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Design of MR dampers to prevent progressive collapse of moment frames

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Abstract. In this paper the progressive collapse resisting capacity of steel moment frames with MR dampers is evaluated, and a preliminary design procedure for the dampers to prevent progressive collapse is suggested. Parametric studies are carried out using a beam-column subassemblage with varying natural period, yield strength, and damper force. Then the progressive collapse potentials of 15-story steel moment frames installed with MR dampers are evaluated by nonlinear dynamic analysis. The analysis results of the model structures showed that the MR dampers are effective in preventing progressive collapse of framed structures subjected to sudden loss of a first story column. The effectiveness is more noticeable in the structure with larger vertical deflection subjected to larger inelastic deformation. The maximum responses of the structure installed with the MR dampers designed to meet a given target dynamic response factor generally coincided well with the target value on the conservative side.

Keywords: MR dampers; moment frames; progressive collapse; nonlinear dynamic analysis

1. Introduction

Magneto-rheological (MR) dampers are semi-active control devices that use MR fluids to produce controllable dampers. They offer the adaptability of active control devices without requiring the associated large power sources, which is particularly critical during seismic events when the main power source to the structure may fail. MR fluids typically consist of magnetically polarizable particles dispersed in mineral or silicone oil. When a magnetic field is applied to the fluids, the fluid becomes a semi-solid and exhibits viscoplastic behavior.

The active and semi-active control of structures with MR dampers has been studied extensively (Soong and Dargush 1997, Spencer *et al*. 1997, Dyke *et al*. 1998, Jansen and Dyke 2000, Yang *et al*. 2002, Lee *et al*. 2010) for protection of structures against seismic load. Lee *et al*. (2007) investigated the applicability of MR dampers for controlling building structures considering soil-structure interaction effects. Park *et al*. (2010) investigated the seismic performance of a building structure installed with an MR damper by using real-time hybrid testing method. Huang *et al*. (2012) investigated the effectiveness of a MR damper as a semi-active control device for the

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This study investigates the application of MR dampers for preventing progressive collapse of a structure subjected to abnormal load which includes any loading condition that is not considered in normal design process but may cause significant damage to structures. The potential abnormal loads are categorized as: aircraft impact, design/construction error, fire, gas explosions, accidental overload, hazardous materials, vehicular collision, bomb explosions, etc (NIST 2006). Progressive collapse has become an important issue in structural design of building structures since collapse of the World Trade Center twin towers in 2001. Analysis procedures and program softwares are developed to simulate collapse behavior of structures (Kaewkulchai and Williamson 2003, Kim *et al*. 2009). The performances against progressive collapse have been studied for steel moment frames (Powell 2005, Kim and Kim 2009) and for reinforced concrete structures (Sassani and Kropelnicki 2007, Yi *et al*. 2008). Recently Kim *et al*. (2013) investigated the progressive collapse performance of structures with viscous dampers.

In this paper the progressive collapse resisting capacity of steel moment frames with MR dampers is evaluated based on an arbitrary column removal scenario recommended in the Alternate Path method of the GSA (2003) and UFC (2013) guidelines. Parametric studies are carried out using a beam-column subassemblage with varying natural period, yield strength, and the force of a MR damper. Then the progressive collapse potentials of 15-story steel moment frames installed with MR dampers are evaluated by nonlinear dynamic analysis. Finally a preliminary design procedure for MR dampers to prevent progressive collapse is suggested based on the results of the parametric study.

2. Modeling of MR dampers

The equation of motion of a structure equipped with MR dampers subjected to external force is represented by

$$\mathrm{M}\ddot{\mathrm{X}}(t) + C\dot{\mathrm{X}}(t) + \mathrm{K}\mathrm{X}(t) = F_o(t) + \mathrm{H}F_{MR}(t) \tag{1}$$

where, **M**, C, and **K** represent the n x n structural mass, damping, and stiffness matrices, respectively; X(t) the $n\times1$ vector of the relative structural displacement to the ground input motion; $F_o(t)$ is the applied load; H is the vector that represents the location of the MR dampers; and $F_{MR}(t)$ is the control force exerted by the MR dampers on the structure. The nonlinear force-velocity relationship of MR dampers has been simulated by various modeling approaches such as Bingham model, bi-viscous model, hysteretic bi-viscous model, and Bouc-Wen model, etc. (Wen 1976, Stanway *et al*. 1987, Gamota and Filisko 1991, Areley *et al*. 1998). The performance of each model is compared by Yang *et al*. (2001), which shows that the difference in structural responses is not significant depending on the models. In this study the behavior of MR dampers is modeled by the Bingham model which consists of a Coulomb friction element placed in parallel with a viscous dashpot. Fig. 1 depicts the schematic description of a single degree-of-freedom system with an MR damper, where K and C_s represent the stiffness and inherent damping of the system, f_d and c_d denote the friction force and damping coefficient of the MR damper, and F_0 is the applied load. In the Bingham model the force generated by the device, F_{MR}, is given by (Spencer *et al*. 1997)

