Assessment of capacity design of columns in steel moment resisting frames with viscous dampers

Theodore L. Karavasilis*

School of Engineering, University of Warwick, Coventry CV4 7AL, United Kingdom

ABSTRACT

Previous research showed that steel moment-resisting frames (MRFs) with viscous dampers may experience column plastic hinges under strong earthquakes and highlighted the need to further assess the efficiency of capacity design rules. To partially address this need, three alternatives of a prototype building having five, 10 and 20 stories are designed according to Eurocode 8 using either steel MRFs or steel MRFs with dampers. Incremental dynamic analysis (IDA) is conducted for all MRFs and their collapse resistance and plastic mechanism is evaluated. The results show that steel MRFs with dampers are prone to column plastic hinging in comparison to steel MRFs. The steel MRFs with dampers are then iteratively re-designed with stricter capacity design rules to achieve a plastic mechanism that is approximately similar to that of steel MRFs. The performance of these re-designed steel MRFs with dampers indicates, that overall, enforcement of stricter capacity design rules for columns is not justified neither from a collapse resistance or a reparability perspective.

Key-words: Capacity design, Steel MRFs, Eurocode 8, Plastic mechanism, Collapse

*Corresponding author. E-mail: T.Karavasilis@warwick.ac.uk

1. Introduction

Conventional seismic-resistant steel structures may experience significant structural and non-structural damage under strong earthquakes due to large story drifts and cyclic plastic deformations in main structural members [1]. Damage results in socio-economic losses (e.g. large repair costs and loss of building occupancy), which are no longer acceptable by modern societies aiming to achieve high levels of earthquake resilience. Therefore, there is an urgent need for codification and widespread implementation of resilient seismic-resistant steel structures that are less vulnerable and easier to repair after strong earthquakes [2].

A well-known class of resilient steel structures is the steel moment-resisting frames (MRFs) with passive dampers [3]. Among the different types of dampers, fluid viscous ones have been extensively studied as they have major advantages including large energy dissipation capacity and peak forces that are out of phase with the peak story drifts of elastic or mildly inelastic frames [4]. Viscous dampers consist of a hollow cylinder fully filled with a fluid and a steel piston with a rod and a piston head. Based on previous dynamic tests, the hysteretic behavior of viscous dampers can be described by [4]:

$$F_{\mathrm{D}} = C \cdot |v|^{a} \cdot \mathrm{sgn}(v) \tag{1}$$

where F_{D} is the damper force output, C is the damping coefficient, v is the velocity across the damper, a is the velocity exponent, and sgn is the signum function. Viscous dampers are typically inserted in steel MRFs by using strong supporting braces, which are designed to be stiff enough so that story drift produces damper deformation rather than brace deformation [3].

A parametric study on the seismic response of yielding single-degree-of-freedom (SDOF) systems evaluated the effect of supplemental viscous damping on peak displacements, residual displacements and absolute accelerations [5]. Researchers proposed predictive formulae for the peak relative velocity of yielding SDOF systems for different levels of supplemental viscous damping [6], while others showed that the nonlinearity of the viscous damper influences the probabilistic seismic response of linear elastic SDOF systems [7, 8]. Research efforts quantified the benefits of using viscous dampers for reducing damage in non-structural components of building structures [9, 10]. Notable experimental studies that validated the superior seismic performance of steel MRFs with viscous dampers include the full-scale shaking table tests conducted by Kasai et al. [11] and the large-scale real-time hybrid simulations conducted by Dong et al. [12].

ASCE 7-10 provides a detailed design procedure for buildings with passive dampers within the framework of the traditional response spectrum and equivalent lateral force methods of analysis [13]. These procedures are iterative and their basis is the use of an equivalent highly damped linear elastic SDOF system, which serves as a substitute of the real yielding frame with dampers. The use of the equivalent linear SDOF system allows the damping system (i.e., the frame that includes the viscous dampers, and their supporting braces and connections) to be designed for three different loading conditions, i.e. those associated with the maximum displacement, maximum velocity and maximum acceleration. The effectiveness of the ASCE 7-10 procedure has been extensively evaluated with seismic simulations on steel MRFs with viscous dampers under the design basis and maximum considered earthquake (DBE and MCE, respectively) intensities in [14, 15]. Guo and Christopoulos [16] proposed an alternative design procedure for multiple target performance objectives utilizing a graphic tool to estimate peak response parameters of yielding structures with passive dampers either by nonlinear response history analyses or by an equivalent linearization procedure.

The author and co-workers explored the design requirements (base shear strength, design drift) which guarantee that a steel MRF with viscous dampers will have seismic collapse resistance similar or higher than that of a special steel MRF [17]. Moreover, they showed that the collapse mode of steel MRFs with viscous dampers is generally identical to that of a special steel MRF, i.e. a sway mechanism with plastic hinges in beams and in column bases. In some cases though, the collapse mode was a combination of plastic hinges in beams and plastic hinges in columns of different stories. Interestingly, a collapse mode characterized by a distinctive soft-story mechanism (i.e. formation of plastic hinges at the top and bottom of columns for a particular story) was also observed for few ground motions (e.g. three out of 44 records). The reason of these unique (for a steel MRF) collapse modes is the high viscous dampers forces that impose high axial force demands to the columns. The aforementioned study, which was based only on a 5-storey building, highlights the need for further research on capacity design of columns and its effect on the collapse resistance of steel MRFs with viscous dampers. Moreover, the seismic intensity beyond which plastic hinges are developed in columns of steel MRFs with viscous dampers should be evaluated since column plastic hinges lead to non-reparable damage, while repair of damage in beam plastic hinges can be addressed by using special bolted fuses at the beam ends [18, 19].

