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Retrofit of RC frames against progressive collapse using prestressing tendons

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SUMMARY

This study investigates the effect of prestressing tendons on the progressive collapse performance of a 6- and 20-story reinforced concrete model structures. According to nonlinear static and dynamic analysis results, the analysis model structures turned out to be vulnerable to progressive collapse caused by sudden loss of a first story column. However, the RC structures reinforced by external prestressing tendons along floor girders showed stable behavior against progressive collapse. The retrofit effect increased as the initial tension and cross-sectional area of tendons increased. The incremental dynamic analyses showed that the seismic performance of the model structure was also enhanced after the retrofit using tendons. Based on analysis results, it was concluded that the retrofit of existing buildings using prestressing tendons could be effective for increasing both progressive collapse resisting capacity and seismic performance of RC framed structures. Copyright © 2011 John Wiley & Sons, Ltd.

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KEY WORDS: post-tension; RC structures; progressive collapse; seismic performance

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. An abnormal load includes any loading condition that is not considered in normal course of design but may cause significant damage to structures. In the USA, the General Service Administration (GSA) presented a practical guideline for design to reduce collapse potential of federal buildings (General Services Administration (GSA), 2003), and the Department of Defence (DoD) also presented a guideline for the new and existing military facilities (UFC 2005). The analysis method recommended in these guidelines is the alternative path method. In this approach, the structure is designed in such a way that if any one component fails, alternate paths are available and a general collapse does not occur. In most cases, design for redundancy requires that a building structure be able to tolerate loss of any one column without collapse. Recently, the performances against progressive collapse have been studied for steel structures (Powell, 2005; Kim and An, 2009; Kim and Kim, 2009; Park and Kim, 2010) and for reinforced concrete structures (Sassani and Kropelnicki, 2007; Tsai *et al*. 2007; Cleland, 2008; Yi *et al*., 2008). Analysis procedures and program software were developed to simulate collapse behavior of structures (Kaewkulchai and Williamson, 2003; Kim *et al*., 2009).

In recent years, external prestressing has become a useful method for strengthening existing structures and has been increasingly used in the construction of segmental bridges. Astaneh (2003) investigated the viability of a steel cable-based system to prevent progressive collapse of buildings by conducting full-scale specimen tests of a single story building and showed that the progressive

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collapse resisting capacity could be enhanced using high-strength cables. El-Ariss (2004) developed a simple analytical model for predicting the flexural behavior of reinforced concrete members with external tendons under service loads accounting for various factors such as eccentricity variations of external tendons and span-to-depth ratios. Ng and Tan (2006a, 2006b) carried out experiments of nine simply supported, externally prestressed beams subjected to two symmetrically applied concentrated loads to evaluate the effect of span-to-depth ratio and second-order effects. Du *et al*. (2008) carried out analytical study on ductile capacity of beams depending on the location of prestressing tendons. Garlock *et al*. (2008) investigated the influence of various design parameters on the seismic response of the post-tensioned steel moment-resisting frame. Kaya and Arslan (2009) investigated the effect of prestressed strand diameters on beam-to-column connections in both experimental and analytical aspects. Those studies mentioned above have confirmed that external prestressing of floor beams can be effective in improving load-resisting capacity of framed structures.

In this study, the progressive collapse potential of reinforced concrete structures retrofitted by prestressing tendons is evaluated. For retrofit the prestressing tendons are placed along the beams, and various levels of prestressing forces are applied to enhance load-resisting capacity of the retrofitted structure. Nonlinear static and dynamic analyses of the model structures subjected to sudden loss of a column are performed to evaluate progressive collapse potential. The seismic performances of the model structures with and without retrofit are also evaluated by incremental dynamic analysis.

2. ANALYSIS MODEL STRUCTURES AND APPLIED LOAD

To validate the effect of external prestressing on preventing progressive collapse of moment frames, the six-story structures with three-by-four bays were prepared. Figure 1 shows the plan shape and elevation view of the six-story analysis model structures designed without considering seismic load. The model structures were designed per Korean Building Code (KBC 2009) with dead and live loads of 4.0 kN/m^2 and 3.5 kN/m^2, respectively. The compressive strength of concrete is 24 MPa, and the

