Progressive collapse resisting capacity of building structures with outrigger trusses

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SUMMARY

In this paper, the progressive collapse potential of building structures with core and outrigger trusses were evaluated using nonlinear static and dynamic analyses. To this end 36-storey analysis model structures composed of RC core walls and perimeter frames connected by outrigger trusses at the top were prepared. The static pushdown analysis of the structure with mega-columns and outrigger trusses showed that the maximum strength reached only about 20% of the load specified in the US General Services Administration guideline when a mega-column in the first storey was removed. According to dynamic analysis results, the vertical displacement monotonically increased until collapse as a result of buckling of some of outrigger truss members. However the structure with outrigger and belt trusses remained stable after a perimeter column was removed. The stability of the structure with mega-columns and outrigger trusses could be achieved by redesigning it with additional belt trusses or with moment connections in interior or exterior frames. Based on the analysis results it was concluded that the dynamic amplification factor of 2.0 recommended in the guidelines provided reasonably conservative results. Copyright © 2010 John Wiley & Sons, Ltd.

1. INTRODUCTION

For buildings taller than a certain height moment frames or braced core may not provide adequate stiffness to resist lateral load such as wind and earthquake loads. In this case the lateral stiffness can be increased by tieing the exterior frames and the shear core together by outrigger trusses. The core and outrigger systems have been widely used as efficient structural systems for tall buildings. Many of super-tall buildings including the Taipei 101 Tower in Taiwan and the Jin Mao Tower in China were built using the system. The placement of outrigger trusses increases the effective depth of the structure and significantly improves the lateral stiffness under lateral load. The columns connected to outriggers resist the rotation and lateral deflection of the core and consequently axial forces are developed in the exterior columns due to lateral load. To make the outriggers and belt trusses adequately stiff, outriggers are made to be at least one or two-stories-deep. It is possible to use truss diagonals extending through several floors to act as outriggers. Sometimes rigidly jointed girders in each floor are used as outriggers between braced core and external columns. The use of belt trusses on the facades, at the same level and perpendicular to the outrigger trusses, further enhances participation of exterior frames in the cantilever behaviour. The belt trusses transform the two-dimensional system of exterior column-outrigger truss-core wall system into a three-dimensional frame system by uniformly distributing vertical outrigger forces to all exterior columns.

After collapse of the World Trade Center twin towers in 2001 the safety of tall buildings against progressive collapse has been one of the primary concern for structural engineers. The progressive collapse refers to the phenomenon that local damage of structural elements caused by abnormal loads results in global collapse of the structure. An abnormal load includes any loading condition that is not considered in normal course of design but may cause significant damage to structures. In the USA,

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the General Service Administration (GSA) presented a practical guideline for design to reduce collapse potential of federal buildings (US GSA, 2003), and the US Department of Defence (DoD) also presented a guideline for the new and existing military facilities (2005). The analysis method recommended in these guidelines is the alternative path method. In this approach, the structure is designed in such a way that if any one component fails, 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. For static analysis those guidelines recommend dynamic amplification factor of 2.0 in the load combination to the bays adjacent to the removed element.

Recently the performance of building structures against progressive collapse has been studied to estimate the safety of structures subjected to sudden loss of a column; nonlinear analyses were carried out to investigate the progressive collapse resisting capacity of moment-resisting frames (Kim and An 2009; Tsi *et al.*, 2009; Kim and Kim, 2009; Kim *et al.*, 2009), braced frames (Kim *et al.*, 2010), tubular structures (Kim and Lee, 2009), and irregular structures (Kim and Hong, 2010). In this paper, the progressive collapse potential of building structures with core and outrigger trusses were evaluated using nonlinear static and dynamic analyses. As analysis model structures, two types of 36-storey structures composed of RC core walls and perimeter frames connected by outrigger trusses at the top were prepared. In addition to the basic structure types, the progressive collapse potentials of various alternative structural design schemes were also investigated. Finally, the validity of the dynamic amplification factor of 2.0 recommended by the GSA guidelines was checked by comparing the nonlinear static and dynamic analysis results.

2. DESIGN OF ANALYSIS MODEL STRUCTURES

The analysis model structures are 36-storey buildings with 36×36 m square plan shape. The structures are composed of reinforce concrete core walls and exterior frames connected together by outrigger trusses at the top. Two types of perimeter frames were designed: (i) perimeter frames with mega-columns (Figure 1(a), Prototype I); and (ii) perimeter frames with ordinary columns tied up by belt trusses at the top storey (Figure 1(b), Prototype II). In Prototype II structure the belt trusses function as a horizontal stiffener of exterior frames by mobilizing other exterior columns to take part in restraining the rotation of the core walls and outrigger trusses. Once an exterior column is accidentally removed the gravity load supported by the removed column is redistributed to adjacent columns by belt truss.

