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Progressive collapse resisting capacity of moment frames with viscous dampers

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SUMMARY

In this paper, the effect of viscous dampers on reducing progressive collapse potential of steel moment frames was evaluated by nonlinear dynamic analysis. Parametric studies were conducted first to evaluate the effects of dampers installed in a steel beam-column subassembly with varying natural period and yield strength on the reduction of progressive collapse potential. Then 15-story moment-resisting frames with three different span lengths were designed with and without viscous dampers, and the effect of viscous dampers was investigated by nonlinear dynamic analysis. According to the parametric study, the vertical displacement generally decreased as the damping ratio of the system increased, and the dampers were effective in both the elastic and the elasto-plastic systems. It was also observed that the effect of the damper increased as the natural period of the structure increased and the strength ratio decreased. The analysis results of 15-story analysis model structures showed that the viscous dampers, originally designed to reduce earthquake-induced vibration, were effective in reducing vertical displacement of the structures caused by sudden removal of a first-story column, and the effect was more predominant in the structure with longer span length. Copyright © 2011 John Wiley & Sons, Ltd.

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KEY WORDS: viscous dampers; progressive collapse; moment frames; nonlinear dynamic analysis

1. INTRODUCTION

Progressive collapse occurs when local failure of a primary structural component leads to the failure of adjoining members and finally to the failure of the partial or whole structure system. It is a dynamic process, usually accompanied by large deformations, in which the collapsing system continually seeks alternative load paths in order to survive. To prevent or reduce the risk of progressive building collapse, many building codes integrated an indirect design approach into the specifications through mandatory strength, ductility and continuity requirements (ACI, 2002). Recently, both the U.S. General Services Administration (GSA, 2003) and the U.S. Department of Defense (DoD, 2005) have issued guidelines for evaluating the progressive collapse hazard, which provides general information about the approach and method of evaluating the progressive collapse potential. In-depth research has been carried out to investigate appropriate analysis methods (Marjanishvili, 2004; Milner *et al*., 2007), energy-based analysis and design procedures (Dusenberry and Hamburge, 2006; Kim and Park, 2008), probability and fragility analysis (Park and Kim, 2010) and performance of various structure systems (Tsai and Lin, 2008; Kim and Lee, 2010) based on an arbitrary column loss scenario.

The progressive collapse, however, has not been considered as one of the normal design loads in most design codes. Therefore, researchers have been concerned with the evaluation of progressive collapse resisting capacity of structures designed for seismic load (Kim and Kim, 2009). There exist

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several alternative methods for the seismic upgrade of a building (ATC, 1996; FEMA, 1997), and among the various methods, supplemental dampers are often chosen because they do not attract higher ground acceleration that requires strengthening of existing columns and foundations.

This study evaluates the progressive collapse resisting capacity of structures with viscous dampers, which are generally installed to reduce wind or earthquake-induced vibration of structures. Parametric study was conducted first with a beam-column subassembly with a viscous damper, and the response of the subassembly was investigated for varying design parameters such as damping ratios, strength ratios and natural periods. Then three-bay 15-story structures were designed with viscous dampers and were analyzed for progressive collapse. As design against progressive collapse is not explicitly specified in most of the current design codes, the dampers were designed to provide 5%, 10% and 20% target damping ratio for lateral vibration caused by seismic load. The progressive collapse resisting capacity of structures with viscous dampers was evaluated using nonlinear time-history analysis based on column removal scenario, and the results were compared with those of the structure without viscous dampers.

2. DYNAMIC ANALYSIS FOR PROGRESSIVE COLLAPSE

In this study, the performance of moment frames installed with viscous dampers subjected to sudden removal of a column was investigated by nonlinear dynamic analysis using the program code SAP 2000 (2004). For nonlinear dynamic analysis, the load combination DL + 0.25LL specified in the GSA (2003) for dynamic analysis was uniformly applied as vertical load in the entire span as shown in Figure 1. In order to carry out dynamic analysis, the member forces of a column, which is to be removed to initiate progressive collapse, are computed before it is removed. Then the column is replaced by the point loads equivalent of its member forces as shown in Figure 1. To simulate the phenomenon that the column is removed by impact or blast, the column member forces are suddenly removed after elapse of a certain time, while the gravity load remained unchanged as shown in Figure 2. In this study, the member reaction forces are increased linearly for 10 s until they reach the specified level, are kept unchanged for 5 s until the system reaches stable condition and are suddenly removed at 15 s to initiate progressive collapse.

3. EFFECT OF DAMPING IN A BEAM-COLUMN SUBASSEMBLY

Parametric studies were carried out with the beam-column subassemblage shown in Figure 3 to investigate the effect of added damping. The structure is composed of two beams fixed at the supports and a column that is to be suddenly lost. A viscous damper is installed vertically at the center of the structure above the column. The beams are H 500 × 200 × 10/16 steel made of SS400 steel (F_y = 235 MPa). Dead load of 5.0 kN/m^2 and live load of 2.5 kN/m^2 are imposed on the beams with the load combination of DL + 0.25LL, and the member forces of the column were computed and were

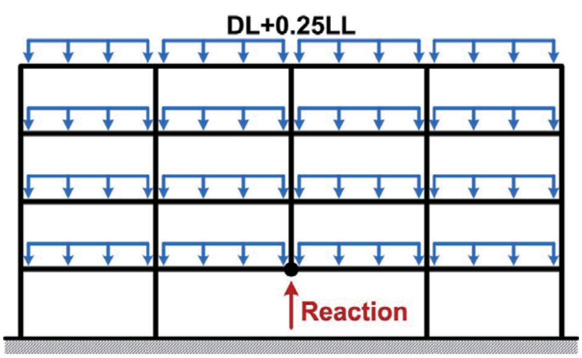

Figure 1. Applied load for dynamic analysis.