$$F_{MR} = f_d \,\mathrm{sgn}(\dot{u}) + c_d \dot{u} \tag{2}$$

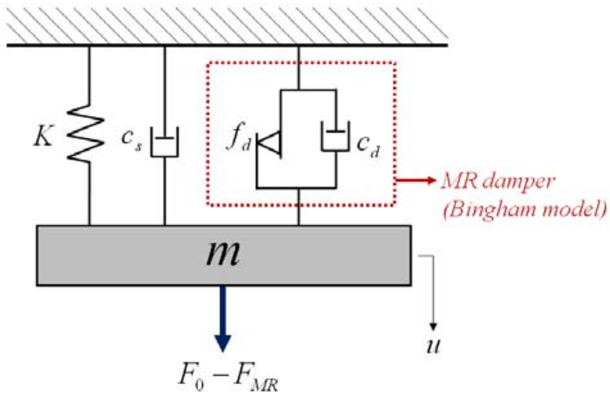


Fig. 1 Mathematical model for a MR damper

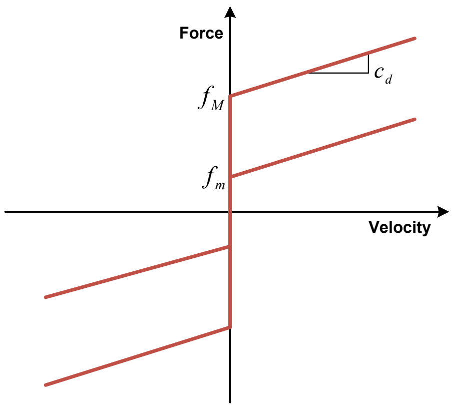


Fig. 2 Force-velocity relationship of Bingham Model

where f_d is the variable friction force; sgn($\dot{u}$) is $\dot{u}/|\dot{u}|$; and c_d is the additional damping coefficient provided by the MR damper. As depicted in Fig. 2 which represents the velocity *vs*. damping force relationship, the friction force of a MR damper f_d can be varied from f_m to f_M by controlling the voltage.

The effectiveness of the MR damper-based control systems for seismic protection of building structures is verified when some semi-active control algorithms are used to mitigate the response of building structures (Lee *et al*. 2010, Jung *et al*. 2006). According to the previous research, the passive-on control turned out to be very effective for response control of structures. In this study the effectiveness of the passive-on control on enhancing progressive collapse resisting capacity of a structure is compared with that of a semi-active control algorithm. The algorithm used in this study is the MHF (Modulated Homogeneous Friction) algorithm which is considered to be suitable for friction dampers (Jansen *et al*. 2000, Park *et al*. 2010). This control strategy is originally developed for variable friction dampers. In this approach, at every occurrence of local extremes in the deformation of the damper, the MR force applied to the frictional interface is updated to a new value. This algorithm is also applicable for MR dampers because the behavior of a MR damper is similar to that of a friction damper. Dyke *et al*. (1997) show that MHF is effective in controlling the relative displacement and acceleration when the structure is subjected to seismic load. The command signal v_i is selected according to the control law (Inaudi 1997)

$$v_i = V_{max} H(f_c - |f_d|) \tag{3}$$

where $V_{\max}$ is the maximum voltage, $H(\bullet)$ is the Heaviside step function, f_d is the capacity of the MR damper, and $f_c = g_i|\Delta(t-s)|$ where $\Delta(t-s)$ is the local extreme value in the deformation of the MR damper, and $s = \{\min u \geq 0 : \dot{\Delta}(t-u) = 0\}$. The proportionality constant g_i has units of stiffness (kN/m). As can be noticed in Eq. (3), the command signal v_i is either 0 or $V_{\max}$ depending on the required control force and the capacity of the MR damper. Therefore the command signal larger than $V_{\max}$ cannot be offered so that the saturation problem of MR damper is prevented. $V_{\max}$ is directly related to the capacity of the MR damper giving the maximum capacity of F_M in case of MHF algorithm.

3. Parametric study using a beam-column subassemblage

To investigate the effectiveness of an MR damper on the progressive collapse resisting capacity of a structure, parametric study is carried out using a beam-column subassemblage shown in Fig. 3. The structure is composed of two continuous beams with fixed ends and a column which is assumed to be suddenly lost. An MR damper is installed above the lost column and is activated when the column is lost. The MR damper used in the parametric study has the same property with the one used in Jung *et al*. (2006). The wide flange section H 594×302×14×23 with yield stress of 235 N/mm^2 is used for beams, and the bi-linear model with post-stiffness of 3% is assumed in the nonlinear dynamic analysis. The dead and live loads are assumed to be 5.0 kN/㎡ and 2.5 kN/㎡, respectively. The maximum capacity of the MR damper, F_M, is 2200 kN, and the damper is operated by the MHF algorithm with the proportionality constant g_i equal to 200 kN/m. The passive-on control is also applied and the results of the two control algorithm are compared. The applied force F_0 in the modeling of the MR damper, shown in Fig. 1, corresponds to the vertical force generated by the sudden loss of the column.