The analysis model structures were designed with dead and live loads of 4.5 and 2.5 kN/cm², respectively. The design seismic load was computed using the design spectral response acceleration parameters S_{DS} and S_{D1} equal to 0.37 and 0.14, respectively, in the International Building Code (2006)
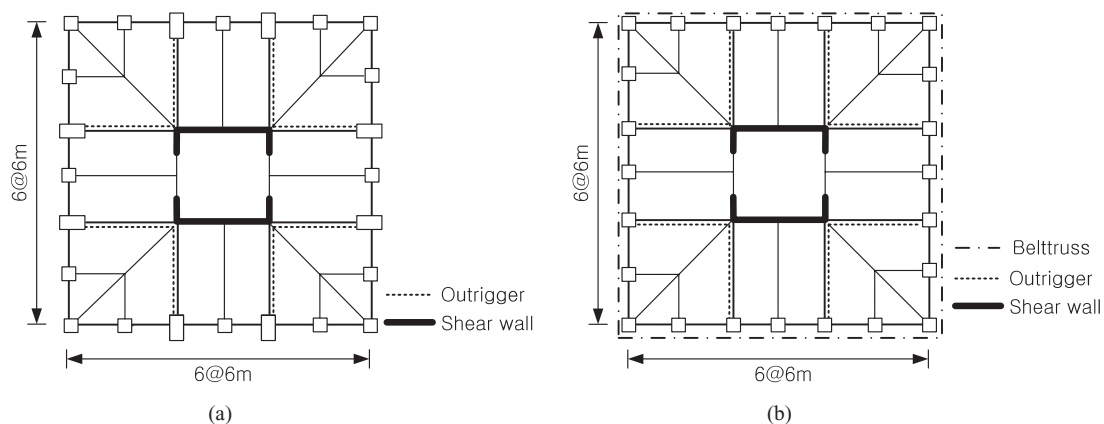

Figure 1. Structural plan of analysis model structures with outrigger trusses: (a) structure with mega-columns and (b) structure with belt trusses.
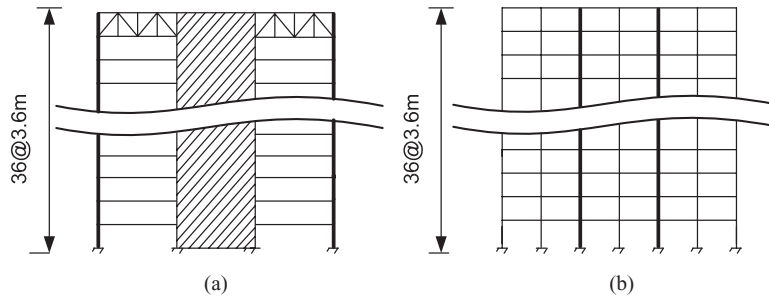
Figure 2. Side view of the structure with mega-columns and outrigger trusses: (a) interior view and (b) exterior view.

format. As enough ductility may not be expected from this type of structure, response modification factor of 3.0 was used in the computation of design seismic load. Figure 2 shows the side view of the structure with mega-columns and outrigger trusses. The mega-columns, two at each side and connected to core walls through outrigger trusses, were designed with concrete-filled steel tube (CFT) columns $1200 \times 600 \times 30$ mm in size having steel yield strength and concrete ultimate strength of 33.0 and 3.5 kN/cm^2, respectively. The other perimeter columns, made of H-400 $\times$ 400 wide flange sections with yield strength of 33 kN/cm^2, were designed only for gravity loads. In the Prototype II structure with outrigger and belt trusses, the exterior columns were designed with H-400 $\times$ 400 sections with their thickness varying along the height. All girders and outrigger trusses were designed with wide flange sections with yield strength of 24 kN/cm^2. The reinforcing bars have ultimate strength of 40.0 kN/cm^2. The columns and exterior girders were designed to have the same size for three consecutive stories. The same members were used for interior girders in all stories. In the Prototype I and II structures all girders are pin connected to columns and to core walls and lateral loads are resisted by the rigidity of the core walls plus the interaction between the core walls and exterior columns.

3. COLLAPSE ANALYSIS OF MODEL STRUCTURES

3.1. Nonlinear static pushdown analysis of the model structures

Nonlinear static Pushdown analysis was carried out first on the model structure with outriggers and mega-columns (Prototype I) with one of the first-storey mega-columns removed using the program code SAP2000 (2004). The procedure accounts for nonlinear effects without sophisticated hysteretic material modelling and is useful in determining elastic and failure limits of the structure. The US GSA (2003) and the US DoD (2005) guidelines proposed the amplification factor of 2 for the static analysis to account for dynamic redistribution of forces. The load combination of the US GSA (2003) for static analysis is 2 (Dead Load + 0.25 × Live Load) and that of the DoD (2005) is 2 (1.2 × Dead Load + 0.5 × Live Load) + 0.2 × Wind Load. In this study, the load combination of the US GSA (2003) was selected for pushdown analysis. This amplified load was applied only in the spans from which a column was removed while unamplified load was applied in the other spans as illustrated in Figure 3.

