THE STRUCTURAL DESIGN OF TALL AND SPECIAL BUILDINGS
Struct. Design Tall Spec. Build. **18**, 455–465 (2009)
Published online 24 June 2008 in Wiley Interscience (www.interscience.wiley.com). **DOI**: 10.1002/tal.448

EVALUATION OF PROGRESSIVE COLLAPSE POTENTIAL OF STEEL MOMENT FRAMES CONSIDERING CATENARY ACTION

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SUMMARY

This study investigated the effect of catenary action on the progressive collapse potential of steel moment framed structures. Non-linear static and dynamic analyses of three- and six-story model structures with and without bracing were carried out following the alternate path method recommended by the General Services Administration 2003. According to the non-linear static push-down analysis results, the contribution of catenary action and the progressive collapse potential of structures increased as the number of story and the number of bay increased. The effect of catenary action increased significantly in braced frames, in which the movement of beam–column joints were fully restrained until the tensile capacity of beams located both sides of the removed column reached their maximum values. The non-linear dynamic analyses showed that the maximum deflection caused by sudden removal of a column decreased when the catenary action was taken into account. Copyright © 2008 John Wiley & Sons, Ltd.

1. INTRODUCTION

The progressive collapse refers to the phenomenon that local damage of structural elements caused by abnormal loads results in global collapse of the structure. Explicit design methods for progressive collapse resistant design can be found in the US government documents such as the US General Services Administration (GSA) 2003 and the United Facilities Criteria (UFC) 2005. The GSA 2003 guidelines provide a methodology to mitigate progressive collapse potential in structures based on the alternate path method (APM). In the UFC (2005) two design approaches are specified, namely the tie force method (TFM) and APM. The former is essentially an indirect design approach, wherein a minimum tie force capacity must be made available in the system to transfer loads from a damaged part to the remainder of the structure. The TFM relies implicitly on the formation of catenary action to mitigate collapse, which is one of the key mechanisms thought to assist a damaged structure to reach an alternative equilibrium configuration is catenary action. In catenary action, elements (e.g., beams and slabs) that are intended to support load in flexure undergo large deformation and have sufficiently stiff and strong anchorages enough to take on load as tension members. The commentary in the ASCE 7-02 (2005) presents general design guidelines and suggestions for improving structural integrity, which includes a catenary action of the floor slab among others. The *Best Practice for Reducing the Potential for Progressive Collapse in Buildings* (NIST, 2007) recommends the catenary action as one of means for upgrading existing buildings. However, specific design guideline for this mode of behaviour is not yet explicitly considered in the design codes and the federal guidelines.

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Hamburger and Whittaker (2004) addressed the advantage of catenary action as follows: first, it is not necessary to provide moment resisting framing at each level of a structure, in order to provide progressive collapse resistance. Second, it is not necessary to have substantial flexural capacity in the horizontal framing, either in the beam section itself or in the connection. Third, it may not be necessary to provide full moment resistance in the horizontal framing, and conventional steel framing may be able to provide progressive collapse resistance as long as connections with sufficient tensile capacity to develop catenary behaviour are provided. Catenary action has been one of major research topics in structural engineering field. For example, Yin and Yang (2005) presented a general analytical method of the catenary action in steel beams at large deflections caused by fire. Astaneh-Asl (2003) carried out 10 tests on a full scale specimen of a one story building with steel cables placed within and other side of the floor. The tests and associated analyses indicated that the catenary action of the cables could economically and efficiently prevent progressive collapse of the floor in the event of removal of one of the exterior columns. Yu and Richard Liew (2005) studied the behaviour of steel beams with the increase of temperature from beam action phase to catenary action phase and until failure. They found that the critical temperature of steel beams could be enhanced by over 200ºC if proper attention was given to the integrity of connections to resist the catenary force. Khandelwal and El-Tawil (2007) carried out finite element analyses of beam–column sub-assemblages of a steel special moment resisting frame to investigate catenary action, and demonstrated the ductility of seismically designed special moment frame connections and their ability to deform in catenary mode. Byfield and Paramasivam (2007) showed that industry standard beam–column connections possess insufficient ductility to accommodate the large floor displacements that occur during catenary action. Sasani and Kropelnicki (2007) carried out experiment of RC continuous beams satisfying the integrity requirements of ACI-318 up to collapse. It was observed that in spite of fracture of beam bottom reinforcement the beam showed significant remaining strength and deformation capacity by the development of catenary action.