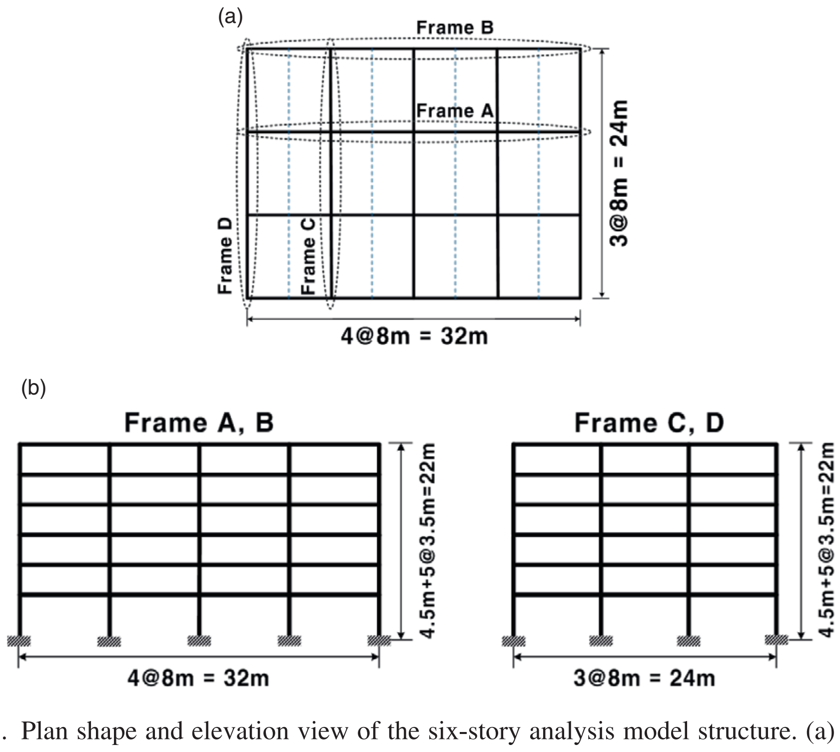

Figure 1. Plan shape and elevation view of the six-story analysis model structure. (a) Plan. (b) Elevation.

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Table 1. Member sizes of Frames A and B.

Frames	Bay	Beams	Columns
A	Int.	600 × 400	500 × 500
	Ext.	600 × 400	450 × 450
B	Int.	550 × 300	500 × 500
	Ext.	550 × 300	450 × 450

Table 2. Initial tension and cross-sectional area of tendons.

Frame	Initial tension (kN)	Tendon
A	2372	12-D17.8 mm (A = 223 mm^2)
B	1170	8-D15.2 mm (A = 165 mm^2)
C	2372	12-D17.8 mm (A = 223 mm^2)
D	1170	8-D15.2 mm (A = 165 mm^2)

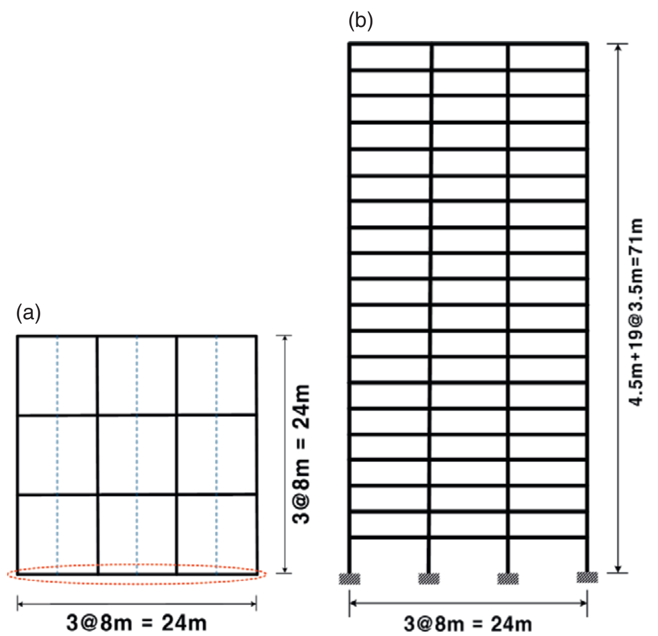

Figure 2. Twenty-story analysis model. (a) Plan. (b) Elevation.

yield stress of re-bars is 400 MPa. Table 1 shows the member sizes of the exterior and interior frames of the model structure, and Table 2 shows the initial tension and cross-sectional area of tendons. The interior frames with T-shaped beams, Frames A and C, and the exterior frames with angle-shaped beams, Frames B and D, were analyzed separately. Figure 2 depicts the structural plan and elevation of the 20-story RC moment frame structure with three bays. The structure was designed considering both gravity and lateral seismic and wind load. The exterior frame marked on the structural plan was separated for analysis. The sizes of the center columns changed from 500 × 500 (mm) in the first story to 700 × 700 in the top story, and those of the corner columns varied from 600 × 600 to 800 × 800. The girder size changed from 500 × 600 to 400 × 600.

Figure 3 illustrates the stress-strain relationship of structural materials used in the analysis. The material model for concrete suggested by Mander *et al*. (1988), which considers the confinement effect of re-bars, was applied for numerical modeling. The behavior of re-bars was modeled by

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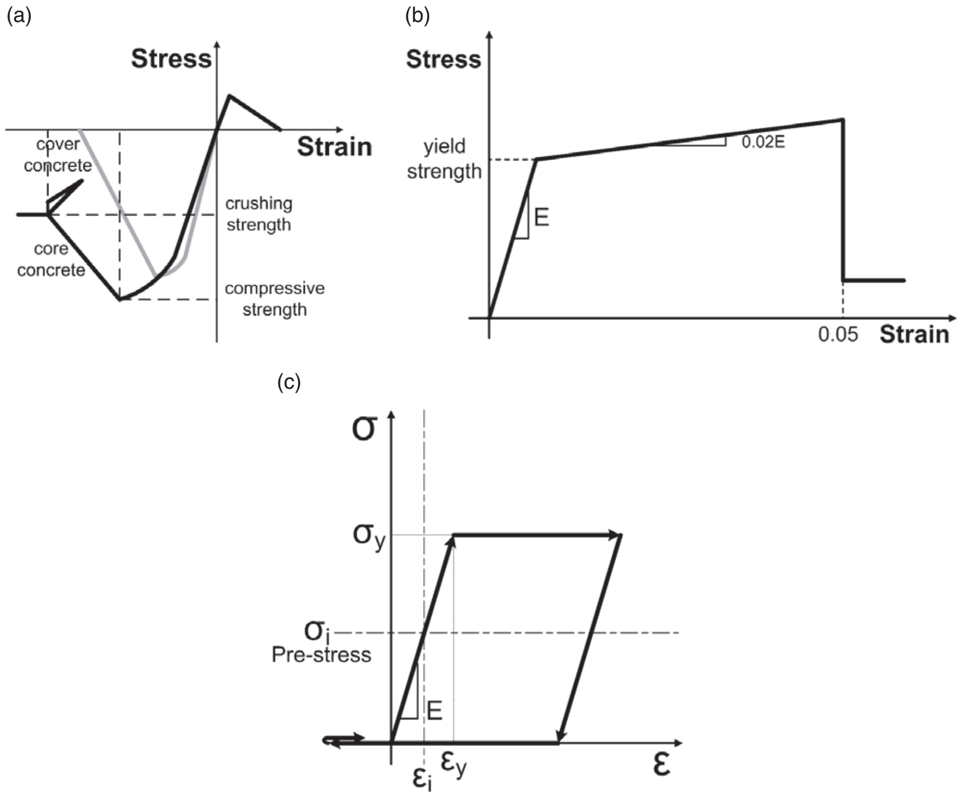

