Magazine of Concrete Research
Volume 64 Issue 1
Analysis of reinforced concrete frames subjected to column loss
Kim and Yu

Magazine of Concrete Research, 2012, **64**(1), 21–33
http://dx.doi.org/10.1680/macr.2012.64.1.21
Paper 1000043
Received 19/02/2010; last revised 10/07/2010; accepted 04/08/2010

ice
Institution of Civil Engineers
publishing

Analysis of reinforced concrete frames subjected to column loss

Jinkoo Kim
Department of Architectural Engineering, Sungkyunkwan University, Suwon, Korea

Jisung Yu
DongYang Structural Engineers Co., Seoul, Korea

The objective of this study is to investigate the progressive collapse potential of reinforced concrete moment frames subjected to sudden loss of a first-storey column. To this end three-, six- and 15-story reinforced concrete moment frames were designed as model structures for analysis with and without considering seismic load, and their progressive collapse potentials were investigated by performing non-linear static and dynamic analyses. It was observed from the analysis results that the catenary action is activated by reinforcing steel and is proportional to the amount of the reinforcement. It was also observed that the amount of stirrup affects the onset of the catenary action and the progressive collapse. According to the non-linear dynamic analysis results the reinforced concrete structures that were not designed for seismic load turned out to be highly vulnerable to progressive collapse, whereas the structures designed considering seismic load showed superior performance against progressive collapse.

Introduction

Progressive collapse is a series of local and global failures due to local damage to structural elements caused by abnormal loads. From a series of accidents it has been observed that, in order to prevent progressive collapse, a structure should have sufficient continuity to offer an alternative path to stability of the structure, even if an element of a vertical load-resisting system is removed. To prevent progressive collapse, the National Building Code of Canada (NRCC, 1995) specifies requirements for the design of major elements, the establishment of connection elements and the ways of providing load transfer paths. Eurocode 1 (CEN, 2002) provides a design standard for the selection of plan types suitable for preventing progressive collapse, and recommends that buildings should be integrated. In the USA, the American Concrete Institute (ACI, 2002) requires structural integrity so that partial damage due to abnormal load does not result in total collapse. The ASCE 7-05 (ASCE, 2005) recommends a design method and load combinations as well as structural integrity. The General Service Administration (GSA) provides a practical design guideline to reduce the collapse potential of federal buildings (GSA, 2003), and the Department of Defense (DoD) also presents a guideline for new and existing DoD buildings (UFC, 2005).

Much research has been carried out regarding the validity and applicability of the various analysis methods recommended in design guidelines for accurate prediction of progressive collapse (Kim *et al.*, 2009a; Marjanishvili and Agnew, 2006; Powell, 2005). A design process to prevent progressive collapse based on a plastic design concept was proposed (Kim and Park, 2008). An analysis–design integrated system was developed for progressive collapse analysis of building structures considering dynamic effects (Kim *et al.*, 2009b). Vlassis *et al.* (2009) investigated progressive collapse of buildings due to impact from failed floors.

In the GSA (2003) and DoD (UFC, 2005) two design approaches are specified, namely the tie force method and the alternate path method. The tie force method relies implicitly on the formation of catenary action to mitigate collapse, which is one of the key mechanisms assisting a damaged structure to reach an alternative equilibrium configuration. 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 to take on load as tension members. *Best Practice for Reducing the Potential for Progressive Collapse in Buildings* (NIST, 2006) recommends the catenary action as one of the means for upgrading existing buildings. Recently, the effect of catenary action on progressive collapse has been investigated by many researchers. Astaneh-Asl (2003) carried out experiments on a full-scale specimen of a single-storey structure to investigate the viability of a steel cable-based system to prevent progressive collapse. Sasani and Kropelnicki (2008) carried out an experiment to study the behaviour of a 3/8 scale model of a continuous perimeter beam in a reinforced concrete (RC) frame structure following the removal of a supporting column. Yi *et al.* (2008) carried out a static experimental study of a three-storey RC frame structure to investigate progressive failure due to the loss of a lower storey column. In the experiment it was observed that after the plastic mechanism has formed, the concrete strain in the compression zone at the beam ends reaches its ultimate compressive strain, and the compressive steel bars are gradually subject to tension with increasing displacement. Recently experiments were carried out with RC beam–column sub-assemblages designed with and without seismic detailing (Choi and Kim, 2010). It was observed that seismically designed RC moment frames could resist progressive collapse by activation of beam catenary force at large displacement.

In the present study the mechanisms and parameters involved in the collapse of a RC sub-assemblage structure subjected to sudden column loss were investigated. Non-linear static and dynamic analyses of RC moment frames were carried out, and the performances of seismic- and non-seismic-designed structures were compared. It was assumed in the analysis that as a result of abnormal load, only a column was lost, while the beam–column joints at the top or bottom of the column remained intact so that full catenary action could be activated.

Catenary action in framed structures

Effect of catenary action in beam–column sub-assemblages

Generally large deformation is involved in the process of progressive collapse caused by sudden removal of a structural member, and the geometric as well as material non-linearity needs to be included in the analysis modelling. Table 1 depicts the stress distribution in a cross-section of a steel beam with and without considering catenary action. When catenary action is not considered the location of the neutral axis does not change and no axial force is induced in the cross-section. However, when catenary action is considered, the neutral axis moves upward in large deformation and the unbalanced forces below and above the neutral axis result in catenary force on the beams.

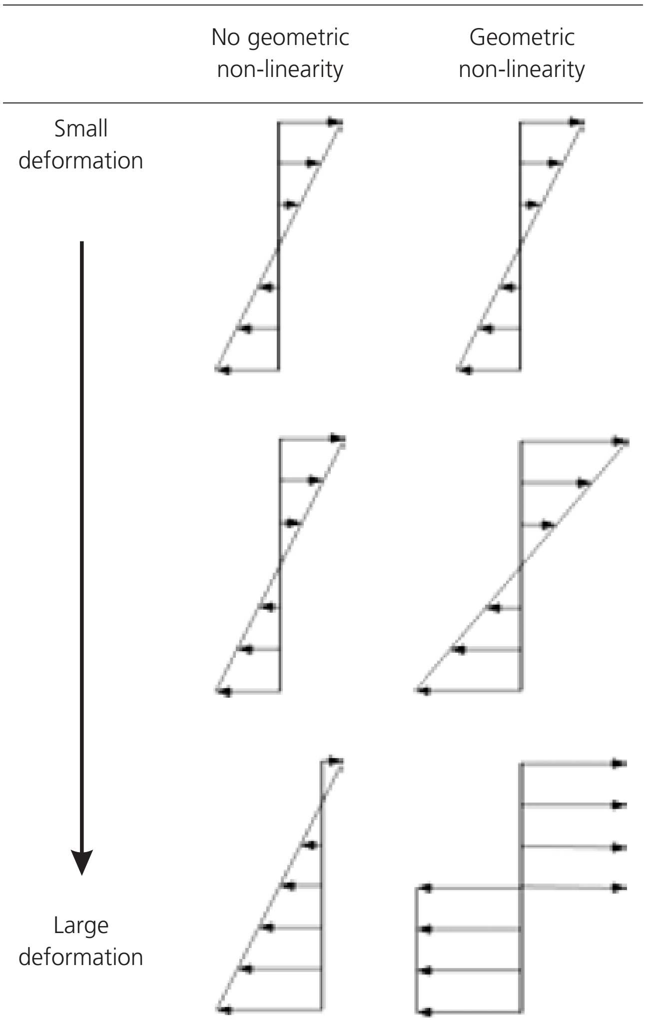

Table 1. Variation of stress in a beam cross-section with and without considering geometric non-linearity