This paper aims to partially answer the research questions raised in the previous paragraph by evaluating the efficiency of the capacity design of columns for three steel MRFs with viscous dampers. Three alternatives of a prototype building having five, 10 and 20 stories are designed using either steel MRFs or steel MRFs with viscous dampers. The steel MRFs with viscous dampers are designed to have significantly higher performance than that of the steel MRFs. Incremental dynamic analyses (IDA) [20] under 44 ground motions are conducted for all the frames and their collapse resistances and plastic mechanisms (with a focus on column plastic hinges) under different drift levels are evaluated and compared. The results show that tall steel MRFs with viscous dampers are prone to column plastic hinging in comparison to steel MRFs. The steel MRFs with viscous dampers are then iteratively re-designed to achieve a plastic mechanism that is approximately similar to that of the steel MRFs. The performance of the redesigned frames is assessed with IDA and the results are quantitatively and qualitatively evaluated to explore whether there is a need for stricter capacity design rules for columns of high-performance steel MRFs with viscous dampers.

2. Prototype building and design of seismic-resistant frames

2.1 Prototype building

Fig. 1 shows the plan view of a prototype 5-bay by 3-bay steel office building. Three alternatives of this building having five, 10 and 20 stories (as shown in Fig. 2) are considered. The building has two perimeter 3-bay seismic-resistant MRFs in the longitudinal direction and two perimeter 1-bay seismic-resistant braced frames in the transverse plan direction. This study focuses on the design of one of the perimeter MRFs in the longitudinal direction. This perimeter MRF is designed as a steel MRF according to Eurocode 8 (EC8) [21] and as a steel MRF with linear viscous dampers.

The models used to perform the designs are based on the centerline dimensions of the steel MRFs without accounting for the finite panel zone dimensions. Beam-column connections are assumed to be rigid, while a rigid diaphragm constraint is imposed at the nodes of each floor to account for the presence of the composite slab. Moreover, a 'lean-on' column is included in the models to account for the P-Δ effects of the gravity loads acting in the tributary plan area (i.e. half of the plan area for one perimeter steel MRF).

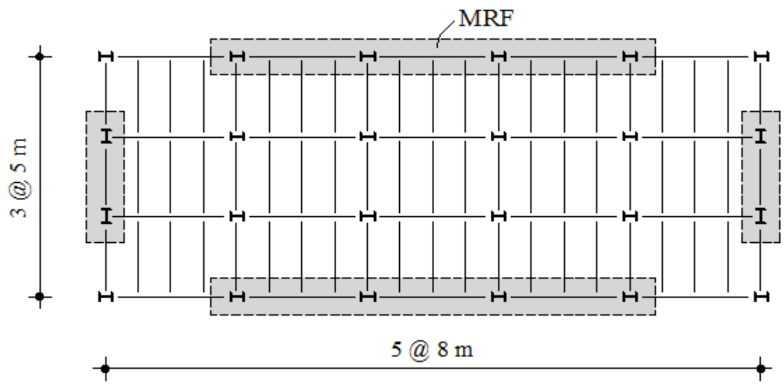

Fig. 1. Plan view of the prototype building

2.2 Design of steel MRFs

The steel MRFs without viscous dampers are designed as high-ductility class according to EC8 [21]. The DBE is expressed by the type 1 EC8 design spectrum for peak ground acceleration equal to 0.35g, ground type B, importance factor II, and behavior factor q equal to 6.5. The steel grade for columns is S355 and for beams is S275. To meet the damage limitation requirement given ductile non-structural elements, the allowable peak story drift, θ_{max}, under the frequently occurred earthquake is equal to 0.75% [21]. The frequently occurred earthquake has an intensity of 40% the DBE, i.e. the v reduction factor is equal to 0.4 according to EC8 [21]. For all the steel MRFs, the story drift sensitivity coefficient θ that accounts for P-Δ effects is limited below 0.20. The weak beam-strong column capacity design rule is enforced by satisfying the condition

$$\sum M_{RC} \geq 1.3 \sum M_{Rb} \tag{2}$$

where ΣM_{RC} is the sum of the plastic moments of resistance of the columns (considers the effect of the axial force in the column) framing a joint and ΣM_{Rb} is the sum of the plastic moments of resistance of the beams framing the same joint.

All designs comply with the specific rules of EC8 for steel MRFs. In particular, the design axial forces in beams are less than 15% of their plastic axial resistance, the design shear forces in beams are less than 50% of their plastic shear resistance, and the design shear forces in columns are less than 50% of their plastic shear resistance. The columns are also checked against axial forces, bending moments and shear forces calculated according to [21]:

$$N_{Ed} = N_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E} \tag{3}$$

$$M_{Ed} = M_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot M_{Ed,E} \tag{4}$$

$$V_{Ed} = V_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot V_{Ed,E} \tag{5}$$

where $N_{Ed,G}$, $M_{Ed,G}$, and $V_{Ed,G}$ are the design values of the axial force, bending moment, and shear force due to non seismic actions; γ_{ov} is the material overstrength factor that is equal to 1.25; and Ω is an overstrength factor which is calculated as the minimum of the ratios of the plastic moment resistance to the internal bending moment under the seismic action of all beams. Design details of the conventional MRFs are provided in Table 1 and in Fig. 2.

2.3 Design of steel MRFs with linear viscous dampers

Linear (a=1; see Equation (1)) viscous dampers are installed in the middle bay of the steel MRFs designed in Section 2.2. Dampers are supported in a horizontal orientation by inverted V braces as shown in Fig. 2. The braces are pinned connected to gusset plates and satisfy the relation $\tau/T < 0.02$ [22], where τ is the relaxation time defined as the ratio C/K_b (K_b is the horizontal stiffness of both braces) and T is the fundamental period of vibration of the steel MRF. The supplemental equivalent viscous damping ratio at the fundamental period of vibration is calculated by [23]:

$$\xi_{eq} = \frac{T}{4\pi} \cdot \frac{\sum_i C_i(\varphi_i - \varphi_{i-1})^2}{\sum_i m_i \varphi_i^2} \tag{6}$$

where φ_i and φ_{i-1} are the first modal displacements of floors i and i-1, respectively, and m_i is the seismic mass of floor i. Damping coefficients are selected to provide a ξ_{eq} equal to 17%. The inherent damping ratio is 3%, and therefore, all steel MRFs with viscous dampers have a total viscous damping ratio, ξ_{tot}, at the fundamental period of vibration equal to 20%. The height-wise distribution of the damping coefficients is selected as proportional to the horizontal story stiffness of the steel MRF as previous research has shown that such distribution is practical and effective in comparison with distributions derived from advanced optimization methods [24].