Most of the previous researches, however, have been focused on the catenary action in beam–column sub-assemblages and the effect of catenary action on the global behaviour of a structure has not been thoroughly investigated yet. In this regard this study is intended to compare the progressive collapse potential of steel moment frames designed per current design codes with and without considering catenary action. Especially the vertical deflections caused by sudden removal of a column and the bending moment and axial force induced in beam elements are evaluated to quantify the effect of catenary action.

2. CATENARY ACTION IN A BEAM–COLUMN SUB-ASSEMBLAGE

2.1 Analysis program code and material models

Figure 1 shows a system composed of two beams deformed after the column located between the beams is removed. The beams are made of H $450 \times 200 \times 9 \times 14$ and are fixed to the boundaries. If the girders are not sufficiently strong to resist the flexural demands resulting from the instantaneous removal of the column in an elastic manner, plastic hinges will form at the two ends of the beams. If the flexural strengths of beams are not sufficient to accomplish stability of the system, the beam will deflect further to mobilize catenary tensile action that, if sufficient, will eventually arrest the collapse. In order for a catenary action to be fully activated in a beam member, the ends of the beam need to be strongly anchored to the joints enough to resist the large axial force generated by the catenary action.

The progressive collapse of a structure is involved with non-linear deformation of structural elements. Therefore non-linear analysis is more preferable to linear analysis to investigate the progressive

DOI: 10.1002/tal

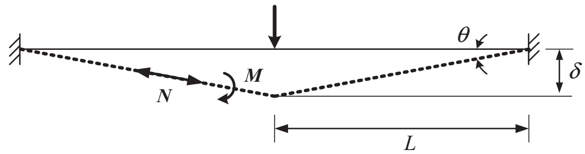

Figure 1. Definition of rotation angle in a beam model

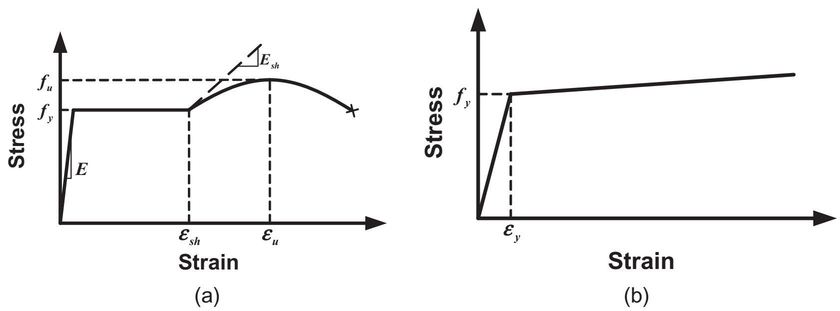

Figure 2. Non-linear material models used in the analysis. (a) Steel 01 model for columns and braces. (b) Reinforcing steel model

collapse potential of structures. To consider the catenary action of beams after removal of a column the geometric non-linearity as well as material non-linearity needs to be included in the analysis modelling. In this study the analysis results with and without considering catenary action are compared using the program code OpenSees (Mazzoni *et al.*, 2006). The cross section of each flange and web of a structural member is divided into 50 fibre elements and each structural member is modelled by four 'non-linear beam column' elements in the longitudinal direction; for the modelling of beam elements without considering catenary action ('no-catenary action' cases) the 'linear' geometric transformation option is used, whereas the 'corotational' geometric transformation is selected for beams analyzed considering catenary action. All columns are modelled using the 'P-Δ' geometric transformation option in the 'non-linear beam column' element. As columns and braces are not subjected to catenary action, the simple 'bi-linear steel 01' model shown in Figure 2(a) is used for the constitutive relation of the material. The post-yield stiffness is assumed to be 2% of the initial stiffness. The beams are modelled by the 'reinforcing steel' model shown in Figure 2(b), which requires such informations as yield stress (f_y); tensile strength (f_u); yield strain (ε_y); beginning of strain hardening (ε_{sh}); elastic modulus (E); and tangent stiffness of strain hardening region (E_{sh}).

