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Progressive collapse-resisting capacity of modular mega-frame buildings

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SUMMARY

In this study, the progressive collapse-resisting capacity of modular mega-frame structures consisting of a few identical subsystems was investigated based on column-loss scenario. Four types of mega-frame structures were designed as basic analysis model structures. According to pushdown analysis results, the mega-frame structure with four corner columns did not satisfy the design guidelines for progressive collapse regardless of the number of subsystems when one of the first-story mega-columns was removed. To enhance the resistance against progressive collapse, we redesigned the basic model structure with four mega-columns by adding additional floor trusses in the transfer floors, adding moment-resisting frames at the perimeter and adding vertical mega-bracing. The pushdown analysis results showed that the schemes with additional mega-braces were most effective in increasing the progressive collapse-resisting capacity of mega-frame buildings. Copyright © 2011 John Wiley & Sons, Ltd.

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KEY WORDS: progressive collapse; structure systems; mega-frame structures; pushdown analysis; tall buildings

1. INTRODUCTION

A mega-frame structure system, which is composed of mega-elements to resist both gravity and lateral loads and subsystems designed only for gravity loads, is considered to be suitable for tall buildings because of its efficiency in resisting lateral loads. Taranath (1997) indicated that the best structure system for a tall building with practical form is a skeletal structure with columns located at the farthest extremity from the building center. To achieve high efficiency, we need to transfer much or all gravity loads into these columns to enhance their capacity for resisting overturning effects due to lateral loads. The ultimate structure for a rectangular building will have just four corner columns interconnected with a shear-resisting system. The main premise behind the ultimate high-efficiency structure is to transfer as much gravity loads as practicable to the columns resisting the overturning moments. This can be achieved by eliminating as many interior columns as possible and maximizing the use of the holding-down power of gravity loads. Within this basic configuration, it is possible to provide a system of transfer floor trusses at approximately every 15th floor, corresponding to the levels at which the low, low-mid, high-mid and high-rise elevators terminate. Since the interior columns carry gravity loads from a limited number of floors, their sizes will be substantially smaller than in a conventional system. Moreover, columns in each subsystem may be located at will to suit the desired interior space planning providing flexibility for mixed use. Even though there would be some premium in the tonnage of steel and fabrication cost for the transfer trusses, the system has advantage in the planning of interior spaces over conventional buildings with large interior columns and core walls.

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The mega-frame systems, however, may be vulnerable to progressive collapse caused by sudden loss of a mega-column because the structural redundancy of a mega-frame is generally limited in comparison with those of conventional buildings. The progressive collapse in buildings refers to the phenomenon that local damage of structural elements caused by abnormal loads results in global collapse of the structure. The analysis method recommended is the alternative path method (General Services Administration (GSA), 2003), in which the structure is designed in such a way that if any one component fails, alternate load 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. Analysis procedures and program software were developed to simulate collapse behavior of structures (Kaewkulchai and Williamson, 2003; Kim *et al.*, 2009). The performances against progressive collapse have been studied for steel moment frames (Powell, 2005; Kim and An, 2009; Kim and Kim, 2009; Park and Kim, 2010) and for reinforced concrete structures (Sassani and Kropelnicki, 2007; Yi *et al.*, 2008). Recently, Kim and Lee (2010) evaluated the progressive collapse-resisting capacity of tubular and diagrid structures and found that the perimeter tube-type structures generally have sufficient capacity for resisting progressive collapse caused by loss of vertical load-resisting elements. It was observed that loss of up to 11% of vertical load-resisting elements from a corner caused progressive collapse in the tubular structures, whereas in the diagrid structures, the ratio reduced to the loss of 8% of gravity load-resisting elements.

This study investigates the progressive collapse-resisting capacity of mega-frame structures composed of many identical subsystems based on column-loss scenario recommended in the GSA guidelines. To this end, the nonlinear static analyses of mega-frames composed of subsystems and mega-elements are carried out by removing one of the mega-columns. Based on the analysis results, various modifications and alternative schemes are investigated to enhance the progressive collapse-resisting capacity of mega-frame buildings.

2. ANALYSIS FOR PROGRESSIVE COLLAPSE

In this paper, the progressive collapse performance of the mega-frame structures was investigated based on the column-loss scenario. The finite element program code MIDAS (2009) was used for structural design and nonlinear static pushdown 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. In this study, pushdown analyses of the model structure were carried out with one of the first-story mega-columns removed. For static analysis, both the GSA 2003 and the Department of Defense (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 1. The DoD guidelines recommend to use a larger gravity load than that recommended by the GSA guidelines and include a wind load in the load combination. The static procedure accounts for nonlinear effects without

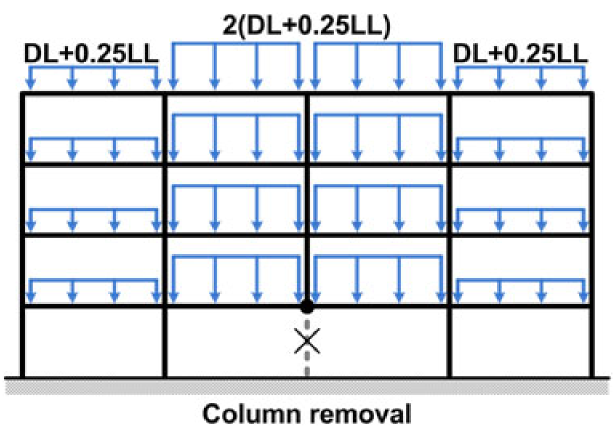

Figure 1. Applied load for static progressive collapse analysis.