In this study push-down analysis was applied by gradually increasing the vertical displacement in the location of the removed column to investigate the resistance of the structure against such deformation. Since this procedure is displacement controlled, there is little chance to diverge. At every step during the push-down analysis (i.e. at each level of the vertical displacement), the amount of equivalent load corresponding to the displacement level was determined. The ratio of the applied load and the GSA-specified load of 2 (Dead Load + 0.25 × Live Load) is referred to as the 'load factor'. The original loading pattern remained unchanged at every step.

As all girders, both interior and exterior, are pin-connected to the mega-column, the outrigger truss located at the top of the structure resists all pushdown force. Figure 4 shows the pushdown curve of
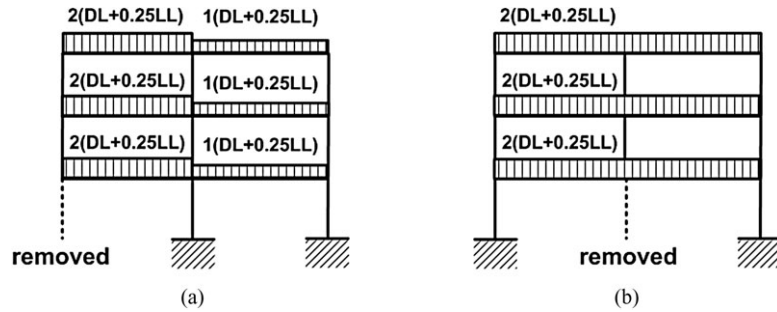
Figure 3. Application of vertical load for nonlinear static analysis: (a) removal of an exterior column and (b) removal of an interior column.
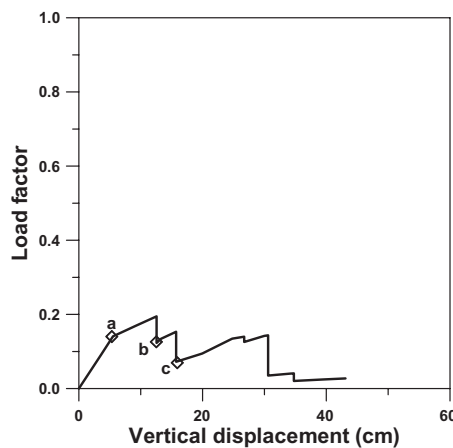

Figure 4. Pushdown curve of the model structure with mega-columns.

the Prototype I structure, where it can be observed that the maximum strength barely reached 20% of the applied load, 2 (Dead Load + 0.25 × Live Load), and the structure has high potential for progressive collapse. Figure 5 shows the yielded or buckled members in the outrigger truss at three levels of vertical displacements. At point 'a' marked on the pushdown curve shown in Figure 4, the upper chord of the outrigger truss connected to the core wall, also marked 'a' in Figure 5, yielded, which caused slight reduction in stiffness as can be observed in the pushdown curve. The strength dropped first at the vertical displacement of 12.5 cm (point 'b') when the upper chord of the truss connected to the mega-column, marked 'b' in Figure 5, buckled. The strength dropped again at point 'c' in Figure 4 when buckling occurred at the lower chord connected to the core wall (the member 'c' in Figure 5).

In the Prototype II structure, structure with outrigger and belt trusses, the exterior frames are composed of uniformly spaced columns tied together at the top by belt trusses. Figure 6 shows the pushdown curve of the Prototype II structure, and Figures 7 and 8 depict the locations of yielded or buckled members when an exterior column and a corner column were removed, respectively. It can be noticed that when an exterior column was removed the ultimate strength of the structure reached almost 1.0. This implies that the structure may be able to resist the gravity load imposed by loss of an exterior column. When an exterior column was removed, the strength dropped rapidly after failure of some members in outrigger trusses. The location of the missing column is shown in Figure 7(a). The strength first dropped at point 'a' in Figure 6 when the upper and the lower chords of the outrigger trusses connected to the core wall yielded and buckled, respectively, and one of the two exterior columns adjacent to the removed column buckled. The strength further decreased as more members in the
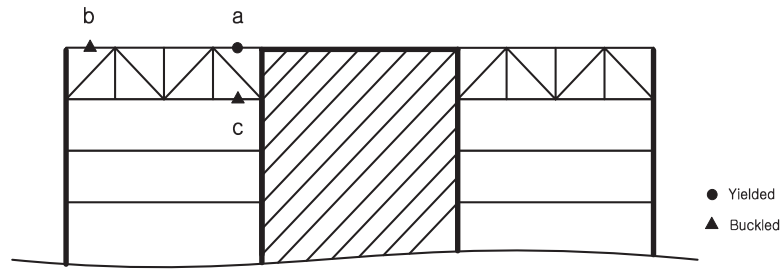
Figure 5. Locations of yielded or buckled members in model structure with mega-columns.
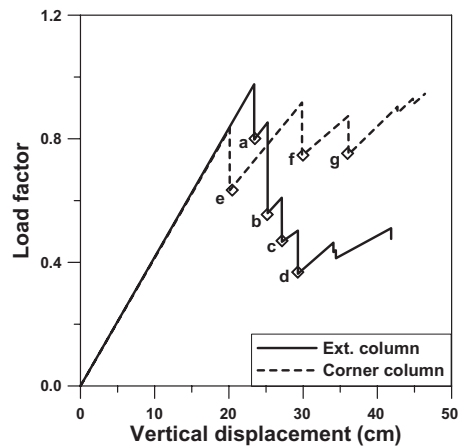