2.2 Push-down analysis of the sub-assemblage

Figure 3(a) shows the member force-rotation relationship of a beam with and without considering catenary action obtained by non-linear static push-down analysis. The horizontal axis represents the rotation angle in radian that is obtained by dividing the vertical deflection with the beam length, *L*. The vertical axis represents the axial tension and bending moment normalized by the yield force and plastic moment, respectively. It can be observed that when catenary action is not considered the bending moment keeps increasing due to strain hardening but axial force is not induced. On the other hand when catenary action is considered the bending moment drops when the rotation angle increases larger than 0·07 rad whereas the axial force keeps increasing. When the rotation angle increases larger

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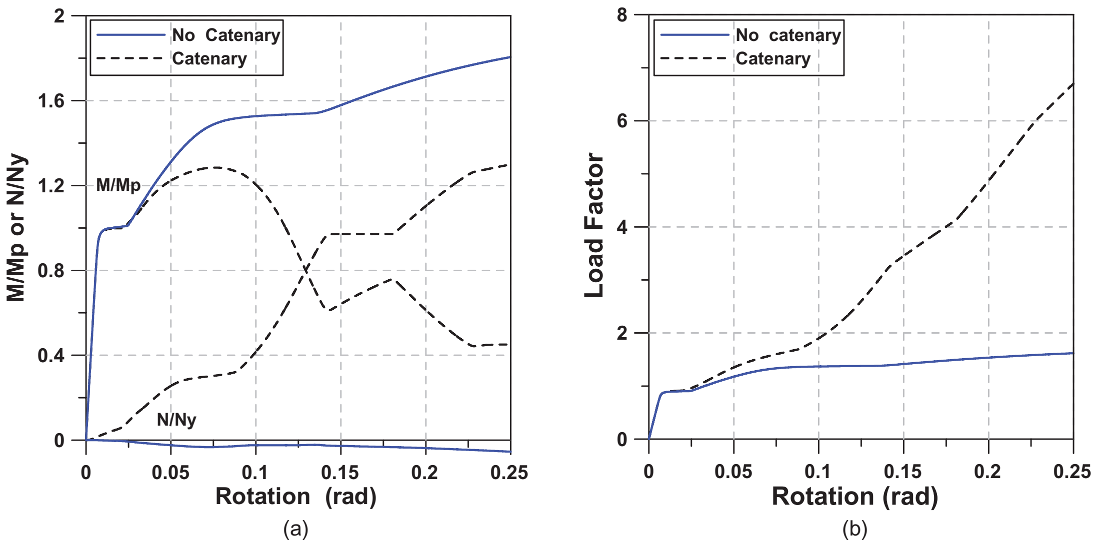

Figure 3. Push-down analysis results of the simple model. (a) Variation of member forces. (b) Push-down curves

than 0·13 rad the axial force becomes more dominant than the bending moment. Figure 3(b) plots the variation of the load factor, which is applied load divided by the design load specified in the guidelines, 2 × (dead load + 0·25 live load). In case catenary action is not considered the load factor does not increase higher than 2·0, whereas when catenary action is considered the load factor continues to increase as rotation angle increases. This implies that when catenary action is fully activated as a result of elaborate preparation of beam–column joints, the beams can resist significantly larger load. The above results are only possible when the beams are strongly connected to the boundaries; and in real structures the contribution of catenary action on resisting the applied load will depend highly on the joint conditions.

3. ANALYSIS OF FRAMED STRUCTURES

3.1 Loading for progressive collapse

For non-linear static push-down analysis the vertical displacement of the beam–column joint in which the lower story column is removed is gradually increased and the corresponding resistance of the system is computed. The load combination recommended by the GSA guidelines is used. In the bays in which the central column is lost the load factor of 2·0 is multiplied to take the dynamic effect into account.

For dynamic analysis the axial force acting on the column to be removed is computed. Then the column is replaced by point loads equivalent of its member forces as shown in Figure 4(a). To simulate the phenomenon that the column is abruptly removed, the member forces are removed after a certain time is elapsed as shown in Figure 4(b), where the variables *P*, *V* and *M* denote the axial force, shear force and the bending moment acting on the column, and *W* is the vertical distributed load. In this study the forces were increased linearly for 5 s until they reached their full amounts, kept unchanged for 2 s until the system reached stable condition, and the upward force was suddenly removed to initiate vertical vibration.