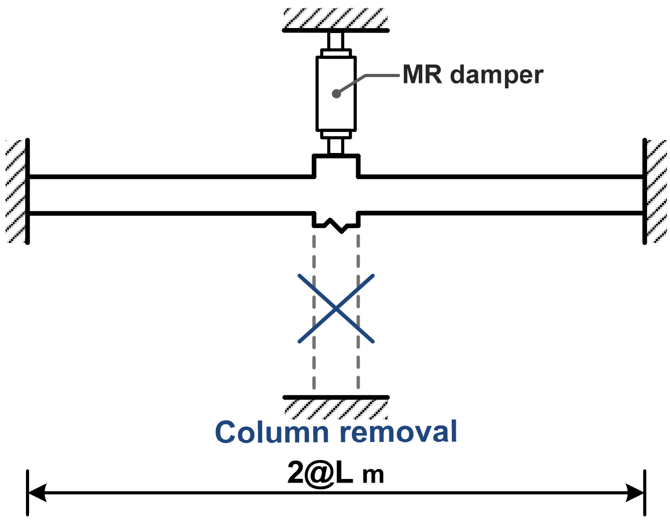


Fig. 3 Beam-column subassemblage for parametric study

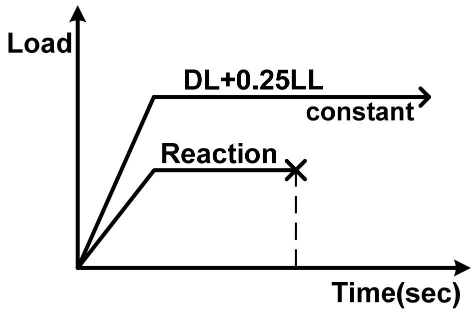


Fig. 4 Time history of applied load for dynamic analysis

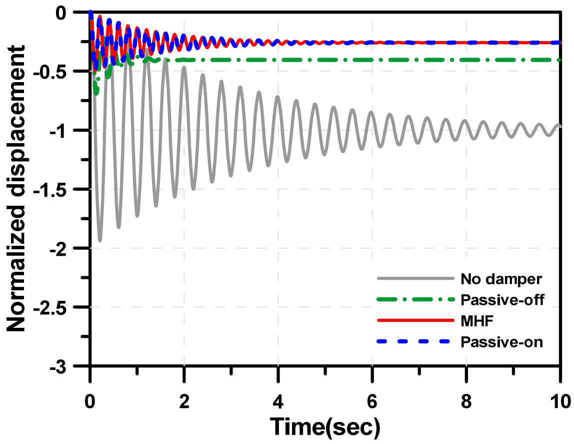


Fig. 5 Time history of normalized displacement without or with a MR damper with different control methods

The collapse behavior of the beam-column subassemblage is investigated through the nonlinear dynamic analysis procedure recommended in the GSA guidelines. In the recommended procedure only material nonlinearity is included and the geometric nonlinearity is not considered. For nonlinear dynamic analysis the load combination DL+0.25LL specified in the GSA 2003 is uniformly applied as vertical load. Then the member forces of a column, which is to be removed to initiate progressive collapse, are computed before it is removed. The column is replaced by the point loads equivalent of its member forces. 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 Fig. 4. In this study the member reaction forces are increased linearly for ten seconds until they reached the specified level, are kept unchanged for five seconds until the system reaches stable condition, and are suddenly removed at fifteen seconds to initiate progressive collapse. The inherent damping ratio is assumed to be 2%, and nonlinear dynamic analysis is carried out using the program code SAP 2000.

Fig. 4 shows the of the vertical displacement time histories of the subassemblage obtained with and without the MR damper. The span length is assumed to be 6 m. Three different control algorithms are applied to control the MR damper; i.e., passive-on, passive-off, and the MHF algorithm. The displacements are normalized with the static displacement. It can be observed that

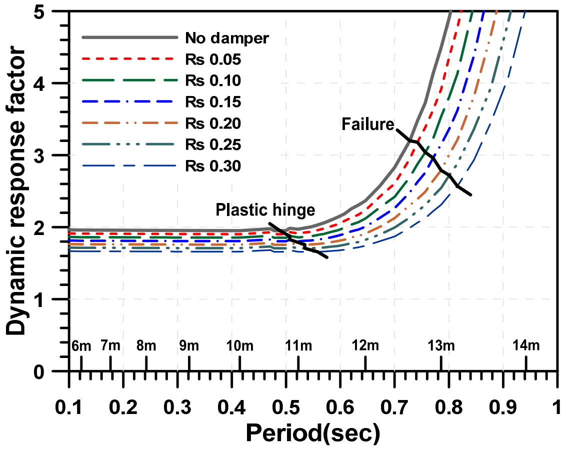
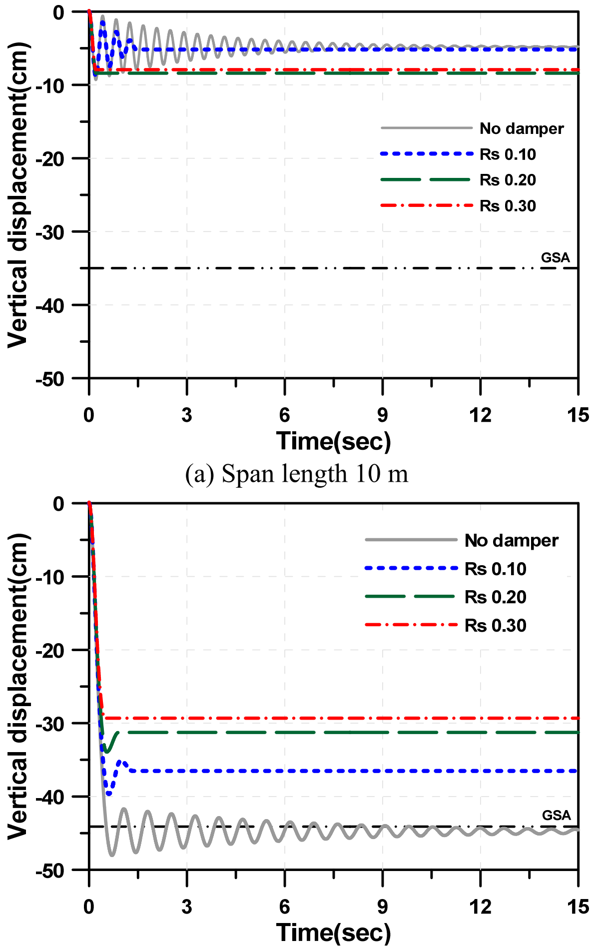