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sophisticated hysteretic material modeling and is useful in determining the elastic and failure limits of the structure. The load combination of the GSA 2003 for static analysis is 2(dead load + 0.25 × live load).

Investigation of the nonlinear behavior of structures must follow the equilibrium path and identify singular points. The load-controlled Newton–Raphson method fails near the limit point, and the displacement control techniques for structural systems may fail for structures exhibiting snap-through or snap-back behavior (Clarke and Hancock, 1990). The arc-length method was developed for structural analysis (Riks, 1979) and was adopted as the main algorithm for pushover or pushdown analysis by the program code MIDAS (2009) to overcome such problems. Unlike in the load control method in which the load is kept constant during a load step or in the displacement control method in which displacement is kept constant during increment, in the arc-length method, the load factor at each iteration is modified so that the solution follows some specified path until convergence is achieved. The basic idea of the method is explained as follows (Memon and Su, 2004): Since the method treats the load factor as a variable, it becomes an additional unknown in equilibrium equations resulting from finite element procedure and yields $(N+1)$ unknowns, where N is the number of elements in the displacement vector. The solution of $(N+1)$ unknowns requires an additional constraint equation expressed in terms of current displacement, load factor and arc length. Simplification of the constraint equation leads to a quadratic equation, whose roots are used for determining the load factor. Proper selection of root is one of the key issues of the method, whose details will be discussed in the subsequent sections. Generally, for the first increment, the trial value of the load factor is assumed as 1/5 or 1/10 of the total load. For further increments, the load factor is computed according to the rate of convergence of the solution process.

3. DESIGN OF MODEL STRUCTURES

To investigate the progressive collapse potential of the mega-frame structures, we designed the four model structures described in Table 1 as per Korea Building Code (2005). The design dead and live loads are 5 kN/m^2 and 2.5 kN/m^2, respectively. The design wind load is computed based on 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 (2006) format. As the mega-frame system is not defined as one of the seismic lateral load-resisting systems in current design codes, the response modification factor (R factor) of 3.0 is used to derive the design seismic load. The base shears resulting from design wind and earthquake loads were determined as 13.7 MN and 21.4 MN, respectively. The SM 520 structural steel with yield stress of 355 N/mm^2 is used for mega-columns and mega-braces, and the SM 400 steel with yield stress of 235 N/mm^2 is used for floor beams.

The model structures include two to four identical subsystems, which are moment frames designed to resist only gravity loads, stacked vertically inside of the mega-frame. Figure 2 shows the plan and the elevation of the mega-frame model structure with three subsystems and with four external mega-columns. Each subsystem is composed of 14 stories and is supported on two-story-high transfer floors, which transfer the gravity load of each subsystem to the exterior mega-columns. In this way, the lateral load-resisting capacity of the model structures against overturning moment can be maximized. In this paper, the transfer floors are designed with two-story deep trusses with diagonal members as shown in Figure 3. In practice, however, the transfer floors could be designed with vierendeel trusses

Table 1. Mega-frame model structures for analysis.

Model	Super columns	Subsystems	Story (height)
1	4	3	51 (206 m)
2	8	3	51 (206 m)
3	4	2	35 (142 m)
4	4	4	67 (270 m)

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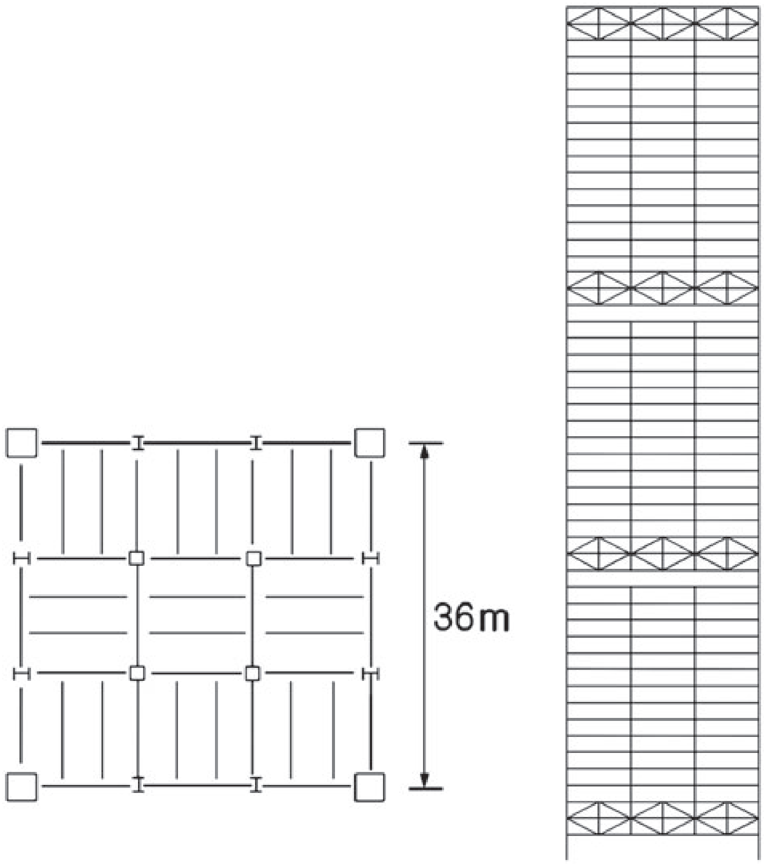

(a) Structural plan (b) Structural elevation

Figure 2. Structural shape of the model structure with four super columns and three identical subsystems.