DOI: 10.1002/tal

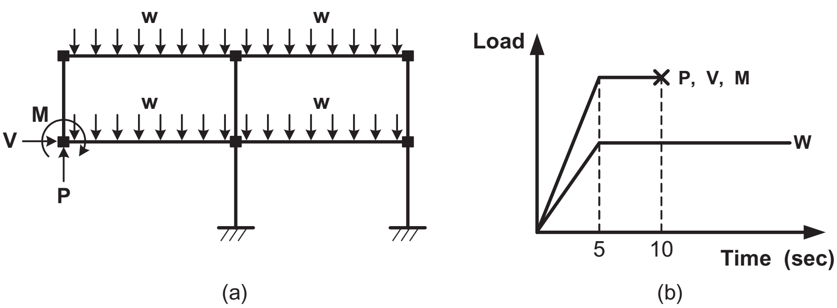

Figure 4. Imposed loads for progressive collapse. (a) Loading on a structure with a column remove. (b) Time history of loads

Table 1. Member size of analysis model (unit: mm) of (a) a three-story structure and (b) a six-story structure

(a)

Members	Size
Beams	H 450 × 200 × 9 × 14
Columns	H 388 × 402 × 15 × 15
Braces	H 100 × 100 × 6 × 8

(b)

Members	Stories	Member size
Beams	1–3	H 500 × 200 × 8 × 15·5
	4–6	H 450 × 200 × 8 × 13·5
Columns	1–3	H 498 × 432 × 35 × 50
	4–6	H 350 × 350 × 12 × 19
Braces	1–6	H 100 × 100 × 6 × 8

3.2 Analysis model structures

Three- and six-story steel moment frames with the structural plan shown in Figure 5 are designed in accordance with the AISC Load Resistance Factor Design (2000). The seismic load used in this study is equivalent of S_{DS} and S_{D1} equal to 0·33 g and 0·18 g, respectively, in the IBC 2006 format. The structures were designed as ordinary steel moment frames with the R-factor of 3·5. The columns and girders are made of SM490 ($F_y = 310$ MPa) and SS400 ($F_y = 240$ MPa) steel, respectively. Table 1 shows the selected structural members. The two-dimensional frame enclosed in the dotted rectangle in Figure 5 is taken out for analysis. The number of bays is varied as two, four and six as shown in Figure 6 to investigate the effect of number of bays on the catenary action. In some model structures braces are installed in both end spans to restrain lateral deformation of the frames. It is assumed that premature failure does not occur in connections so that full catenary action can be realized.

DOI: 10.1002/tal

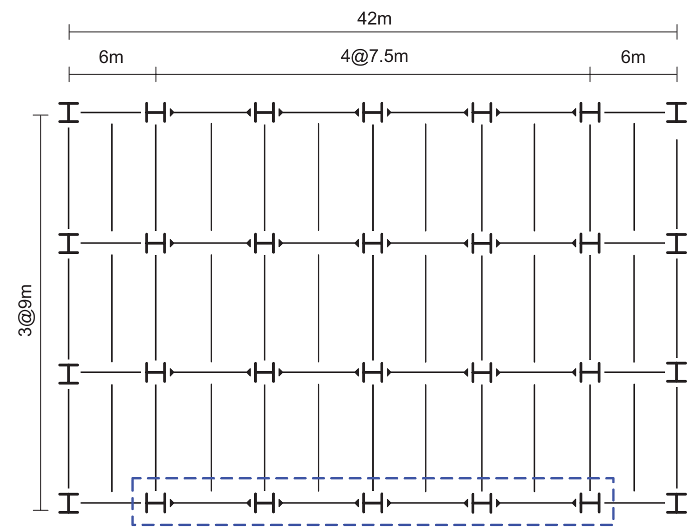

Figure 5. Structural plan of the model structure

3.3 Non-linear static push-down analysis

Figure 7 shows the bending moment or axial force induced in the beams obtained by push-down analysis of model structures. In all cases the centre column in the first story is removed to initiate progressive collapse. As the analysis results without considering catenary action are quite similar in all models regardless of the number of bays, only the results of the model with four bays are plotted in the figure. It can be noticed that the 'no catenary action' models behave like the simple two-beam model with fixed boundaries analyzed without catenary action. When catenary action is not activated no axial force is induced in the beams. On the other hand the curves for bending moments and axial forces of the model structures with catenary action considered deviate from those of 'no-catenary' models at the beam rotation of 0·065 rad and 0·008 rad, respectively. As the numbers of bay and story increase the axial force in the beams increases, which implies that a large axial force or catenary action is induced when there exists a strong restraint at both sides of the structure against deformation towards the centre of the structure. As the axial forces in the beams increase, however, the bending moments induced in beams decrease. In the three-story structures, both with and without braces, bending moment is more dominant than axial force in resisting progressive collapse. In the six-story structures, however, the axial force becomes more dominant when the beam rotation exceeds about 0·25 rad. As the number of story increases the size of the lower story columns also increases and does the restraint for catenary action. It can be expected that as the lateral stiffness of a structure increases the contributions of bending moment and catenary action will become similar to those of simple two-beam model (Figure 3(a)).