Fig. 6 Dynamic response factor of the subassemblage with a MR damper subjected to sudden loss of the column

the maximum displacement of the structure obtained from the dynamic analysis reaches almost twice the static displacement when no damper is applied. When the MR damper with the maximum capacity of 2200 kN is applied using the three different control algorithms, the displacement is generally reduced. It is observed that the maximum reduction of displacement is achieved by using the passive-on and the MHF algorithm, and that the passive-on control, which always applies the maximum damper force, and the semi-active MHF algorithm result in the similar results. The passive-off control, which is the MR damper, F_m, with the minimum capacity of 1100 kN, results in slightly larger displacement.

Parametric studies of the beam-column subassemblage are carried out for design variables such as natural frequency and damping force. The natural periods of the subassemblage are varied by changing the length of the beams. The post-yield stiffness of the beams is assumed to be 3% of the initial stiffness. Nonlinear dynamic analyses are carried out by suddenly removing the column as recommended in the guidelines. Fig. 6 depicts the dynamic response factor which is the ratio of the maximum displacement and the displacement obtained by linear static analysis. R_s is the damping force of the MR damper normalized by the applied gravity load (DL+0.25LL). The subassemblage is defined as failed when the maximum rotation of the beams exceeds 0.035 radian as recommended in the GSA guidelines for a flexural member. The analysis results show that before the formation of plastic hinges the dynamic response factors are less than 2.0. The factors increase significantly as the natural periods of the subassemblages become larger than about 0.5 second. In a linear elastic system without the damper, the dynamic response is twice the static response. The ratio gets close to 1.0 as the damping ratio increases. The decrease in the response ratio is more pronounced in the inelastic systems, and the natural periods at which plastic hinges and failure occur increase as the damping force increases. This implies that the progressive collapse-resisting capacity of the beam-column subassemblage increases due to the installation of the MR damper.

Fig. 7 shows the vertical displacement time history of the subassemblage with 10 m and 12.6 m span lengths. The inherent damping ratio, ζ_i, is assumed to be 2% of the critical damping. The normalized damping force, R_s, is varied from 10% to 30% of the gravity load. It is observed the subassemblage with 10m span lengths remains elastic after removal of the column and the maximum displacement is far less than the limit state specified in the GSA guidelines. In this case



(a) Span length 10 m

(b) Span length 12.6 m

Fig. 7 Vertical displacement time history of the subassemblage

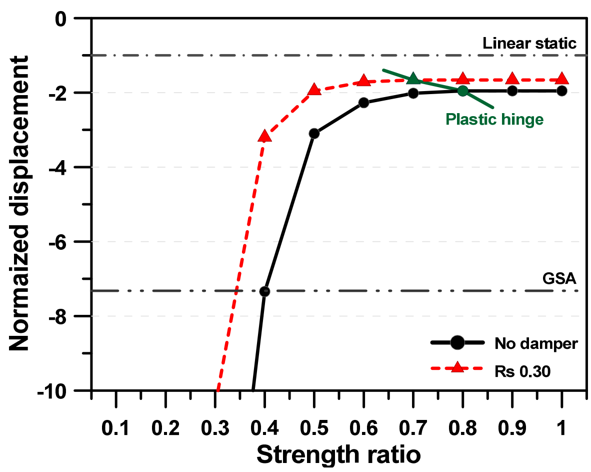


Fig. 8 Normalized displacement of the subassemblage with various strength ratios

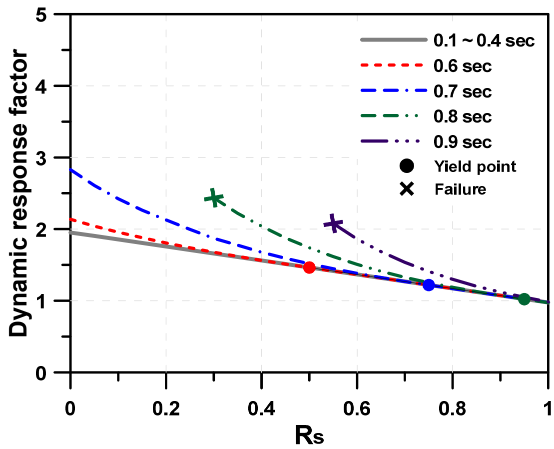


Fig. 9 Dynamic response factor of the subassemblage with various natural periods and damper forces

the final displacement rather increases when a MR damper is installed. When the span length increases to 12.6 m the vertical displacement increases significantly due to the formation of plastic hinges exceeding the specified limitation. In this case the MR damper is quite effective in decreasing the vertical displacement and thus the preventing progressive collapse of the system.