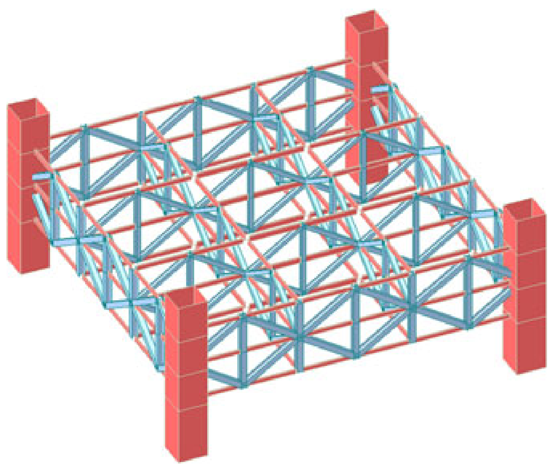

Figure 3. Configuration of the two-story-high transfer floor.

without diagonal members spanning in two directions for the full width and length of the building. Where appropriate, the transfer levels can be made into sky lobbies or other forms of common areas.

The analysis models 1 and 2 have three subsystems and thus are of the same height of 206 m with 51 stories including the transfer floors. Model 1 is designed with four square mega-columns with a

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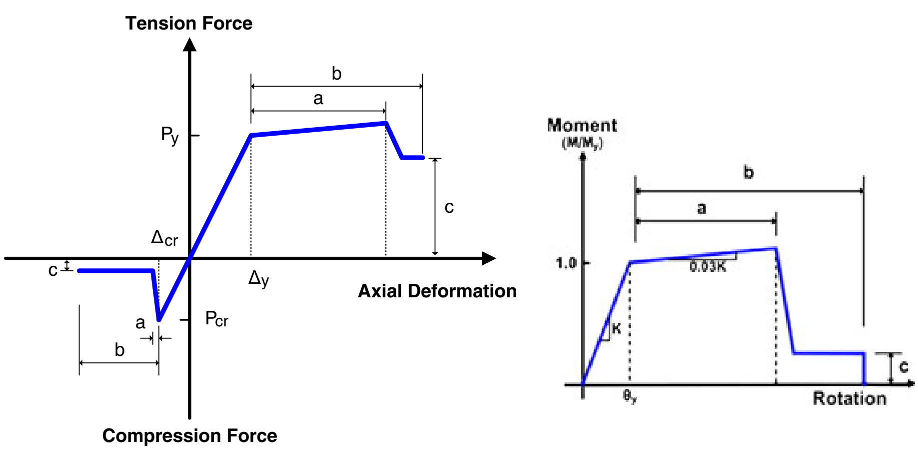
size of 3.1 × 3.1 m and a thickness of 10 cm located at the four corners, and model 2 has eight columns with two columns at each side of the structure with a size of 3.0 × 2.5 m and a thickness of 10 cm. Models 3 and 4 have four mega-columns but are composed of two and four subsystems, respectively. The interior columns are disconnected between each subsystem to induce all the gravity loads into the exterior mega-columns. The model structures have a uniform story height of 4 m and a span length of 12 m. The exterior mega-columns are designed with box columns with dimensions of 3.1 m × 3.1 m at the first story. The pin-connected interior columns and girders in each subsystem are designed with H-shaped members to resist only the gravity loads. The fundamental natural period of the structure with three subsystems and four mega-columns is estimated to be 6.1 s, which is generally longer than that of a typical building with equivalent size. For nonlinear analysis of truss and bracing members, the generalized load–deformation curves recommended in the Federal Emergency Management Agency (FEMA) 274 (1997) is used, which is based on the phenomenological model proposed by Jain and Goel (1978). For bending members, the skeleton curve provided in the FEMA 356 (2000) is used. Figures 4(a, b) show the load–deformation relationship used in this study. The parameters *a*, *b* and *c* vary depending on the width–thickness ratio of the structural members. In this study, $a = 6.5$, $b = 8.5$ and $c = 0.4$ with postyield stiffness of 3% are generally used for modeling of bending members based on the guidelines provided in Tables 5-6 and 5-7 of the FEMA 356. For braces, $a = 11$, $b = 14$ and $c = 0.8$ are used for tension members and $a = 0.01$, $b = 8$ and $c = 0.2$ for compression members based on Table 5-8 of FEMA 273.

