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PROGRESSIVE COLLAPSE RESISTING CAPACITY OF TUBE-TYPE STRUCTURES

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SUMMARY

In this study, the progressive collapse potential of tube-type buildings, such as diagrid and tubular structures, composed of lateral load-resisting perimeter frames and internal pin-connected gravity frames, was evaluated by nonlinear static and dynamic analyses. To this end, 36- and 54-storey structures were designed as analysis models and progressive collapse analyses were carried out by removing first-storey columns. According to the analysis results, the progressive collapse of tube-type analysis model buildings occurred when perimeter columns corresponding to more than 11% of all member cross-sectional areas were removed from one side of the structures. When the diagonals located around a corner were removed, the ratio was reduced to 8%. It was observed that the corner columns in the diagrid system helped prevent the propagation of member failure all around the perimeter. Copyright © 2009 John Wiley & Sons, Ltd.

1. INTRODUCTION

Tubular structures have been widely used as efficient structural systems for tall buildings. Many supertall buildings, including the 110-storey Sears Tower and the 100-storey John Hancock Building in Chicago were built using tubular systems. Typical tubular structures are composed of perimeter moment-resisting frames that resist all the lateral loads and internal frames supporting only gravity loads. Idealized tubular structures will behave as a hollow tube responding predominantly in a bending mode of lateral deformation. Since the distance between the lateral load-resisting elements is maximized, the lateral load-resisting capacity of tubular structures is superior to typical moment-resisting frames. The tubular behaviour also allows considerable freedom in the architectural planning of interior space. Tubular behaviour is achieved by placing external columns 1 to 3 m to as much as 6 m apart, with the depth of spandrel girders varying from 1·0 to 1·5 m (Taranath, 1998). Further improvement in the lateral load-resisting capacity can be obtained by using diagonal members in the exterior moment frames to induce truss action in the perimeter of the structure.

The structural efficiency of a box-type tubular structure decreases by shear lag effect, which is the nonlinear distribution of stress across the building sides. In recent years, some high-rise buildings, such as the Swiss Re Building in London and the Hearst Tower in New York, have been constructed with triangulated exterior members, which are known as diagrid systems. It is a particular form of space truss mixed with the tubular system, and the diagonal grid makes the structure stable even without any vertical column around 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).

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The progressive collapse refers to the phenomenon that local damage of structural elements caused by abnormal loads results in the global collapse of the structure. An abnormal load includes any loading condition that is not considered in the normal course of design but may cause significant damage to structures. To prevent the progressive collapse caused by abnormal loads, the National Building Code of Canada (1996) specified requirements for design of major elements, establishment of connection elements and ways of providing load transfer paths. The Eurocode 1 (2002) presented a design standard for selecting plan types for preventing progressive collapse, and recommended that buildings should be integrated. In the USA, the General Service Administration (GSA) presented a practical guideline for design to reduce the collapse potential of federal buildings (GSA, 2003), and the Department of Defence (DoD) also presented a guideline for the new and existing DoD buildings (DoD, 2005). The analysis method recommended in these guidelines is the alternative path method. In 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 the loss of any one column without collapsing. For static analysis, those guidelines recommend a dynamic amplification factor of 2·0 in the load combination to the bays adjacent to the removed element.

Previous research on tube-type structures generally focused on wind-induced responses (Connor and Pouangaree, 1992; Koran, 1994), shear lag effect (Leonard, 2007), simplified design methodology (Moon, 2007) and inelastic seismic performance (Kim *et al.*, 2008). Also, research effort on the progressive collapse of steel structures has been focused on the performance of moment frames (Khandelwal and El-Tawil, 2008; Kim and An, 2008; Kim and Park, 2008). However, no research has been conducted yet to investigate the progressive collapse potential of tube-type structures.

In this paper, the progressive collapse potential of tube-type structures was evaluated using nonlinear static and dynamic analyses. To this end, 36- and 54-storey framed-tube and diagrid structures with the same plan shapes and storey heights were prepared. The validity of the dynamic amplification factor of 2·0 recommended by the GSA guidelines was investigated first by comparing the analysis results obtained from nonlinear static and dynamic procedures.

2. DESIGN OF MODEL STRUCTURES AND ANALYSIS MODELLING

The analysis models are the 36- and 54-storey diagrid and tubular structures shown in Figure 1. The structures have square plans symmetric in two perpendicular axes. Two types of diagrid structures were designed: structures with and without corner columns. According to Moon (2007), the diagrid with external braces having 65° to 75° slope is the most efficient in resisting lateral load. Based on his research, the external braces in the diagrid model structures, which were spaced 6 m, were designed to have 67° slope. The tube structures were designed to have deep spandrel girders and 3·6-m spaced external columns. Every model structure has the uniform story height of 3·6 m. The interior frames of both the diagrid and tubular structures were designed only for gravity load, and thus were pin-connected. The design dead and live loads were 4·0 kN/m^2 and 2·5 kN/m^2, respectively, and the design wind load was computed based on the basic wind speed of 30 m/s in an exposure *A* area. The design seismic load was computed based on the seismic coefficients of S_{DS} = 0·37 and S_{D1} = 0·15, with response modification factor of 3·0 in the IBC 2006 format. In all model structures, columns were made of SM490 (F_y = 325 N/mm^2) steel, and girders and braces were made of SS400 (F_y = 245 N/mm^2) steel.

For nonlinear analysis, the beams and columns were modelled by beam elements and the braces were modelled by truss elements. The nonlinear behaviour of structural members is depicted in Figure 2, which is suggested in the Federal Emergency Management Agency (FEMA)-356 (FEMA, 2000), where P_y is the yield strength, θ is the rotation angle and Δ is the displacement. For braces, the

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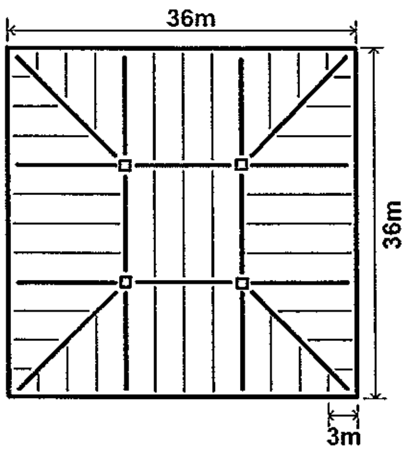
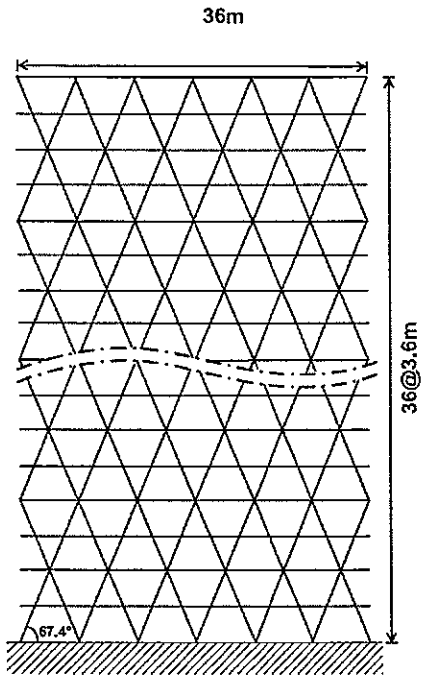
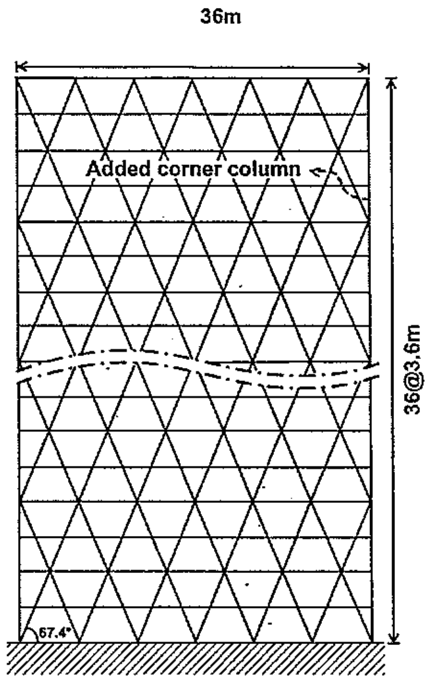