Figure 3. Stress–strain relationship of structural materials. (a) Concrete. (b) Steel re-bars. (c) Tendons.

bi-linear curves with the post-yield stiffness 2% of the initial stiffness. The Grade 270 strands (f_{pu} = 1862 MPa) with diameters of 12.7 mm (A_s = 112 mm^2), 15.2 mm (A_s = 165 mm^2) and 17.8 mm (A_s = 223 mm^2) were used for retrofit. Nonlinear static and dynamic analyses were carried out using the program code OpeenSees (Mazzoni *et al*., 2006).

The progressive collapse potential of the model structures were evaluated by removing one of the first story columns. In nonlinear static analysis, the load combination, 2(Dead Load + 0.25 Live Load), was applied in the spans from which a column was removed as recommended by the GSA guidelines (General Services Administration (GSA), 2003) as depicted in Figure 4(a). For nonlinear dynamic analysis, the load combination of (Dead Load + 0.25 Live Load) was imposed in all spans (Figure 4(b)). To carry out dynamic analysis, the axial force acting on a column was computed first before the column was removed. Then the column was replaced by point loads equivalent of its member forces as shown in Figure 4(b). To simulate the effect of a column abruptly removed, the member forces were suddenly removed after a few seconds were elapsed as shown in Figure 4(c), where the imposed gravity load, *W,* is remained constant throughout the analysis.

3. RETROFIT OF MODEL STRUCTURES USING HIGH-STRENGTH TENDONS

Figure 5 shows a beam-column sub-assemblage with a reinforcing tendon applied along the beams. When the column located between the two beams is lost, the upward recovery force P_{sv}, generated by the tendon, is given in Equation (1):

$$P_{SV} = 2\left(\frac{EA\Delta \sin^2\theta}{l} + P_e \sin\theta\right) \tag{1}$$

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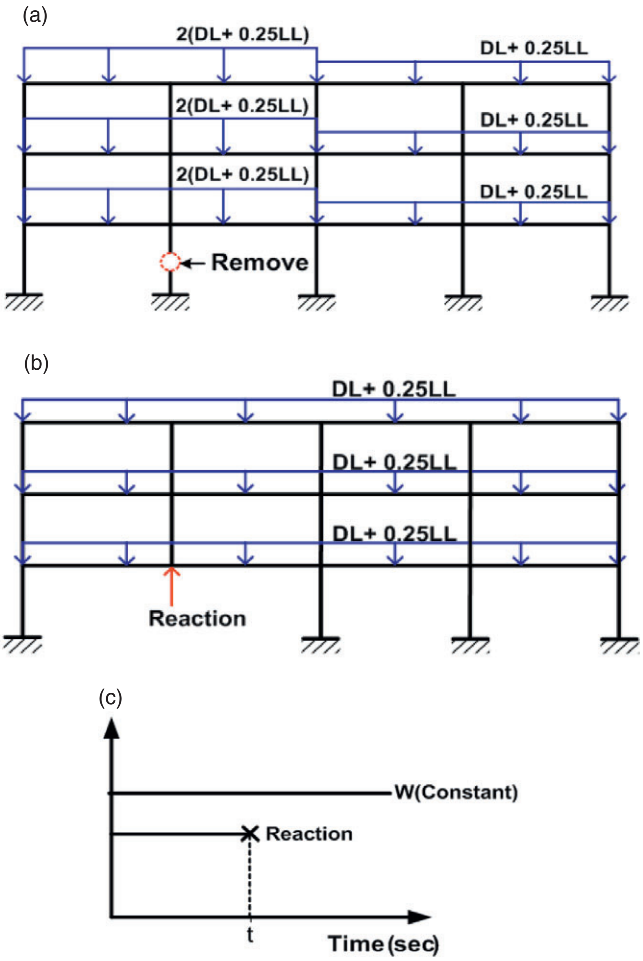

Figure 4. Applied load for analysis of progressive collapse. (a) Static analysis. (b) Dynamic analysis. (c) Time history of the gravity load and column force for dynamic analysis.