4. ANALYSIS RESULTS OF MODEL STRUCTURES

To evaluate the progressive collapse-resisting capacity of the four model structures listed in Table 1, we conducted nonlinear static pushdown analyses. The locations of the removed column are shown in Figure 5(a, b) for the structure with four and eight mega-columns, respectively. The pushdown analyses were carried out by gradually increasing the vertical displacement in the location of the removed column and estimating the applied load required to produce such deformation. The ratio of the applied load and the GSA-specified load combination of 2(dead load + 0.25 × live load) is referred to as the load factor. Figure 6 shows the pushdown curves of the four model structures, where it can be observed that only the maximum strength of the structure with eight mega-column (model 2) reaches the load factor of 1.0 and the maximum load factors of the other model structures range between 0.63 and 0.75. This implies that except for the model structure with eight mega-columns, all the model structures with four mega-columns have potential for progressive collapse in case one of the mega-columns is suddenly removed. As expected, the number of mega-columns may contribute significantly to the safety of a mega-frame structure against progressive collapse. It can also be



(a) Braces (b) Bending members

Figure 4. Moment–rotation relationship of structural members.

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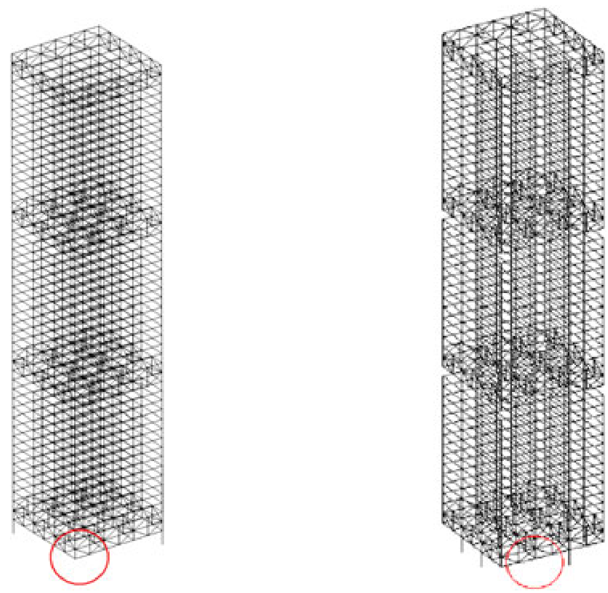

(a) Mega-frame with four mega-columns (b) Mega-frame with eight mega-columns

Figure 5. Location of the removed column in the model structure with three subsystems.

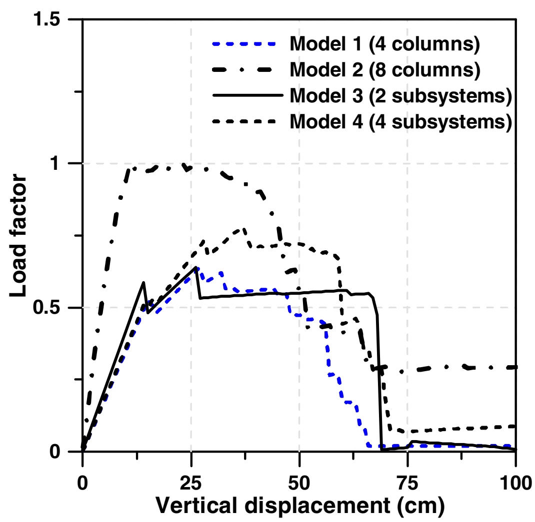

Figure 6. Pushdown curves of mega-frame analysis model structures.

noticed that the maximum strength of the structure with four subsystems is slightly higher than the structures with two or three subsystems.

Figure 7 illustrates the locations of buckled or yielded members at three different vertical displacement levels. In the figure, the circles with different colors represent the deformation levels as depicted in Figure 8. It is observed that buckling or yielding first occurs in the bracing members at the lowest transfer floors, and as the vertical deflection further increases, the members in the upper transfer floors undergo inelastic deformation. The sudden drops of the load in the pushdown curves, as observed in Figure 6, occur simultaneously with the buckling or yielding of bracing members in the transfer floors. Final collapse occurs when plastic hinges form in the mega-columns. Based on the analysis results, it seems to be necessary to reinforce the transfer floors or to provide more alternate load paths by adding additional structural members to provide more alternate load paths for the mega-frame structures with four corner mega-columns.

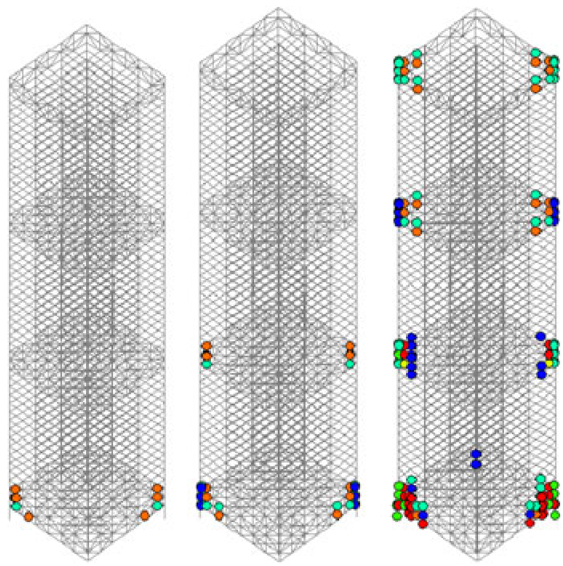

Figure 7. Plastic hinge formation in the model structures at three levels of vertical displacement (*D*).