Figure 8 shows the push-down curves of the model structures analyzed with and without considering catenary action. The load factor, which is the vertical load normalized by the load specified in the GSA guidelines (2 (DL + 0·25LL)), is plotted against rotation of the beam. As the pushover curves of the 'no-catenary action' models did not vary with number of bays, only the results of the four-bay models were plotted. When the catenary action is considered the push-down curves form upper bounds and the load factors increase as the number of bays increases. It can be observed that the push-down curves of the braced frames start to deviate from those of the 'no-catenary action' models at lower

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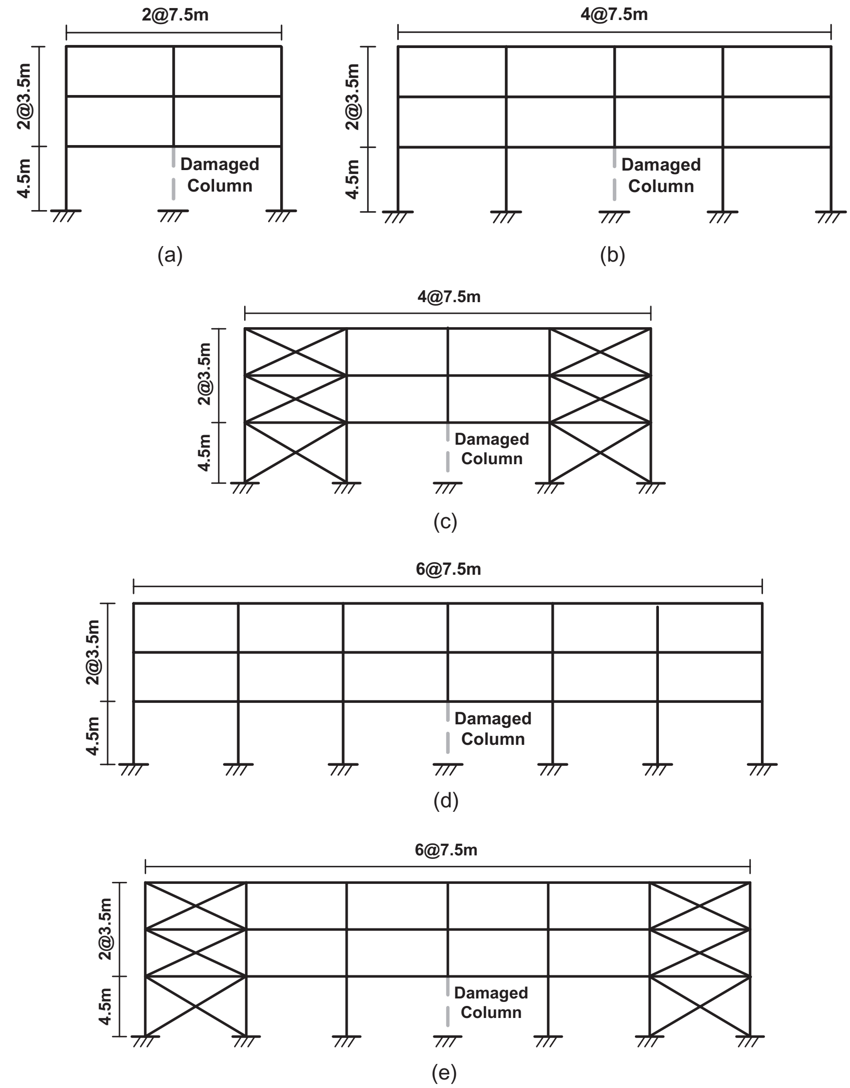

Figure 6. Three-story analysis model structures. (a) Two-bay model. (b) Four-bay model. (c) Four-bay model with braces. (d) Six-bay model. (e) Six-bay model with braces

values of beam rotation than the model structures without braces; that is, they vary depending on the constraint for lateral movement. For example the push-down curve of the three-story two-bay model with catenary action deviates from that of the three-story two-bay without catenary action at 0·07 rad, whereas the curve of the model six-story six-bay braced frame with catenary action deviates from that of the six-story six-bay braced frame without catenary action at 0·03 rad. It also can be noticed that the yield point of the push-down curve increases as the number of story increases, which is natural considering that as the number of story increases the overall redundancy and the number of elements resisting the progressive collapse also increases.