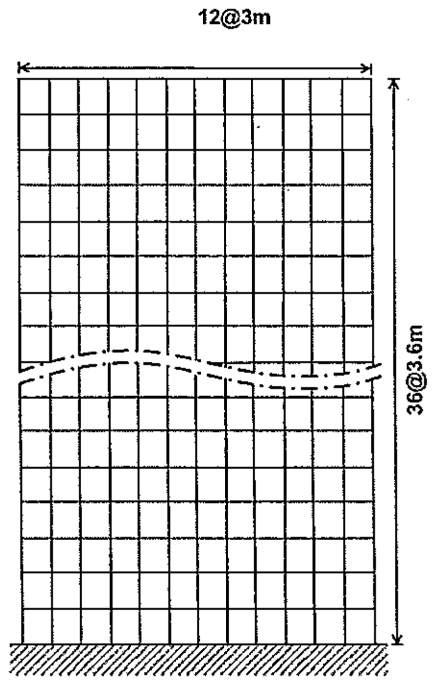
(a) Structural plan



(b) Elevation of diagrid without corner columns



(c) Elevation of diagrid with corner columns



(d) Elevation of tubular structure

Figure 1. Structural plan and elevations of 36-storey model structures

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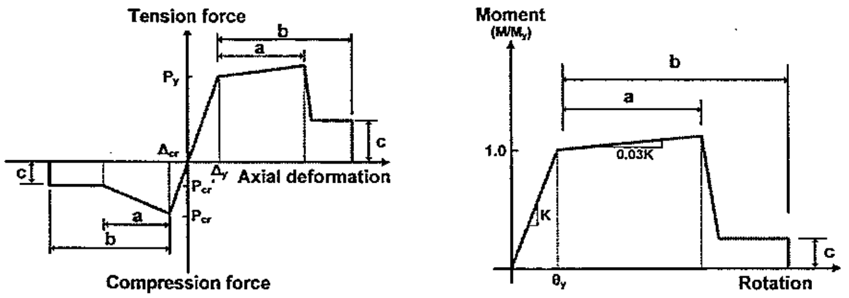

(a) Braces (b) Bending members

Figure 2. Force–deformation relationships of structural members

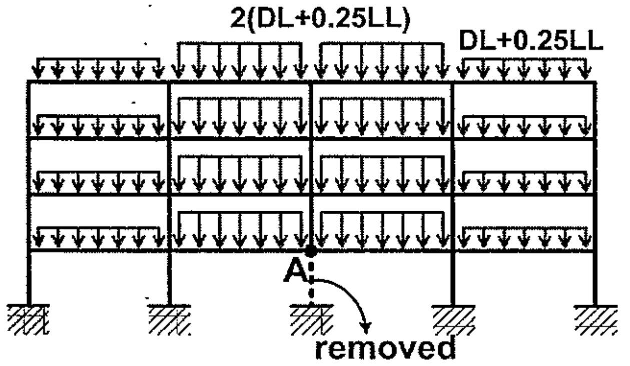

Figure 3. Applied gravity load for nonlinear static analysis (GSA guidelines)

post-buckling strength (P'_{cr}) was determined to be 40% or 20% of the buckling strength (P_{cr}), depending on the width–thickness ratio.

3. ANALYSIS PROCEDURES AND DYNAMIC AMPLIFICATION FACTORS FOR STATIC ANALYSIS

Progressive collapse is generally initiated by the sudden loss of one, or many, structural members. The load combination of the GSA2003 for static analysis is 2 (Dead Load + 0·25 × Live Load). This amplified load is applied only in the spans in which a column is removed, while unamplified load is applied in the other spans (Figure 3). For dynamic analysis, the amplification factor is not applied, and the load combination (Dead Load + 0·25 × Live Load) is imposed in every span (Figure 4). In this study, the displacement-controlled pushdown analyses and nonlinear dynamic analyses were carried out using the program code SAP2000 (SAP2000, 2004) to investigate the progressive collapse potential of the model structures.

The advantage of the pushdown procedure is its ability to account for nonlinear effects without sophisticated hysteretic material modelling and time-consuming time-history analysis. The disadvantage is the inability to consider dynamic effects caused by sudden removal of columns. At each analysis step of pushdown analysis, the amount of equivalent load corresponding to the given displace-

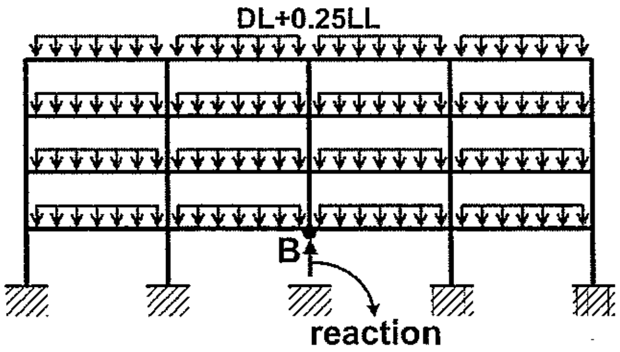

Figure 4. Applied gravity load for nonlinear dynamic analysis (GSA guidelines)

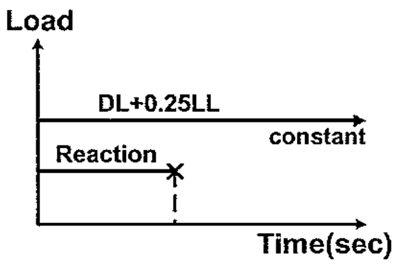

Figure 5. Time history of applied loads for dynamic analysis

ment level was determined and was imposed on all beams with its original loading pattern unchanged. The displacement level was gradually increased until the structure became unstable. The original loading pattern remained unchanged at every analysis step. The computed load level at each step is referred to as the 'load factor', which is the ratio of the equivalent load at that analysis step to the full gravity load specified in the guidelines.