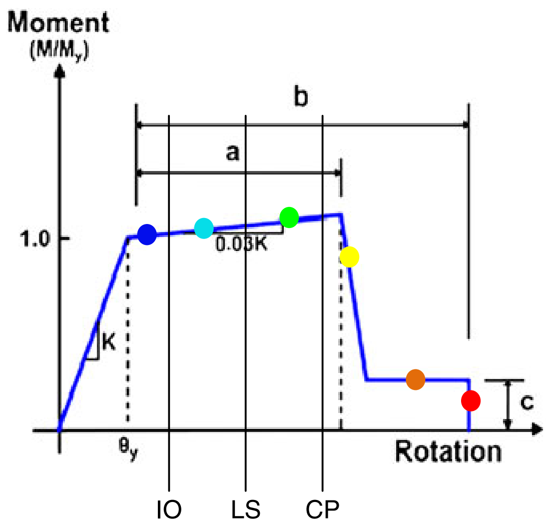

Figure 8. Deformation levels of plastic hinges.

5. REDESIGN OF MODEL STRUCTURES

The basic model structure with three subsystems and four corner mega-columns, which turned out to be vulnerable to progressive collapse, was redesigned to incorporate additional structural members to increase the structural redundancy and consequently the alternate load paths against progressive collapse. Table 2 shows the five different revised schemes designed using the same design loads as used for the basic structure. The member sizes of the revised structure with external X bracing are presented in Table 3. The first revised scheme, model 5, is to add two-story deep diagonal trusses in the transfer floors as shown in Figure 9. The second scheme is to design the exterior beam–column joints as rigid connections. The other three schemes, models 7 to 9, utilize the mega-bracing with the height of the whole subsystem in the interior or exterior of the structure. Figure 10 shows the configurations of the revised model structures with exterior inverse V

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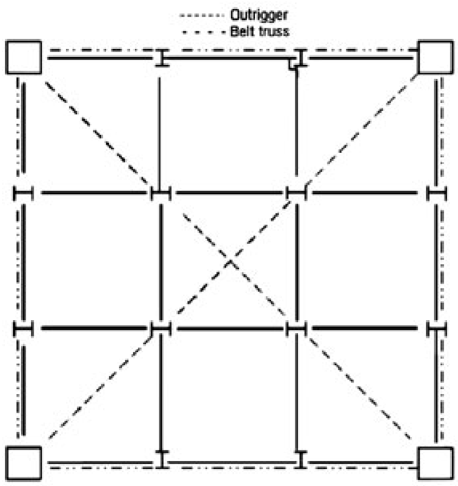
Table 2. Alternative design schemes for enhancing progressive collapse-resisting capacity.

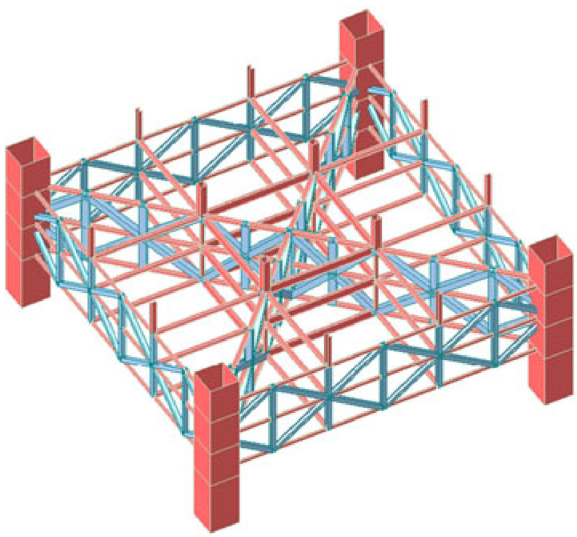
Model	Alternative design schemes
5	Addition of horizontal diagonal trusses
6	Exterior moment frames
7	Exterior mega-bracing (inverse V)
8	Exterior mega-bracing (X shape)
9	Interior mega-bracing

Table 3. Sizes of mega-columns and mega-bracing in the structure with external X bracing.

Story	Columns	Braces
1–3	3 m × 3 m × 85 mm	
4–18	2.5 m × 2.5 m × 65 mm	0.8 m × 0.8 m × 40 mm
19–26	1.8 m × 1.8 m × 40 mm	
27–34	1.5 m × 1.5 m × 40 mm	
35–51	1 m × 1 m × 20 mm	0.68 m × 0.68 m × 30 mm



(a) Structural plan



(b) Three-dimensional view

Figure 9. Configuration of the redesigned transfer floors (model 5).

(Figure 10(a)), X-shape (Figure 10(b)) and interior diagonal (Figure 10(c)) mega-bracing. The mega-braces were modeled as separate truss elements in each story with pin-connected end conditions. The buckling strength was computed considering both in-plane and out-of-plane buckling modes. For the horizontal truss elements in the transfer floors, only in-plane buckling was considered based on the assumption that the out-of-plane buckling was prevented by the floor diaphragm. The force–deformation relationship of the bracing elements recommended by the FEMA 274 is shown in Figure 4(a).