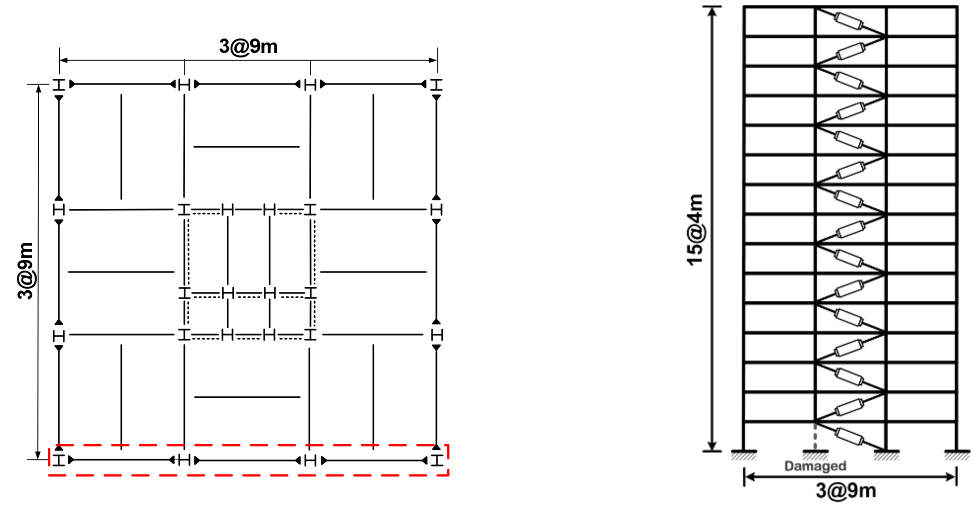
Fig. 8 shows the vertical displacement of the subassemblage with 10 m span length with varying strength of the beams. The displacement is normalized by the displacement obtained by linear static analysis at the strength ratio of 1.0. The damper force is assumed to be 30% of the gravity load. It can be observed that when the damper is not installed the dynamic response ratio increases rapidly as the strength ratio decreases below 0.7. At the strength ratio of 0.4 the rotations of the beams exceed the GSA specified failure criterion of 0.035 rad and the system is defined as failed. The failure is delayed until the strength ratio drops to about 0.3 with installation of the MR damper. The figure shows that the decrease in the normalized displacement is more pronounced in the inelastic system.

Fig. 9 depicts the dynamic response factor of the subassemblage with various span lengths and the damping forces of the MR damper. The system with natural periods ranging from 0.1 to 0.4 second shows linear behavior after the column is removed. In the linear elastic cases the dynamic response factors of the systems with the same damper force are identical regardless of the natural periods. As the natural period of the structure with no damper increases more than 0.5 second the dynamic response factor increases significantly, implying occurrence of inelastic deformation. For the same MR damper the dynamic amplification of the displacement increases as the natural period increases, and as the damper force increases the dynamic response factor approaches 1.0. Based on the analysis results it may be possible to find out the minimum damping force of the MR damper to prevent failure of yielding in case of sudden column loss

4. Progressive collapse of structures with MR dampers

4.1 Design and analysis modeling of model structures

The multi-story analysis structures for application of MR dampers are the 15-story moment



(a) Structural plan of the model structure

(b) Elevation of the model structure and the location of the removed column

Fig. 10 Analysis Model structure with 9 m span length

Table 1 Member size of analysis models (Unit: mm)

Story	Ext. columns	Int. columns	Beams
(a) 6m span model			
1~3	H 298×299×9/14	H 344×348×10/16	H 244×175×7/11
4~6	H 250×255×14/14	H 300×300×10/15	H 244×175×7/11
7~9	H 250×250×9/14	H 294×302×12/12	H 244×175×7/11
10~12	H 244×252×11/11	H 250×255×14/14	H 244×175×7/11
13~15	H 208×202×10/16	H 200×200×8/12	H 244×175×7/11
(b) 9m span model			
1~3	H 406×403×16/24	H 428×407×20/35	H 386×299×9/14
4~6	H 400×400×13/21	H 414×405×18/28	H 386×299×9/14
7~9	H 394×405×18/18	H 400×408×21/21	H 386×299×9/14
10~12	H 350×350×12/19	H 350×350×12/19	H 386×299×9/14
13~15	H 350×350×12/19	H 300×305×15/15	H 386×299×9/14
(c) 12m span model			
1~3	H 458×417×30/50	H 498×432×45/70	H 594×302×14/23
4~6	H 458×417×30/50	H 498×432×45/70	H 594×302×14/23
7~9	H 458×417×30/50	H 458×417×30/50	H 406×403×16/24
10~12	H 428×407×20/35	H 428×417×30/50	H 406×403×16/24
13~15	H 428×407×20/35	H 350×357×19/19	H 406×403×16/24

frames with 6 m, 9 m, and 12 m span lengths with uniform story height of 4 m. Only the perimeter frames are designed as moment frames to resist lateral loads, and the interior gravity load-resisting frames are simply connected. The plan shape of the prototype structure is shown in Fig. 10(a), and only one of the exterior frames is separated for analysis. The SM490 steel with yield stress of 325 MPa is used for columns and the SS400 steel with yield stress of 235 MPa is used for beams. Dead and live loads of 5.0 kN/m^2 and 2.5 kN/m^2, respectively, are used as gravity load, and the seismic load with S_{DS} and S_{D1} of 0.44 g and 0.23 g, respectively, in IBC format are applied for structural design. The member sizes of the model structure are shown in Table 1. Identical MR dampers in passive-on control are installed in each story of the two-dimensional frame in the mid-bay as shown in Fig. 10(b). The damper force is expressed as a portion of the gravity load imposed on the first story column to be removed. Three different damper forces, 10% (R_s=0.1), 20%, and 30% of the column force, are used in the analysis. Table 2 shows the normalized damper forces applied in the model structures.