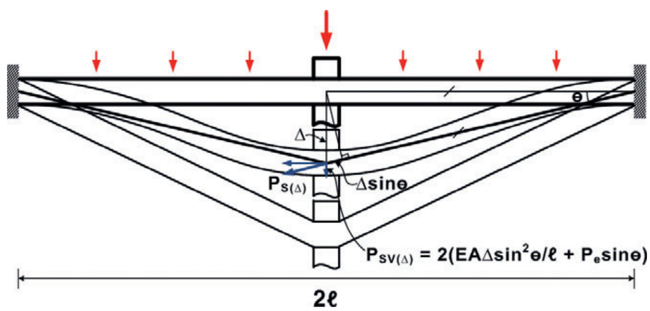

Figure 5. Recovery force of tendons when a column is lost.

where P_e is an effective initial tension, E is the elastic modulus, A is the cross-sectional area of tendons. It can be noticed that the recovery force depends on the initial tension, tendon size, span length and the slope of tendons.

For reinforcement of the model structure, tendons were placed parallel to or in X-shape along both sides of the beams as shown in Figure 6. Nonlinear static pushdown analyses were carried out with the Frame A of the six-story model structure reinforced with tendons along the beams in all stories.

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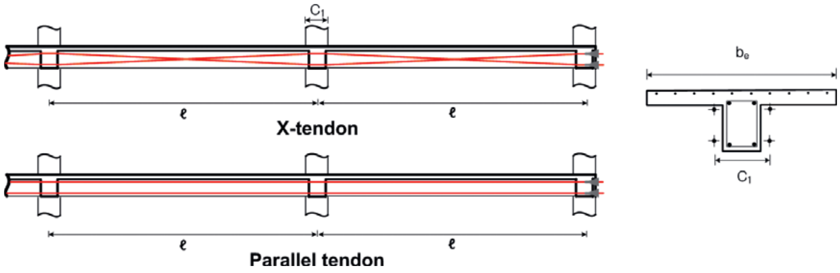

Figure 6. Placement of tendons along the beams.

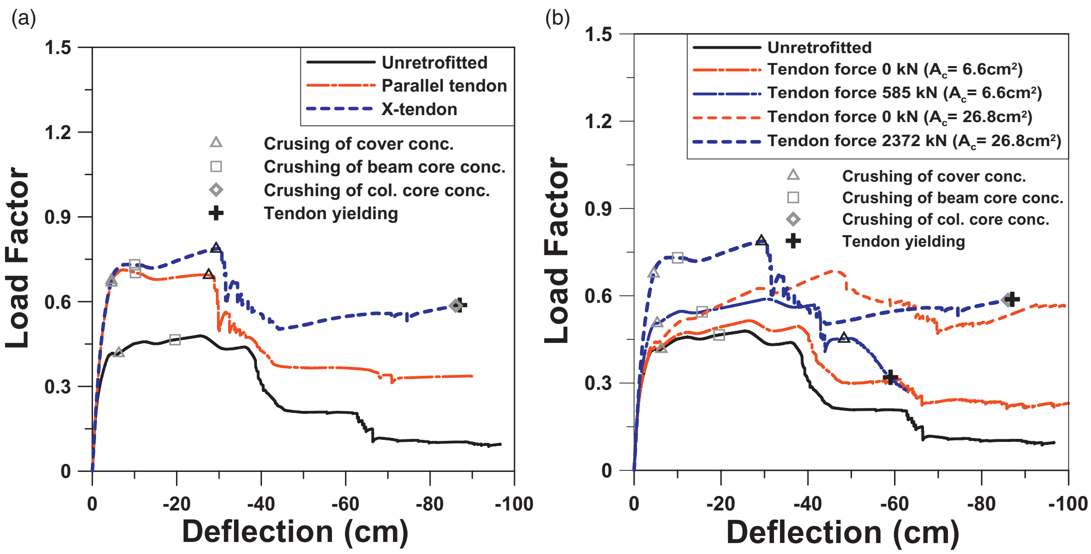

Figure 7. Pushdown curve of Frame A with an interior column removed. (a) Pushdown curves depending on tendon shape. (b) Pushdown curve depending on initial tension (X-tendon).

Figure 7(a) shows the analysis results of the structure without and with tendons subjected to loss of a first-story interior column. The initial prestressing tension imposed on the tendons was 2372 kN. According to the analysis results, the maximum strength increased significantly after the tendons were installed. The strength of the model structure increased until the cover concrete of beams reached its ultimate strain, 0.003. The triangles marked on the left-hand side of the pushdown curves indicate crushing of cover concrete at beam ends, while the triangles at the right-hand sides of the pushdown curves denote crushing of cover concrete at the center of the beam. The figure shows that as the vertical displacement increased the strain of cover concrete at beam ends reached the limit state first. As the vertical displacement further increased the core concrete at beam ends started to reach 0.003. When the beams were reinforced by high-strength tendons, the cover concrete at the center of beams also started to reach the limit state. After that moment, the flexural strengths of the beams were almost lost, and the strength suddenly dropped. Then catenary action was initiated at the beams, and the load factor kept increasing again until yielding of tendons. The structure not reinforced by tendons failed before the strain at the beam center reached 0.003. At large displacement, the strength of the structure with tendons kept increasing due to catenary action of tendons. For the same cross-sectional area of tendons, the X-shape installation scheme turned out to be more effective than the parallel scheme. The catenary action of tendons is more predominant in structures with X-tendons. Figure 7(b) shows the pushdown curves of the structure with two types of tendon cross-sectional areas and two initial tension forces for each tendon size. Tendons were installed along the beams in X shape. It can be observed that the maximum strength increases as the initial tension of tendons increases.