For dynamic analysis, all member forces were obtained first from full structural models subjected to the applied load. The structure was then remodelled with some of the first-storey columns removed and with member forces of the lost columns applied to the structure as lamp forces to maintain equilibrium (Figure 5). In this way, the progressive collapse analyses started when the structures were already deformed by the applied load. In the dynamic analysis, the inherent viscous damping ratio was assumed to be 5%.

The GSA2003 and the DoD2005 guidelines proposed the amplification factor of 2 for static analysis to account for dynamic redistribution of forces. Powell (2005) compared the static and dynamic analysis methods and found that the amplification factor of 2·0 regulated in the static analysis can display a conservative result. Ruth *et al.* (2006) found that a factor of 1·5 better represents the dynamic effect, especially for steel moment frames.

In a diagrid structure in which gravity load-resisting elements are not oriented parallel with the direction of the gravity load, the load-transfer mechanism will be more complicated. Also, in a tubular structure with closely spaced columns and deep spandrel girders, the load path for gravity load may be different from that of typical moment-resisting frames. Besides, the dynamic amplification factor recommended by the guidelines is recommended to be applied to low- to medium-rise structures. Therefore, the dynamic amplification of vertical load in tall tube-type structures may differ from that of a low-rise moment frame.

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Figure 6 shows the ratios of the maximum member forces obtained by linear dynamic and static analyses of: (a) a four-storey moment-resisting frame with a centre column removed; (b) a 36-storey diagrid structure with a pair of braces removed from the centre of a side; (c) a 36-storey diagrid structure with three pairs of braces removed from the centre of a side; and (d) a 36-storey tubular structure with five columns removed from the centre of a side. It can be observed that in the four-storey moment-resisting frame, the amplification of member force exceeding 2·0 occurred only in the beam ends connected to the columns located right above the lost column. When a pair of first-storey braces was removed from the diagrid system, a large amplification of member force occurred only at the two braces in the second storey right above the lost braces. In the other members, the amplification factors were less than 1·14. When three pairs of first-storey braces were removed from the same diagrid structure, the dynamic amplification factors of the six braces connected to the removed braces in the second storey reached 1·56 to 2·32. At above the fifth storey, the amplification factors reduced to less than 1·26. The analysis results indicate that the dynamic amplification of member force as large as 2·0 is limited to a few members located nearby the lost members. Therefore, to double the gravity load in every storey will result in a too conservative performance evaluation of the model structures. In addition, it is not clear on which bays the amplified loads should be applied, especially in diagrid structures with slanted vertical members. In this case, the impact is spread to many adjacent bays when a member is suddenly removed. Based on the above observation, it was concluded that to apply the amplification factor of 2·0 in all beams located above the lost column might not be reasonable in the progressive collapse analysis of high-rise diagrid and tubular structures. Therefore, in this study, uniform gravity load was applied in all spans without considering dynamic amplification of load. In this case, the static analysis results may be slightly non-conservative.

4. PROGRESSIVE COLLAPSE POTENTIAL OF MODEL STRUCTURES

Collapse analyses of the model structures were carried out using the program code SAP2000. The collapse of structures was initiated by removing a few vertical members located in the first storey. Figure 7 shows the braces and columns removed from the model structures. Nonlinear static and dynamic analyses were carried out to investigate the progressive collapse potential of the model structures.

4.1 Performance of the diagrid structures

Figures 8–10 plot the pushdown curves and vertical displacement time histories of the 36-storey diagrid model structures with one, three and five pairs of external columns removed, respectively. The horizontal axis of the figures represents the vertical deflection of the beam-removed column joint, and the vertical axis represents the load factor, which is the applied load divided by the specified gravity load combination. A load factor larger than 1·0 implies that the structure remained stable after the columns were removed, and therefore, progressive collapse did not occur. It was observed that most structural members remained elastic until the vertical load reached its maximum value. When the maximum load was reached, buckling occurred in some diagonal members, and the structure failed by progressive collapse. The figures show that as the number of removed braces increased, the maximum load factor decreased. When five pairs of external braces were removed from the centre of a side, the maximum load factor of the diagrid structure was slightly greater than 1·0. It was observed that the removal of more than five pairs of first-storey braces lead to the progressive collapse of the structure. The time histories of vertical displacement obtained by nonlinear dynamic analysis confirmed that the structure remained stable after the removal of the five pairs of braces. This confirms the validity of using a dynamic amplification factor of 1·0 in all spans. In the figures, the dotted lines represent the vertical displacements obtained by static analysis. It can be observed that when one or

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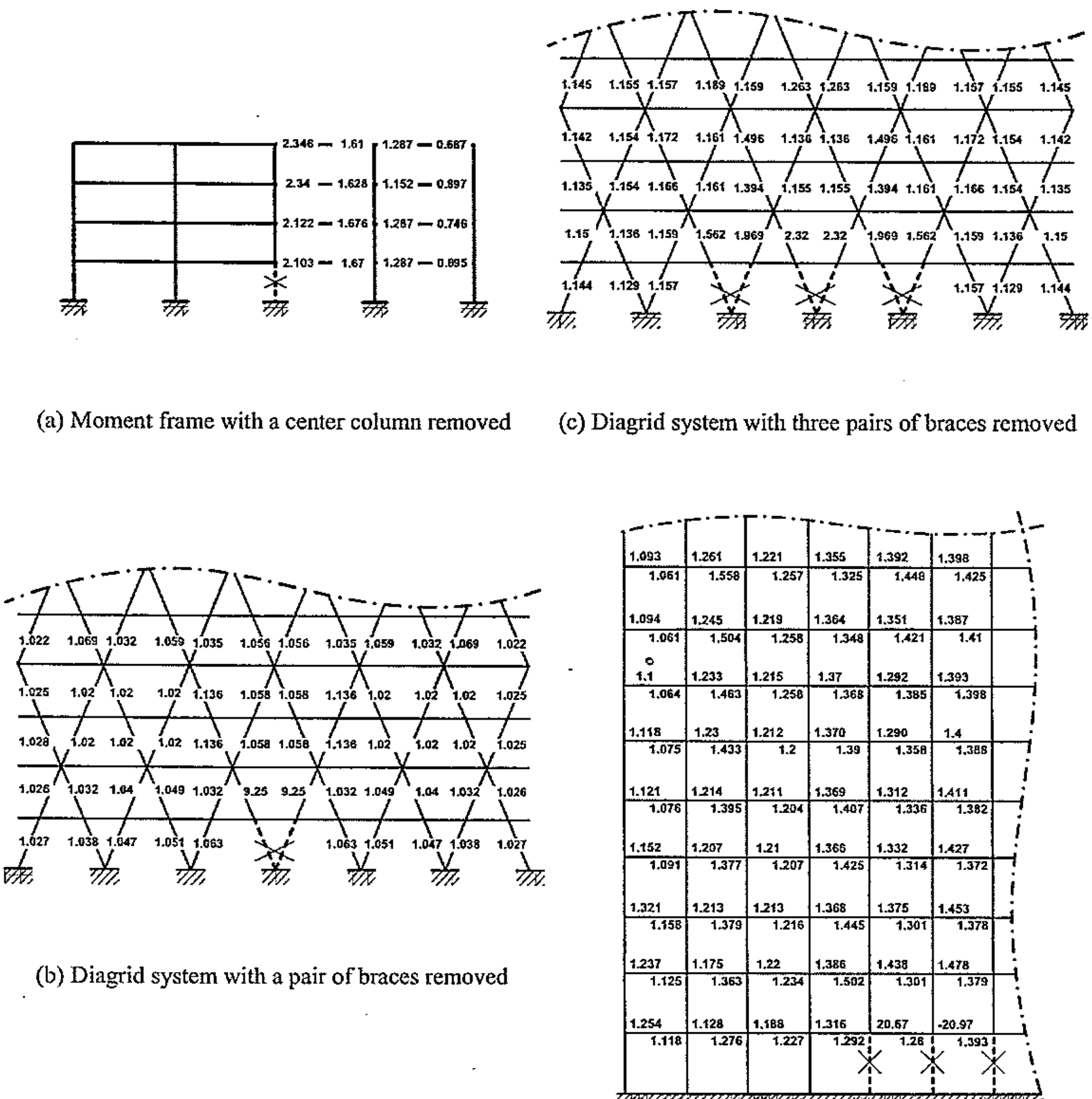