In this section the non-linear static pushdown analyses of steel and RC beam–column sub-assemblages, depicted in Figure 1, were conducted to demonstrate the effect of catenary action on the structural response and member forces. The beam–column sub-assemblages were designed to have similar yield strength when subjected to removal of the centre column. They have fixed boundary conditions at both beam ends and the column located between the two beams was assumed to be removed. The beams of the steel sub-assemblage are composed of H-sections, dimensions $250 \times 125 \times 6 \times 9$ mm, and the cross-sections of the RC beams were 300×400 (mm) in size. In the analysis, the effect of sudden loss of a column was simulated by the vertical point load ***P***. Throughout the study the program code OpenSees (Mazzoni *et al.*, 2007) was used for non-linear analyses, in which the cross-section of a structural member was modelled by many fibre elements. For modelling of beam elements without considering catenary action, the 'linear' geometric transformation option was used, whereas the 'corotational' geometric transformation was selected for beams analysed considering catenary action. The hysteretic behaviour of structural steel and reinforcing bars was modelled by the 'hysteretic material' as shown in Figure 2(a). The cover concrete and the core concrete were modelled by the 'concrete01' and the 'concrete02' materials, respectively, as

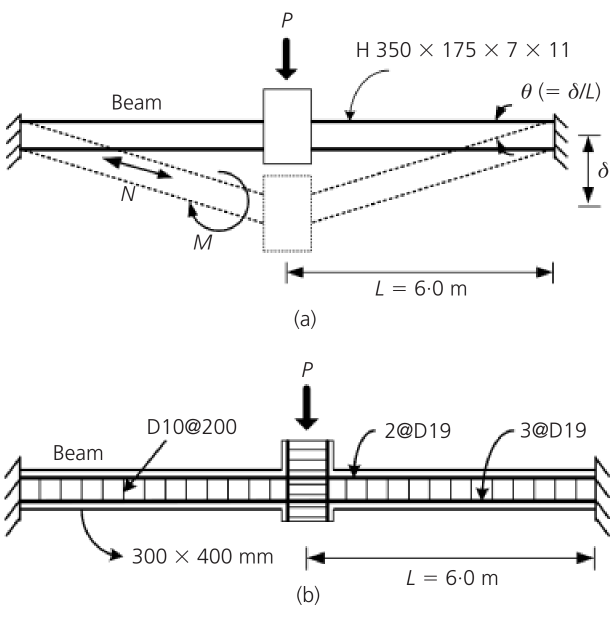

Figure 1. Beam–column sub-assemblage analysis model with fixed ends: (a) steel sub-assemblage; (b) reinforced concrete sub-assemblage

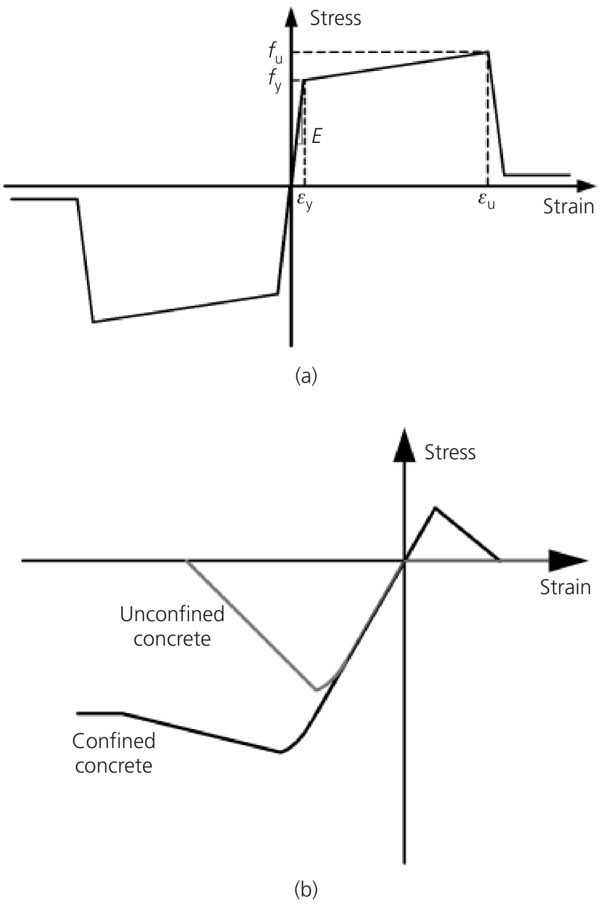

Figure 2. Stress–strain relationship of structural materials

shown in Figure 2(b). The behaviours of cover and core concrete were modelled based on Mander *et al.* (1988).

Figure 3 shows the non-linear static pushdown analysis results of the steel sub-assemblage structure. It can be observed in Figure 3(a) that, when the catenary action was considered, the progressive collapse-resisting capacity of the steel sub-assemblage became significantly higher than the capacity obtained without considering catenary action. Without catenary action, the bending moment kept increasing due to strain hardening but no axial force was induced in the beams, as shown in Figure 3(b). On the other hand, when catenary action was considered, the bending moment dropped immediately after the plastic hinge formed, while the axial force kept increasing. When the vertical displacement increased to larger than about 45 cm the axial force became more dominant than the bending moment in resisting progressive collapse. However, it should be pointed out that, for a catenary action to be activated, the beam–column connections need to have large deformation capacity.

Figure 4 depicts the pushdown analysis results of the RC beam–column sub-assemblage shown in Figure 4(b). It can be observed that the pushdown curve obtained without considering catenary action is higher than the curve obtained considering geometrical non-linearity up to the vertical displacement of about 70 cm. It was observed that, when catenary action was considered, immediately after the peak strength was reached the strength decreased rapidly due to concrete crushing. Then the strength increased again due to activation of catenary force. At the point of lowest strength the compression steel started to undergo tension and the beams began to act as tension members. It is interesting to note that when catenary action was considered the strength of the RC beam–column sub-assemblage decreased after yielding of the sysem and increased again when the catenary force was activated, whereas in the steel sub-assemblage the strength kept increasing