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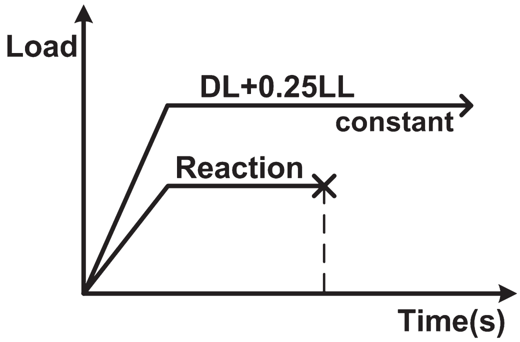

Figure 2. Time history of applied load for dynamic analysis.

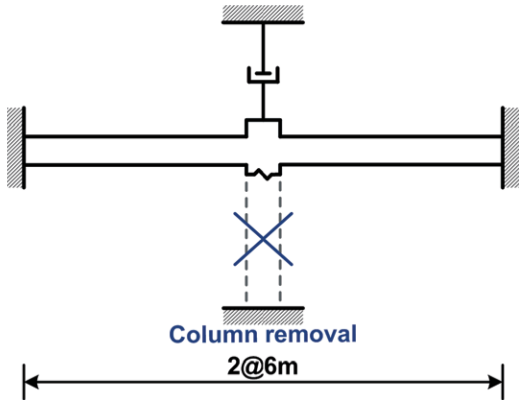

Figure 3. Beam-column subassembly for parametric study.

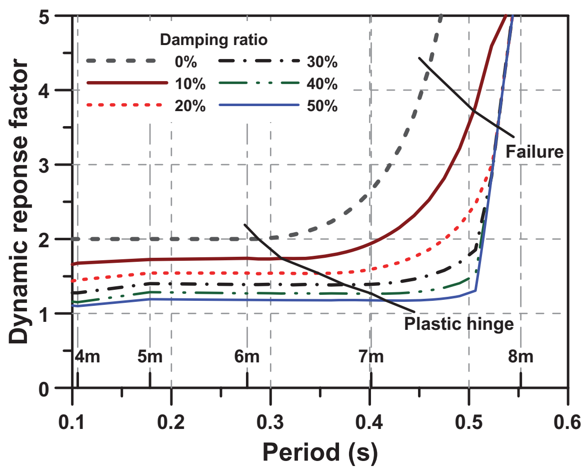

Figure 4. Dynamic response factors of the subassembly with various natural periods subjected to sudden loss of the column.

replaced by the applied forces. Bilinear moment-rotation relationship with post-yield stiffness of 2% was assumed for beams. Vertical vibration was initiated by suddenly removing the column member forces, while the gravity load remains unchanged.

Figure 4 shows the dynamic analysis results of the subassembly with various damping ratios of the added viscous damper subjected to sudden loss of the column. The ordinate and the abscissa represent

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the dynamic response factor and the natural period, respectively. The length of the beam corresponding to the natural period is also shown along the abscissa. The dynamic response factors are the maximum vertical displacements obtained by nonlinear dynamic analyses divided by those obtained by linear static analyses. The natural period of the structure is varied by changing the beam length from 4 m to 8 m. It was observed that the structure experienced inelastic deformation as a result of sudden removal of the column when the length of the beams is longer than 6.2 m. The figure shows that when there is no damping, the dynamic response factor is 2.0 when the beams are in elastic stage and increases after formation of plastic hinges. As the damping ratio increases, the dynamic response factor keeps decreasing toward 1.0, which implies that no dynamic effect occurs due to installation of the damper.

Figure 5 shows the vertical displacement time histories of the subassembly with the span length of 6 m and 6.5 m. The damping ratio of 2% is provided when there is no viscous damper attached to the system and only inherent damping exists. It can be observed that as the damping ratio increases the amplitude of vibration decreases more quickly. In the subassembly with 6-m span length, the final displacements are almost the same regardless of the damping ratio, which implies that the system is elastic. However in the structure with 6.5-m span length, which experiences inelastic deformation due to sudden loss of the column, the final as well as the maximum displacements are reduced due to the installation of the damper. It also can be seen that the residual displacement due to yielding also decreases as the damping ratio increases.

Figure 6 shows the displacement time-histories of the subassembly with 6-m span length and with 30% damping ratio of the damper. The yield strength of the 6-m beams was reduced to 80% and 60%

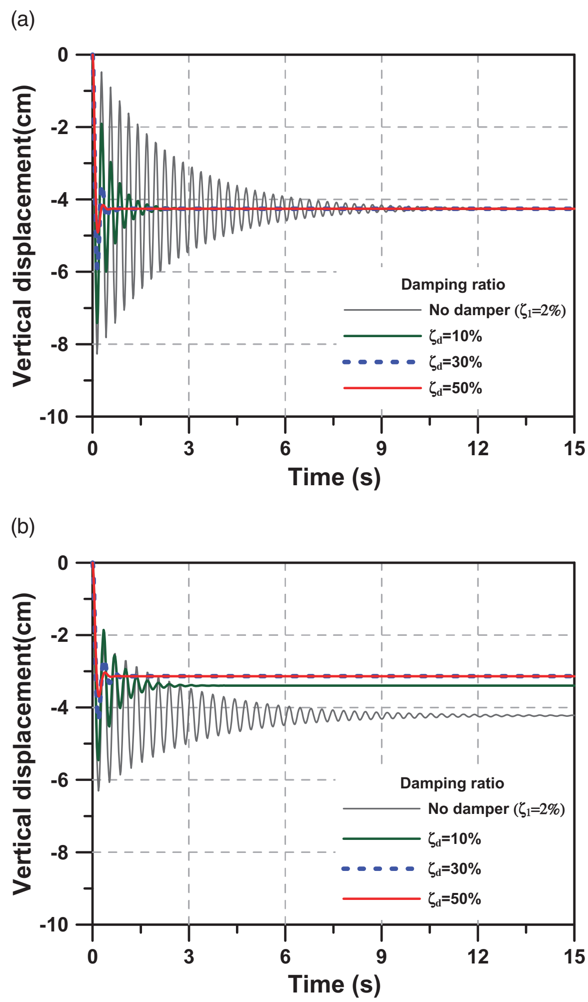

Figure 5. Vertical displacement time history of the subassembly. (a) Span, 6 m. (b) Span, 6.5 m.