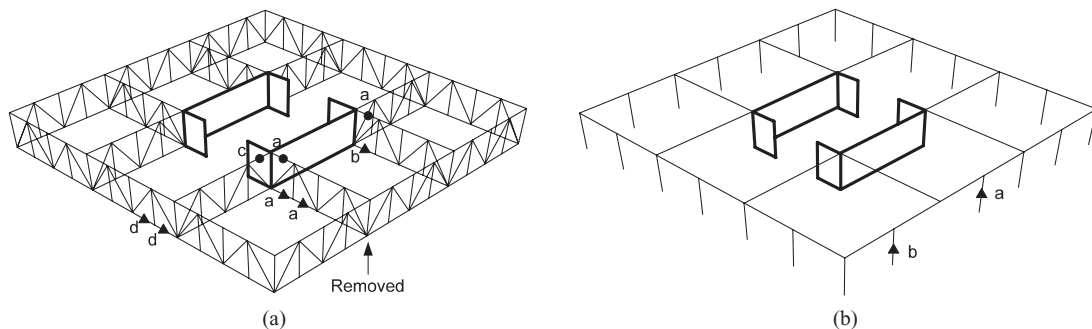Figure 6. Pushdown curve of the model structure with belt trusses.

Figure 7. Yielded or buckled members in Prototype II structure when an exterior column was removed: (a) at outrigger and belt trusses and (b) at first-storey columns.

outrigger trusses failed and the other adjacent column buckled (point 'b' and 'c'). At point 'd' two lower chords of the belt truss buckled. With further increase of the vertical displacement the strength increased again when both lower and upper chords began to be subjected to tension. The locations of yielded or buckled members in the outrigger and belt trusses corresponding to each displacement step are shown in Figure 7(a). Figure 7(b) shows the location of buckled columns at loading step 'a' and 'b' in Figure 6.

When a corner column was removed from the Prototype II structure, the maximum strength turned out to be slightly less than when an exterior column was removed as can be observed in the dotted pushdown curve shown in Figure 6. However in this case the structure deformed symmetrically and
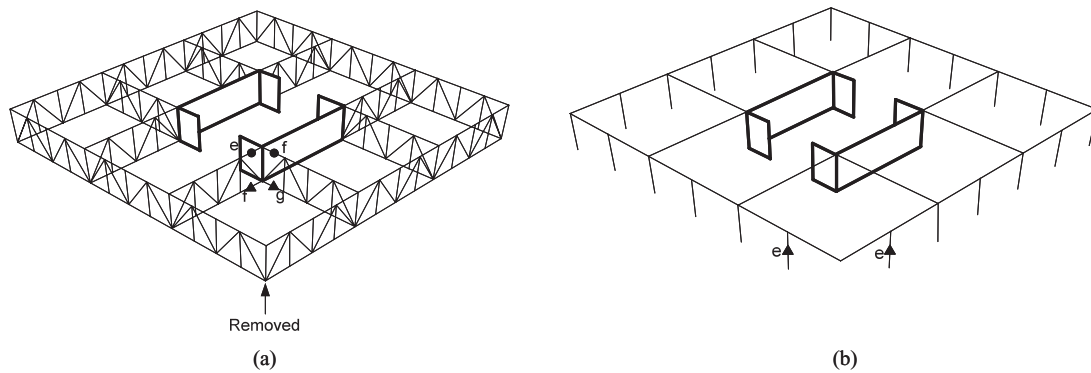
(a) (b)

Figure 8. Yielded or buckled members in Prototype II structure when a corner column was removed: (a) at outrigger and belt trusses and (b) at first-storey columns.

showed more ductile behaviour. At point 'e' in the pushdown curve the strength dropped rapidly after the two adjacent columns buckled and an upper chord of the outrigger truss yielded as shown in Figure 8. The strength increased again until other members in the outrigger trusses failed at point 'f' in the pushover curve. At point 'g' the strength dropped again due to buckling of a lower chord of the outrigger truss.

In comparison with the model structure with mega-columns (Prototype I) in which the yielded or buckled members are confined to the outrigger truss directly connected to the column line with a missing first-storey column, the failed members in Prototype II structure are more widely distributed to adjacent outrigger trusses and belt trusses. This indicates that in the structure with belt trusses around the building perimeter the influence of column removal is not confined to the frame to which the removed column belongs, but spreads to wider range of the building perimeter. This leads to the much higher overall strength of the Prototype II structure than that of the Prototype I structure.