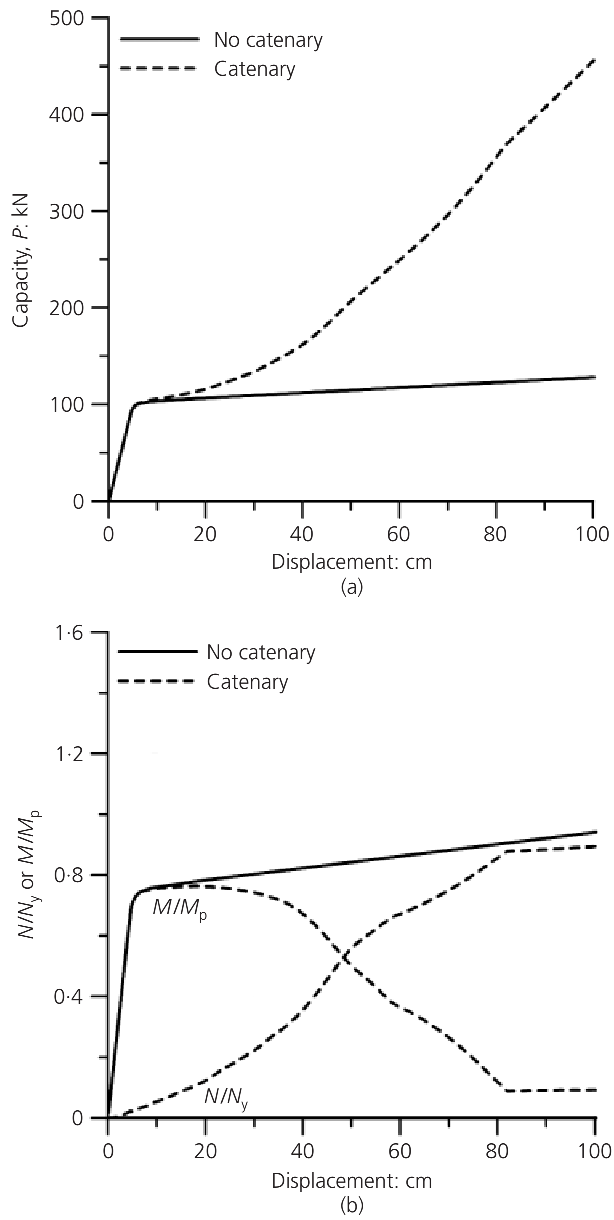

Figure 3. Pushdown analysis results of the steel sub-assemblage model

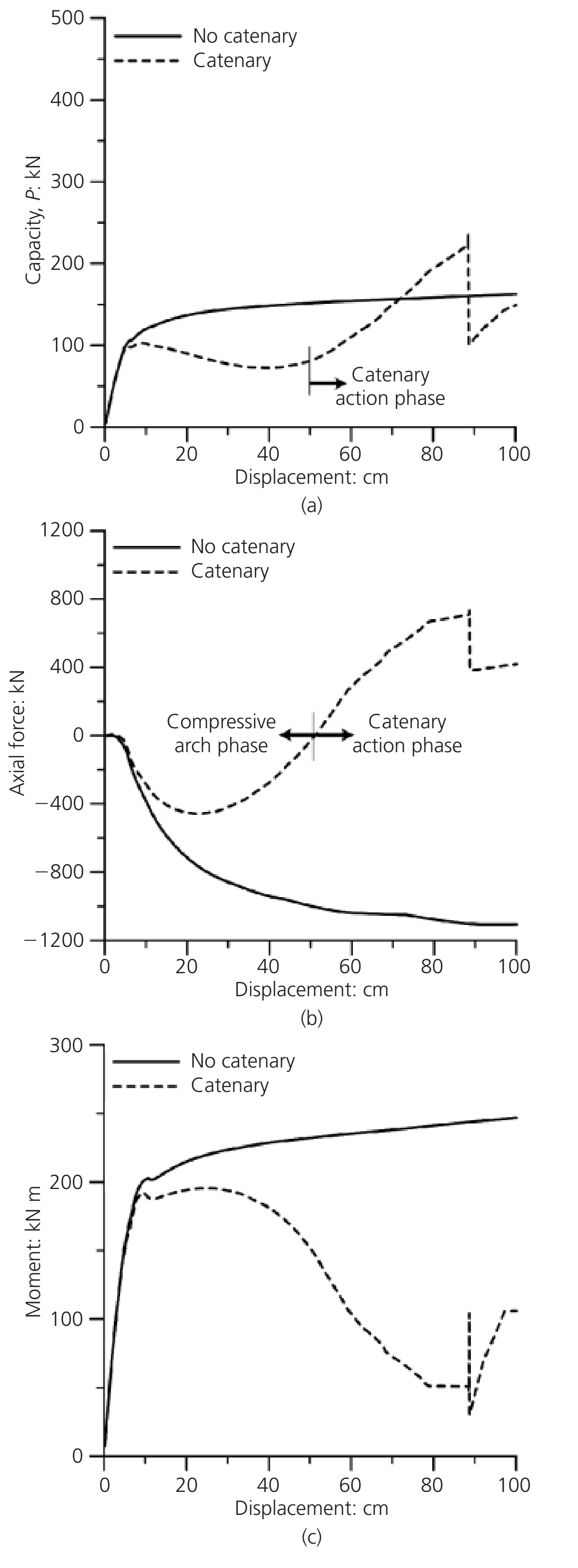

Figure 4. Pushdown analysis results of the RC sub-assemblage model: (a) pushdown curves; (b) variation of axial force; (c) variation of bending moment

after yielding due to catenary action, as observed in Figure 3. When a RC beam is under large deflection, the neutral axis moves upward, as shown in Table 1. In this case the concrete below the neutral axis cannot resist any tensile load. When the whole cross-section is under tension, only the reinforcing bars resist external load as tension members. This process corresponds to the down slope of the pushdown curve in Figure 4(a) and results in lower strength than that predicted by the analytical model in which catenary action is not considered. Figure 4(b) shows the variation of axial force as the vertical displacement increased with and without catenary action. It can be observed that, when catenary action was not considered, the axial force kept decreasing as vertical displacement increased due to a compressive arch action. However, when catenary action was included in the analysis, the axial force first decreased at the compressive arch phase; then it increased due to catenary action and, finally, the whole cross-section was subjected to tensile force. This variation of axial force was also observed in the experiment of continuous beams with fixed ends carried out by Orton (2007). It can be observed in Figures 4(b) and 4(c) that, as the catenary force started to be activated, the bending moment resisted by the beams began to decrease.

Comparison with experimental results of RC structures

The validity of the modelling technique for catenary action used in this study was verified by comparing the analysis results with those of experiments conducted by Sasani and Kropelnicki (2008) and Yi *et al.* (2008). Sasani and Kropelnicki (2008) conducted a RC beam–column sub-assemblage test to study the behaviour of beams bridging over a removed column. Figure 5(a) compares the vertical force–displacement relationships obtained from the experiment of Sasani and Kropelnicki and simulated by the analytical modelling. When catenary action was not considered, the analytical model overestimated the force–displacement relationship obtained by experiment, especially after the vertical displacement exceeded about 5 cm, whereas the analytical model including geometrical non-linearity predicted the experimental result more precisely. It was observed that the analytical model with geometrical non-linearity could predict the rapid decrease in strength caused by concrete crushing and the subsequent increase in strength again due to activation of catenary force observed in the experiment. As the bond slip of reinforcing bars observed in the experiment was not considered in the analytical modelling, the analytical pushdown curve slightly overestimated the experimental data of Sasani and Kropelnicki. Yi *et al.* (2008) conducted an experiment to investigate progressive failure of a three-storey RC frame due to the loss of a first storey column. The test results for the pushdown curve are compared with analysis results in Figure 5(b). As in the sub-assemablage test described above, the results of the analytical modelling considering catenary action better predicted the experimental results than the model not considering catenary action. It is reported in the experiment that severe concrete crushing was observed as the vertical displacement exceeded 70 mm and tension cracks penetrated through the compression zones, indicating the formation of the catenary

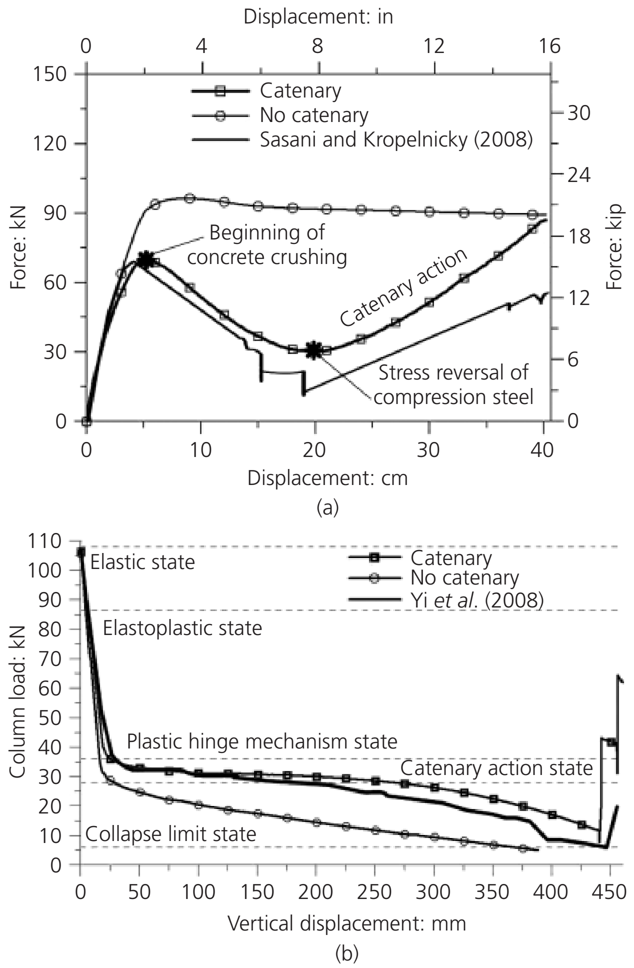

Figure 5. Comparison of force–displacement relationship obtained by experiments and numerical simulation: (a) RC beam–column sub-assembly of Sasani and Kropelnicki (2008); (b) three-storey RC frame tested by Yi *et al.* (2008)

mechanism in the beams. At this point the measured strain in the upper steel bars changed from compression to tension. These phenomena were also observed in the analytical modelling when catenary action was considered. In this three-storey frame specimen the abrupt change in strength occurring in the sub-assemblage specimen was not observed in both the experiment and the analysis.