To carry out nonlinear dynamic analysis of the model structures, the material model of the structural members recommended by the FEMA-356 (2003) is used. Fig. 11(a) shows the bending moment *vs*. rotation angle relationship of the flexural members. The coefficients used to define the nonlinear behavior (*a, b* and *c*) are computed considering the width-thickness ratios of the structural members, and are summarized in Table 3 for each model structure. Fig. 11(b) indicates the deformation levels corresponding to each performance point such as the first yield, immediate occupancy (IO), life safety (LS), collapse prevention (CP), collapse, and fracture specified in the FEMA-356 (2003). The inherent damping ratio of the structure is assumed to be 2% of the critical damping.

4.2 Performance of the model structures subjected to sudden column removal

Nonlinear dynamic analyses of the model structures are carried out using the program code SAP2000 (2004) with one of the first story interior columns suddenly removed. Fig. 12 shows the vertical displacement time histories of the model structures without and with MR dampers with three different damping forces. The linear static analysis results and the failure limit states specified in the GSA guidelines (2003) are also plotted in the figures. According to the GSA guidelines a flexural member in a moment frame is considered to be failed when the maximum rotation exceeds 0.035 radian, which corresponds to 21 cm, 31.5 cm, and 42 cm in the model structure with 6 m, 9 m, and 12 m span length, respectively. The maximum displacements obtained from the analyses are summarized in Table 4. It can be observed in the analysis results

Table 2 Damping force of the MR damper installed in the model structures

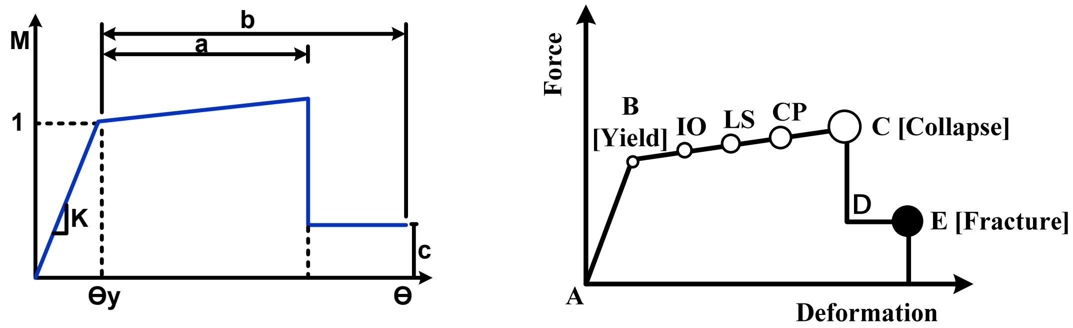
Span	Damper force (kN)		
	R_s 0.1	R_s 0.2	R_s 0.3
6 m	10.1	20.3	30.4
9 m	26.1	52.1	78.2
12 m	46.6	93.1	139.7

Table 3 Coefficients for defining nonlinear behavior of flexural members

Span	Story	Parameters		
		a	*b*	*c*
6m	1~15	8.42	10.42	0.55
9m	1~15	4	6	0.2
12m	1~6	9	11	0.6
	7~15	7.28	9.28	0.46

Table 4 Maximum vertical displacements (cm)

Damping	Span length		
	6 m	9 m	12 m
Static	−10.54	−13.61	−17.68
No damper	−32.10	−44.83	−62.88
R_s 0.1	−12.85	−20.55	−39.93
R_s 0.2	−12.56	−19.28	−36.83
R_s 0.3	−12.27	−18.10	−34.19



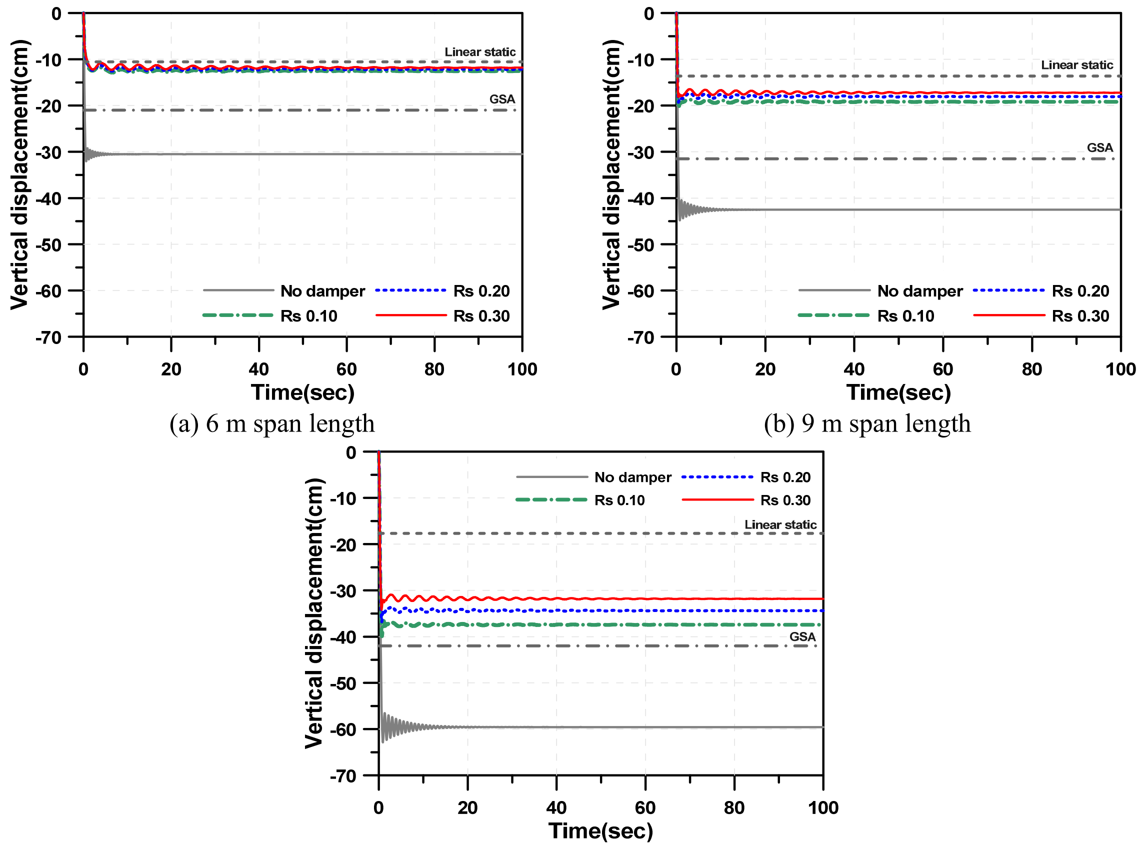
(a) Force-deformation relationship (b) Definition of performance points