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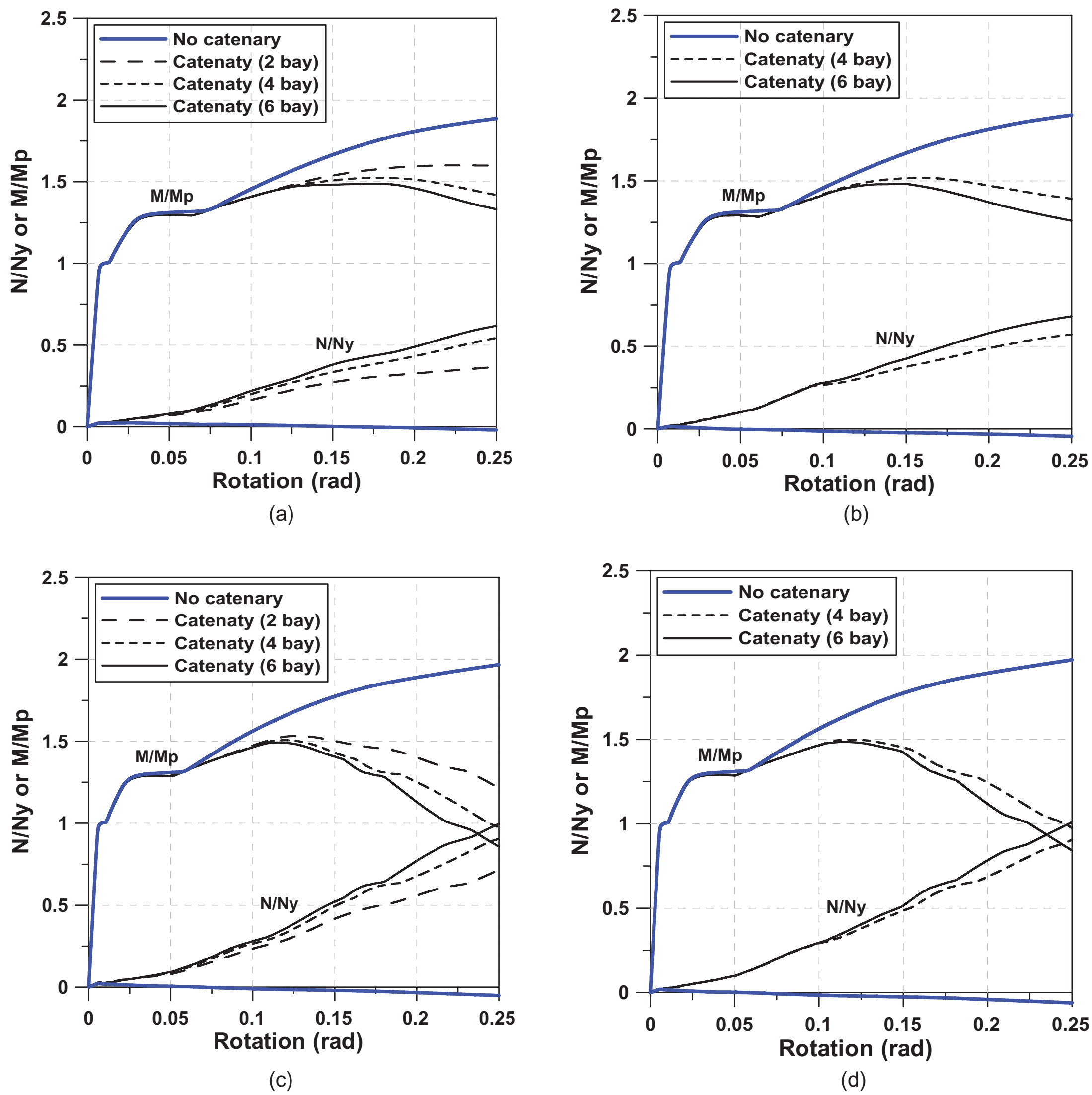

Figure 7. Variation of axial force and bending moment of model structures obtained from push-down analysis. (a) Three-story models without braces. (b) Three-story models with braces. (c) Six-story models without braces. (d) Six-story models with braces