Parameters affecting performance of a RC subassemblage

In the previous section it was observed that the progressive collapse-resisting capacity of a beam–column sub-assemblage depended largely on the activation of catenary action. In this section, pushdown analysis of a RC beam–column sub-assembalge was conducted to discover the important parameters affecting catenary action. The RC sub-assemblage has the same span length and boundary conditions as the beam–column sub-assemblage shown in Figure 1, and has a rectangular cross-section of 400 mm × 500 mm. Table 2 shows the design variables used for parametric study. Figure 6 presents the non-linear static pushdown analysis results of the model structure with and without considering catenary action. Two types of concrete ultimate strength, 21 MPa and 30 MPa, were used in the analysis. Figure 6(a) shows that, when catenary action was not considered (NCA), the load-resisting capacity increased as the ultimate strength of the concrete increased. However, when catenary action was considered (WCA) the increase in load-resisting capacity was almost neglible, especially in large deformations, in which only reinforcing bars resisted external load by catenary action. It can be observed in Figure 6(b) that the bending moment imposed on the beams slightly increased with the increase in concrete strength when the vertical deflection was less than 50 cm. However, the axial force induced in the beams did not change with the variation of concrete strength.

Figure 7 depicts the pushdown curves of the sub-assemblage structure with various numbers of tension reinforcing bars (D19) when the number of compression steel bars was fixed to two. It can be observed that as the number of tension rebars increased, progressive collapse-resisting capacity also increased. It can also be noticed that the catenary action was first activated at a vertical deflection of around 40 cm, and that when there was no tension rebar the catenary action was not activated.

Figure 8 shows the variation of pushdown curves depending on yield stress of tension rebars. Reinforcing steels with yield stress of 300 MPa, 400 MPa and 500 MPa were used in the analysis. It can be observed that as yield stress increases the progressive collapse-resisting capacity and the effect of catenary action also increase. In comparison with the results with those presented in Fig. 7, it can be concluded that the increase of yield stress of tension rebars has an effect similar to the increase in the number of tension rebars.

Figure 9 shows the pushdown curves of the sub-assemblage structure with varying number of compression rebars when the number of tension rebars is fixed to three. The analysis results were similar to those obtained with varying number of tension reinforcing bars and fixed compression steel shown in Figure 7. The difference is that when there were no tension rebars, catenary

Variables	Values
Concrete compressive strength	21 MPa 30 MPa
Number of tension steel bars (D19)	0 2 4
Stirrups (D10@100 mm)	Without stirrups (NS) With stirrups (S)

Table 2. Design variables of sub-assemblage model

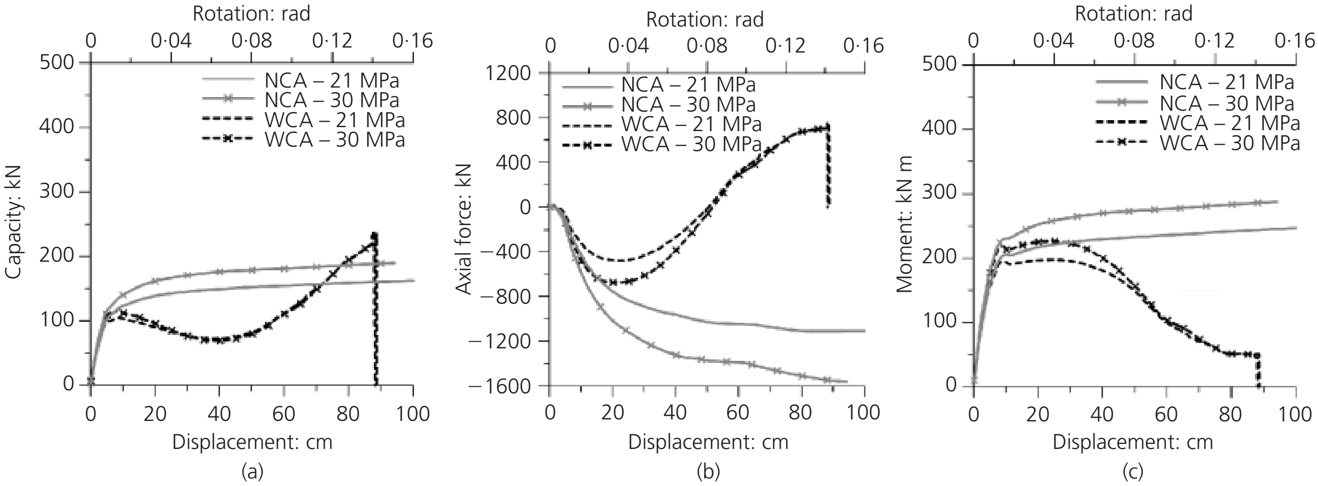

Figure 6. Pushdown analysis results of a RC beam–column sub-assemblage with ultimate strengths of 21 and 30 MPa (NCA: no catenary action; WCA: with catenary action)

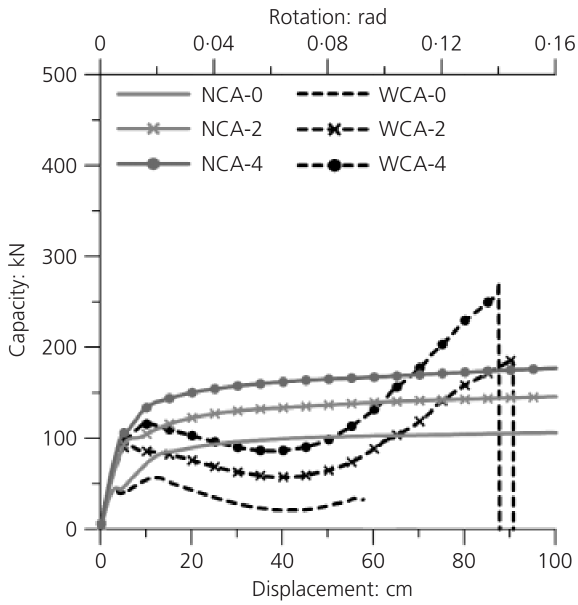

Figure 7. Pushdown analysis results of a RC beam–column sub-assemblage with different numbers of rebars

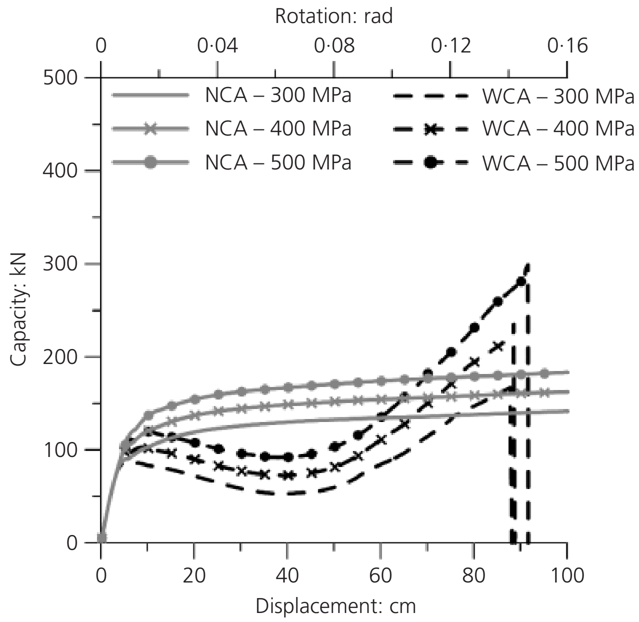

Figure 8. Pushdown analysis results of a RC beam–column sub-assemblage with different yield strength of rebars (NCA: no catenary action; WCA: with catenary action)

action was not activated, while catenary action was still observed when no compression steel was placed. This implies that catenary action is activated by the existence of tension reinforcing steel and the placement of compression steel helps to increase the collapse-resisting capacity through catenary action.

The effect of shear stirrups on the progressive collapse-resisting capacity of the analysis model is plotted in Figure 10. The load-resisting capacity of the structure without stirrups dropped rapidly due to crushing of concrete immediately after the maximum strength was reached, whereas in the structure with D10@100 mm shear stirrups the crushing of concrete and consequently the activation of catenary action occurred at larger deflection. However, as the amount of reinforcing steel was the same, the ultimate capacity of the structure at large deformation did not change significantly.