Fig. 11 Nonlinear modeling of a flexural member

that in the model structures without MR dampers both the maximum and the final displacements exceed the limit states and the structures are considered as failed due to progressive collapse. The linear static analysis results are significantly smaller than those of the nonlinear dynamic analysis, and the difference increases as the span length increases. After the installation of the MR dampers with the damping force equivalent of 10% of the column gravity load, the maximum vertical displacements are reduced to 40% (6 m span), 46% (9 m span), and 63.5% (12 m span) of the maximum displacements obtained without the dampers. As the damping force increases the displacements further decrease but the effect is not significant.

Fig. 13 depicts the formation of plastic hinges in the model structure with 9 m span length without and with the MR dampers. It can be observed that plastic hinges corresponding to the collapse prevention state formed in the lower story beams of the structures without the dampers when one of the interior columns is suddenly lost. When the MR dampers are installed the plastic rotation in the beam ends are reduced to below immediate occupancy (IO) state and in many locations plastic hinges disappeared.

Fig. 14 depicts the dynamic response factors of the model structures as a function of the damping force of the MR dampers. It can be observed that as the damping force increases the dynamic response factor generally decreases toward 1.0, and that as the beam length increases the response factor also increases. The minimum amount of damper force required to prevent progressive collapse is also indicated in the figure. It turns out that the collapse can be prevented when the minimum damper force of R_s=0.05 is provided in the structures with 6 m and 9 m span lengths, and the damper force of R_s=0.15 is provided in the structure with 12 m span length. It also can be observed that the maximum vertical displacements of the structures decrease only slightly



(a) 6 m span length

(b) 9 m span length

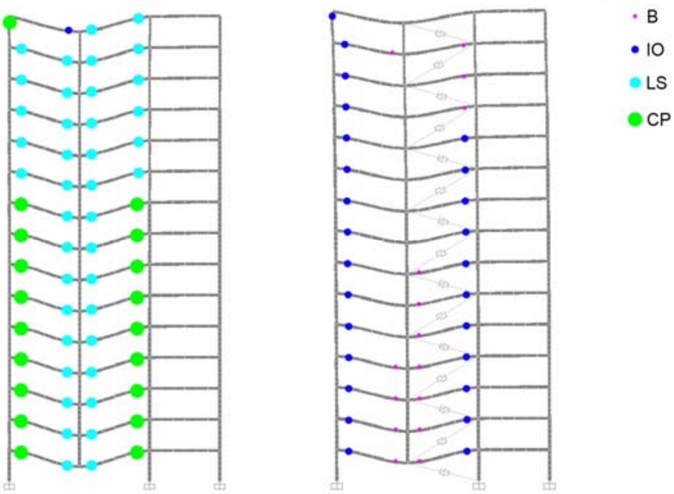
(c) 12 m span length

Fig. 12 Time history of the vertical displacement of the model structures subjected to sudden loss of an exterior column

as the damping force further increases above a certain level; i.e., a saturation level exists in each model structure above which the effect of the MR dampers does not increase in proportion to the amount of the damping force. It can be noticed that the saturation level increases as the span length increases.

4.3 Preliminary design procedure for MR dampers

In this section the amount of damping force required to achieve a target dynamic response is obtained based on the parametric analysis results of the beam-column subassembalge. From the dynamic response factor corresponding to each natural period and damper force of the subassemblage shown in Fig. 6, it can be determined whether the dynamic response factor and therefore the maximum vertical displacement of a structure satisfy the limit state (failure criterion) of the GSA guidelines or not. Once it turns out that the structure fails as a result of the sudden column removal, the damper force required to meet a desired dynamic response can be obtained from the figure. Once the required damper force is obtained, it is uniformly distributed to each story of the structure. Nonlinear dynamic analysis is carried out to check whether the added



(a) Without dampers (b) With MR dampers

Fig. 13 Plastic hinge formation of the model structure with 9m span length

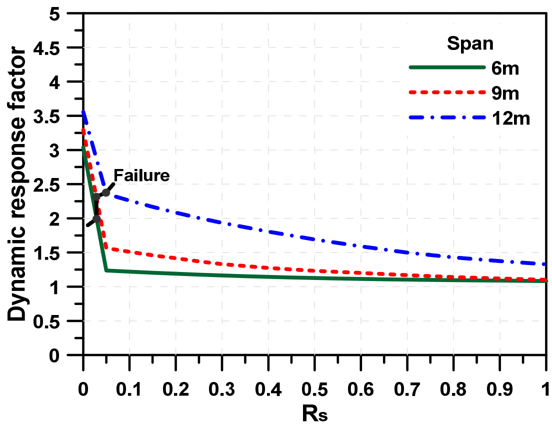


Fig. 14 Dynamic response factors of the model structures with MR dampers with various damping force

dampers satisfy the target response. As the parametric study results presented in Fig. 6 are obtained from analysis of a subassembalge, which is a single degree-of-freedom system, the procedure may produce approximate solution when applied to multi-story structures. In this sense the determined damper force may be considered as a preliminary design for the multi-story structure and therefore needs to be refined for final design. The design process for MR dampers to prevent progressive collapse is summarized as follows:

Step 1: Carry out modal analysis of the structure after removing a column and obtain fundamental vibration mode for the vertical vibration.