(a) Moment frame with a center column removed

(c) Diagrid system with three pairs of braces removed

(b) Diagrid system with a pair of braces removed

(d) Tubular system with five columns removed

Figure 6. Ratio of member forces obtained from dynamic and static analyses

three pairs of braces were removed, the displacements computed by static and dynamic analyses are quite similar to each other. However, when five pairs of braces were removed, at which the load factor obtained from the pushdown analysis is close to 1·0, the final vertical displacement obtained from dynamic analysis is much larger than the static displacement. This implies that significant inelastic deformation occurred due to dynamic effect.

Figure 11 plots the nonlinear static and dynamic analysis results of the diagrid structure when five pairs of braces located around a corner were removed. Figure 11(a) shows that the maximum load factor is less than 1·0, and thus the structure is not safe for progressive collapse. The time history of the vertical displacement shows that the vertical displacement became unbounded, and progressive collapse occurred.

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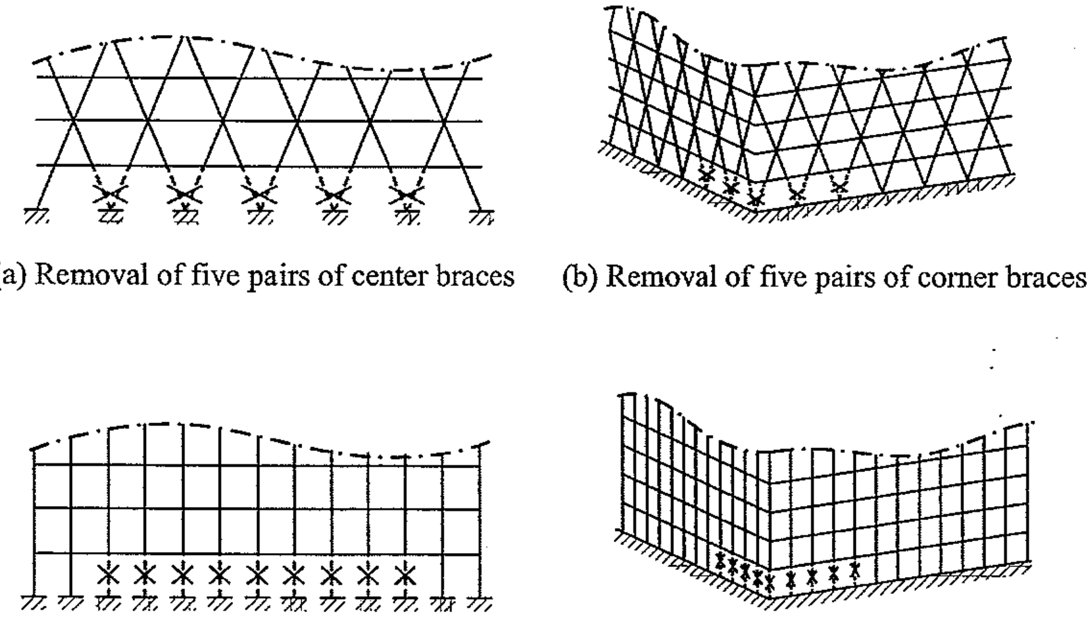
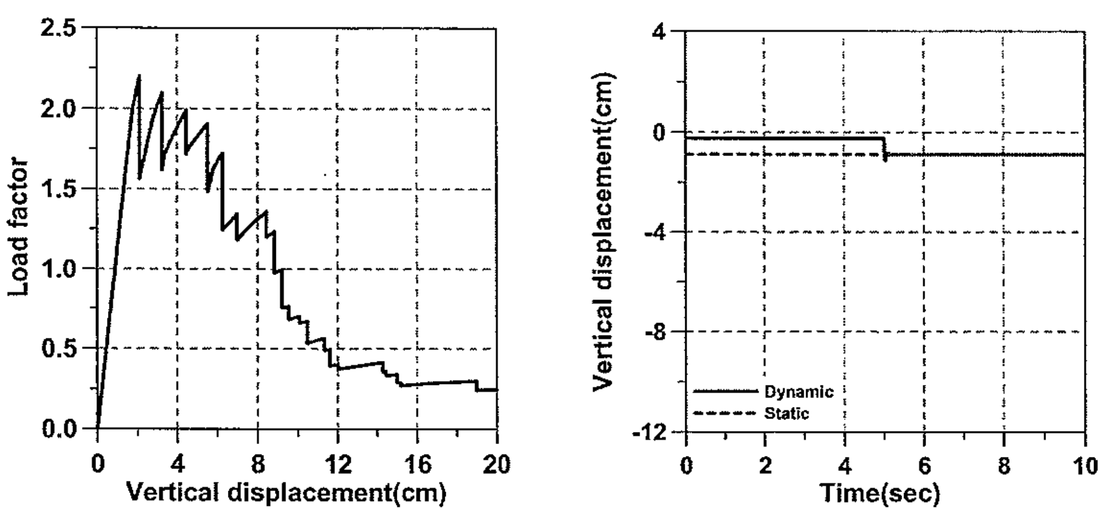


(a) Removal of five pairs of center braces (b) Removal of five pairs of corner braces

(c) Removal of nine center columns (d) Removal of nine corner columns

Figure 7. Locations of removed elements



(a) Nonlinear static analysis (b) Nonlinear dynamic analysis

Figure 8. Analysis results of the 36-storey diagrid structure with a pair of braces removed from center of a side