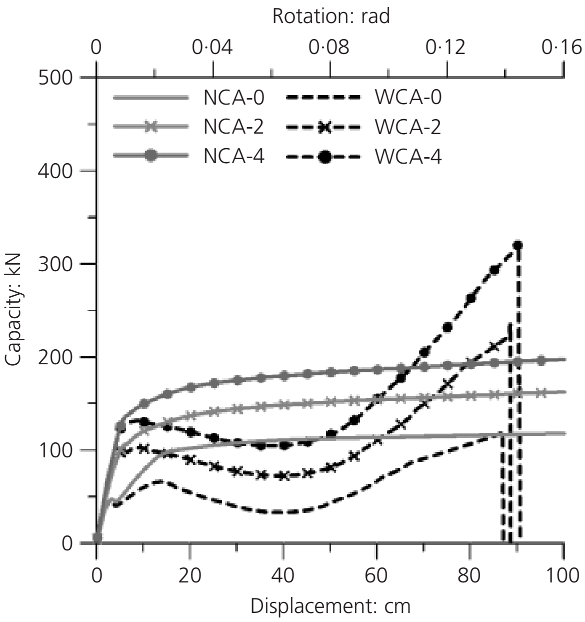

Figure 9. Pushdown analysis results of a RC beam–column sub-assemblage with different amounts of compression steel

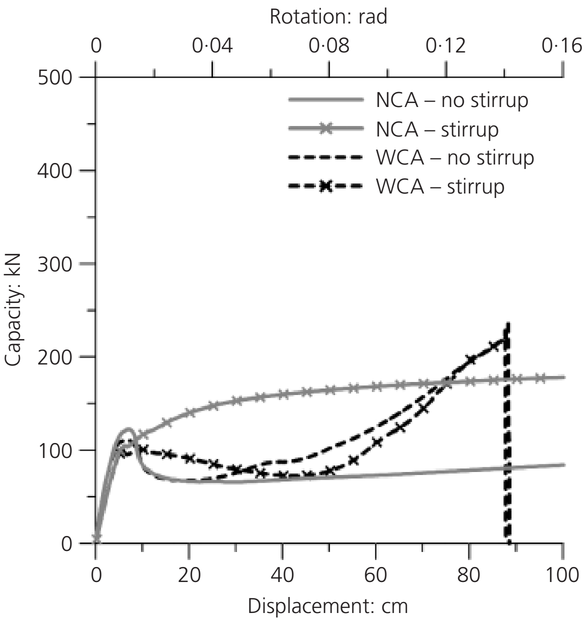

Figure 10. Pushdown analysis results of the sub-assemblage with and without stirrups

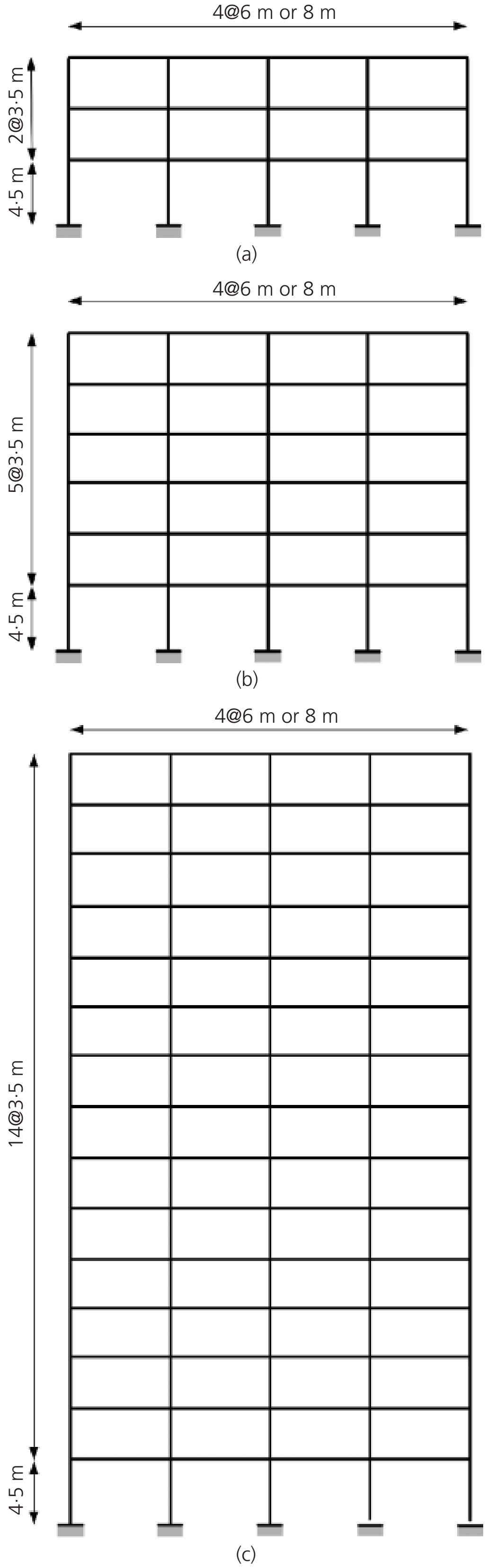

Figure 11. Elevation of model structures for analysis: (a) three-storey; (b) six-storey; (c) 15-storey

Collapse performance of RC framed structures

Design and modelling of analysis structures

In this section the detailed information regarding example structures and analysis methods for progressive collapse are provided. The analysis models, shown in Figure 11, are three-, six-, and 15-storey RC moment frames designed with (SMRFs) and without (OMRFs) considering seismic load. The former corresponds to a special moment-resisting frame (SMRF) and the latter corresponds to an ordinary moment-resisting frame (OMRF). The structures are external frames taken out of three- by four-bay moment-resisting frames with the plan dimensions shown in Figure 12. In SMRFs, the sum of the nominal flexural strengths of the columns framing into a joint was designed to be at least 1·2 times larger than that of the beams framing into the joint to ensure a strong column–weak beam mechanism. The response modification factors corresponding to SMRFs and OMRFs are 8 and 3, respectively. The model structures were designed to have two different span lengths, 6 m and 8 m. The design dead and live loads were 4·5 kN/m^2 and 2·5 kN/m^2, respectively. Structural members were designed in accordance with the ACI 318-02 (ACI, 2002) and seismic load was determined based on the 2006 international building code (IBC) (ICC, 2006). The coefficients for seismic design load are presented in Table 3. Table 4 shows the member size and rebars of the three-storey model structures for analysis. Figure 13 depicts the rebar placement in beams in the seismic-designed and non-seismic-designed model structures. It can be observed that in the structures designed with seismic load both the top and bottom rebars are continuous, whereas in the non-seismic-designed structures they are discontinuous in the beam–column joints. In the model structures for analysis, member sizes were varied in every three storeys.