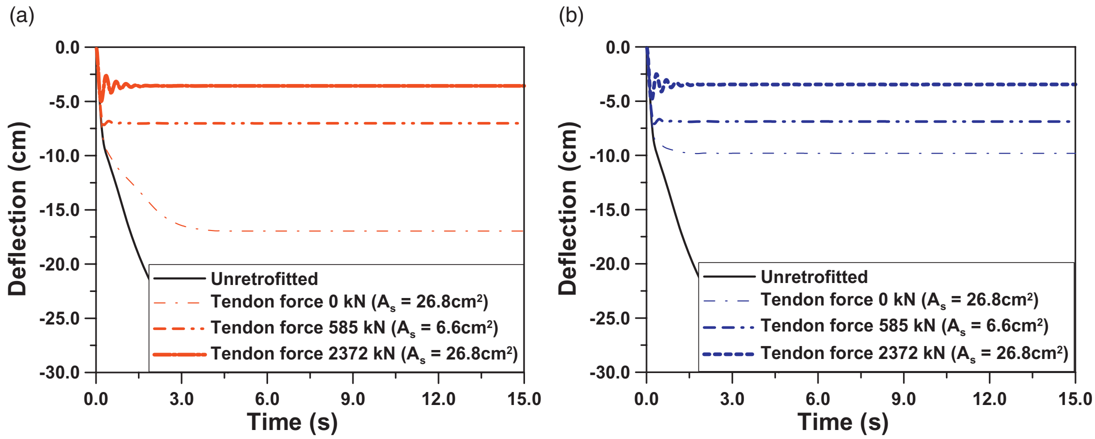

Figure 8. Time-histories of vertical deflection of Frame A. (a) Parallel tendon. (b) X-tendon.

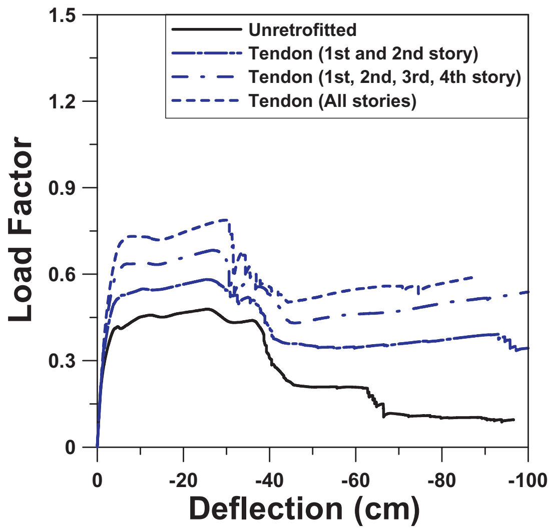

Figure 9. Pushdown curves of Frame A with and without X-tendons.

Figure 8 depicts the nonlinear time history analysis results of the Frame A subjected to sudden loss of a first story interior column. It can be observed that the structure not reinforced by the cables failed right after the column was removed, whereas the structure with reinforcing tendons remained stable. The vertical displacement decreased as the initial tension and cross-sectional area of the tendons increased. It was observed that when the tendons with the initial tension of 2372 kN were installed along the beams the structure remained elastic after the column loss, whereas large inelastic deformation occurred when initial tension was not applied. The comparison of Figure 8(a, b) shows that the vertical displacement of the structure with X-type tendons turned out to be smaller than that of the structure with parallel tendons.

Figure 9 shows the vertical load-displacement relationship of the Frame A structure obtained from nonlinear static pushdown analysis, where it can be observed that as the number of the stories with added tendons increased the maximum load factor increased.

Figures 10 and 11 show the pushdown curves of the Frames B and D composed of angle-shaped beams without and with X-type tendons. The initial tension of 1170 kN was applied to the tendons.

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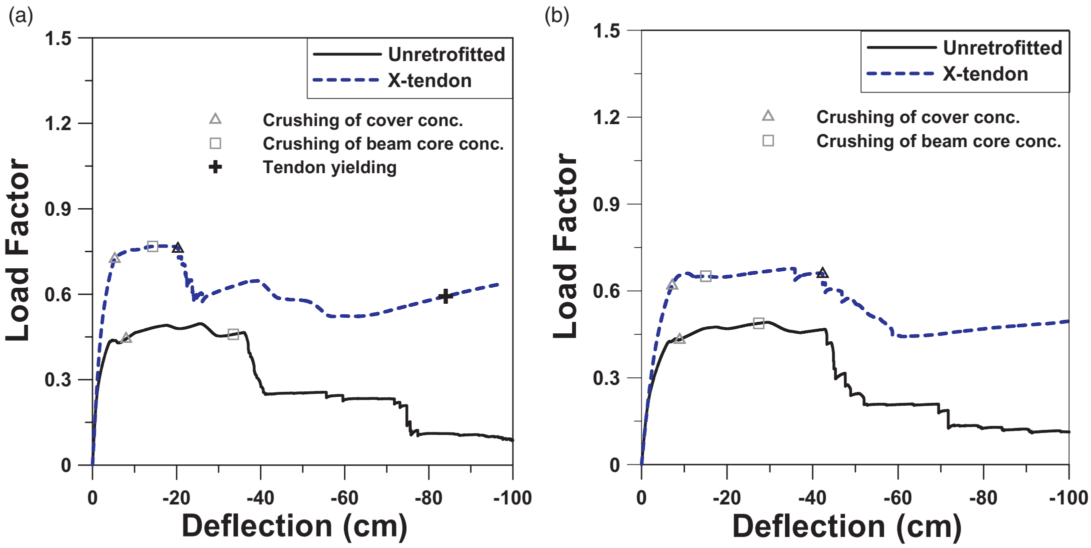

Figure 10. Pushdown curves of Frame B with and without tendons. (a) Removal of an interior column. (b) Removal of a corner column.