3.4 *Non-linear dynamic analysis*

Figure 9 shows the displacement time history analysis results of the model structures with the catenary action of beams considered. The maximum deflections of the 'no-catenary action' cases are also plotted in the figure, which are quite similar to one another regardless of the number of bays or whether braces are installed or not. It can be observed that the maximum deflections of the three-story structures with the catenary action considered are smaller than those obtained without considering catenary action. The six-bay model with braces showed the smallest deflection. In the six-story structures the vertical deflections are reduced significantly compared with those in the three-story structures. However, the

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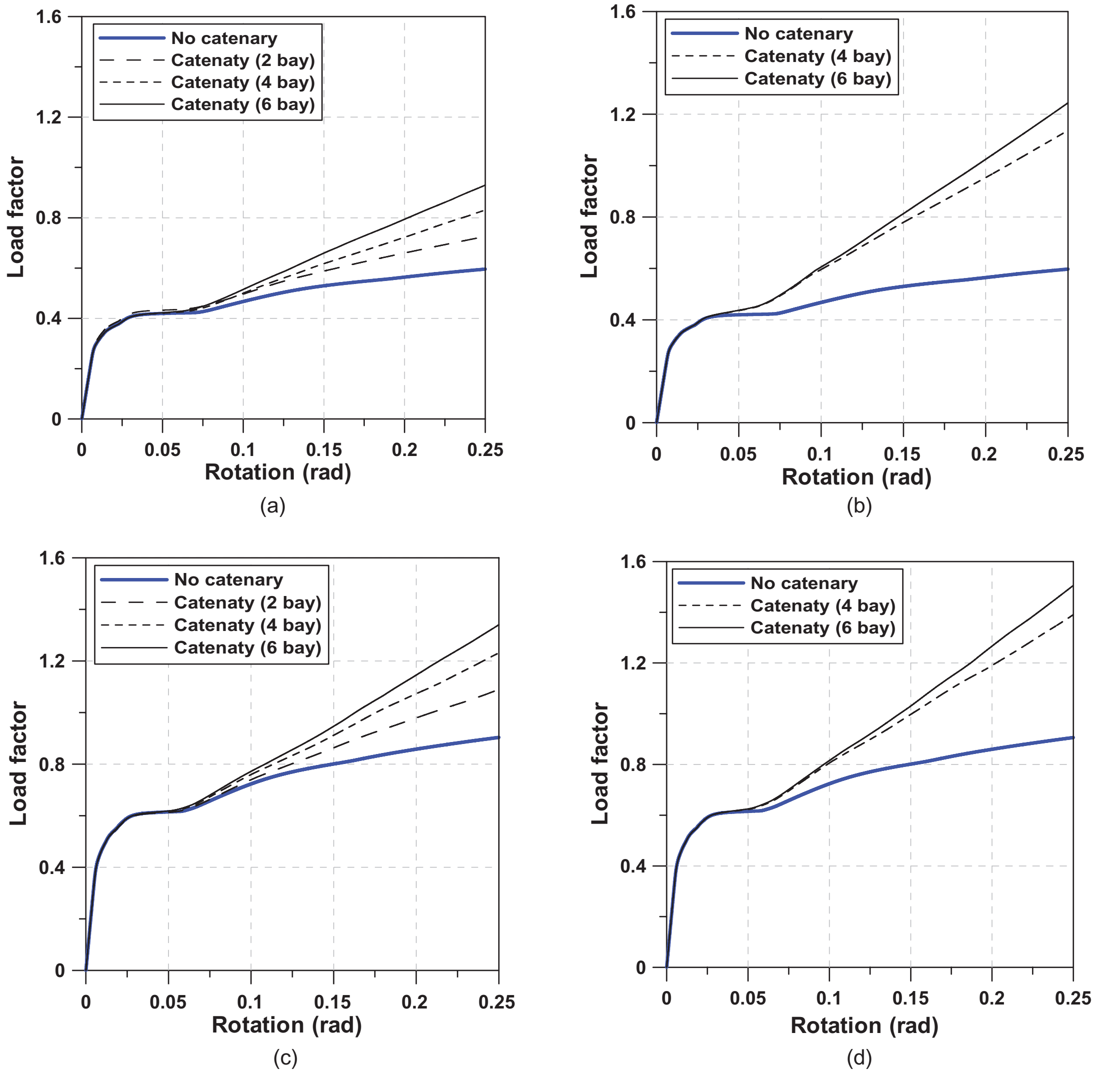