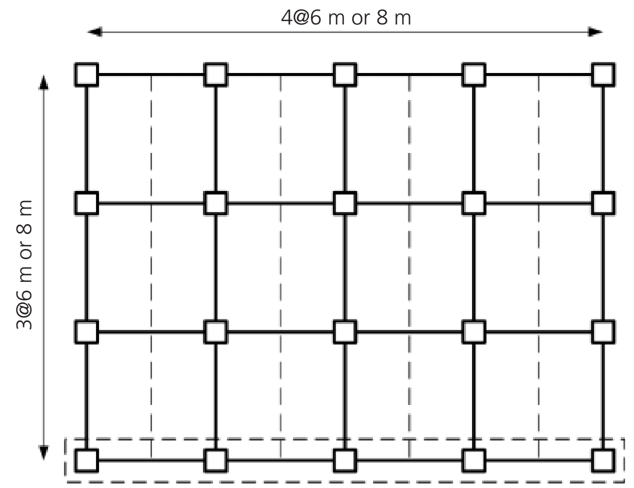

Figure 12. Structural plan of model structure for analysis

Figure 14 shows the gravity load applied on the model structures for progressive collapse analysis. For static analysis, the GSA (2003) recommends using a dynamic amplification factor of 2·0 in load combination in the bay from which a column is removed,

Site class	Stiff soil (class D; $F_a = 1{\cdot}0$, $F_v = 1{\cdot}5$)
Design earthquake	$S_{DS} = 1{\cdot}07\boldsymbol{g}$, $S_{D1} = 0{\cdot}79\boldsymbol{g}$
Seismic use group	Group I: $I_E = 1{\cdot}0$ (three-storey, six-storey)
	Group II: $I_E = 1{\cdot}25$ (15-storey)
Seismic design category	D

Table 3. Seismic design variables for model structures

Type	Storey	Span: m	Member size: mm	Reinforcing steel			
				Ends		Mid-span	
				Bottom	Top	Bottom	Top
Non-seismic designed structure	1~3 storeys	6	350 × 450	3-D13	3-D16	4-D13	2-D13
				D10@190		D10@190	
		8	450 × 550	2-D19	4-D19	5-D16	2-D16
				D10@240		D10@240	
Seismic designed structure	1~3 storeys	6	450 × 550	2-D22	2-D25	2-D22	2-D19
				D10@110		D10@240	
		8	450 × 600	3-D19	4-D22	3-D19	2-D19
				D10@120		D10@250	

Table 4. Member size and reinforcement of beams in three-storey structures

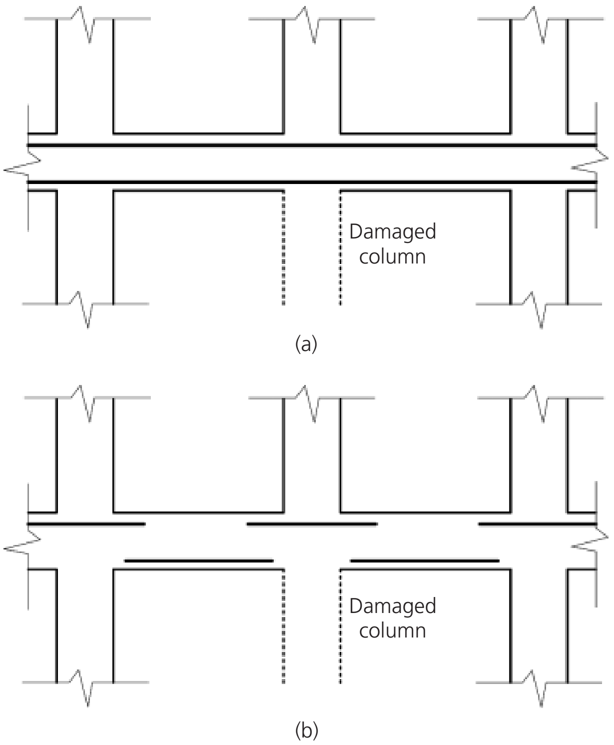

Figure 13. Rebar placement in model structures: (a) seismic design; (b) non-seismic design

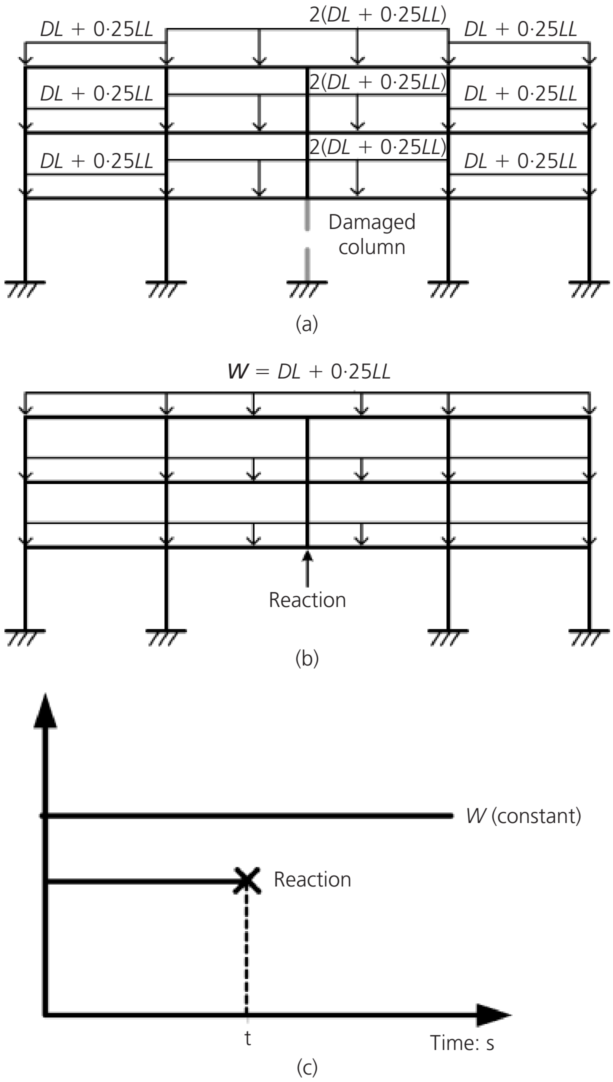

Figure 14. Applied load for progressive collapse analysis: (a) gravity load applied for non-linear static analysis; (b) gravity load applied for non-linear dynamic analysis; (c) application of vertical load for dynamic analysis

as shown in Figure 14(a), but for dynamic analysis no amplification factor is applied. To carry out dynamic analysis, the axial force acting on a column is computed first before the column is removed. Then the column is replaced by point loads which are the equivalent of its member forces, as shown in Figure 14(b). To simulate the phenomenon that the column was abruptly removed, the member forces were removed after 7 s had elapsed, as shown in Figure 14(c), where W is the vertical load, and reactions are axial force, shear force and bending moment equivalent to the member forces of the removed column. For dynamic analysis, a 2% damping ratio was estimated using the Rayleigh method.

Pushdown analysis results

Figures 15 and 16 present non-linear static analysis results of the model structures. The analyses were carried out until the vertical deflection over beam length became 0·2 rad, in consideration of the fact that the test specimens of Sasani and Kropelnicki (2008) and Yi *et al.* (2008) failed at a beam deflection of about 0·195 rad. The progressive collapse-resisting capacity was expressed as a load factor corresponding to a certain vertical deflection level. The load factor of 1·0 denotes the loading state specified in the GSA guideline including the dynamic amplification factor, 2 × (dead load + 0·25 × live load).

Figure 15 shows the pushdown curves of the three-storey RC frames, where it can be observed that the maximum load factor of the structure designed without considering seismic load did not reach 0·4 and showed a large possibility for collapse. However, the maximum load factor of the structure designed with seismic load reached close to 1·0. When a corner column was removed the consideration of catenary action did not affect the pushdown curve significantly. However, when a middle column was removed the progressive collapse-resisting capacity was somewhat reduced when catenary action was considered. The same phenomenon was also observed in the six- and the 15-storey structures. It can be observed that as the span length increased from 6 m to 8 m the pushdown curves decreased significantly, and even in the seismic-designed structure with 8 m span the maximum load factor was much less than 1·0. No significant difference was observed in the