The response spectrum procedure of ASCE 7-10 [13] is used to check the resistance of the internal columns of the steel MRFs, which are in the force path of the viscous dampers. In particular, these columns are checked against the requirements of Equations (2)-(5) of EC8 for the stages of maximum displacement, maximum velocity, and maximum acceleration under the DBE according to ASCE 7-10 [13]. This additional design check did not result in any change of the internal column cross-sections. Design details of the steel MRFs with viscous dampers are provided in Table 1 and in Fig. 2. The last column of Table 1 shows that supplemental damping reduces the peak story drift under the DBE, $\theta_{max,DBE}$, by 42%, i.e. the MRFs with viscous dampers are designed for a higher performance than that of the conventional MRFs.

Table 1. Design details of the steel MRFs with and without viscous dampers

Frame	T (sec)	ξ_{tot} (%)	$\theta_{max,DBE}$ (%)
MRF			
5-story	1.27		1.84
10-story	2.42	3	1.50
20-story	3.75		1.00
MRF with dampers			
5-story	1.27		1.06
10-story	2.42	20	0.87
20-story	3.75		0.58

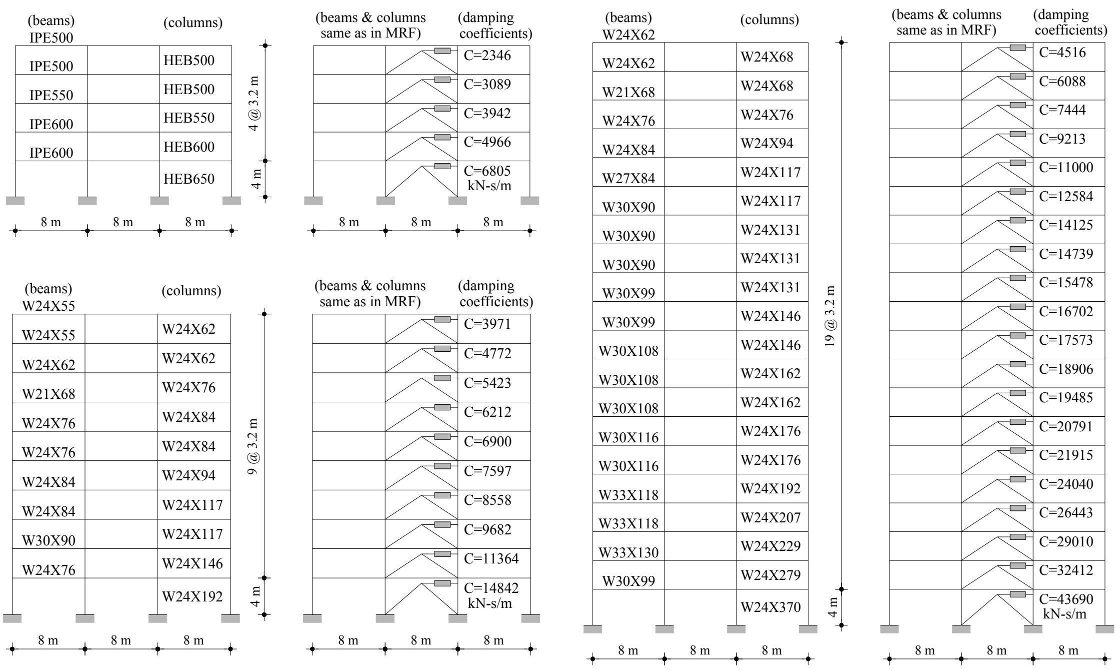

Fig. 2. Elevation view and design details of the steel MRFs with and without viscous dampers

3. Models for nonlinear dynamic analysis and earthquake ground motions

Nonlinear models for the steel MRFs with and without viscous dampers are developed in OpenSees [25]. The columns are modeled as nonlinear force-based beam-column fiber elements with bilinear elastoplastic stress-strain behavior. The assumption of stable hysteresis for the columns is justified by the fact that heavy columns with webs and flanges of low slenderness do not show cyclic deterioration even under large drifts [26]. Beams are modeled as elastic elements with zero length flexural plastic hinges at their ends. The zero length plastic hinges are represented by rotational springs that exhibit strength and stiffness deterioration to simulate beam flange inelastic buckling. The properties of these springs are calculated by using the available predictive equations in [27]. Panel zones are modeled using the Krawinkler model [28]. A rigid diaphragm constraint is imposed at the nodes of each floor to account for the presence of the composite slab, while a 'lean-on' column is included in the models to account for the P-Δ effects of the gravity loads acting in the tributary plan area of the steel MRF.

The viscous dampers are modeled with zero length viscous elements (dashpots), while the supporting braces are modeled with elastic braces. The damper limit states, which occur when the piston reaches its stroke limit during earthquake response, are not considered. Damper limit states should be considered in the assessment of the collapse resistance of frames equipped with viscous dampers having limited stroke [29]. Typical stroke limits in the dampers available in the market ranges from ±80 to ±130mm and strokes can be extensible up to ±900mm upon request [30]. With an extended stroke limit, the dampers of the steel MRFs examined in this study would not reach their limit states even under very large drifts. Therefore, the analytical models in this study are valid under the aforementioned condition.