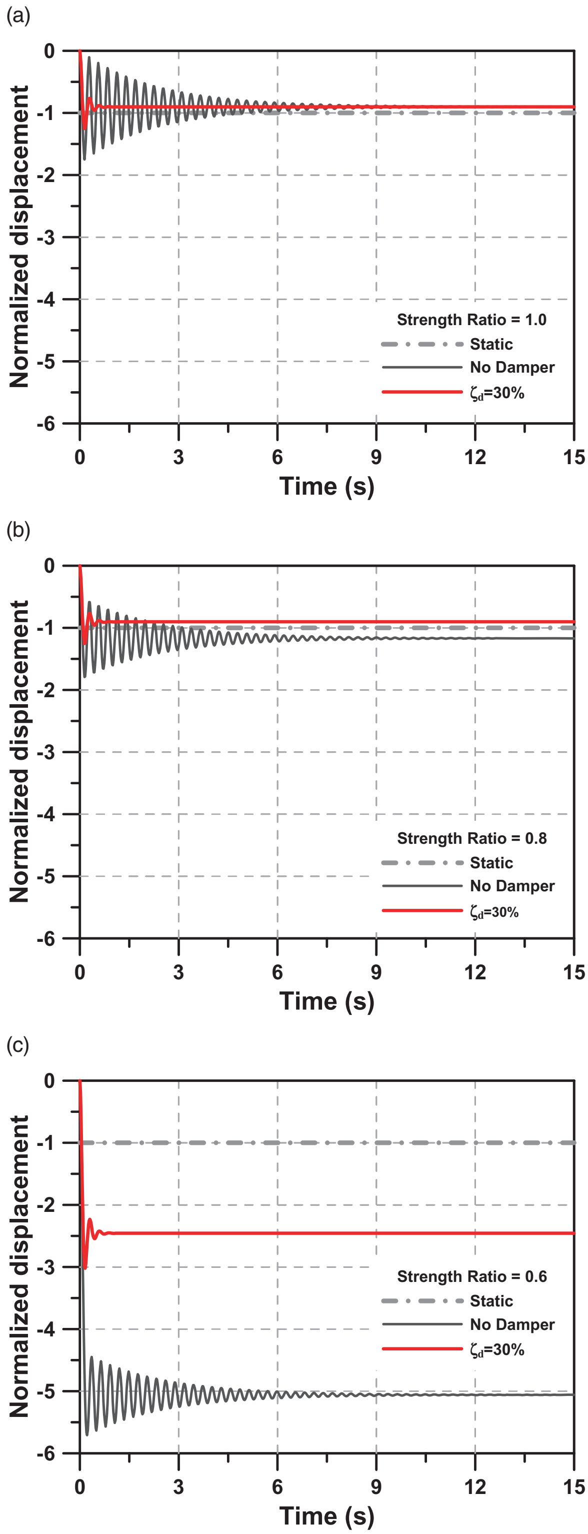

Figure 6. Normalized displacement of the subassembly with various strength ratios. (a) Strength ratio, 1.0. (b) Strength ratio, 0.8. (c) Strength ratio, 0.6.

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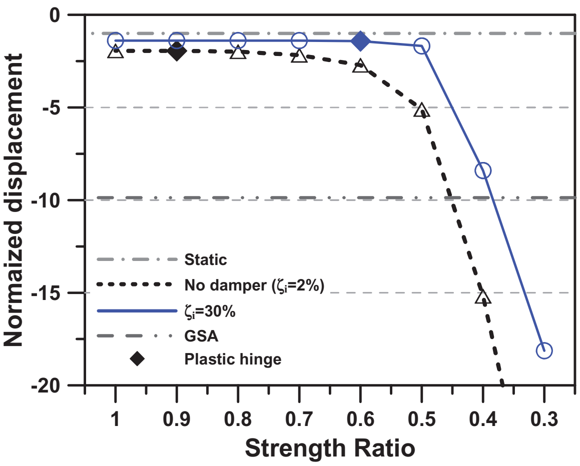

Figure 7. Normalized displacement of the subassembly with various strength ratios.

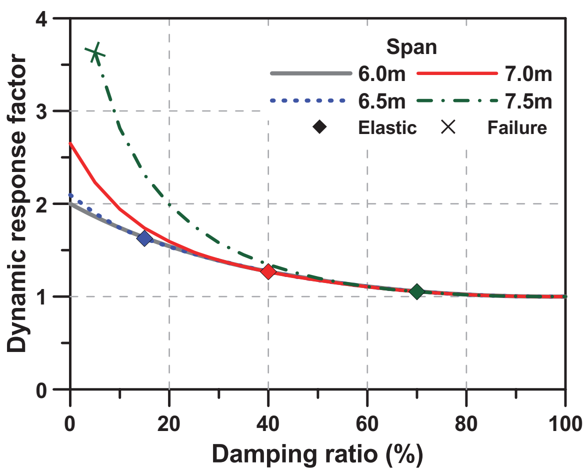

Figure 8. Variation of dynamic response factors as a function of damping ratio.

to observe the effect of yielding and damping on the vertical displacement. The structure without the added damper has 2% inherent damping. The vertical displacement was normalized by the displacement obtained by linear static analysis. It can be observed that when the strength ratio is 1.0, the displacements obtained by the static and dynamic analyses are almost identical. However, as the strength ratio decreases, the displacement obtained by the dynamic analysis increases significantly and so does the effect of added damping. The normalized maximum dynamic displacements of the 6-m span subassembly structure with and without dampers are plotted for various strength ratios in Figure 7. When there is no damper, plastic hinge formed in the structure with strength ratio of 0.9, whereas it formed first at the structure with strength ratio of 0.6 when the damper is installed. As the strength ratio decreases to a value less than 0.5, the maximum member rotation exceeds the limit state specified in the GSA guidelines, and the substructure without the viscous damper are considered as failed when the column is removed. However, the vertical displacement of the structure with the viscous damper remained within allowable range. Therefore, it can be expected that the damper can be more effective in preventing progressive collapse of structures that undergo significant inelastic deformation.