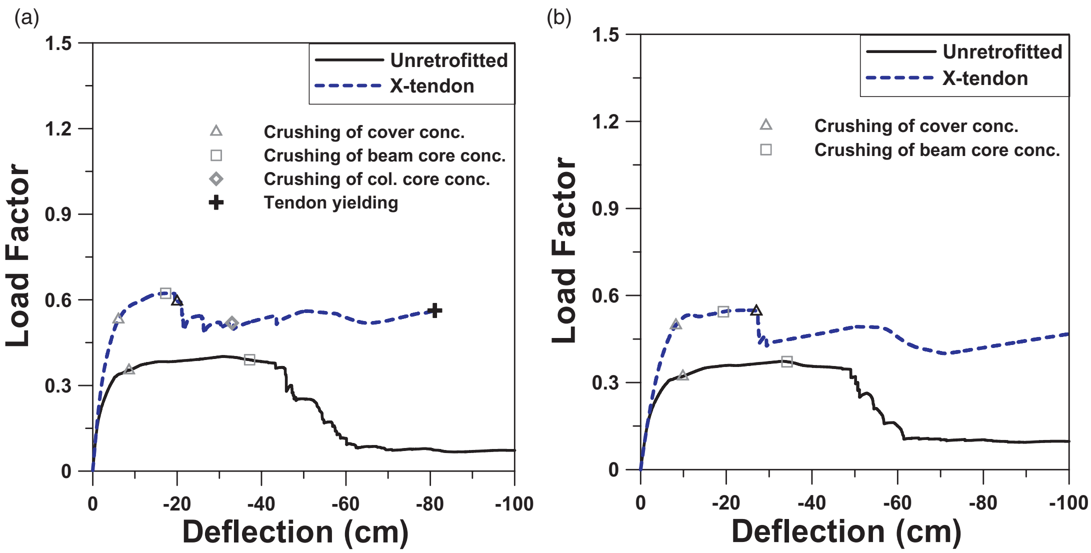

Figure 11. Pushdown curves of Frame D with and without tendons. (a) Removal of an interior column. (b) Removal of a corner column.

It can be noticed that the structures with tendons showed higher load factor than that for the structure without tendons. The increase in load factor was higher when the interior column was removed than when the corner column was removed. This implies that the effect of tendon reinforcement may be higher when an interior column was removed. The load factors obtained for Frame B, which is composed of four bays, were slightly higher than those obtained for Frame D, which has three bays. When an interior column was removed, the tendons started to yield when the vertical deflection exceeded 80 cm. The tendons remained elastic when a corner column was removed.

Figures 12 and 13 show the time histories of the vertical deflections of Frames B and D without and with X-type tendons caused by sudden loss of a column. It can be observed that the structures reinforced by the tendons remained stable after the column was removed, whereas the unretrifitted

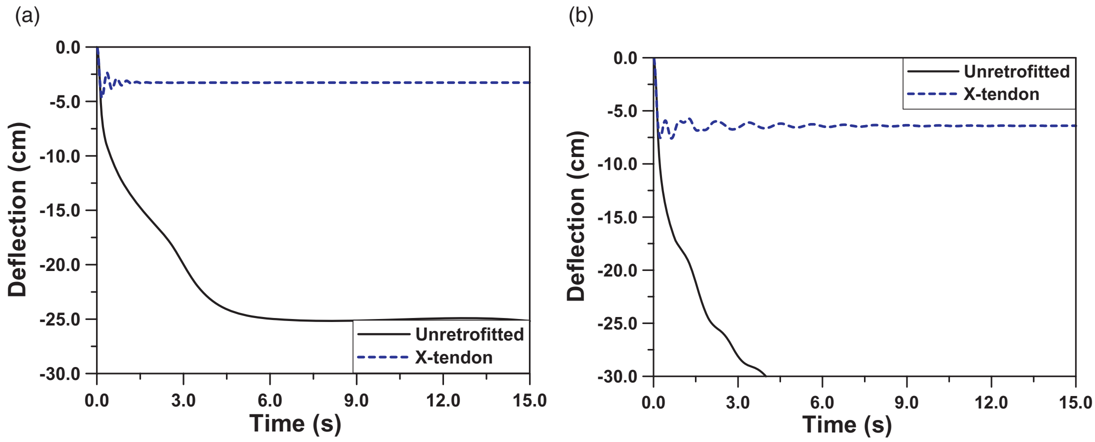

Figure 12. Vertical deflection time-histories of Frame B. (a) Removal of an interior column. (b) Removal of a corner column.