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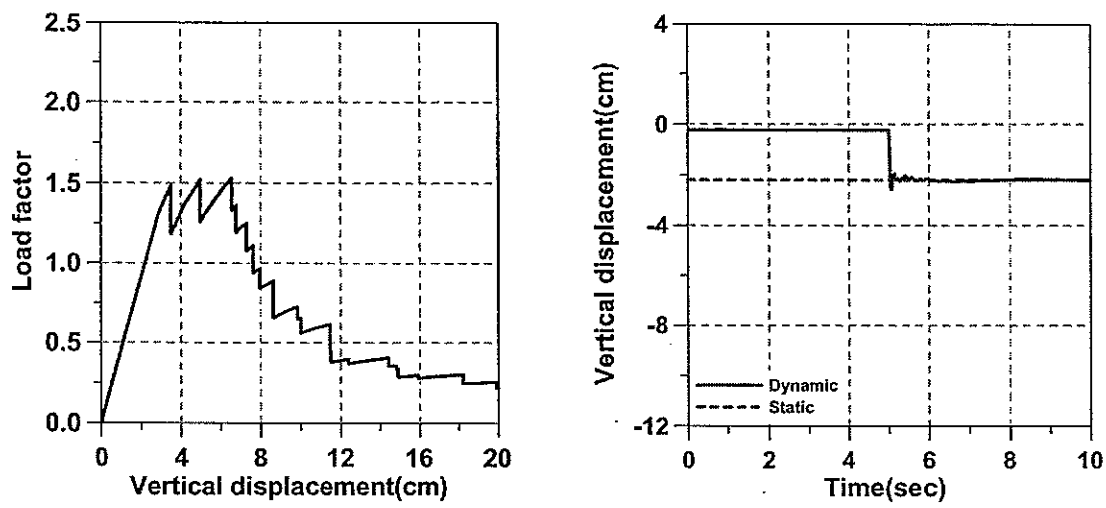
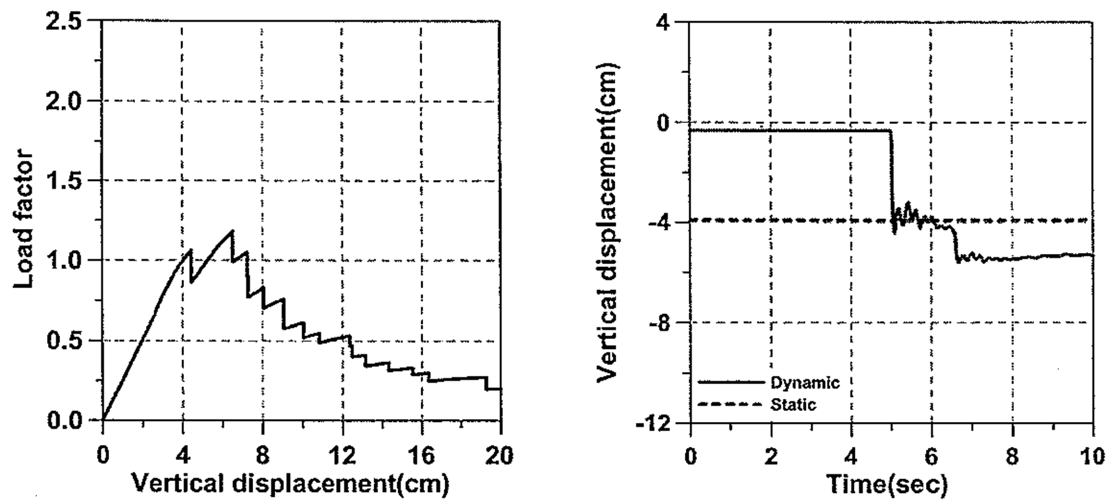


(a) Nonlinear static analysis (b) Nonlinear dynamic analysis

Figure 9. Analysis results of the 36-storey diagrid structure with three pairs of braces removed from center of a side



(a) Nonlinear static analysis (b) Nonlinear dynamic analysis

Figure 10. Analysis results of the 36-storey diagrid structure with five pairs of braces removed from center of a side

Figures 12 and 13 show the analysis results of the 36-storey diagrid structure with corner columns when five pairs of braces were removed from the centre of a side and from around a corner, respectively. Even though a slight increase in maximum load factor was observed compared with the analysis results of the structure without corner columns, the overall trends for vertical displacement and progressive collapse were the same. Figure 14 compares the failure modes of the diagrid structures designed with and without corner columns when five pairs of braces were removed from around a corner. It can be noticed that the failure modes are quite different depending on the existence of corner

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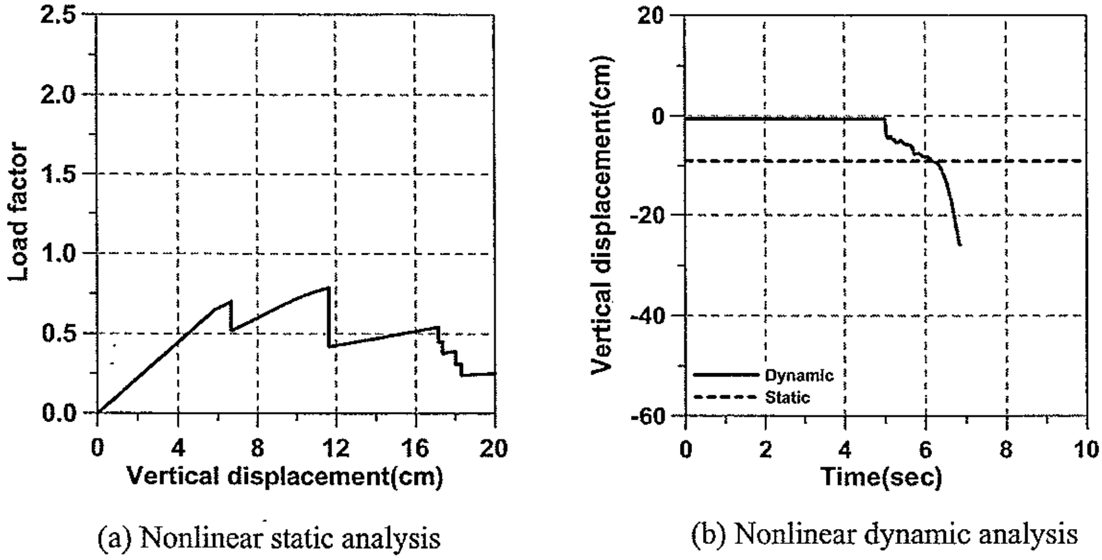

Figure 11. Analysis results of the 36-storey diagrid structure with five pairs of braces removed from a corner

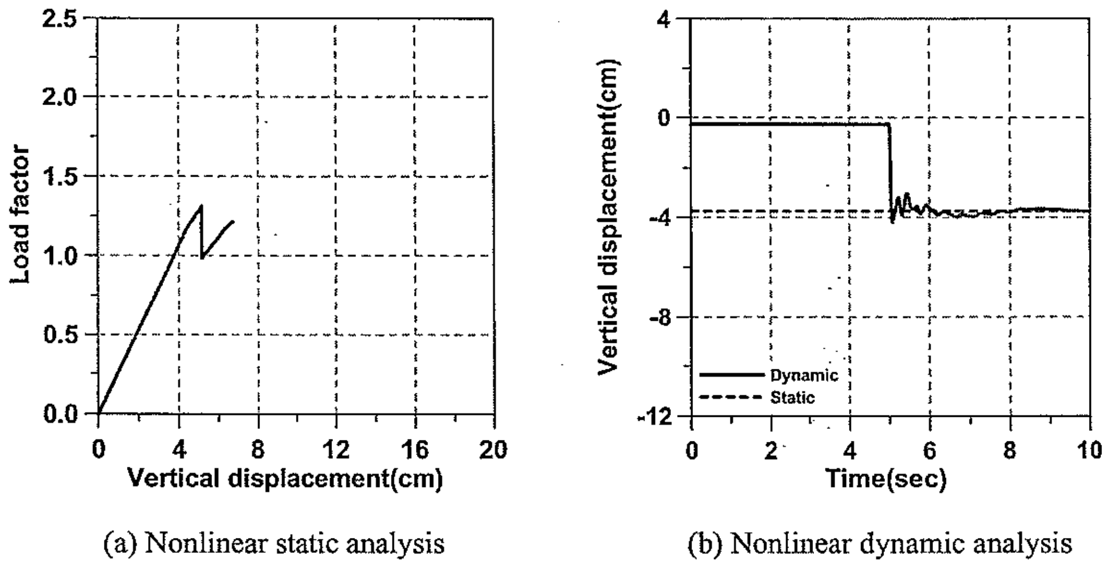

Figure 12. Analysis results of the 36-storey diagrid structure (with corner columns) with five pairs of braces removed from center of a side