Figure 8 plots the dynamic response factor, which is the absolute values of the normalized displacement, of the subsystem with various span lengths. The horizontal axis represents the damping ratio

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of the viscous damper. It can be observed that when there is no damper, the dynamic response factor is higher than 2.0, which is specified in the GSA guidelines for considering dynamic effect in static analysis. The dynamic response factor increases as the span length increases. This implies that the dynamic response factor is larger in the yielding structure than in the elastic structure as observed in Figure 7. The structure with 7.5-m span length without the dampers even failed when the column was suddenly removed. However, as the damping ratio increases, the dynamic response factor gradually approaches to 1.0. It can be noticed that the structures become elastic when the damping ratio of the system increases higher than a certain value. The structure with 6.5-m span length behaves elastically after sudden loss of the column when the damping ratio reaches about 15% of the critical damping, whereas the structure with 7.5-m span length becomes elastic when the damping ratio increases above 70% of the critical damping.

4. EFFECT OF VISCOUS DAMPERS IN A MULTISTORY BUILDINGS

4.1. Structural modeling and analysis procedure

As analysis model structures, three-bay 15-story framed structures were designed. Figure 9 shows the structural plan of the analysis model. Only the perimeter frames were designed as moment frames to resist lateral loads, and the interior gravity load-resisting frames were simply connected. The perimeter frame enclosed in the dotted rectangle was separated for analysis. To compare the effect of dampers on structures with different span lengths, the structures with three different span lengths, 6 m, 9 m and 12 m, having uniform story height of 4 m were prepared. The SM490 steel with yield stress of 325 MPa was used for columns, and the SS400 steel with yield stress of 235 MPa was used for beams. Dead and live loads of 5.0 kN/m^2 and 2.5 kN/m^2, respectively, were used as gravity load, and the seismic load of S_{DS} and S_{D1} of 0.44 *g* and 0.23 *g*, respectively, were applied for structural design. The member sizes of the model structure were shown in Table 1. The vertical fundamental vibration period of the model structure with 6-m span length turned out to be 0.21 s, and those of the structures with 9 m and 12 m span lengths were 0.26 and 0.28 s, respectively. The dampers, which are located in the mid-bay and are uniformly distributed throughout the building height as shown in Figure 10, were designed to provide approximately 5%, 10% and 20% of critical damping for lateral seismic motion using the following equation (Kim and Choi, 2006):

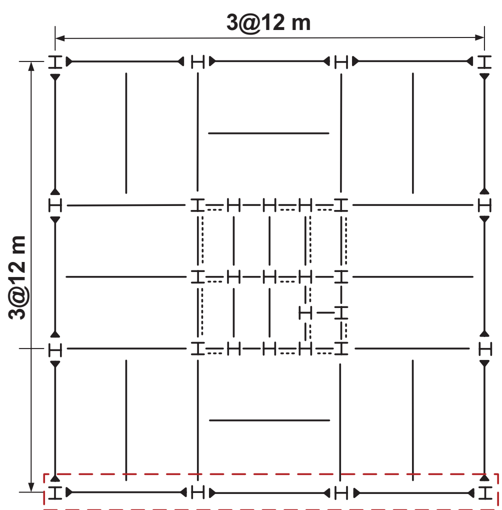

Figure 9. Structural plan of the analysis model structure with 12-m span length.

Table 1. Member size of analysis models (unit: mm).

Story	Exterior columns	Interior columns	Beams
(a) 6-m span model			
1–3	H 304 × 301 11/17	H 350 × 357 × 19/19	H 350 × 175 × 7/11
4–6	H 300 × 300 × 10/15	H 344 × 348 × 10/16	H 350 × 175 × 7/11
7–9	H 298 × 299 × 9/14	H 304 × 301 × 11/17	H 354 × 176 × 8/13
10–12	H 250 × 255 × 14/14	H 300 × 300 × 10/15	H 354 × 176 × 8/13
13–15	H 200 × 204 × 12/12	H 200 × 204 × 12/12	H 350 × 175 × 7/11
(b) 9-m span model			
1–3	H 428 × 407 × 20/35	H 458 × 417 × 30/50	H 482 × 300 × 11/15
4–6	H 406 × 403 × 16/24	H 428 × 407 × 20/35	H 482 × 300 × 11/15
7–9	H 400 × 408 × 21/21	H 406 × 403 × 16/24	H 482 × 300 × 11/15
10–12	H 394 × 405 × 18/18	H 394 × 398 × 11/18	H 482 × 300 × 11/15
13–15	H 394 × 398 × 11/18	H 300 × 305 × 15/15	H 482 × 300 × 11/15
(c) 12-m span model.			
1–3	H 458 × 417 × 30/50	H 458 × 417 × 30/50	H 588 × 300 × 12/20
4–6	H 428 × 407 × 20/35	H 458 × 417 × 30/50	H 588 × 300 × 12/20
7–9	H 414 × 405 × 18/28	H 428 × 407 × 20/35	H 588 × 300 × 12/20
10–12	H 406 × 403 × 16/24	H 406 × 403 × 16/24	H 588 × 300 × 12/20
13–15	H 406 × 403 × 16/24	H 310 × 305 × 15/20	H 588 × 300 × 12/20