The Newmark method with constant acceleration is used to integrate the equations of motion of the steel MRFs under a ground motion excitation. The Newton method with tangent stiffness is used to minimize the unbalanced forces within each integration time step. A Rayleigh damping matrix is used to model the inherent 3% damping ratio at the first two modes of vibration. A nonlinear force-controlled static analysis under gravity loads is first performed and then nonlinear dynamic analysis is executed.

A set of 22 pairs of recorded far-field ground motions developed in the FEMA P695 [31] is used for nonlinear dynamic analysis. All the records are recorded on stiff soil or on soft rock, while event magnitudes range from 6.5 to 7.6. None of the records exhibits near-fault pulse-like characteristics. The 5% damped spectral acceleration at the fundamental period of the structure, $S_a(T_1)$, is selected as the ground motion record intensity measure [32] .

4. Collapse fragilities

Incremental dynamic analysis (IDA) [20] is employed for assessing the collapse potential of the steel MRFs with and without viscous dampers. In this method, $S_a(T_1)$ is systematically scaled up in increments until the steel MRF becomes globally unstable and drifts increase without bound given a very small increment of $S_a(T_1)$. The procedure described in [17] is employed to detect the actual $S_a(T_1)$ value leading to collapse of a frame under a specific ground motion. By repeating this procedure for all 44 ground motions, the collapse fragility curve of a frame is obtained by fitting a lognormal distribution to the 44 $S_a(T_1)$ values associated with collapse.

Fig. 3 shows the collapse fragility curves of all frames, where $S_a(T_1)$ is normalized by $S_{a,MCE}(T_1)$, i.e. the MCE spectral acceleration at T_1. Beyond just simplifying the discussion to follow, this normalization also simplifies the comparison of frames having different fundamental periods [17, 33]. The $S_a(T_1)$ at 50% probability of collapse is $3.75 \cdot S_{a,MCE}$ for the 5-story MRF, $6.25 \cdot S_{a,MCE}$ for the 5-story MRF with viscous dampers, $2.5 \cdot S_{a,MCE}$ for the 10-story MRF, $4.5 \cdot S_{a,MCE}$ for the 10-story MRF with viscous dampers, $3.63 \cdot S_{a,MCE}$ for the 20-story MRF, and $6.25 \cdot S_{a,MCE}$ for the 20-story MRF with viscous dampers. The aforementioned values show that supplemental viscous damping significantly increases (i.e. by 60 - 80%) the collapse resistance of steel MRFs.

It is interesting to note that the 5-story MRF has similar collapse resistance with that of the 20-story MRF, while the 10-story MRF has lower collapse resistance. To provide further insight in these results, Fig. 4 plots the base shear coefficient (V/W; V is the base shear and W the seismic weight) – roof drift (θ_r) behavior of the steel MRFs from nonlinear monotonic static (pushover) analysis under an inverted triangular lateral load distribution. The pushover curves show that P-Delta effects are more pronounced as the number of stories increases and that the drift levels at which the pushover curve of the 10- and 20-story MRFs intersects the horizontal axis (leading to negative restoring forces for positive drifts) are significantly lower than that of the 5-story MRF. This explains why the 10-story MRF has lower collapse resistance than the 5-story steel MRF but raises a question for the high collapse resistance of the 20-storey MRF. The reason for the high collapse resistance of the 20-story MRF is hidden in its long period of vibration (see Table 1) and the conservatism of EC8 in designing long period structures. In particular, EC8 [21] imposes a lower bound factor β (typically equal to 0.2) for the horizontal design spectrum, and thus, the 20-story MRF was designed for a seismic intensity higher than that expressed by the elastic spectrum, which is used to normalize the horizontal axis in Fig. 3.

