when the right-hand side corner column was suddenly removed as can be observed in Figure 16(b).

3.3. *Structures with outrigger/belt trusses*

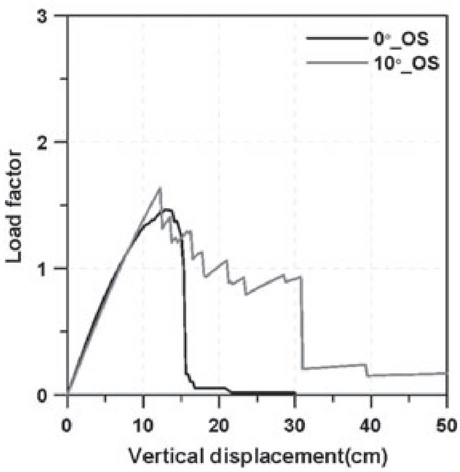
Figure 17 shows the nonlinear static pushdown analysis results of the structures with outrigger and belt trusses at the top stories. Pushdown curves of the regular and the 10° tilted structures subjected to loss of one, three and five columns from a corner were presented in Figure 17(a–c), respectively. When a corner column was removed, the maximum strength of the tilted structure turned out to be higher than that of the regular structure. This corresponds well with the results of the moment-resisting frames. When three and five columns were removed from a corner of the tilted side, both the regular and the tilted structures showed similar results. In the case where five columns were removed from the center of a side, the stiffness and strength of the tilted structure were slightly larger than those of the regular structure due mainly to the increased member sizes to satisfy the code requirement. Due to the stiffening effect of outrigger and belt trusses, the overall strength increased significantly compared with those of moment-resisting framed structures. Figure 18 shows the plastic hinge formation due to removal of five columns from a corner right before collapse. It can be observed that in the regular structure, the plastic hinges formed around the corner from which the columns were removed, whereas



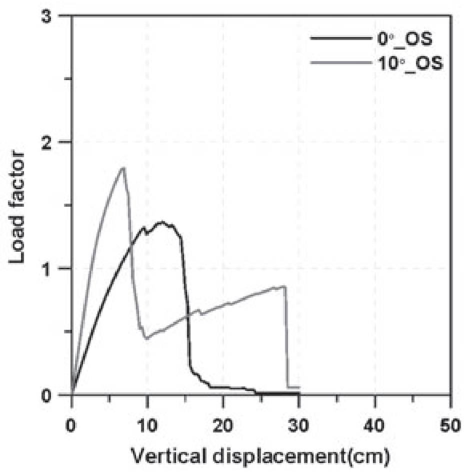
(a) Removal of one corner column



(b) Removal of three columns from a corner



(c) Removal of five columns from a corner



(d) Removal of five columns from a side

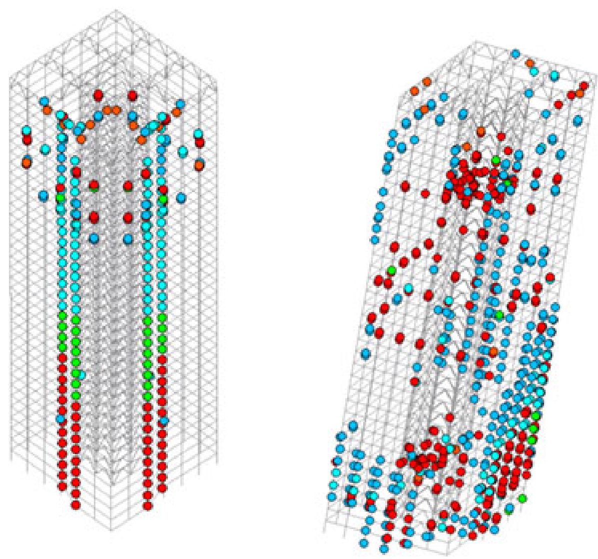
Figure 17. Pushdown curves of the structures with outrigger trusses.

DOI: 10.1002/tal

in the tilted structure the plastic hinges were more widely distributed around the building. This also may contribute to the increased strength of the tilted structure.

3.4. Tubular and diagrid structures

Nonlinear static analyses were conducted with the tubular and the diagrid structures subjected to loss of five corner columns and three pairs of diagrids in the first story, and the results are presented in Figures 19 and 20, respectively. Figure 19 shows the pushdown curves of the tubular structures, where it can be observed that the strength and the ductility of the tilted structure are higher than those of the regular structure. The pushdown curves of the diagrid structures presented in Figure 20 show that the maximum strength of the tilted structure is slightly higher than that of the regular structure. As observed in the previous cases, the strengths of the tilted structures turned out to be higher than those of the regular structures.