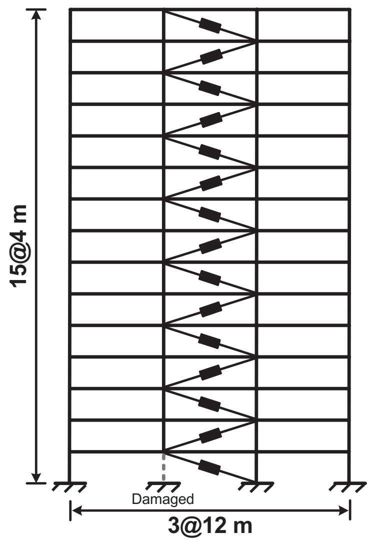

Figure 10. Fifteen-story structure with viscous dampers subjected to sudden loss of an interior column.

$$\zeta_d = \frac{T\sum_{i=1}^{N} C_i \cos^2\theta_i (\Delta_i - \Delta_{i-1})^2}{4\pi \sum_{i=1}^{N} m_i \Delta_i^2} \quad (1)$$

where ζ_d is the damping ratio contributed from the viscous dampers, T is the fundamental natural period of the structure, C_i is the damping coefficient of the damper located in the ith story, θ is the slope of the damper as shown in Figure 11, m_i is the modal mass of the ith story and Δ_i is the maximum displacement of the ith story. In this study, the modal displacements of the first mode were used for the maximum displacement. A single damper with the same damping capacity is located in each story. Table 2 shows the damping coefficients of the dampers computed to provide the three different target damping ratios, and Table 3 shows the corresponding vertical damping ratios obtained by free vibration analysis with a first-story column suddenly removed.

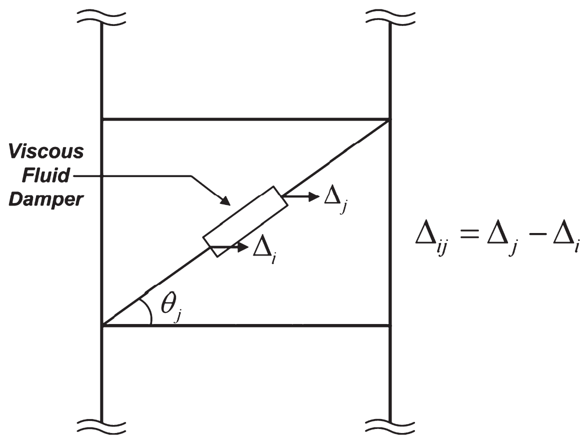

Figure 11. Relative displacement of a viscous damper.

Table 2. Damping coefficients C_d of the dampers corresponding to the target damping ratios (kN s/cm).

	Span lengths		
Damping ratios (%)	6 m	9 m	12 m
5	6.7	15.4	25.4
10	13.4	31	50.5
20	27	61.2	100.8

Table 3. Vertical damping ratios corresponding to target lateral damping ratios.

Span length	Target lateral damping ratio	Vertical damping ratio
6	5	6
	10	10
	20	15
9	5	7
	10	11
	20	16
12	5	7
	10	10
	20	16

The material model of the structural members recommended by the FEMA-356 (2000) for nonlinear analysis was used to simulate nonlinear behavior of the model structures. Figure 12 shows the bending moment versus rotation angle relationship of the material model and the coefficients used to define the nonlinear behavior (*a, b* and *c*) are computed as a = 8.419, b = 10.419 and c = 0.554 for the structure with 6-m span length considering the width–thickness ratios of the structural members. The inherent damping ratio of the structure was assumed to be 2%.

Nonlinear static pushdown analyses of the model structure were carried out first with one of the first-story interior columns removed. The static procedure accounts for nonlinear effects without sophisticated hysteretic material modeling and is useful in determining elastic and failure limits of the structure. The GSA (2003) guidelines proposed the amplification factor of 2 for the static analysis to account for dynamic redistribution of forces. The load combination of the GSA (2003) for static analysis is 2(dead load + 0.25 × live load). This amplified load was applied only in the spans from which a column was removed, while unamplified load was applied in the other spans. In this study, pushdown analysis was applied by gradually increasing the vertical displacement in the location of the removed column to investigate the resistance of the structure against such deformation. Since this

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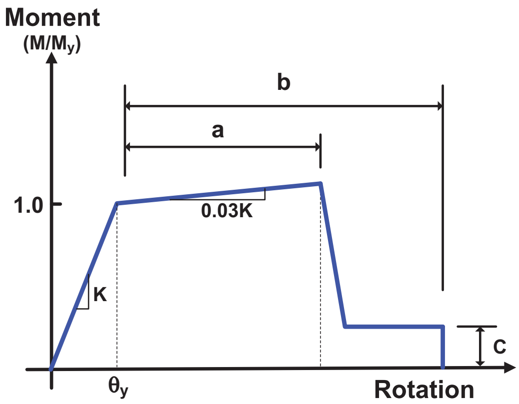

Figure 12. Moment–rotation relationship of flexural members.