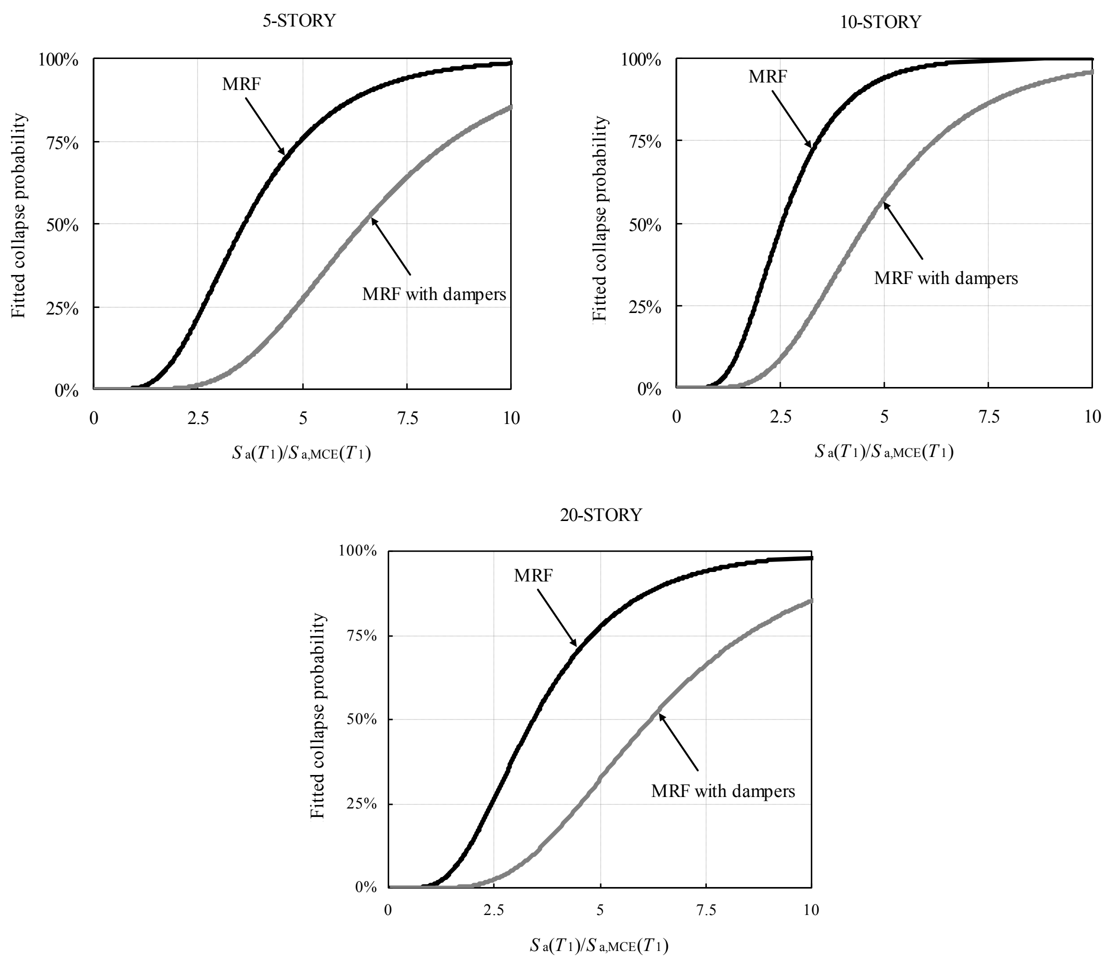

Fig. 3. Collapse fragilities of the steel MRFs with and without viscous dampers

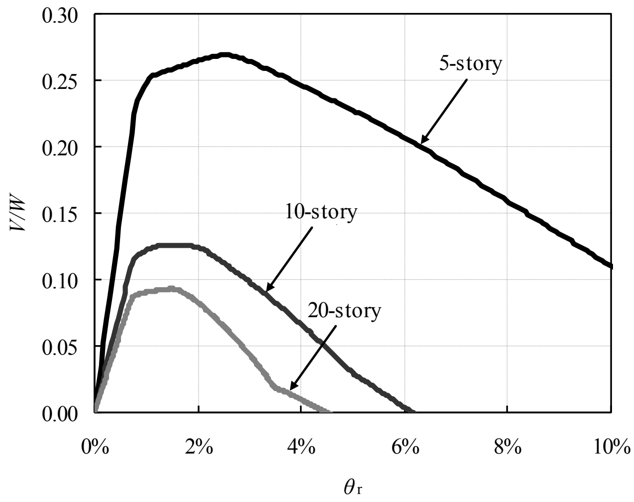

Fig. 4. Pushover curves for the steel MRFs

5. Assessment of capacity design of columns for steel MRFs with viscous dampers

In this section, the effectiveness of the capacity design of columns for steel MRFs with viscous dampers is assessed and compared to that for steel MRFs without viscous dampers through post-processing of the IDA results. The assessment is based on the number of plastic hinges that develop in the columns. The steel MRFs with viscous dampers experience significantly lower story drifts (and therefore, significantly lower column bending moments) than those of the conventional MRFs for a given seismic intensity level as shown in Table 1. Therefore, fair comparisons of the effectiveness of the capacity design of columns among steel MRFs with and without viscous dampers can be achieved by recording the number of column plastic hinges for specific θ_{max} levels. For this reason, linear interpolation on the IDA results is performed to calculate the number of plastic hinges in the columns of a steel MRF subjected to a specific ground motion at different θ_{max} levels. Then, the median value of the number of the plastic hinges in the columns of a steel MRF at a specific θ_{max}

level is calculated from the IDA results for the 44 ground motions. The aforementioned median values are divided by the total number of possible column plastic hinge locations to enable a fair comparison among the percentage of columns developing plastic hinges in steel MRFs of different stories. The total number of possible column plastic hinge locations is 32, 72 and 152 for the 5-story, 10-story and 20-story steel MRFs, respectively, without considering the bottom of the first story columns and the top of the last story columns.