3.2. Nonlinear dynamic analysis of the model structures

Nonlinear dynamic analyses of the model structures were carried out using the program code SAP 2000 with the same first-storey column suddenly removed. To carry out dynamic analysis, hysteretic behaviours of plastic hinges were defined based on Federal Emergency Management Agency and American Society of Civil Engineers (2000). The Jain–Goel model (BSSC, 1997) was used to define the behaviour of braces. For nonlinear dynamic analysis the load DL + 0.25 LL was uniformly applied in the entire spans. In order to carry out dynamic analysis the member forces of a column, which is to be removed to initiate progressive collapse, were computed before it is removed. Then the column was replaced by point loads equivalent of its member forces. In order to simulate the phenomenon that the column was abruptly removed, the member forces were suddenly removed a few seconds after their application while the applied load remained unchanged.

Figure 9 depicts the displacement time histories of the model structures caused by sudden removal of a mega-column. It can be observed that right after one of the mega-columns was removed from the Prototype I structure the vertical displacement monotonically increased until failure. This implies that progressive collapse occurred due to sudden removal of a mega-column. However, in the structure with belt trusses (Prototype II) the vertical displacement reached maximum value of 25.1 cm, and after a few oscillations the structure became stable at the vertical displacement of 15.0 cm. Figure 10 shows the locations of yielded or buckled members in the Prototype I structure obtained by dynamic analysis at vertical displacement of 21.3 cm. Comparing with Figure 5 it can be observed that the locations of yielded or buckled members obtained by dynamic analysis are similar to those obtained by pushdown analysis. The same phenomenon was observed in the Prototype II structure. Figure 11 shows the locations of yielded or buckled members obtained from both static and dynamic analyses at the vertical displacement of 25.3 cm.
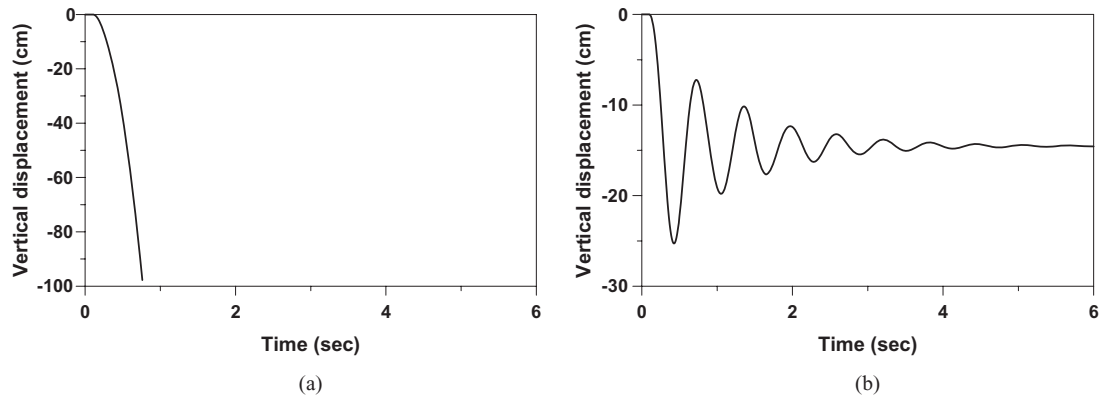
Figure 9. Vertical displacement time histories of model structures: (a) structure with outrigger and mega-column and (b) structure with outrigger and belt truss.
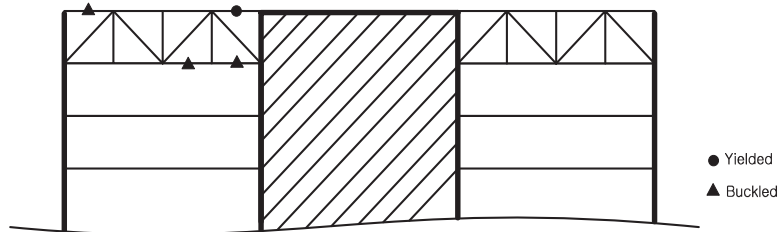

Figure 10. Locations of yielded or buckled members in model structure with mega-columns obtained by dynamic analysis at vertical displacement of 21.3 cm.
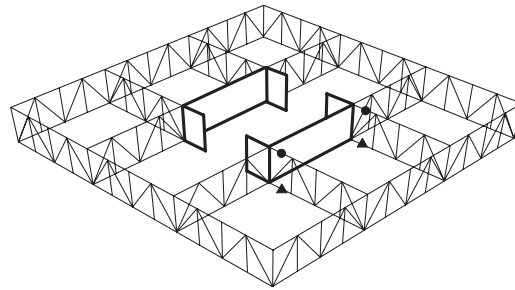

Figure 11. Locations of yielded or buckled members in Prototype II structure at vertical displacement of 25.3 cm obtained from both static and dynamic analyses.