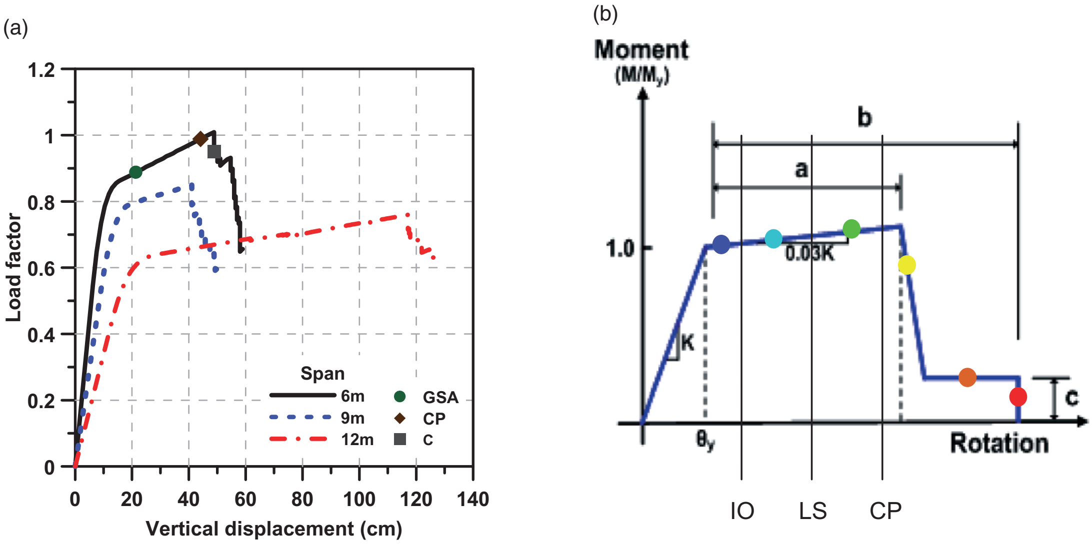

Figure 13. Pushdown analysis results of model structures. (a) Pushdown curves. (b) Member deformation levels.

procedure is displacement controlled, there is little chance to diverge. At every step during the pushdown analysis, i.e. at each level of the vertical displacement, the amount of equivalent load corresponding to the displacement level was determined. The ratio of the applied load and the GSA-specified load combination of 2(dead load + 0.25 × live load) is referred to as the 'load factor'. The original loading pattern remained unchanged at every step.

Nonlinear dynamic analyses of the model structures were carried out using the program code SAP 2000 (2004) with a first-story column suddenly removed. For nonlinear dynamic analysis, the load DL + 0.25LL was uniformly applied in the entire spans. In order to carry out dynamic analysis, the member forces of a column, which is to be removed to initiate progressive collapse, were computed before it is removed. Then the column was replaced by point loads equivalent of its member forces. In order to simulate the phenomenon that the column was abruptly removed, the member forces were suddenly removed a few seconds after their application while the applied load remained unchanged.

4.2. Analysis results

Figure 13(a) shows the pushdown curves of the model structures without dampers, in which the load factor versus vertical displacement relationships were plotted. The load factor greater than 1.0 implies

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Table 4. Limit states specified in the GSA 2003.

Component	Ductility (μ)	Rotation (rad)
Steel beams	20	0.21
Steel columns (tension controls)	20	0.21
Steel columns (compression controls)	1	–
Steel frame	–	0.035

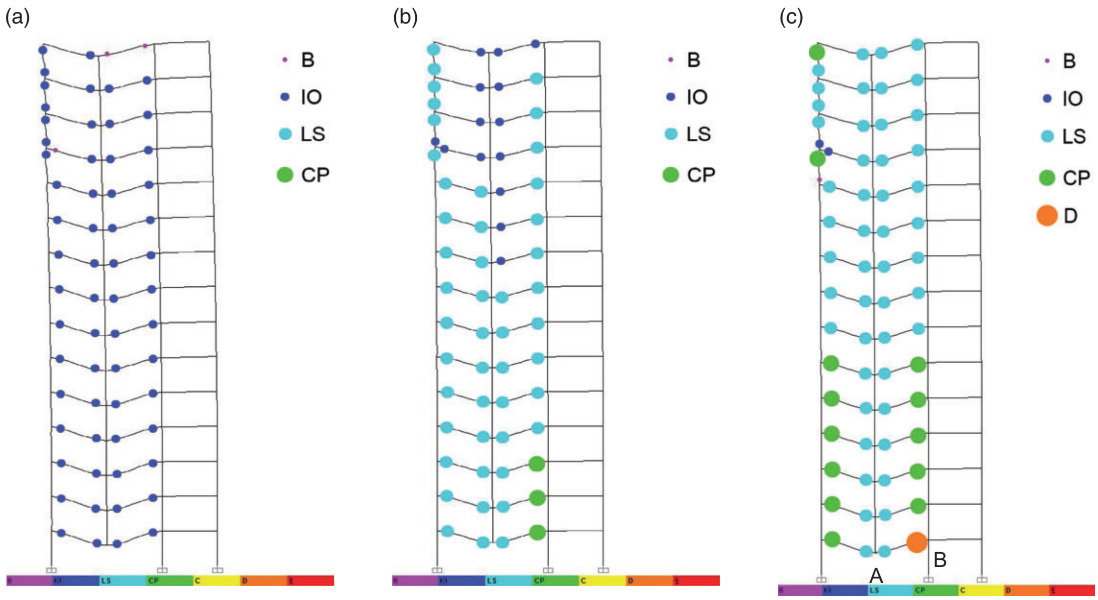

Figure 14. Plastic hinge formation of 6-m span structure at various deformation levels. (a) At GSA limit state. (b) At collapse prevention state. (c) At collapse state.

that the structure has enough strength against progressive collapse caused by sudden loss of a column. It can be observed that the load factor of the structure with 6-m span exceeds 1.0, whereas those of the structures with 9-m and 12-m span lengths are less than 1.0. According to the GSA guidelines shown in Table 4, the failure criterion for a beam is the rotation angle of 0.035 rad. This corresponds to vertical deflection of 21 cm and 31.5 cm for the model structure with 6-m and 9-m span length, respectively. In the pushdown curve for the structure with 6-m span length, the limit states recommended in the GSA guidelines and the collapse prevention (CP) and the collapse (C) limit states specified in the FEMA-356 are indicated. The CP and C points marked in the pushdown curve correspond to the deformation states that the plastic hinge rotation of a member reached CP and C points, respectively, shown in the bending moment–member rotation model of Figure 13(b). It can be observed that the structure can resist higher load after the GSA recommended limit state is reached. This can be confirmed in Figure 14, which depicts the plastic hinge formation of the 6-m span structure at three different deformation levels marked in Figure 13(a). It can be noticed that in most members, the plastic deformation corresponds to immediate occupancy (IO) level when the GSA limit state of vertical displacement is reached. It also can be observed that the pushdown curve suddenly drops when C (collapse) level plastic deformation occurred at the beam right above the lost column.