columns: in the structure without corner columns, the failure lines, initiated at the second-storey braces, continued to expand around the other corners, whereas in the structure with corner columns, the failure lines stopped at the other corner columns. Figure 15 shows the pushdown curves of the 54-story diagrid structure (with no corner column) with five pairs of the first story braces removed. It can be observed that the 54-story structure has slightly higher strength than the 36-story diagrid structure with its pushdown curves shown in Figures 10 and 11.

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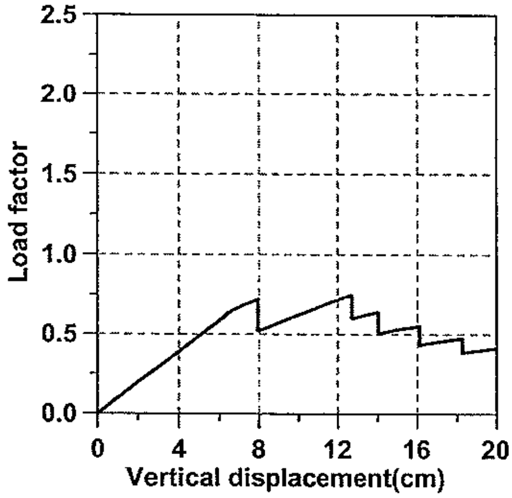
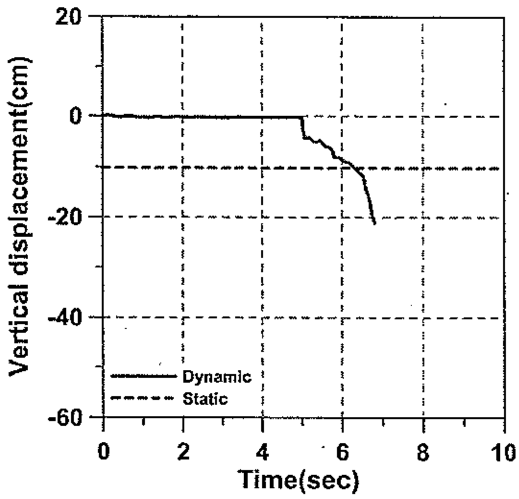
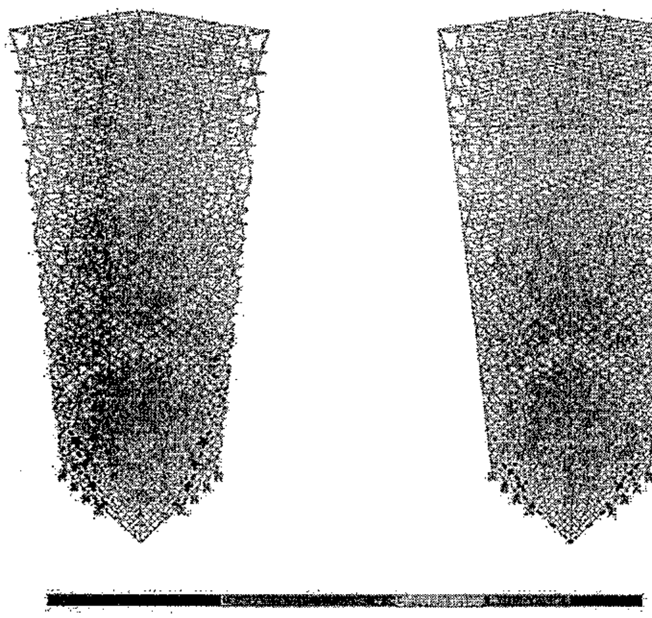


(a) Nonlinear static analysis



(b) Nonlinear dynamic analysis

Figure 13. Analysis results of the 36-storey diagrid structure (with corner columns) with five pairs of braces removed from a corner



(a) Diagrid without corner columns (b) Diagrid with corner columns

Figure 14. Plastic hinge formation in the 36-storey diagrid structure when five pairs of braces are removed from a corner

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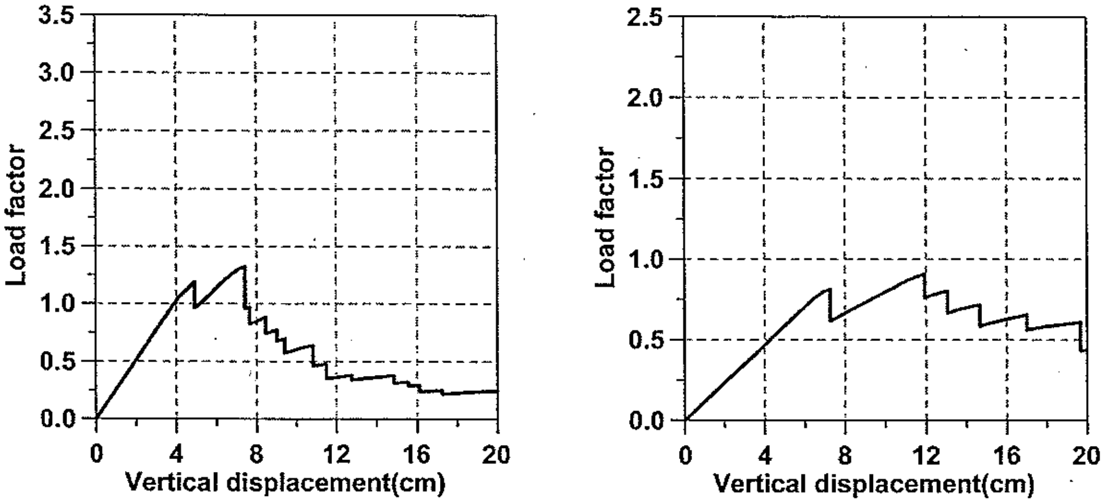
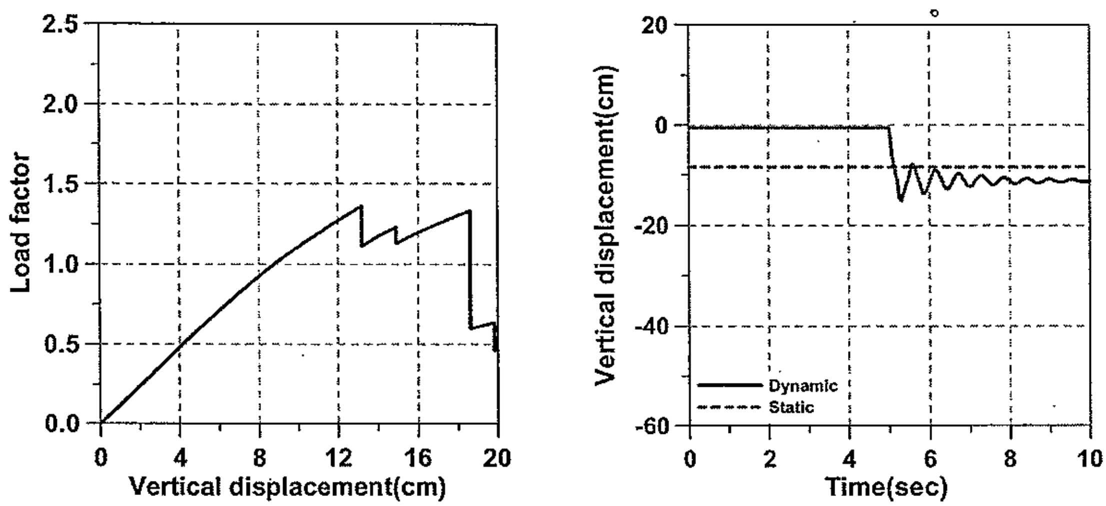