4. PERFORMANCE OF REDESIGNED STRUCTURES

It was observed in the previous section that the Prototype I structure with core walls and mega-columns connected by outrigger trusses failed progressively after removal of one of the mega-columns. As the exterior frames of this model structure are pin-connected, only the outrigger truss connected to the missing mega-column resists against progressive collapse. Both the static and dynamic analysis results showed that the outrigger trusses, designed per current design codes, might not have enough strength to resist progressive collapse initiated by sudden loss of a column.

In this study three alternative design schemes for preventing progressive collapse were investigated: (i) use of two-storey outrigger trusses; (ii) use of rigid girder-column connections in the
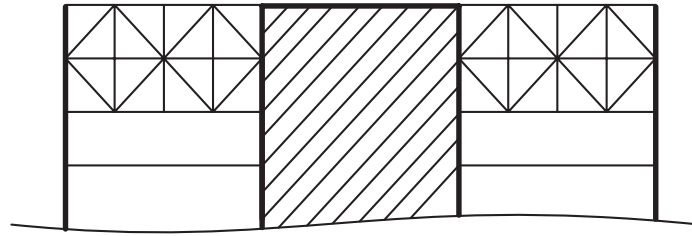
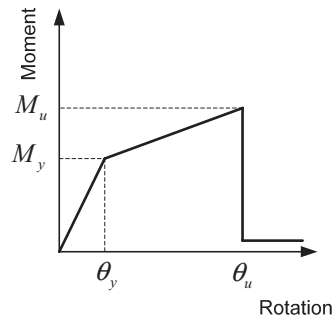Figure 12. Redesigned structure with two-storey outrigger trusses.

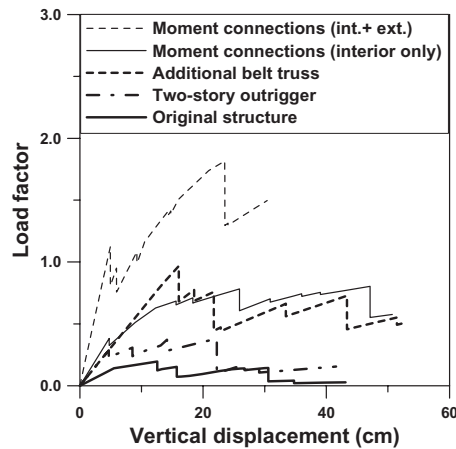Figure 13. Bending moment–rotation relationship of a rigidly connected girder end.

Figure 14. Pushdown curves of the redesigned structures.

interior and exterior frames; and (iii) addition of belt trusses at the top storey. They were redesigned using the same design loads as used to design the original model structure. Figure 12 illustrates the redesigned structure with two-storey-deep outrigger trusses. In the schemes with moment-connected girder-column connections, the bending moment-girder end rotation relationship shown in Figure 13 was used using the ultimate rotation angle and the post-yield stiffness ratio of 0.035 and 0.02, respectively.

Figure 14 depicts the pushdown curves of the redesigned structures. Even though all the redesigned structures showed higher strength than the original Prototype I structure, the strength of the structure with two-storey high outrigger trusses was not high enough to prevent progressive collapse. The structure with moment connected girder-column connections showed the highest strength well above the load factor of 1.0. The strength of the structure with belt trusses in addition to the outrigger trusses
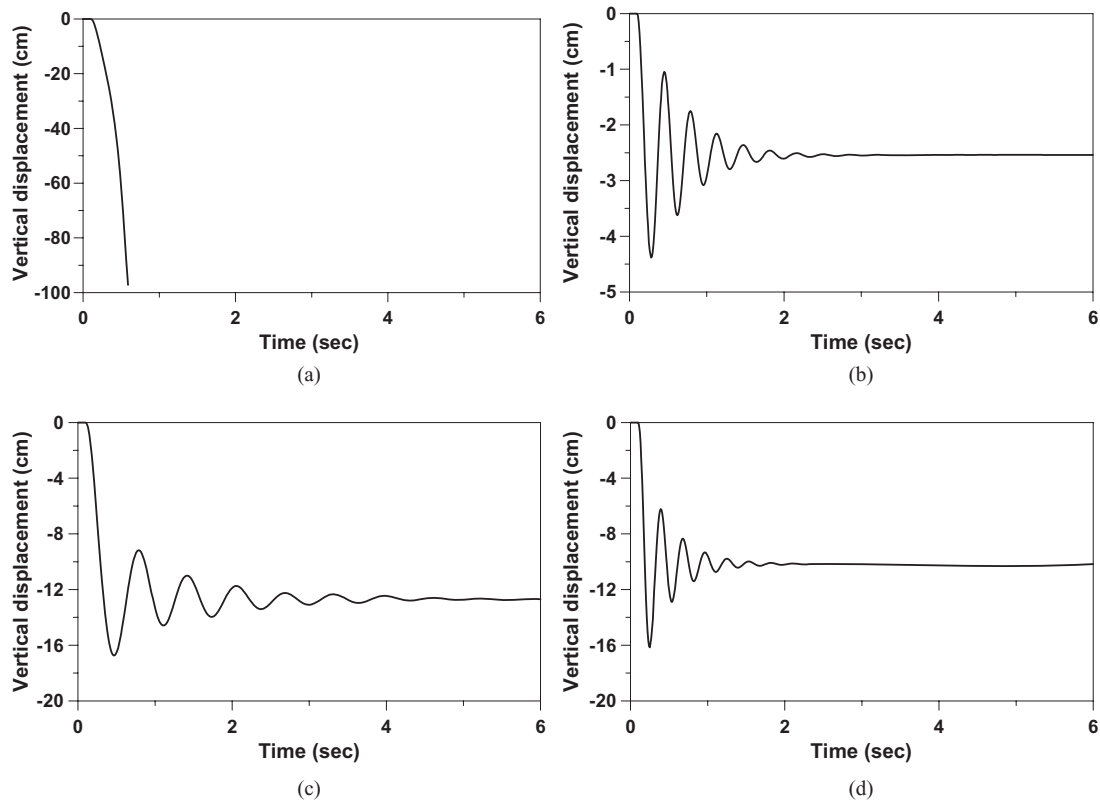
Figure 15. Time histories of vertical displacements: (a) two-storey outrigger trusses;
(b) moment connection of interior/exterior frames; (c) moment connection of interior frames;
and (d) additional belt trusses.