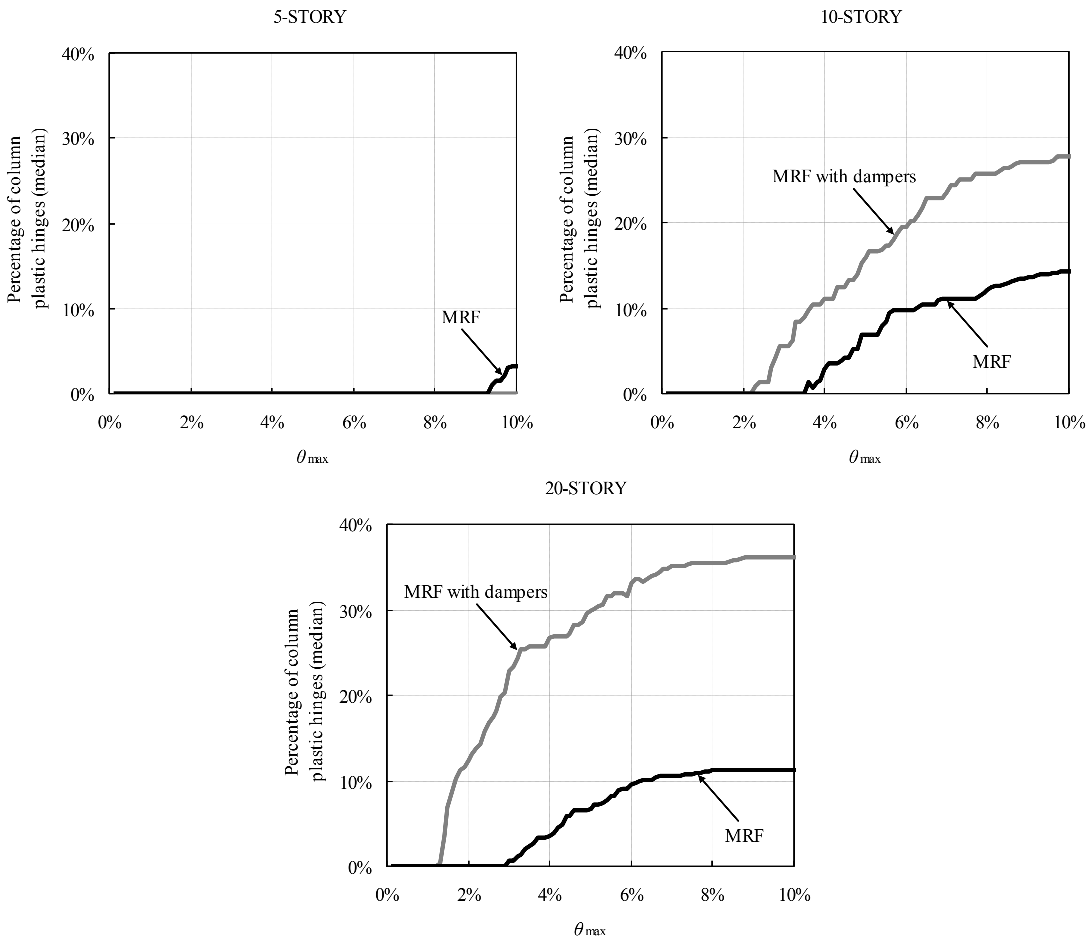

Fig.5. Percentage of column plastic hinges in the steel MRFs with and without viscous dampers

Fig. 5 shows the median value of the percentage of the column plastic hinges against θ_{max} for the steel MRFs with and without viscous dampers. The 5-story steel MRF with viscous dampers shows no plastic hinges in columns, while the steel MRF shows a very low percentage of plastic hinges at drifts higher than 9%. These results indicate that the capacity design of columns for the 5-story steel MRF with viscous dampers is adequate. The 10-story and the 20-story steel MRFs with viscous dampers have higher percentages of column plastic hinges in comparison to those of the steel MRFs. More specifically, the percentage of the column plastic hinges at 10% drift (i.e. a drift value associated with the near-collapse behavior of a steel MRF) is 14% for the 10-story steel MRF, 28% for the 10-story steel MRF with viscous dampers, 11% for the 20-story steel MRF, and 37% for the 20-story steel MRF with viscous dampers. These results indicate that a stricter capacity design rule for steel MRFs with viscous dampers could possibly result in improvement of their collapse resistance (see Section 6 where this issue is investigated). Moreover, it is important to note that none of the steel MRFs with viscous dampers experiences column plastic hinges under the DBE and MCE seismic intensities. This observation indicates that the capacity design rules for columns of steel MRFs with viscous dampers are adequate in terms of reparability.

6. Performance of enhanced steel MRFs with viscous dampers

To explore whether stricter capacity design rules could enhance the seismic performance of steel MRFs with viscous dampers, the interior columns of the 10-story and 20-story steel MRFs with viscous dampers were re-designed. An iterative procedure is followed for that purpose, where the sections of the interior columns experiencing plastic hinges are increased until the median value of the number of column plastic hinges in the re-designed MRF with viscous dampers (referred to as enhanced MRF with viscous dampers) approximately approaches the corresponding values of the MRF without viscous dampers. Design details of the enhanced MRFs with viscous dampers are given in Table 2 and Fig. 6.

Fig. 7 shows the percentage of column plastic hinges against θ_{max} for the MRFs, the MRFs with viscous dampers, and the enhanced MRFs with viscous dampers. The results show that the both the 10-story and 20-story enhanced MRFs with viscous dampers experience significantly less column plastic hinges than those of the corresponding MRF with viscous dampers. In addition, their behavior in terms of column plastic hinges at different θ_{max} levels is similar to that of the corresponding MRFs.

Fig. 8 shows the locations of the column plastic hinges at θ_{max} equal to 3% for the 10-storey MRF, MRF with viscous dampers, and enhanced MRF with viscous dampers subjected to the same ground motion. The MRF experiences column plastic hinges only at its base. The MRF with dampers experiences column plastic hinges at the base and 16 additional plastic hinges in the first six stories. The enhanced MRF with viscous dampers experiences plastic hinges at the base and only one plastic hinge in the fourth story.

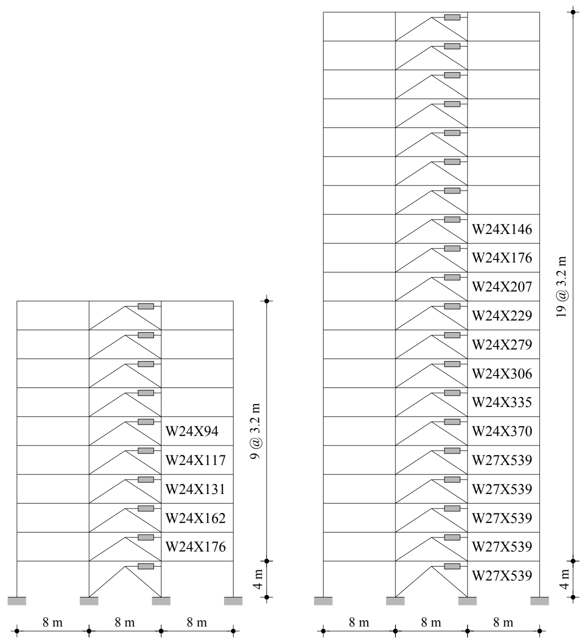

Fig. 6. Elevation view and design details of the enhanced steel MRFs with viscous dampers (beams and columns are the same with those of the corresponding MRFs apart from the interior columns indicated in this figure)

Table 2. Design details of the enhanced MRFs with viscous dampers

MRF with dampers	T(sec)	ξ_{tot} (%)	$\theta_{max,DBE}$ (%)
10-storey	2.39	20	0.86
20-storey	3.56	20	0.55

Fig. 9 shows the collapse fragility curves of the 20-story MRF, MRF with viscous dampers, and enhanced MRF with viscous dampers. The results show that the refined capacity design of columns results in higher collapse resistance. In particular, the seismic intensity at 50% probability of collapse of the enhanced MRF with viscous dampers is 10% higher than that of the MRF with viscous dampers. The refined capacity design of columns did not result in notable changes of the fragility curve of the 10-story MRF with viscous dampers. Considering the superior collapse resistance of the steel MRFs with viscous dampers in comparison to steel MRFs, the results indicate that a refined capacity design rule for high-performance steel MRFs with viscous dampers with the goal of achieving a plastic collapse mechanism similar to that of steel MRFs without dampers is not justified. However, this statement may not be true for light-weight steel MRFs with viscous dampers designed to have similar performance with conventional steel MRFs and further investigation are needed in this direction.