(a) Removed from a center (b) Removed from a corner

Figure 15. Pushdown curves of the 54-storey diagrid structure with five pairs of braces removed



(a) Nonlinear static analysis (b) Nonlinear dynamic analysis

Figure 16. Analysis results of the 36-storey tubular structure with nine columns removed from center of a side

4.2 *Performance of the tubular structure*

The same analyses were performed with the tubular structure by removing first-storey columns from the centre of a side and from a corner. Figure 16 shows the nonlinear static and dynamic analysis results of the tubular structure with nine centre columns removed from a side, where it can be observed that the maximum load factor of pushdown curve exceeded 1·0 and the time history of the vertical displacement remained stable. As in the diagrid structures, the vertical displacement obtained by dynamic analysis turned out to be larger than the displacement obtained by static analysis due to

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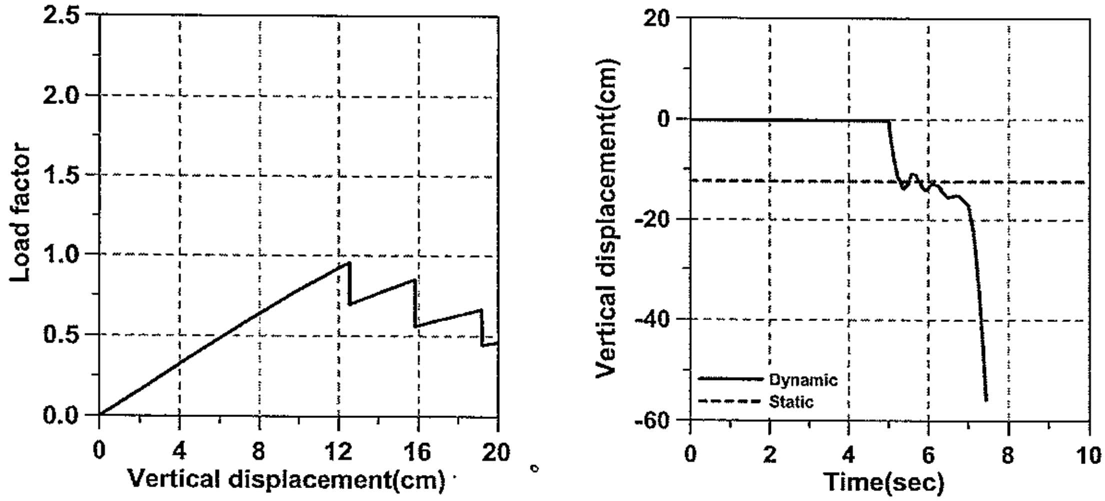
dynamic effect. Figure 17 shows the analysis results of the same structure when nine columns were removed from a corner. In this case, the maximum load factor turned out to be less than 1·0, which implies that the capacity of the structure to resist progressive collapse is less than the load effect. This is confirmed by the unbounded vertical displacement obtained by dynamic analysis. The damaged members are depicted in Figure 18, where it can be observed that damage is widely spread along the



(a) Nonlinear static analysis (b) Nonlinear dynamic analysis

Figure 17. Analysis results of the 36-storey tubular structure with nine columns removed from a corner

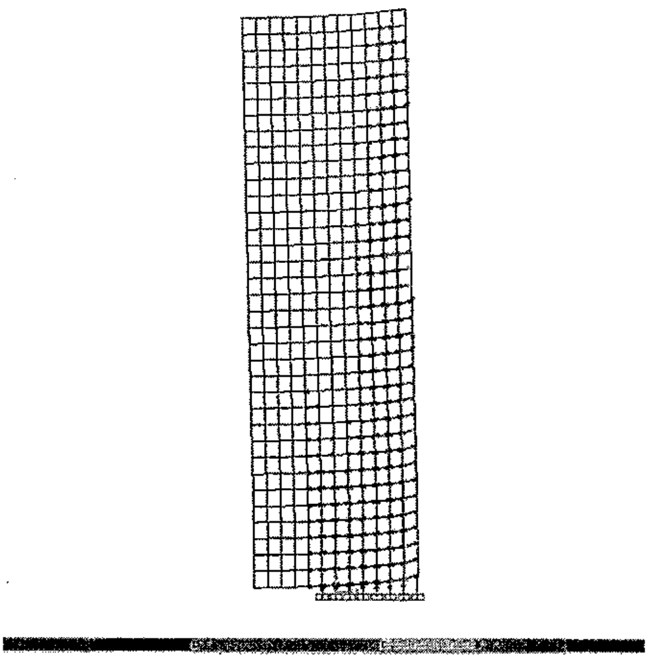


Figure 18. Plastic hinge formation in 36-storey tubular structure with nine columns removed from a corner