reached almost 1.0, which implies that the structure can sustain the specified gravity load of 2.0 (DL + 0.25 LL) without collapse. It can be noticed that the structure behaved elastically until the maximum strength was reached. The structure with moment connected interior frames reached 0.8 and is considered to have a good chance to remain stable after a mega-column is removed. The turned out to be true through dynamic analysis.

Figure 15(a) shows the time history of vertical displacement caused by sudden removal of a mega-column from the structure with two levels of outrigger trusses depicted in Figure 12. It was observed that as soon as the column was removed member failure of outrigger truss followed and the structure failed without oscillation. The structures redesigned with moment connected interior and exterior frames showed stable behaviour with small vertical displacement as shown in Figure 15(b). Figure 15(c) and (d) shows that the structure with moment-connected interior frames and the structure with additional belt trusses also remained stable after one of the mega-columns was removed. In comparison with the pushdown analysis results shown in Figure 14, it can be observed that the structure with maximum load factor less than 0.5, the structure with two-storey outrigger trusses, failed in dynamic analysis. However the structures with maximum load factor larger than 0.7 remained stable after removal of a mega-column. It was also observed that both the static and the dynamic analyses results in similar yielding or buckling pattern in the outrigger and belt trusses. Therefore based on the analysis results obtained in this study, it can be concluded that the US GSA and the US DoD recommended dynamic amplification factor of 2 for the static analysis leads to reasonably accurate but somewhat conservative results.

Table 1 compares the maximum displacements obtained from dynamic analyses with those computed from pushdown analyses at load factor of 0.5 (i.e. displacements under the static load of DL + 0.25 LL are compared with those under the same load applied dynamically) when an exterior

Table 1. Comparison of vertical displacements obtained from dynamic and static analysis at load factor of 0.5.

Model structures	Pushdown analysis (cm)	Dynamic analysis (cm)	Displacement ratio
Prototype II	7.9	16.2	2.05
Prototype I + int. moment connections	8.4	16.7	1.99
Prototype I + int./ext. moment connection	2.7	4.4	1.63
Prototype I + belt trusses	12.1	25.1	2.07

Table 2. Comparison of load factors at the same static/dynamic vertical displacements.

Model structures	Static analysis		Dynamic analysis	
Model structures	Disp. (cm)	Load factor	Disp. (cm)	Load factor
Prototype II	16.2	0.69	16.2	1
Prototype I + interior moment connections	16.7	0.68	16.7	1
Prototype I + int./ext. moment connections	4.4	1.07	4.4	1
Prototype I + belt trusses	25.1	0.85	25.1	1

column was removed. As the maximum strengths of the Prototype I structure and the redesigned structure with two-storey outriggers did not reach 0.5 as results of pushdown analyses, they were not included in the comparison. It can be observed that in most cases the maximum displacements obtained from dynamic analyses are approximately twice as large as those obtained from static analyses at the same loading level. This confirms the validity of the dynamic amplification factor of 2.0 recommended in the guidelines, especially when a structure behaves nearly elastically under the specified loading condition. However, when structures are subjected to large inelastic deformation under the specified gravity load, the amplification factor of 2.0 may not be valid. Table 2 shows the load factors of static analyses when the displacements obtained by static analyses reached the maximum displacements obtained from dynamic analyses. Except for the structure redesigned with interior and exterior moment connections, the maximum load factors did not reach 1.0; i.e. the imposed loads on the structures were less than the specified load of 2.0 (DL + 0.25 LL). This implies that the specified amplification factor of 2.0 generally results in conservative results. That is to say, if a structure can remain stable under the gravity load of 2.0 (DL + 0.25 LL) applied statically on the bays adjacent to the removed column, there is a fairly good chance that the structure will remain safe against progressive collapse caused by sudden removal of a column. This statement can be confirmed by comparing the pushdown curve (Figure 14) and the time history analysis result (Figure 15(b)) of the model structure with moment-connected interior frames, where it can be observed that even though the maximum load factor of the pushdown curve did not reach 1.0 the structure remained stable under dynamic load. Such an observation corresponds well with the finding of Powell (2005).