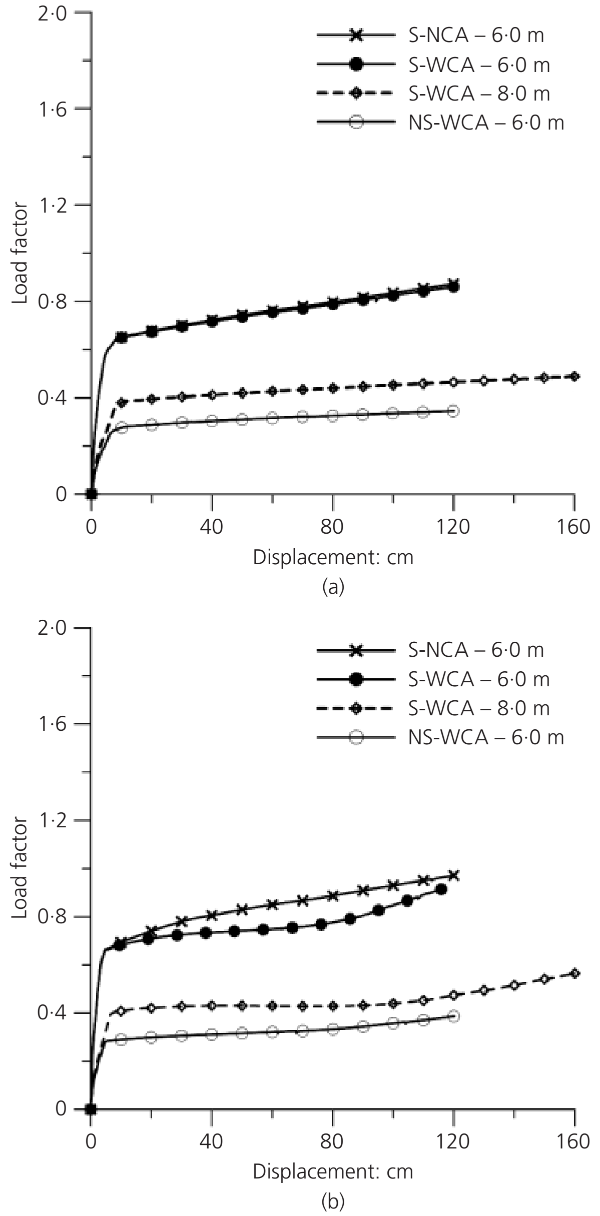

Figure 15. Pushdown analysis results of three-storey structures (S: seismic design; NS: non-seismic design)

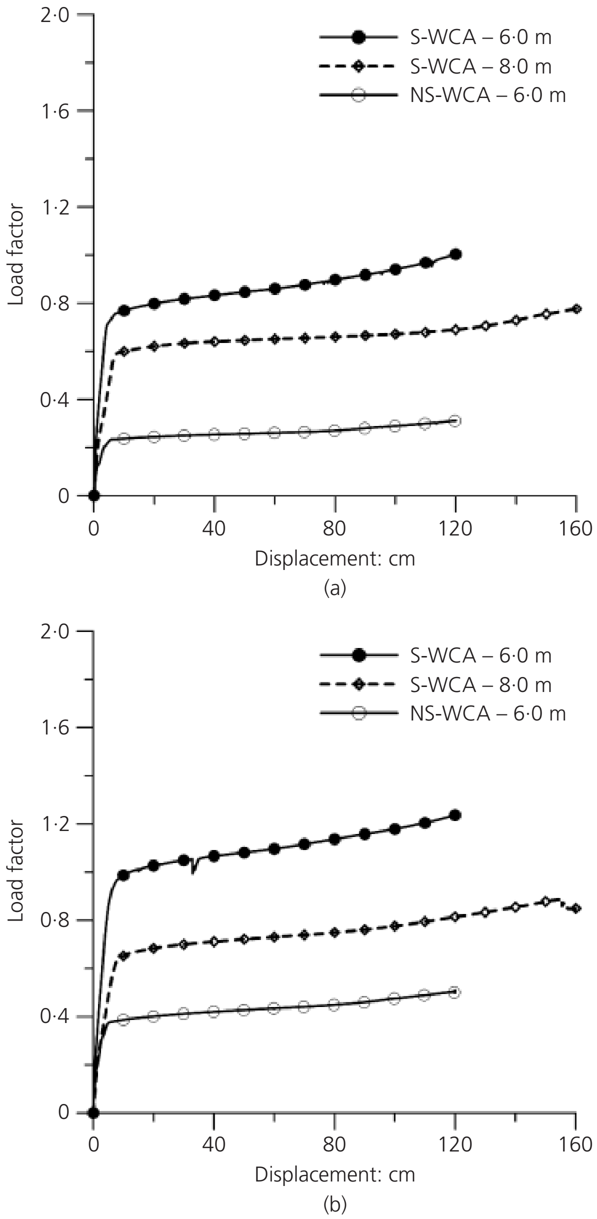

Figure 16. Pushdown analysis results of six- and 15-storey structures designed with seismic load when the middle column was removed

progressive collapse-resisting capacity between the removal of corner and middle columns.

Figure 16 depicts pushdown curves of the six- and 15-storey structures with 6 m and 8 m span lengths when a middle column was removed, where it can be observed that as the number of storey increased, the progressive collapse potential decreased. The maximum load factors of non-seismic-designed structures were less than 0·4 and the structures showed a high possibility of progressive collapse. On the other hand, the load factors of the structures designed with seismic load increased more than twice as much as those of non-seismic-designed structures.

Non-linear dynamic analysis results

Figures 17 and 18 show the non-linear dynamic analysis results of the model structures. It can be observed that all model structures designed without considering seismic load failed when the corner column or the middle column was removed, whereas the structures designed with seismic load remained stable. This was expected from the pushdown analysis results, where the