(a) Vertical structure (b) Inclined structure

Figure 18. Plastic hinge formation of the structure with belt and outrigger trusses.

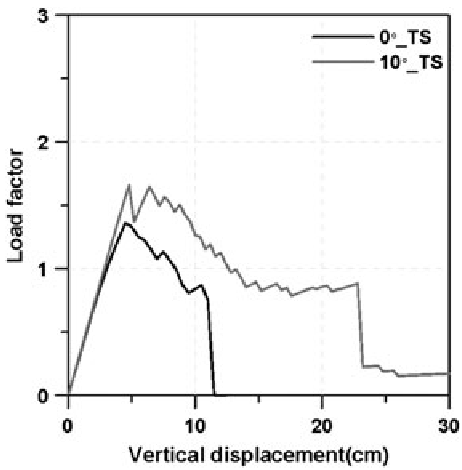


Figure 19. Pushdown curves of tubular structures with five columns removed from a corner.

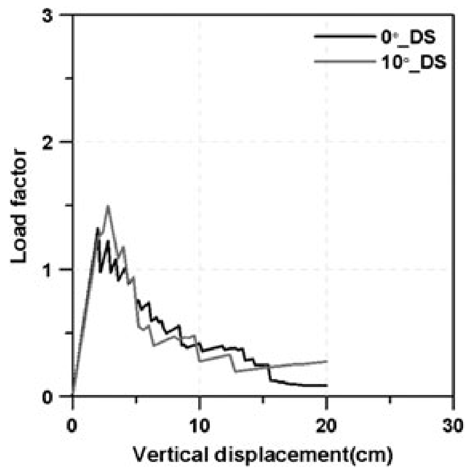
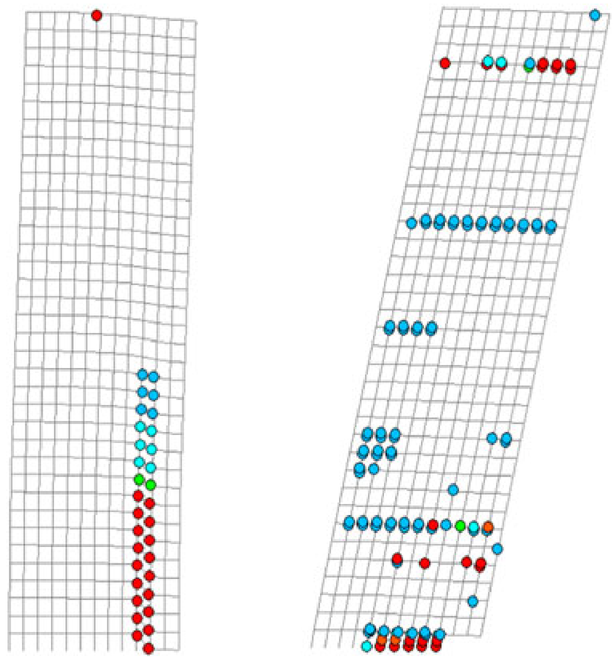


Figure 20. Pushdown curves of diagrid structures with three pairs of diagrid removed from a corner.

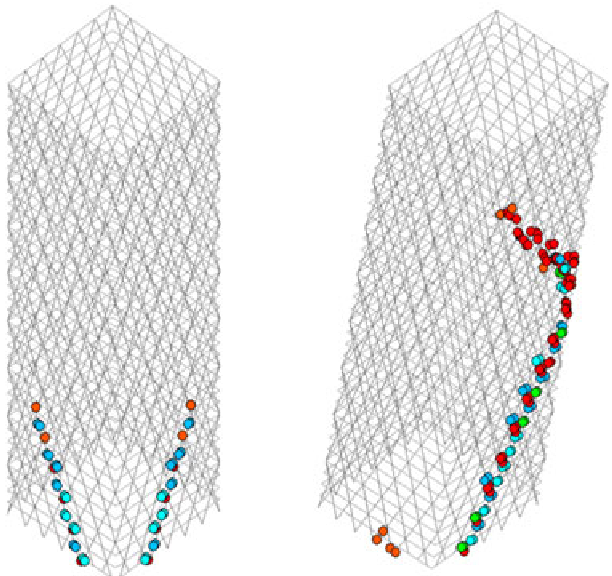


(a) 0° (b) 10°

Figure 21. Plastic hinge formation of tubular structures with five columns removed from a corner.

The plastic hinge formations of the tubular and diagrid structures are depicted in Figures 21 and 22, respectively. It can be observed that compared with the regular structures, the plastic hinge of the tilted structures are more widely and asymmetrically distributed. This implies that more structural elements participate in resisting progressive collapse, which participated in the increase in the overall strength of the tilted structures.