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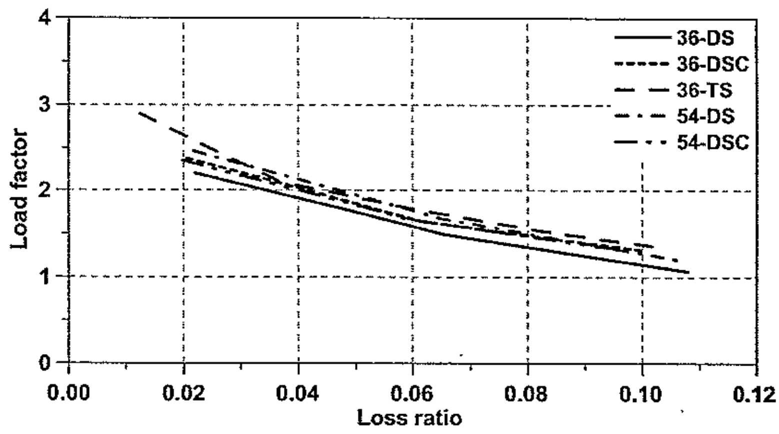
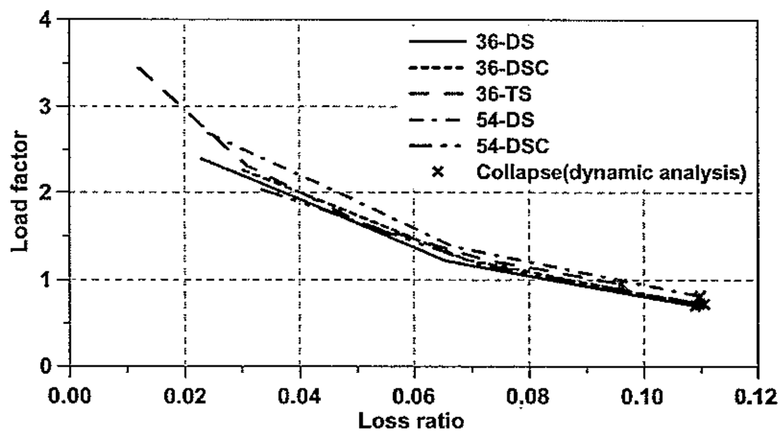
height of the structure. This is different from the failure mode of the diagrid structure, where members were damaged along lines going around the building.

4.3 Load factor versus loss ratio of vertical elements

In order to compare the collapse behaviour of the diagrid and tubular structures, the maximum load factors and the maximum vertical displacements were plotted as a function of the loss ratio, which is the number of removed vertical elements divided by the total number of vertical elements. Figure 19 shows the load factor versus loss ratio obtained by nonlinear static pushdown analyses when vertical members were removed from the centre of a building side (Figure 19(a)) and from a corner (Figure 19(b)). It can be observed that as the loss ratio increases, the maximum load factor decreases. The



(a) Removal of center elements



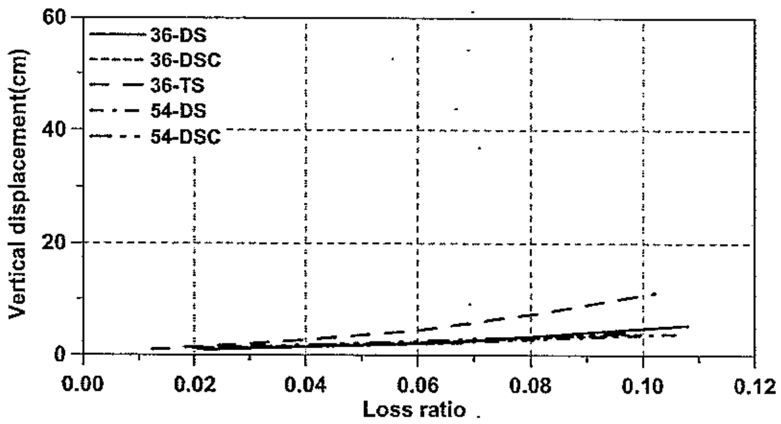
(b) Removal of corner elements

Figure 19. Load factor versus loss ratio of vertical elements in diagrid structures (DS), diagrid structures with corner columns (DSC) and tubular structure (TS)

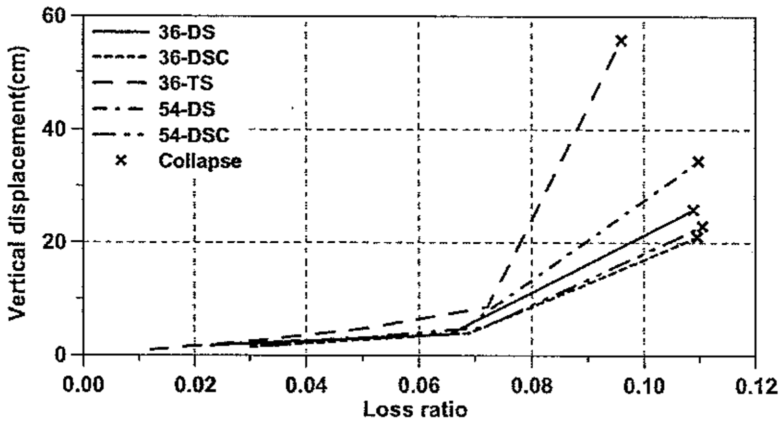
DOI: 10.1002/tal

load factor reached 1·0 at a loss ratio of about 0·11 when columns or braces were removed from a centre, and at about 0·08 when they were removed from a corner. Generally, the tubular structure showed a slightly larger load factor than the diagrid structure at a given loss ratio. In the 36-storey diagrid structures, the structure with corner columns showed larger load factors than the structure without corner columns. However, the opposite is true in the 54-storey structures.

Figure 20 depicts the maximum vertical displacements versus loss ratio of the vertical elements of the model structures obtained by nonlinear dynamic analyses. It can be observed that the displacement of the tubular structure is larger than those of the diagrid structures at a give loss ratio, and that displacements of the diagrid structures with corner columns are slightly smaller than those of the structures without corner columns both in the 36- and 54-storey diagrid structures. When more than 7% of vertical members were removed from a corner of the tubular structure, the vertical displacement increased very rapidly, and failure occurred at the loss ratio of 9·5%. At the same loading condition, the diagrid structures failed when loss ratio reached about 11%.



(a) Removal of center elements



(b) Removal of corner elements

Figure 20. Vertical displacement versus loss ratio of vertical elements obtained by nonlinear dynamic analyses

DOI: 10.1002/tal

4.4 *Reinforcement of internal frames*

In tube type structures, all lateral forces are resisted by external structures, and the internal frames are designed only for gravity load. Sometimes, the internal gravity load-resisting frames are hinge-connected and are vulnerable to progressive collapse. In this case, the resistance to progressive collapse can be increased by designing internal frames with moment connections or by installing strong horizontal members, such as outrigger trusses, at the top of the structure.

5. CONCLUSIONS

This study investigated the progressive collapse potential of high-rise tube-type structures in which lateral load-resisting systems are located at the perimeter of the structures. Two different types of diagrid structures, with and without corner columns, and a tubular structure with closely spaced external columns and deep spandrel girders, were considered for analysis. In the nonlinear static pushdown analysis, any dynamic amplification factor was not applied in the load combination based on the observation that the amplification of member force was not significant in the diagrid and tubular model structures.

The analysis results showed that tube-type buildings generally had high resistance to progressive collapse caused by the sudden loss of external members. The progressive collapse of tube-type buildings tended to occur when perimeter columns corresponding to more than 11% of all vertical members were removed from a side of the diagrid and tubular structures. When the diagonals located around a corner were removed, the number was reduced to 8%. It was observed that the addition of corner columns in the diagrid system did not contribute significantly to an increase in maximum strength for progressive collapse, but helped prevent the failed members from propagating all around the perimeter. It was also observed that the progressive collapse-resisting capacity of 54-storey diagrid structures were slightly higher than that of 36-storey structures. In most cases, the nonlinear static and dynamic analysis results corresponded well with each other in the prediction of progressive collapse.

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