Step 2: Read dynamic response factor corresponding to the natural period of the structure in Fig. 6.

Step 3: If the dynamic response factor exceeds the failure point, obtain the damper force required to meet a desired dynamic response from the figure.

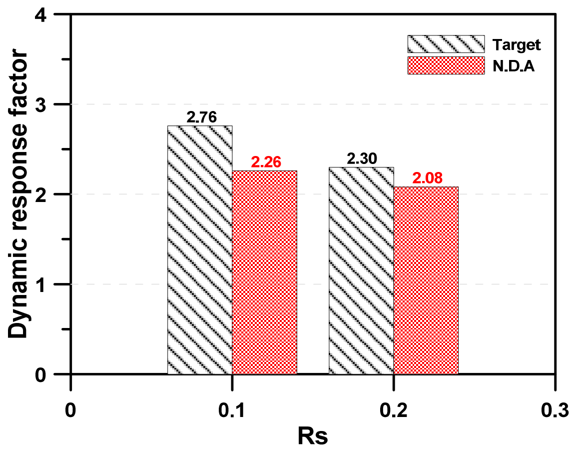


Fig. 15 Comparison of the target dynamic response factors of the model structure with 12m span length with those obtained from nonlinear dynamic analyses

Step 4: Evenly distribute the required damping force to each story of the structure and determine appropriate MR damper.

Step 5: Carry out nonlinear dynamic analysis of the structure with dampers by suddenly removing a column and check whether the maximum displacement (dynamic response factor) is less than the limit state.

Step 6: If the limit state is still exceeded, repeat from Step 1 to obtain the additional damping force required to satisfy the limit state.

The preliminary design procedure is applied to the 15-story model structure with 12 m span length. The fundamental natural period of the structure is computed as 0.78 second, and according to Fig. 6 the dynamic response factor corresponding to the specific natural period is approximately 4.5. This exceeds the failure limit state of 2.76 which corresponds to the maximum beam rotation of 0.035 radian specified in the GSA guidelines. To prevent failure of the structure, target dynamic response factor is set to 2.76 and the corresponding damper force of R_s=0.2 is obtained from Fig. 6. For comparison another set of MR damper is designed based on the target dynamic response factor of 2.3 and the corresponding R_s of 0.3. The required damper forces are evenly distributed to each story of the model structure and the maximum displacement of the structure with each of the two sets of MR dampers is obtained by nonlinear dynamic analysis after removing one of the first-story interior columns. The maximum dynamic response factors of the model structures with the MR dampers designed by the above procedure are compared with the given target values in Fig. 17. It can be observed that the maximum responses of the system installed with two different sets of MR dampers generally coincide well with the target values on the conservative side. The errors are 18% and 10% for the target response factors of 2.76 and 2.30, respectively.

5. Conclusions

In this paper the progressive collapse resisting capacity of steel moment frames with MR dampers was evaluated, and a preliminary design procedure for the dampers to prevent progressive collapse of framed structures was suggested. The effect of damper force on the dynamic response of a steel beam-column subassemblage was evaluated after sudden removal of a column following the Alternate Path approach. Parametric studies were carried out with varying natural period, yield strength, and damper force. Then the progressive collapse potentials of 15-story steel moment frames installed with MR dampers were evaluated by nonlinear dynamic analysis. Finally a design procedure was proposed to estimate the required damper force of MR dampers to achieve a desired target response based on the parametric study of the beam-column subassemblage.

According to the results of the parametric study, the dynamic response factor decreased toward 1.0 as the MR damper force increased. The effectiveness of the MR dampers became more pronounced in the structures with longer natural periods and in the structures subjected to larger inelastic deformation. The analysis results of the 15-story structures showed that the dampers were effective in preventing progressive collapse of the model structures subjected to sudden loss of a first story column. The effectiveness was more noticeable in the structure with 12 m span length with larger vertical deflection, which corresponded to the results of the parametric study. The maximum responses of the structure installed with the MR dampers designed to meet a given target dynamic response factor generally coincided well with the target value on the conservative side.

In the current design practice dampers are generally used to reduce wind or earthquake induced vibration. In case MR dampers are designed to enhance structural capacity against progressive collapse as well as wind or earthquake load, the more realistic design procedure is to design dampers to satisfy structural performance for wind or earthquake load first following current design codes, and then to check the progressive collapse potential of the structure based on the guidelines. If the performance against progressive collapse turns out to be unsatisfactory, then the amount of additional damping force required to satisfy the limit state for progressive collapse can be obtained by following the design procedure proposed in this study. The proposed design procedure can be used not only to design new buildings against progressive collapse but also to enhance progressive collapse resisting capacity of existing structures in which conventional retrofit techniques may not be applicable.

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