The vertical displacement time histories of the model structures with and without viscous dampers were obtained by nonlinear dynamic analysis and are shown in Figure 15. It can be observed that no structure failed due to sudden loss of the column. However, the maximum and the final values for vertical displacement vary depending on the span length. The structure with 12-m span length showed the largest vertical displacement as expected from the lowest maximum strength obtained from

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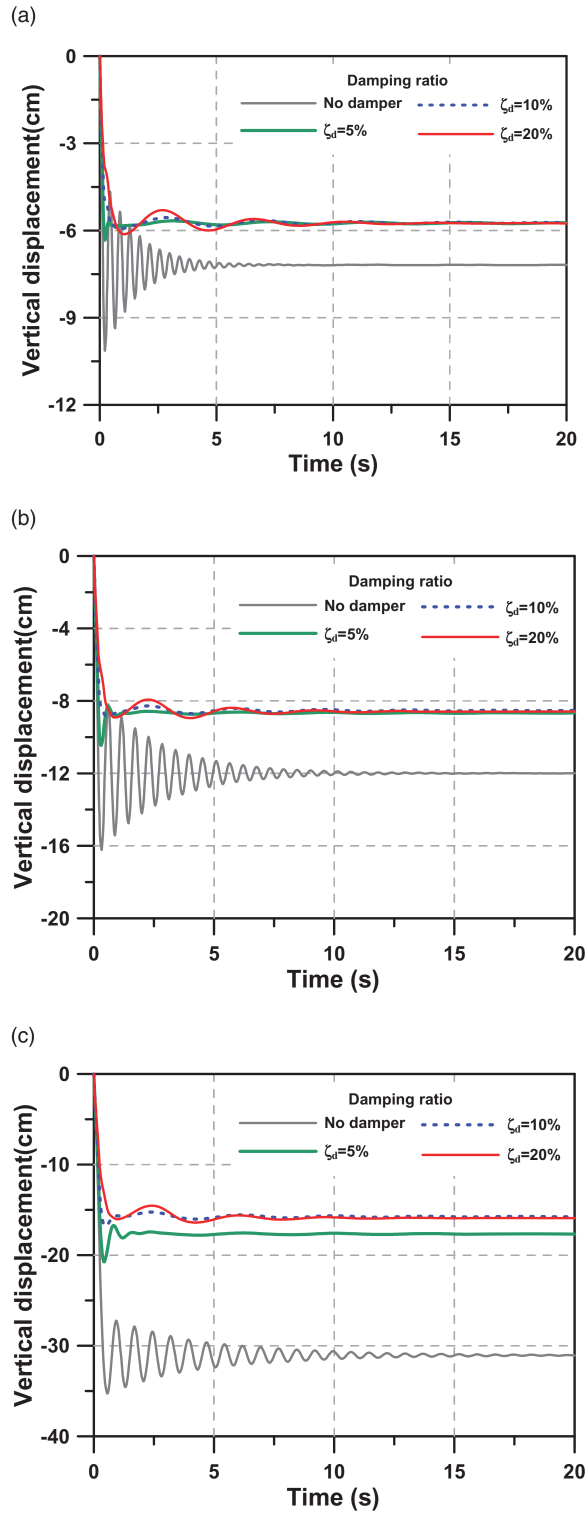

Figure 15. Vertical displacements of model structures with and without dampers. (a) Structure with 6-m span. (b) Structure with 9-m span. (c) Structure with 12-m span.

pushdown analysis. According to the analysis results, the maximum displacements of the model structures did not exceed those limit states. It is observed that, when the dampers designed to provide 20% of the critical damping are installed, the maximum displacements of the structures with 6-m and 9-m span lengths decreased from 10.1 cm and 16.2 cm to 6.1 cm and 9.0 cm, respectively. The final displacements also reduced from 7.2 cm and 12.0 cm to 5.8 cm and 8.6 cm, respectively. Similar behavior was observed in the structure with 12-m span. As the viscous dampers were assumed to have no stiffness, the reduction of final displacement implies that the structures without dampers experienced inelastic deformation when an interior column was suddenly removed. It can be noticed in the figure that due to the installation of dampers, the amplitudes of vibration were significantly reduced at the beginning of vertical vibration. The dampers are most effective in reducing the vertical deflection caused by sudden loss of a column in the structure with 12-m span length, which showed largest deflection and plastic deformation. This coincides well with the results of the parametric study shown in Figure 4 to 6.

Figure 16 depicts the dynamic response factors of the model structures as a function of the damping coefficient of the damper. It can be observed that as the damping coefficient increases, the dynamic response factor generally decreases, and that as the beam length increased the effect of viscous dampers also increased. However, as can be observed in Table 5, which shows the maximum vertical displacements of the model structures with various damping ratios, the maximum vertical displacements of the structures, especially the structures with 6-m and 9-m span lengths, slightly increase as the damping coefficient (and thus damping ratio) increases above the saturation level. The saturation level for a damping ratio increased as the span length increases, which was also observed in the analysis of the subassembly structure as shown in Figure 5.

Figure 17 depicts the locations of plastic hinges in the model structures without and with the dampers at maximum displacements. It can be observed that a lot of plastic hinges formed in the

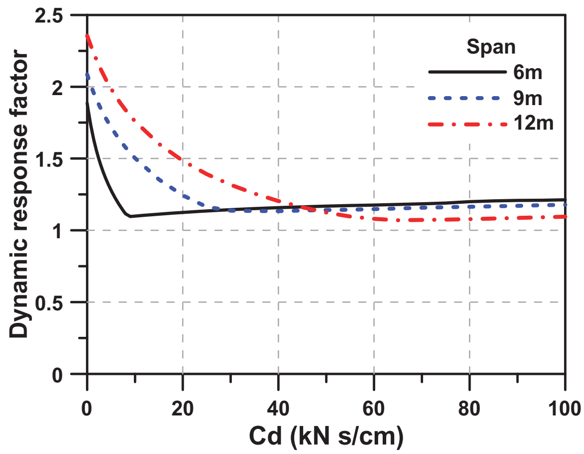

Figure 16. Dynamic response factor of model structures with various damping coefficients.