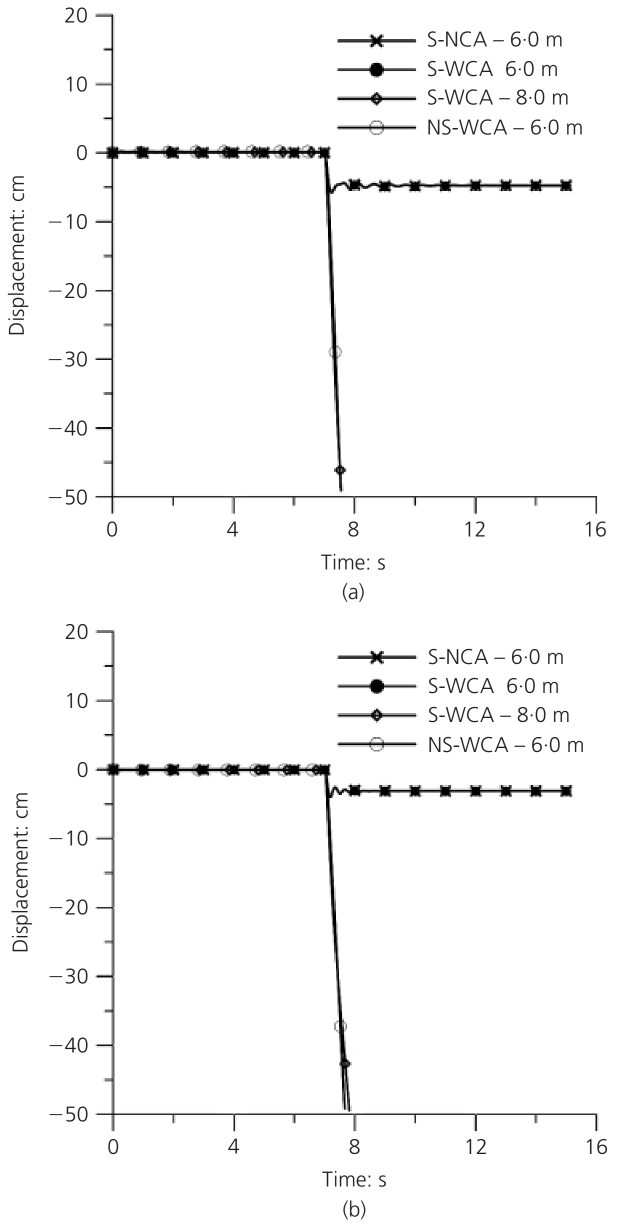

Figure 17. Non-linear dynamic analysis results of three-storey structures

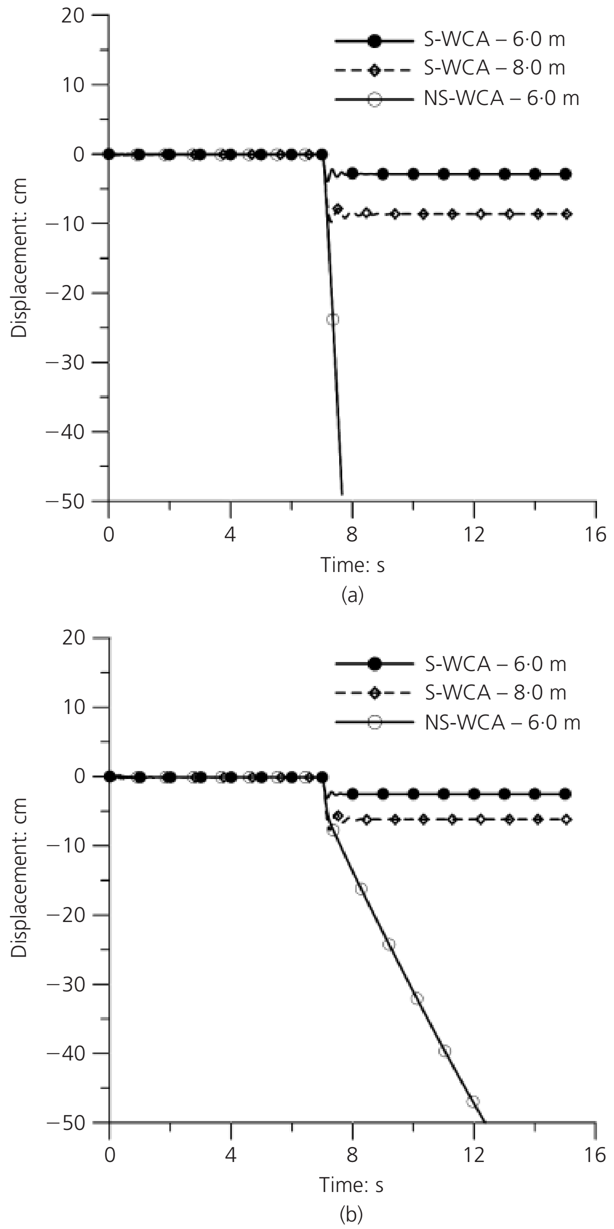

Figure 18. Non-linear dynamic analysis results of six- and 15-storey structures when the middle column was removed

maximum load factors of the non-seismic-designed structure turned out to be less than 0·5. The vertical deflection of the seismic-designed structures decreased as the number of storeys increased and the span length decreased. The deflection was larger when one of the centre columns was removed than when a corner column was removed. These results correspond well with the non-linear static pushdown analysis results obtained above. It was observed that the consideration of catenary action did not affect the vertical displacement of the model structures significantly, since the structures reached equilibrium condition at vertical displacements of less than 10 cm and the catenary force was not yet activated.

Conclusions

In this study the progressive collapse potential of RC moment frame structures was investigated considering catenary action. To this end, three-, six- and 15-storey RC moment frames were designed as analysis model structures with and without considering seismic load, and their progressive collapse potentials were compared by performing non-linear static and dynamic analyses.

The non-linear static analysis results of beam–column sub-assemblages obtained without considering catenary action turned out to overestimate the test results significantly. The activation of catenary action depended significantly on the amount of reinforcing steel, but not on the ultimate strength of the concrete. The number of stirrups affected the onset point of catenary action.

In a multi-storey RC framed structure the effect of catenary action was not so prominent as in the analysis of a beam–column sub-assemblage, owing to more flexible beam boundary conditions. The progressive collapse potential of a framed structure increased as span length increased and as the number of storeys decreased. Compared with when a middle column was removed, the progressive collapse potential increased when a corner column was removed. The non-seismic-designed RC moment frame structures turned out to have high progressive collapse potential, whereas the seismic-designed structures showed enough strength and ductility to resist progressive collapse caused by sudden removal of a first storey column. In comparison with RC moment-resisting frames designed with the same loading condition, steel moment frames showed more stable and ductile behaviour, especially when catenary action was considered.

Whether catenary action will be fully activated or not depends largely on the connection strength. In this study it was assumed that the beam–column connections had enough strength to sustain the catenary force induced in the beams at large deformation. It was also assumed in the analysis that only a column was lost, while the beam–column joints at the top or bottom of the column remained intact. However, there is always the danger that beam–column joints could be ripped out under extreme loading, leaving no catenary action possible. Further research on connection details and catenary force will be necessary to evaluate the progressive collapse-resisting capability of RC structures.

Acknowledgement

This research was supported by a grant (Code No. '09 R&D A01) from the Cutting-Edge Urban Redevelopment Program funded by the Korean Ministry of Land, Transport, and Maritime Affairs.

REFERENCES

ACI (American Concrete Institute) (2002) *Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (318R-02)*. American Concrete Institute, Farmington Hill, Michigan, USA, ACI 318.

ASCE (American Society of Civil Engineers) (2005) *Minimum Design Loads for Buildings and Other Structures*. American Society of Civil Engineers, Reston, Virginia, USA, ASCE 7-05.

Astaneh-Asl A (2003) Progressive collapse prevention in new and existing buildings. In *Proceedings of the 9th Arab Structural Engineering Conference, Abu Dhabi*, 1001–1008.

CEN (European Committee for Standardisation) (2002) *Eurocode 1, Actions on Structures*. European Committee for Standardization, Brussels, Belgium.

Choi H and Kim J (2010) Progressive collapse-resisting capacity of reinforced concrete beam-column subassemblage. *Magazine of Concrete Research* **63(4)**: 297–310.

GSA (US General Services Administration) (2003) *Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects*. The US General Services Administration, Washington, DC.

ICC (International Code Council) (2006) *International Building Code*. International Code Council, Falls Church, Virginia.

Kim J and Park J (2008) Design of steel moment frames considering progressive collapse. *Steel and Composite Structures* **8(1)**: 2008: 85–98.

Kim T, Kim J and Park J (2009a) Investigation of progressive collapse-resisting capability of steel moment frames using push-down analysis. *Journal of Performance of Constructed Facilities* **23(5)**: 327–335.

Kim H, Kim J and An D (2009b) Development of integrated system for progressive collapse analysis of building structures considering dynamic effects. *Advances in Engineering Software Archive* **40(1)**: 1–8.

Mander JB, Priestley MJN and Park R (1988) Theoretical stress–strain model for confined concrete. *Journal of Structural Engineering* **113(8)**: 1804–1826.

Marjanishvili SM and Agnew E (2006) Comparison of various procedures for progressive collapse analysis. *Journal of Performance of Constructed Facilities* **20(4)**: 365–374.

Mazzoni S, McKenna F, Scott MH, Fenves GL (2007) *Open System for Earthquake Engineering Simulation, User Command-Language Manual*. Pacific Earthquake Engineering Research Center, Berkeley, California.

NRCC (National Research Council of Canada) (1995) *National Building Code of Canada*. National Research Council of Canada, Ottawa, Canada.

NIST (National Institute of Standards and Technology) (2006) *Best Practices for Reducing the Potential for Progressive Collapse in Buildings*. NIST, Gaithersburg, Maryland, USA.

Orton SL (2007) *Development of a CFRP System to Provide Continuity in Existing Reinforced Concrete Buildings Vulnerable to Progressive Collapse*. PhD thesis, University of Texas at Austin, USA.

Powell G (2005) Progressive collapse: Case study using nonlinear analysis. *Proceedings of the 2005 Structures Congress and the 2005 Forensic Engineering Symposium*, New York, USA.

Sasani M and Kropelnicki J (2008) Progressive collapse analysis of an RC structure. *The Structural Design of Tall and Special Buildings* **17(4)**: 757–771.

UFC (Unified Facilities Criteria) (2005) *Design of Buildings to Resist Progressive Collapse*. Department of Defense, USA, UFC4-023-03.

Vlassis AG, Issuddin BA, Elghazouli AY and Nethercot DA (2009) Progressive collapse of multi-story buildings due to failed floor impact. *Engineering Structures* **31(7)**: 1522–1534.

Yi WJ, He QF, Xiao Y and Kunnath SK (2008) Experimental study on progressive collapse-resistant behavior of reinforced concrete frame structures. *ACI Structural Journal* **105(4)**: 433–439

WHAT DO YOU THINK?

To discuss this paper, please submit up to 500 words to the editor at www.editorialmanager.com/macr by 1 July 2012. Your contribution will be forwarded to the author(s) for a reply and, if considered appropriate by the editorial panel, will be published as a discussion in a future issue of the journal.