Figure 23 shows the variation of the maximum load factors depending on the loss ratio of the exterior columns or diagrids. The loss ratio represents the summation of the axial forces of the removed columns/diagrids divided by the summation of the first-story column/diagrid forces, which is identical to the summation of the imposed gravity load. With the graphs, it would be possible to determine the minimum number of columns required to initiate progressive collapse of the structures. It can be



(a) 0° (b) 10°

Figure 22. Plastic hinge formation of diagrid structures with three pairs of diagrids removed from a corner (load factor = 0.6).

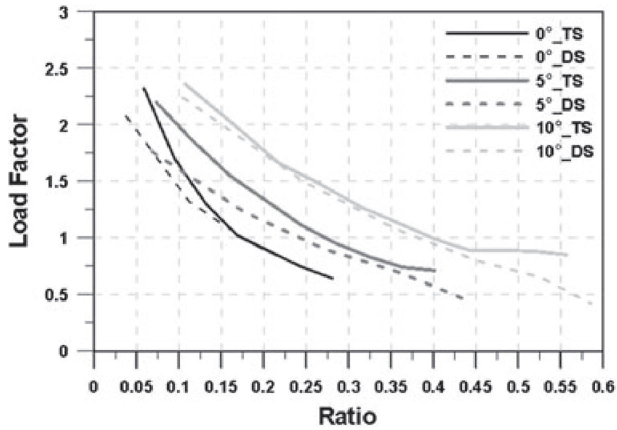
observed that as the tilting angle increases, the maximum load factor corresponding to a given loss ratio generally increases regardless of the locations of the removed elements. The load factors of the tubular structures are generally higher than those of the diagrid structures at the same loss ratio. When columns/diagrids were eliminated from a corner of the tilted side (right-hand side in Figure 4), the maximum load factors of the regular structures decreased below 1.0 as the loss ratios increased approximately above 1.5. In the 10° tilted structures, however, the load factors decreased below 1.0 as the loss ratio increased more than approximately 2.5.

4. CONCLUSIONS

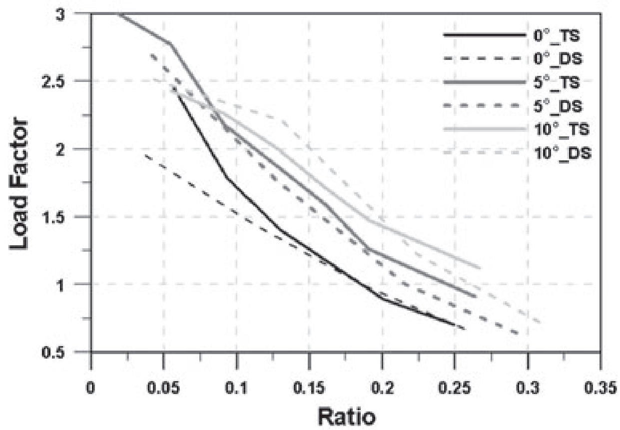
In this study, the progressive collapse resisting capacities of structure systems typically used in the design of building structures were evaluated. With the analysis results obtained in this study, it is concluded that the tilted building structures, once they are properly designed to satisfy a given design code, may have at least equivalent capacity for resisting progressive collapse caused by sudden loss of a column. To draw more generalized conclusion on the progressive collapse potential of tilted buildings, however, it would be necessary to investigate buildings with wider range of design parameters such as number of story, slenderness ratios, structure systems and building shapes. The analysis results are summarized as follows.

The tilting of the structures does not decrease the progressive collapse resisting capacity due mainly to the increased member sizes demanded by the increased *p*-delta effect. Another reason for the increase in the resisting capacity of tilted structures may be the more widely distributed plastic hinges, which implies that more structural members participate in resisting progressive collapse. This arises from the unsymmetric configuration of the tilted structures. In the tilted moment-resisting frames with vertical interior columns, the plastic hinges formed only in the bays from which a column was removed, whereas in the tilted structures with inclined interior columns, the plastic hinges were distributed more widely in the adjacent bays. The tilted tubular model structure showed slightly better performance against progressive collapse than the tilted diagrid structure. This, however, does not imply that the tubular structures are more effective in resisting progressive collapse than the diagrid structures considering the increased steel tonnage required to meet the design code.