Table 5. Maximum vertical displacements of the model structures with various damping ratios and span lengths (unit: cm).

	Span length (cm)		
Damping ratios	6 m	9 m	12 m
Static	−5.7	−8.5	−14.6
No damper	−10.14	−16.23	−35.27
5%	−6.33	−10.46	−20.74
10%	−5.95	−8.85	−16.79
20%	−6.12	−8.95	−16.40

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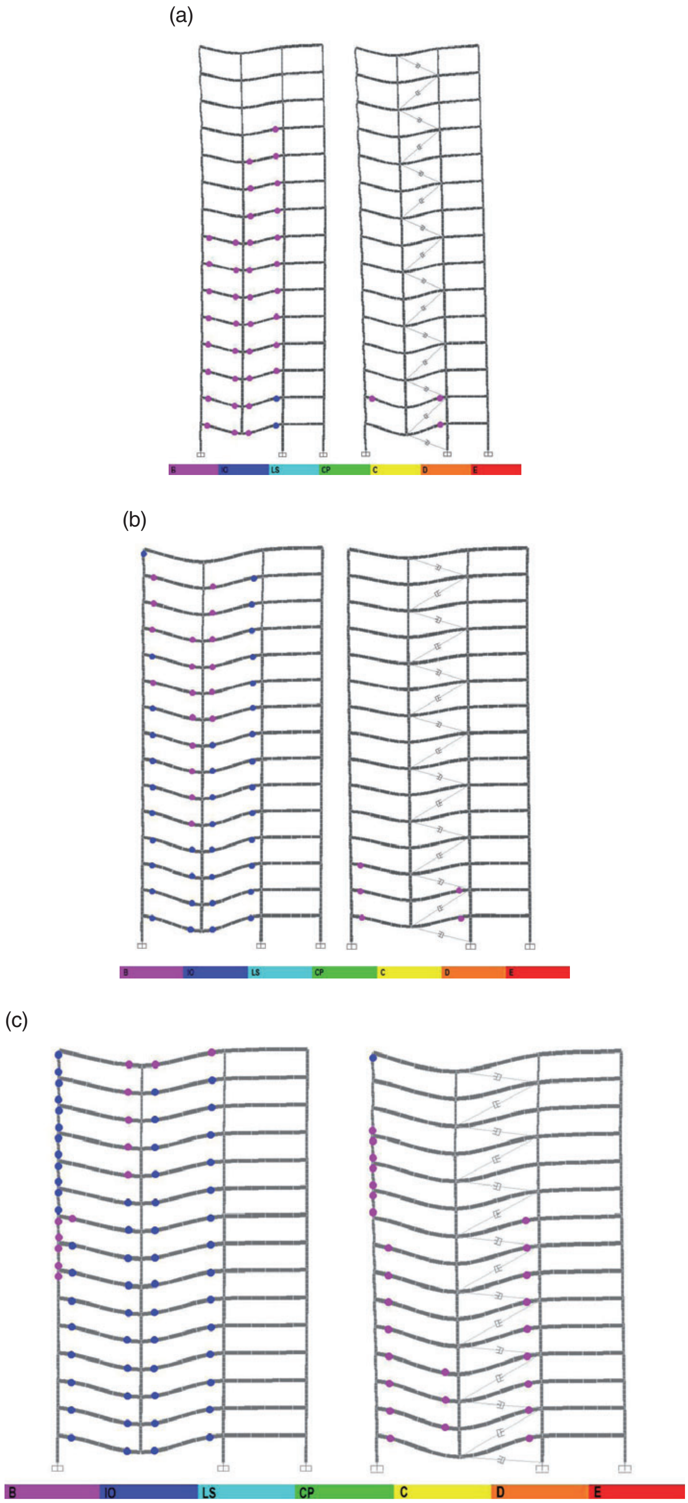

Figure 17. Plastic hinge formation in model structures without and with dampers. (a) Structure with 6-m span. (b) Structure with 9-m span. (c) Structure with 12-m span.

lower story beams of the structures without dampers when one of the interior columns was suddenly lost. More beams yielded in the structure with longer span length. However, in the structures with added viscous dampers, less plastic hinges formed and the amount of plastic rotation was significantly reduced.

5. CONCLUSIONS

In this study, the progressive collapse resisting capacity of structures with viscous dampers was evaluated using nonlinear time-history analysis based on column removal scenario, and the results were compared with those of the structure without viscous dampers. First, the effects of dampers installed in steel beam-column subassemblages with varying natural period and yield strength were evaluated after sudden removal of the column. Then three-bay 15-story structures were designed with viscous dampers and were analyzed for progressive collapse.

According to the parametric study, the vertical displacement generally decreased as the damping ratio of the system increased, and the dampers were effective in both the elastic and the elasto-plastic systems. The analysis results showed that when there was no damping, the dynamic response factor was 2.0 when the beams were in elastic stage and increased after formation of plastic hinges. As the damping ratio increased, the dynamic response factor kept decreasing toward 1.0. It was also observed that the effect of the damper increased as the natural period of the structure increased and the strength ratio decreased. The nonlinear dynamic analysis results of the 15-story analysis models showed that the dampers, designed to reduce earthquake-induced vibration, were effective in reducing vertical displacement of the structure when a column was suddenly removed from the bay at which the dampers were installed. The effect was more predominant in the structure with 12-m span length, which showed largest deflection and plastic deformation. This result coincided well with the observation made in the parametric study. As the damping coefficient of the viscous dampers increased, the dynamic response for vertical displacement decreased until the damping coefficient reached a saturation level. The saturation level for damping ratio increased as the span length increased, which was also observed in the analysis of the subassembly structure.

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