Figure 8. Push-down curves of the model structures with and without considering catenary action. (a) Three-story models without braces. (b) Three-story models with braces. (c) Six-story models without braces. (d) Six-story models with braces

differences between the 'catenary' and 'no-catenary' cases are not so significant as in the three-story structures. The reason can be found in the push-down curves shown in Figure 8; it can be observed that the maximum deflection of the six-story 'no catenary' models, 19·48 cm (0·0260 rad), occurs before the push-down curves of the 'catenary action' models start to bifurcate from those of the 'no-catenary' models, which is about 0·03 rad. Therefore in cases where the final deformation is less than the bifurcation point, the catenary action may not contribute to the maximum deflection of a structure with a column suddenly lost. However, when the bifurcation point is smaller than the maximum deflection the consideration of the catenary action will result in increased resistance to progressive collapse.

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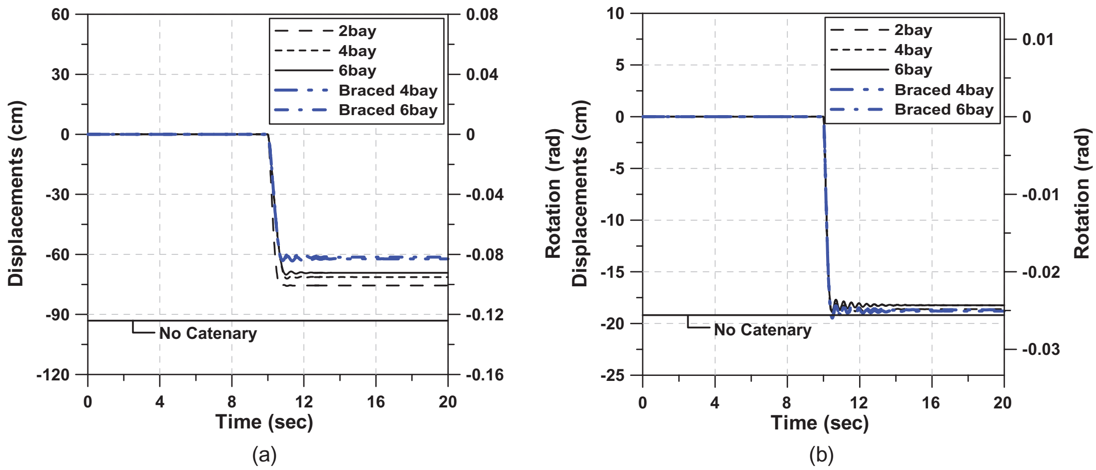

Figure 9. Displacement time histories of model structures obtained from non-linear dynamic analysis. (a) Three-story structures. (b) Six-story structures

4. CONCLUSIONS

This study investigated the effect of catenary action on the progressive collapse potential of steel moment framed structures. Non-linear static and dynamic analyses of three- and six-story model structures with and without bracing were carried out following the alternate path method recommended by the GSA 2003.

When catenary action was considered the non-linear static push-down curves of steel moment frames formed upper bounds of the curves obtained without considering catenary action. The effect of catenary action increased as the constraint for lateral movement, such as additional bays or braces, increased. As the number of story increased the yield point and strength also increased; however the variation of the number of story did not affect the catenary action significantly. It was observed that the push-down curves of the frames reinforced by braces considering catenary action started to deviate from those of the 'no-catenary action' models at lower values of beam rotation than the model structures without braces. The maximum deflections of the structures obtained from dynamic analysis generally decreased when catenary action was considered. However, in cases where the final deformation is less than the bifurcation point obtained from push-down analysis, the catenary action may not contribute significantly to the maximum deflection. But when the bifurcation point is smaller than the maximum deflection, the consideration of the catenary action will result in smaller responses.

Finally, it should be pointed out that the analysis results of this paper were obtained based on the assumption that the beam–column joints were strong enough to activate full catenary action of beams. However, this assumption is not based on experimental observation and may not be true in normal conditions, especially in structures with non-seismic joints. Therefore for more accurate evaluation of progressive collapse potential of a structure, further study is still required to understand the relationship between the joint conditions and catenary action when a column is suddenly removed.

ACKNOWLEDGEMENT

This work was supported by the Basic Research Program of the Korea Science & Engineering Foundation (Grant No. R0A-2006-000-10234-0). The authors appreciate this financial support.

DOI: 10.1002/tal

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DOI: 10.1002/tal