Figure 11 shows the pushdown curves of the revised structures with three subsystems and four mega-columns. The dotted curve represents the pushdown curve of the basic model structure, which has four transfer trusses in each orthogonal direction of the transfer floors and pin-connected

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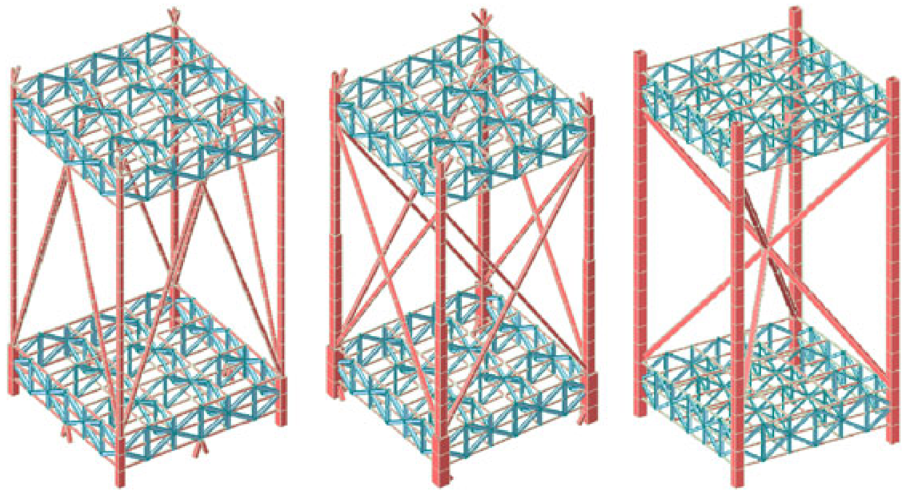

Figure 10. Three-dimensional view of the subsystem with mega-bracing.

girder–column joints. Figure 11(a) compares the pushdown curves of the basic structure and the structure with its transfer floors reinforced with two additional diagonal trusses, where it can be observed that the maximum load factor increases slightly as a result of the addition of the diagonal trusses in the transfer floors. Figure 11(b) shows the pushdown curve of the revised structure with moment connection of exterior frames. Not only the exterior beam–column connections of the subsystems but also the connections between the subsystems and the mega-frame are designed with rigid joints. The results show that the increase in strength due to the moment connection is not significant in comparison with the strength of the basic structure, but the ductility is slightly increased. Both redesign schemes fail to enhance the maximum load factor (strength) above 1.0, the level required in the GSA (2003) guidelines for safety against progressive collapse. Figure 11(c–e) depict the pushdown curves of the model structures redesigned with exterior inverse V, X-shape mega-bracing and interior diagonal mega-bracing, respectively. The use of mega-bracing is effective in reducing the amount of structural steel for the same design load as shown in Table 4. In all three cases, the maximum load factors exceed 1.0, and the safety against progressive collapseis ensured when one of the mega-columns is lost. The revised model structure with interior mega-bracing, model 9, shows the largest vertical displacement before collapse. It was observed that in the structures with inverse V and X-shape bracing, where braces are placed at the perimeter of the structures, the plastic hinges mostly formed in the lower subsystem of the structures including the mega-columns, whereas they were more widely distributed throughout the building height in the structure with internal mega-bracing. This implies that a larger number of structural members participate in resisting progressive collapse in the structure with internal mega-bracing. It is also observed that yielding in mega-columns occurs only at transfer floor levels. These contribute to the larger displacement capacity of model 9 as observed in Figure 11(e).

The pushdown curve of the structure with four corner mega-columns with the moment connection of the beam–column connections, presented in Figure 11(b), shows that the moment connection is not effective in enhancing the progressive collapse-resisting capacity of the basic model structure. Figure 12 shows the pushdown curves of the mega-frame structures with eight mega-columns with and without the moment connection of the perimeter beam–column joints. In this case, the moment connection results in significant increase in the maximum load factor. The difference in the effectiveness of the moment connections originates from the distance between the mega-columns. The stiffness of the vierendeel panel formed by the exterior moment frame of a subsystem and two mega-columns increases as the distance between the two adjacent mega-columns decreases.