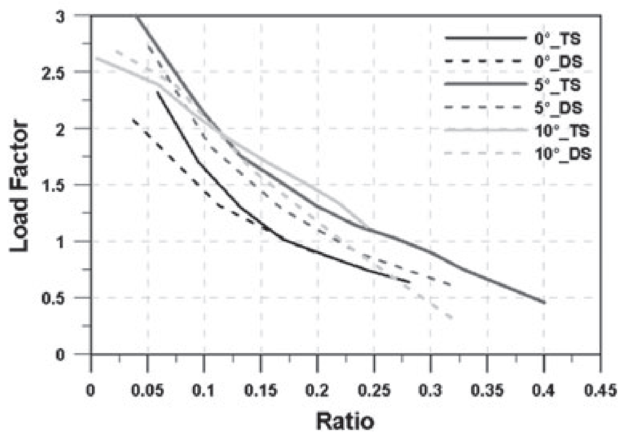
DOI: 10.1002/tal



(a) Removal of elements from a corner of the tilted side



(b) Removal of elements from the center of a side parallel to the tilting direction



(c) Removal of elements from a corner of the obtuse side

Figure 23. Variation of maximum load factors depending on the ratio of the removed elements.

ACKNOWLEDGEMENTS

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

REFERENCES

Al-Ali AK, Krawinkler H. 1998. Effects of vertical irregularities on seismic behavior of building structures, Report No. 130, Dept. Civil and Environmental Eng., Stanford University.

Alashker Y, El-Tawil S. 2011. A design-oriented model for collapse resistance of composite floors subjected to column loss, *J. Constructional Steel Research* **67**: 84–92.

Almusallam TH, Elsanadedy HM, Abbas H, Ngo T, Mendis P. 2010. Numerical analysis for progressive collapse potential of a typical framed concrete building, *International Journal of Civil & Environmental Engineer.* **10**(2): 40–46.

FEMA. 2000. Prestandard and commentary for the seismic rehabilitation of buildings, FEMA-356, Federal Emergency Management Agency, Washington, DC.

FEMA 274, 1997. NEHRP Commentary on the guidelines for the Seismic Rehabilitation of buildings, Federal Emergency Management Agency, Washington, D.C.

GSA. 2003. Progressive collapse analysis and design guidelines for new federal office buildings and major modernization project, The US General Service Administrations.

Izzuddin BA, Vlassis AG, Elghazouli AY, Nethercot DA. 2008. Progressive collapse of multi-story buildings due to sudden column loss—part I: simplified assessment framework, *Engineering Structures* **30**: 1308–1318.

Jain AK, Goel SC. 1978. Inelastic cyclic behavior of bracing members and seismic response of braced frames of different proportions, Report No. UMEE78R3, Department of Civil Engineering, University of Michigan, Ann Arbor, Michigan.

KBC-2009. 2009. Korea Building Codes, Architectural Institute of Korea.

Kim J, Hong S. 2011. Progressive collapse performance of irregular buildings, *The Structural Design of Tall and Special Buildings* **20**(6): 721–734.

Kim J, Lee Y. 2010. Progressive collapse resisting capacity of tube-type structures. *The Structural Design of Tall and Special Buildings* **19**(7): 761–777.

Kim J, Lee Y, Choi H. 2011. Progressive collapse resisting capacity of braced frames. *The Structural Design of Tall and Special Buildings* **20**(2): 257–270.

Leonard, J.. 2007. Investigation of shear lag effect in high-rise buildings with diagrid system, Master's Thesis, Dept. Civil and Environmental Eng., MIT.

Moon K. S.. 2007. Diagrid structural systems for tall buildings, characteristics and methodology for preliminary design. *The Structural Design of Tall and Special Buildings* **16**: 205–230.

SAP2000. 2004. Structural analysis program, computers and structures, Berkeley.

Sarkar P., Prasad AM, Menon D. 2010. Vertical geometric irregularity in stepped building frames. *Engineering Structures* **32**: 2175–2182.

Scott D, Farnsworth D, Jackson M, Clark M. 2007. The effects of complex geometry on tall towers. *The structural design of tall and special buildings* **16**: 441–455.

Starossek U. 2006. "Progressive collapse of structures: Nomenclature and procedures.", IABSE, Structural Engineering International, **16**(2): May 2006.

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

Vollers K. 2008. Morphological scheme of second-generation non-orthogonal high-rises, CTBUH 8th World Congress, Dubai.

AUTHORS' BIOGRAPHIES

Jinkoo Kim received his BS and MS from Seoul National University in Korea, and MS and PhD from Dept. of Civil and Environmental Engineering, Massachusetts Institute of Technology. His research interests include performance evaluation and retrofit of building structures against earthquake loads and progressive collapse.

Min-Kang Jung received BS from Yonsei University and MS from Dept. of Megastructures, Sungkyunkwan University, Korea. Currently, he is an engineer in the Samsung Engineering and Construction, Ltd.