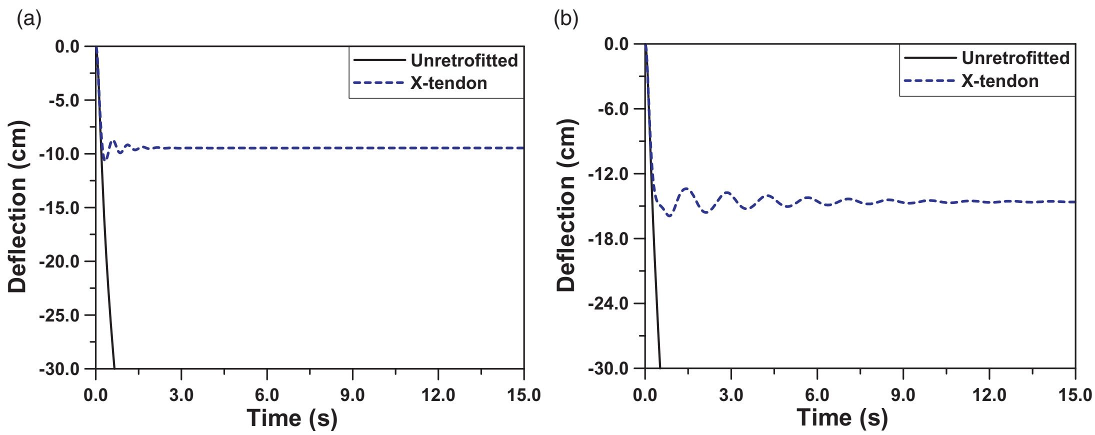

Figure 13. Time-histories vertical deflection of Frame D. (a) Removal of an interior column. (b) Removal of a corner column.

structures failed right after the removal of the column. This can be expected from the pushdown curves shown in Figures 10 and 11, in which the maximum load factors of the structures with reinforcing tendons reached at least 0.6, whereas those of the unretrifitted structures were less than 0.4. This implies that the GSA recommended dynamic response factor of 2.0 to ensure safety against progressive collapse may be too conservative for RC moment frames considered in this study. The dynamic response factor of 2.0 corresponds to the maximum load factor of 1.0 in this study because the imposed load was already doubled for static analysis considering dynamic effect.

Figure 14 shows the pushdown curves and the time-history analysis results of the 20-story model structure without and with retrofit when one of the two middle columns is removed. For retrofit against progressive collapse, high-strength tendons with initial tension of 2.375 MN were applied either in every story or in every four stories. It can be observed in the pushdown curves that the maximum strength of the unretrofitted structure is less than 0.5, whereas those of the structures retrofitted with tendons in every four stories and in every story increased to 0.62 and 0.87, respectively. The nonlinear dynamic analysis results show that the vertical displacement of the unretrofitted structure is unbounded, whereas those of the retrofitted structures remain stable after the column is suddenly removed.

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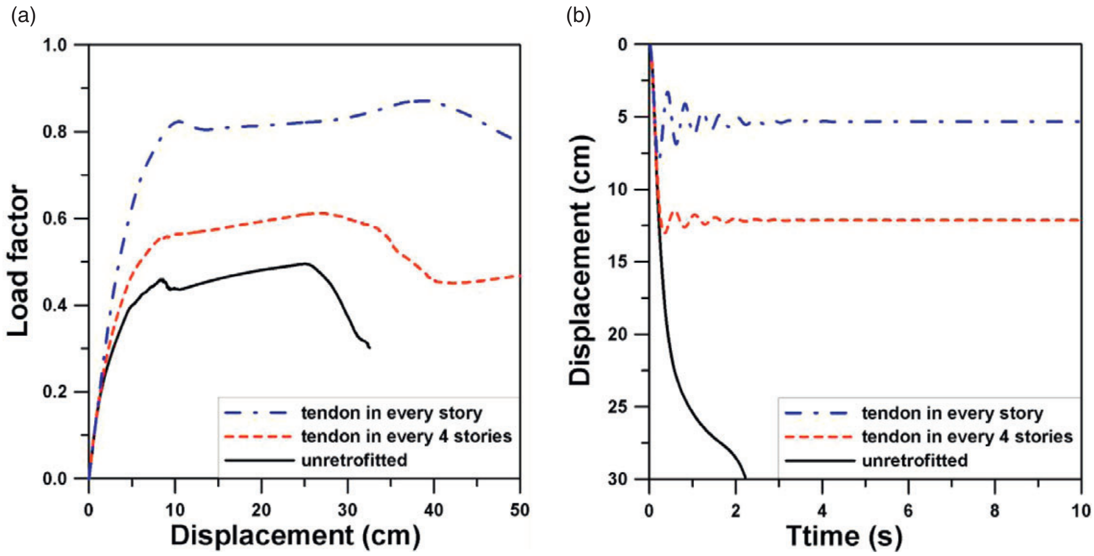

Figure 14. Analysis results of the 20-story structure. (a) Pushdown curves. (b) Time histories of vertical displacements.

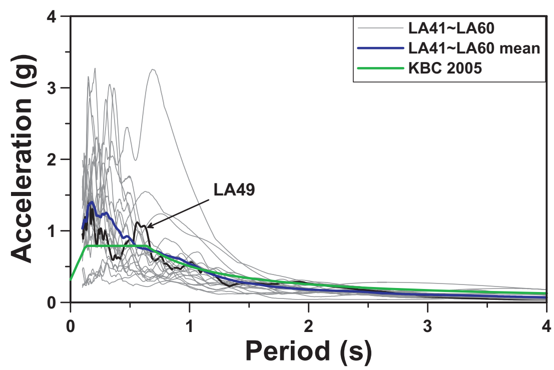

Figure 15. Response spectra of LA41–LA60 earthquakes.