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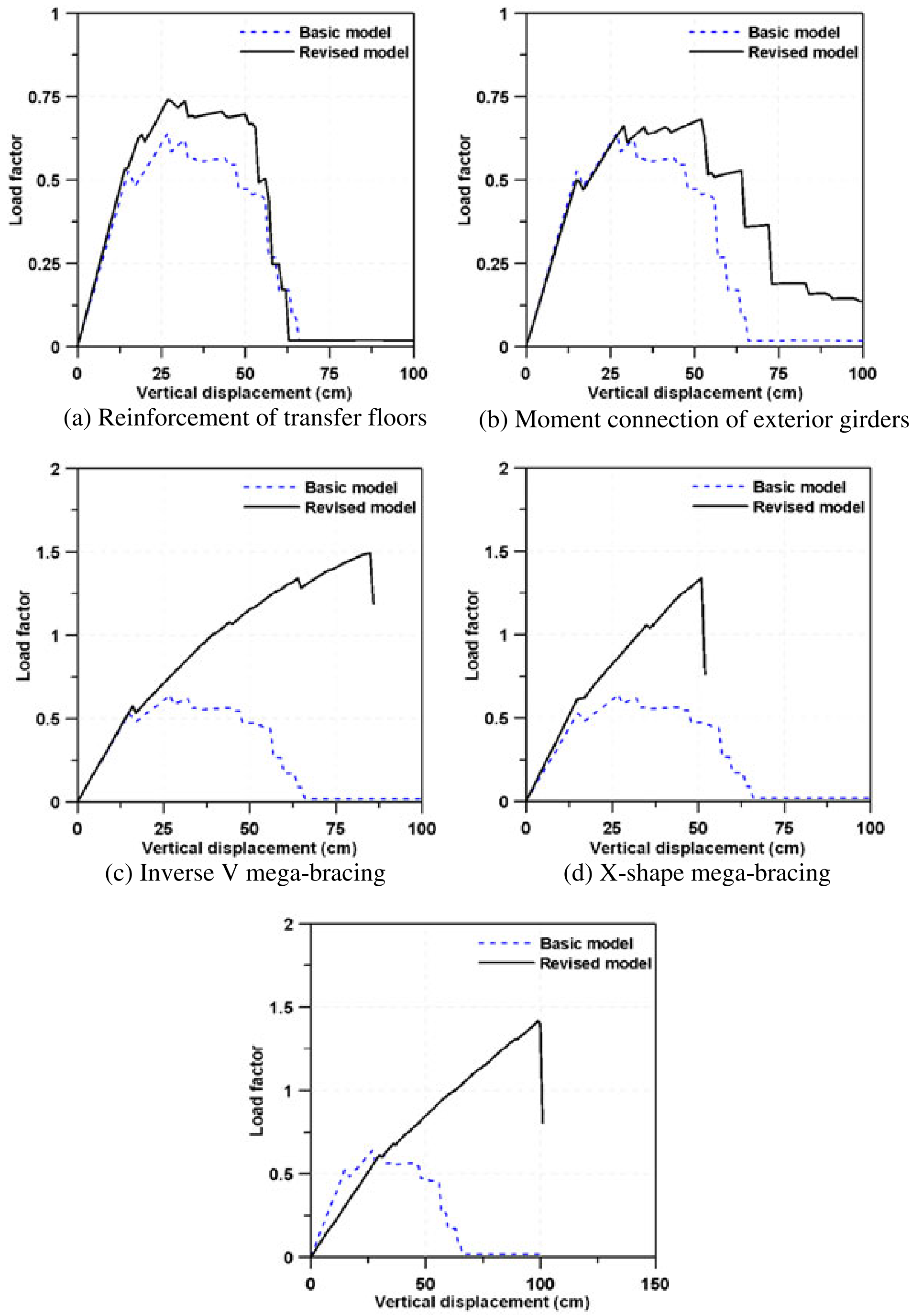

(a) Reinforcement of transfer floors (b) Moment connection of exterior girders

(c) Inverse V mega-bracing (d) X-shape mega-bracing

(e) Model with internal mega-bracing

Figure 11. Pushdown curves of the basic model structure with three subsystems and four mega-columns and the structures revised with various schemes.

Therefore, the effectiveness of the exterior moment frames depends on the number of perimeter mega-columns.

In the structural design of tall buildings, the direction of floor beams is generally altered throughout the stories in such a way that the floor gravity load is distributed to each column as evenly as possible. Figure 13 shows the locations of beams in the model structures. In floors above and below that story,

Table 4. Maximum load factors and steel tonnage of the basic and revised models.

Model	Load factor	Steel (MN)
1	0.66	141.8
5	0.74	148.5
6	0.70	143.2
7	1.50	91.9
8	1.34	104.0
9	1.42	119.5

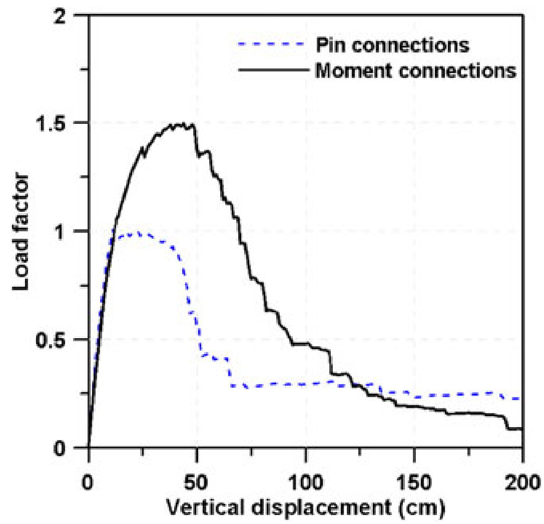

Figure 12. Pushdown curves of the mega-frame structures with eight mega-columns with and without moment connection of perimeter girders.

the beams are located in the orthogonal directions to prevent nonsymmetric distribution of the floor load. Figure 14 compares the pushdown curves of the basic structures with three subsystems and four mega-columns with uniform layout of floor beams and with orthogonal layout of beams in alternate floors. It can be seen that the structure with uniform beam layout throughout the stories shows smaller ductility at failure when a mega-column is removed. This is due to the fact that the load paths for gravity load are not symmetric when one of the mega-columns is removed. It was observed in the plastic hinge formation of the basic model structure with uniform beam layout that the plastic hinges formed in a slightly unsymmetrical manner due to unsymmetrical application of gravity load. However, the direction of floor beams may not be important when the removal of a column causes unsymmetrical failure mode. Figure 15 shows the pushdown curves of the model structures with eight mega-columns with and without changing the beam layout throughout the stories. In this case, the floor beam layout does not affect significantly the maximum strength and ductility. The removal of a mega-column causes highly unsymmetrical deformation mode, and therefore, the slightly uneven distribution of floor gravity load does not affect the overall behavior of the structure.