Figure 16 depicts the bending moment diagrams and the ratio of the bending moments obtained by dynamic and static analyses under the loading of DL + 0.25 LL when an exterior column was removed from the first storey of the Prototype I structure redesigned with both the interior and exterior moment frames. It can be observed from Figure 16(a) that the bending moment is largest in the girders right above the missing column and keeps decreasing until no bending moment occurs in the girders located above the 11th storey due to the redundancy of the structure. Figure 16(b) shows that the dynamic amplification factors ranges from 1.60 to 1.76, which are less than the guideline-recommended value of 2.0. This also verifies the previous observation that the recommended amplification factor may be relevant to guarantee safety against progressive collapse caused by loss of a first-storey column.
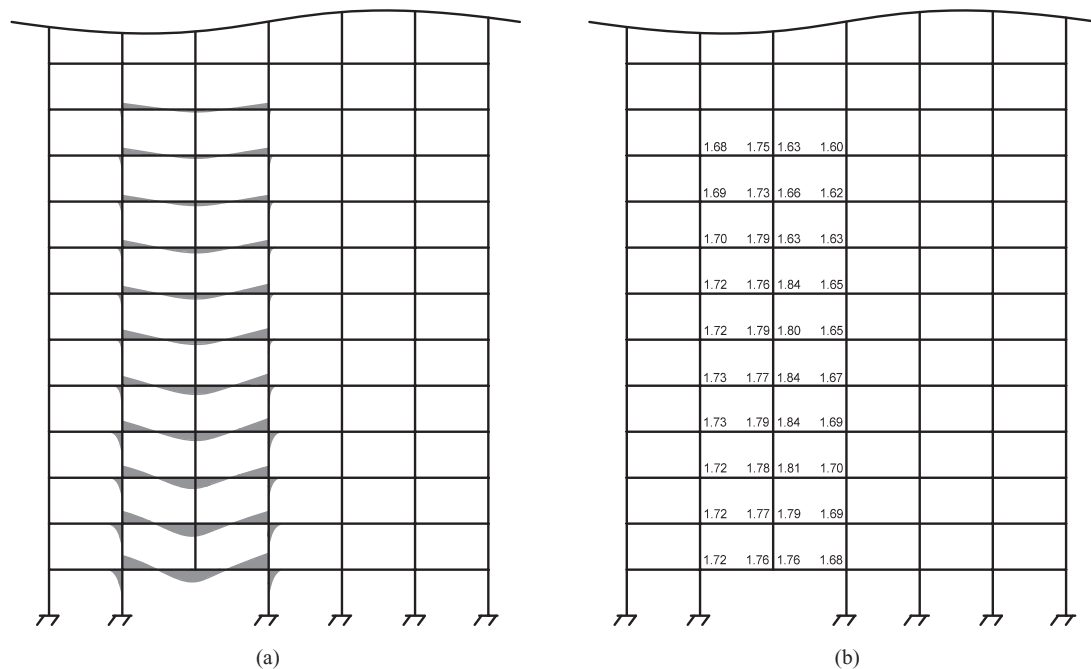
Figure 16. Dynamic amplification factors in Prototype Structure I with interior and exterior moment frames: (a) bending moment diagram and (b) dynamic amplification factors.

5. CONCLUSIONS

This study investigated the progressive collapse potential of 36-storey building structures with RC core walls and outrigger trusses as a major lateral load-resisting system. Two types of perimeter frames were designed: perimeter frames with mega-columns and perimeter frames with belt trusses at the top storey.

The static pushdown analysis of the structure with mega-columns and outrigger trusses showed that the maximum strength reached only about 20% of the load specified in the GSA guideline when a mega-column was removed. The dynamic analysis showed that the vertical displacement monotonically increased until collapse when a mega-column was suddenly removed. However, the structure with outrigger and belt trusses remained stable after a perimeter column was removed. In this case the maximum load factor obtained from pushdown analysis reached almost 1.0.

The progressive collapse resisting capacity of the structure with mega-columns and core walls connected by outrigger trusses could be enhanced by providing additional redundancy to the key elements such as mega-columns. It was observed that redesigning the structure with additional belt trusses or with moment connected interior/exterior frames significantly enhanced robustness of the structure. Moreover, based on the comparison of static and dynamic analysis results, it was concluded that the dynamic the amplification factor of 2.0 recommended in the guidelines provided reasonably conservative results.

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