4. SEISMIC PERFORMANCE OF RETROFITTED STRUCTURES

Currently, the design or retrofit of structures against progressive collapse is generally carried out only for military facilities and government offices. For most buildings, the abnormal loads such as blast or collision are not included in current design codes. Therefore, for most low- to medium-rise buildings, the primary concern for structural design is to guarantee seismic safety rather than to prevent progressive collapse.

In the previous section, the effect of tendon-reinforcing scheme to enhance progressive collapse resisting capacity was verified. In this section, the seismic performance of the six-story structure retrofitted by the tendons with cross sectional area of 26.8 cm^2 and initial tension of 2372 kN was evaluated by nonlinear time-history analyses. The input seismic loads are the 20 earthquake ground motions, LA41 to 60, developed for the SAC Phase Ⅱ Program (Somerville *et al*., 1997). The earthquake records were scaled to fit the design spectrum of the Korea Building Code as shown in Figure 15. According to the eigenvalue analysis results, the natural periods of the model structure did not change after the retrofit by tendons.

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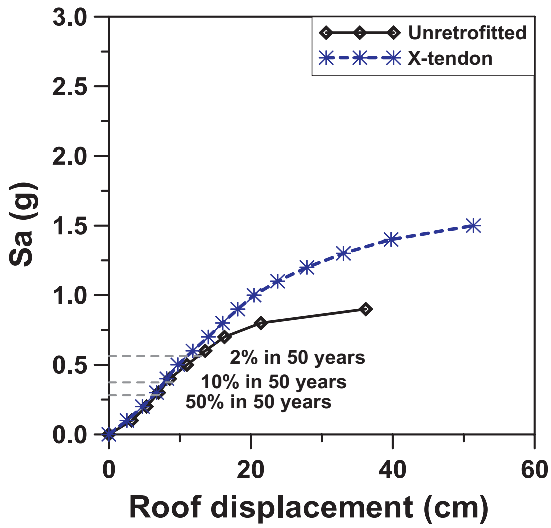

Figure 16. Spectral acceleration–roof displacement relationship of Frame A of model structures obtained by incremental dynamic analyses using LA49 earthquake.

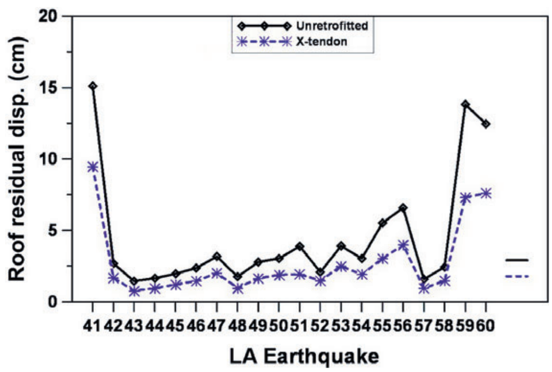

Figure 17. Roof residual displacement of Frame A obtained by nonlinear dynamic analysis.

To evaluate the load–displacement relationships of the model structures without and with tendon retrofit, the incremental dynamic analysis (IDA) procedure proposed by Vamvatsikos and Cornell (2002) was applied. A series of nonlinear dynamic analyses were carried out by gradually increasing the amplitude of LA49 earthquake in such a way that the response spectral acceleration value corresponding to the fundamental period increased by 0.1 g. Figure 16 shows the IDA curves representing spectral acceleration versus roof displacement relationship of the model structures. The spectral values corresponding to the earthquakes with probability of occurrence of 2%, 10% and 50% in 50 years are also indicated in the figures. It can be observed that the strength of the structure retrofitted by X-type tendons increased by about 60% after the structure was retrofitted by the tendons.

Figure 17 plots the permanent displacement at the roof story before and after the retrofit obtained by nonlinear dynamic time-history analyses using the 20 earthquake records. It can be observed that after the retrofit, the permanent displacements due to inelastic deformation decreased for all earthquake records. This implies that by introducing prestressing into existing beams the inelastic deformation or structural damage caused by earthquakes can be effectively reduced.

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5. CONCLUSIONS

In this study, the effect of prestressing tendons on the progressive collapse performance of 6- and 20-story reinforced concrete model structure was evaluated by nonlinear static and dynamic analysis. The high-strength tendons were installed along the beams, and initial tensions were applied to the tendons. According to nonlinear static and dynamic analysis results, the analysis model structure before retrofit turned out to be vulnerable to progressive collapse caused by sudden loss of a first-story column. The progressive collapse resisting capacity of the four-bay frame was slightly higher than that of the three-bay frame. However, the RC structures reinforced by external prestressing tendons along floor girders showed stable behavior against progressive collapse. The retrofit effect increased as the initial tension and cross-sectional area of tendons increased. The incremental dynamic analyses using 20 earthquake records showed that the seismic performance of the model structure was also enhanced after the retrofit using tendons. Therefore, based on analysis results, it was concluded that the retrofit of existing buildings using prestressing tendons could be effective for enhancing progressive collapse resisting capacity as well as seismic performance.

6. ACKNOWLEDGMENTS

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