6. CONCLUSIONS

This study investigated the progressive collapse-resisting capacity of mega-frame structures composed of many identical subsystems based on column-loss scenario. To this end, nonlinear analyses of mega-frames composed of various numbers of subsystems and mega-columns were carried out by removing

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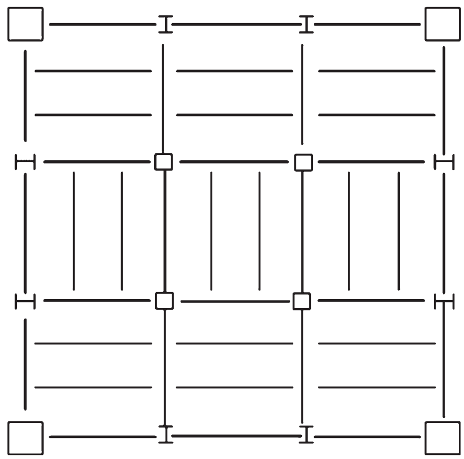

Figure 13. Placement of floor beams.

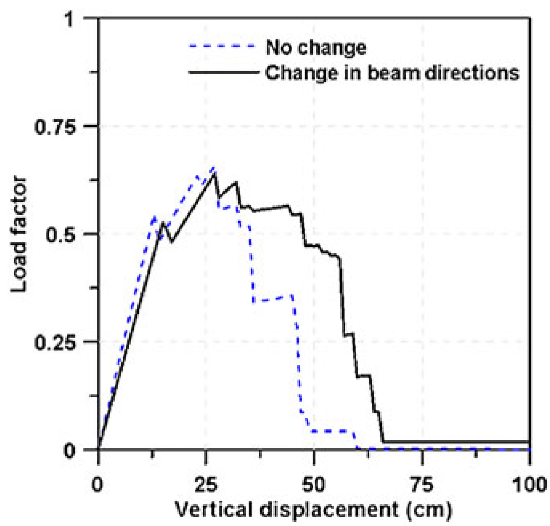

Figure 14. Pushdown curves of the basic structures with four mega-columns with different configuration of floor beams.

one of the first-story mega-columns. Based on the analysis results, various alternative schemes were investigated to enhance the progressive collapse-resisting capacity of mega-frame buildings.

According to pushdown analysis results, the mega-frame structure with four corner columns did not satisfy the design guidelines for progressive collapse regardless of the number of subsystems when one of the first-story mega-columns was removed. To enhance the resistance against progressive collapse, we redesigned the basic model structure with four mega-columns by adding additional floor trusses in the transfer floors, adding moment-resisting frames at the perimeter and adding vertical interior or exterior mega-bracing. The pushdown analysis results showed that the schemes with additional mega-bracing were most effective in increasing the progressive collapse-resisting capacity of mega-frame buildings with additional benefit of smaller requirement of structural steel. It was shown that by installing mega-bracing, more structural members participate in resisting progressive collapse. In this study, each element of the mega-bracing was jointed to the beams in every story in the substructure, which resulted in significant increase in structural redundancy. This contributed to the enhanced overall strength and ductility of the structure with mega-bracing. However, in practice, the mega-bracing is connected to the structure only at the intersection with columns for economy, ease

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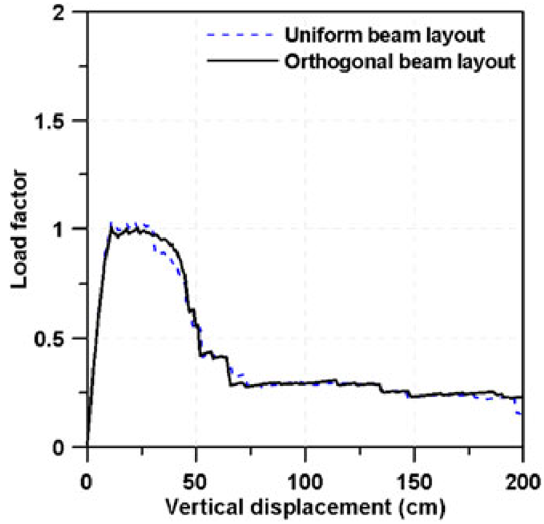

Figure 15. Pushdown curves of the model structures with eight mega-columns with and without changing beam directions along the height.

of construction and some structural considerations like preventing unbalanced force in beam due to failure of bracing in lateral force. The revised scheme with adding exterior vierendeel frames was not effective in the model structure with four mega-columns since the distance between the mega-columns was too wide. The story-wise layout of floor beams turned out to be important in case the failure mode due to the removal of a column would be symmetric. Based on the analysis results, it is recommended that the exterior or interior mega-bracing be used in the design of mega-frame structures to enhance the overall redundancy and consequently the progressive collapse-resisting capacity of the structure.

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