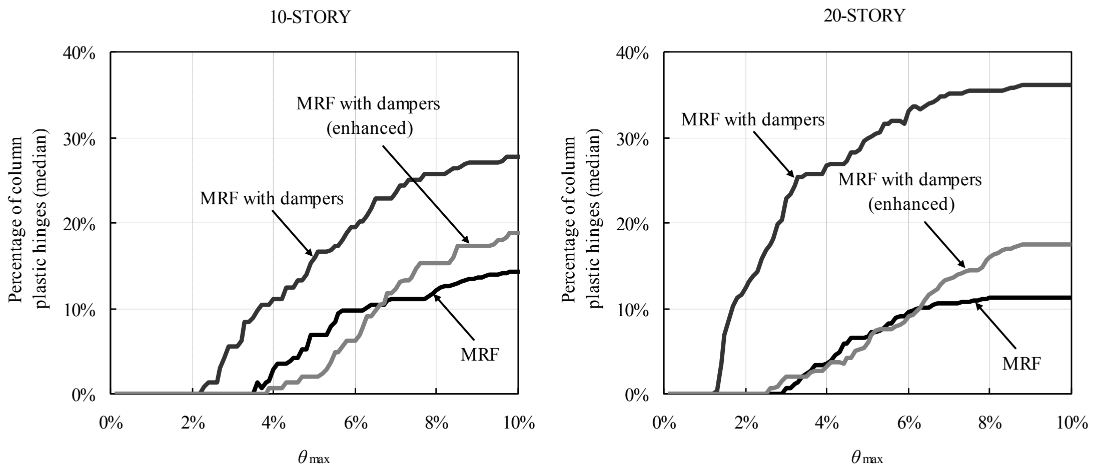

Fig. 7. Percentage of column plastic hinges in the steel MRFs, the steel MRFs with viscous dampers, and the enhanced steel MRFs with viscous dampers

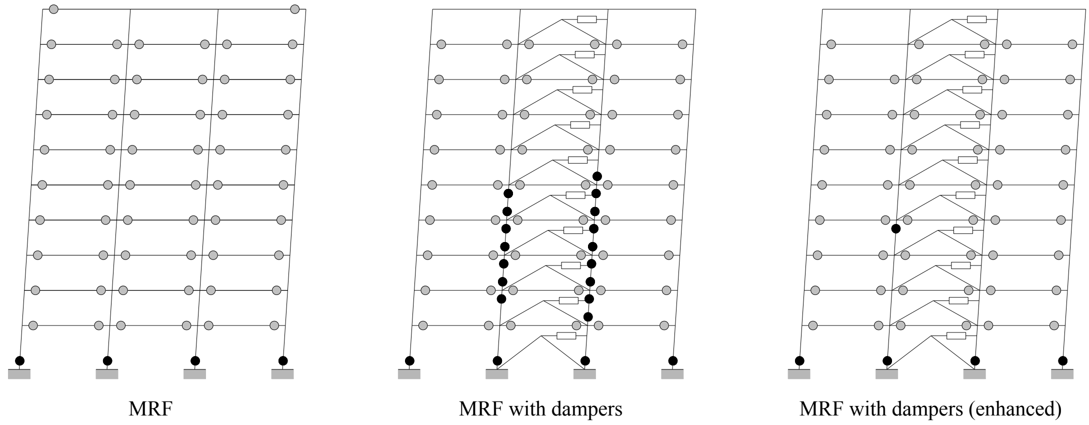

Fig. 8. Locations of plastic hinges at $\theta_{\max}$ equal to 3% for the 10-storey MRF, MRF with viscous dampers, and enhanced MRF with viscous dampers

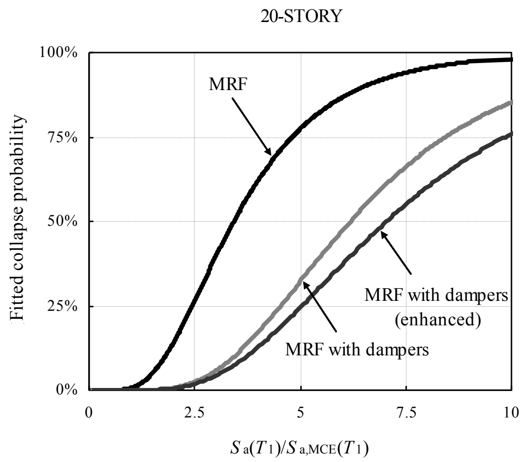

Fig. 9. Collapse fragilities of the 20-story MRF, MRF with viscous dampers, and enhanced MRF with viscous dampers

7. Conclusions

Three alternatives of a prototype building having five, 10 and 20 stories were designed according to Eurocode 8 using either steel MRFs or steel MRFs with viscous dampers. The steel MRFs with viscous dampers were designed to have significantly better drift performance than that of the steel MRFs. Incremental dynamic analysis under 44 ground motions were conducted for all the frames and their collapse resistances as well as their plastic mechanisms (with a focus on column plastic hinges) under different drift levels were evaluated and compared. The steel MRFs with viscous dampers were also iteratively re-designed to achieve a plastic mechanism that is approximately similar to that of the steel MRFs. The performance of these redesigned MRFs with viscous dampers was also assessed with incremental dynamic analyses and the results were quantitatively and qualitatively evaluated to justify whether or not stricter capacity design rules for the columns of high-performance steel MRFs with viscous damper are needed. Based on the results presented in the paper, the following conclusions are drawn:

1. The 5-story steel MRF with viscous dampers is not prone to column plastic hinging and develops a global plastic mechanism identical to that of a steel MRF.
2. The 10-story and the 20-story steel MRFs with viscous dampers have higher percentages of column plastic hinges in comparison to those of the steel MRFs.
3. None of the steel MRFs with viscous dampers experiences column plastic hinges under the design and maximum considered earthquakes, and therefore, the capacity design rules for columns are adequate in terms of reparability.
4. Stricter capacity design of columns results in tall steel MRFs with viscous dampers having similar plastic collapse mechanisms with those of steel MRFs.
5. The benefit of stricter capacity design of columns in terms of the collapse resistance of high-performance steel MRFs with viscous dampers is modest, e.g. increases of only up to 10% were found.
6. Overall, the need for stricter capacity design rules for columns of high-performance steel MRFs with viscous dampers is not justified neither from a collapse resistance or a reparability perspective.
7. It is important to highlight that the conclusions of this paper may not be valid for steel MRFs with viscous dampers that are designed to have drift performance similar to that of steel MRFs.

REFERENCES

1. Clifton C, Bruneau M, MacRae G, Leon R, Fussel R. Steel structures damage from the Christchurch earthquake series of 2010 and 2011. Bulletin of the New Zealand Society for Earthquake Engineering 2011; 44(4):297-318.

2. Christopoulos C, Filiatrault A. Principles of passive supplemental damping and isolation. IUSS press 2006.
3. Symans M, Charney F, Whittaker A, Constantinou M, Kircher C, Johnson M, McNamara R. Energy Dissipation Systems for Seismic Applications: Current Practice and Recent Developments. Journal of Structural Engineering, ASCE 2008; 134(1):3-21.
4. Seleemah A, Constantinou MC. Investigation of seismic response of buildings with linear and nonlinear fluid viscous dampers. Report No. NCEER 97–0004, National Center for Earthquake Engineering Research, State Univ. of New York at Buffalo, Buffalo, N.Y., 1997.
5. Karavasilis TL, Seo C-Y. Seismic structural and non-structural performance evaluation of highly damped self-centering and conventional systems. Engineering Structures 2011; 33: 2248-2258.
6. Hatzigeorgiou GD, Papagiannopoulos GA. Inelastic velocity ratio. Earthquake Engineering and Structural Dynamics 2012; 41(14):2025-2041.
7. Tubaldi E, Ragni L, Dall'Asta A. Probabilistic seismic response of linear systems equipped with nonlinear viscous dampers. Earthquake Engineering and Structural Dynamics 2015; 44(1):101-120.
8. Dall'Asta A, Tubaldi E, Ragni L. Influence of the nonlinear behavior of viscous dampers on the seismic demand hazard of building frames. Earthquake Engineering and Structural Dynamics 2016; 45(1):149-169.
9. Pavlou E, Constantinou MC. Response of nonstructural components in structures with damping systems. Journal of Structural Engineering, ASCE 2006; 132(7):1108–1117.
10. Wanitkorkul A, Filiatrault A. Influence of passive supplemental damping systems on structural and nonstructural seismic fragilities of a steel building. Engineering Structures 2008; 30(3):675–682.
11. Kasai K, Motoyui S, Ozaki H, Ishii M, Ito H, Kajiwara K, Hikino T. Full-scale tests of passively-controlled 5-story steel building using E-Defense shake table. Part 1: Test concept, method, and building specimen. Proceedings of the sixth international conference on behaviour of steel structures in seismic areas. Philadelphia, PA, 2009.

12. Dong B, Sause R, Ricles JM. Seismic response and performance of a steel MRF building with nonlinear viscous dampers under DBE and MCE. . Journal of Structural Engineering, ASCE 2016; in press
13. ASCE 7-10. Minimum Design Loads for Buildings and Other Structures. American Society of Civil Engineers, Reston, Virginia, 2010.
14. Ramirez OM, Constantinou MC, Gomez JD, Whittaker AS, Chrysostomou CZ. Evaluation of simplified methods of analysis of yielding structures with damping systems. Earthquake Spectra 2002; 18(3):501-530.
15. Ramirez OM, Constantinou MC, Whittaker AS, Kircher CA, Chrysostomou CZ. Elastic and inelastic seismic response of buildings with damping systems. Earthquake Spectra 2002; 18(3):531-547.
16. Guo, JWW., Christopoulos, C. (2013), "Performance Spectra-based Design Method for the Seismic Design of Structures Equipped with Passive Supplemental Damping Systems", Earthquake Engineering and Structural Dynamics, 42(6), 935–952
17. Seo C-Y, Karavasilis TL, Ricles JM, Sause R. Seismic performance and probabilistic collapse resistance of steel moment resisting frames with fluid viscous dampers. Earthquake Engineering and Structural Dynamics 2014; 43(14):2135-2154.
18. Balut N, Gioncu V. Suggestion for an improved 'dog-bone' solution. 4th International Conference STESSA, Naples, Italy; 2003.
19. Baiguera M, Vasdravellis G, Karavasilis TL. Dual seismic-resistant steel frame with high post-yield stiffness energy-dissipative braces for residual drift reduction. Journal of Constructional Steel Research 2016; 122:198-212.
20. Vamvatsikos D, Cornell C. A. Incremental dynamic analysis. Earthquake Engineering and Structural Dynamics 2002; 31(3):491–514.
21. Eurocode 8. Design of Structures for Earthquake Resistance. 2004
22. Lin W, Chopra A. Earthquake Response of Elastic Single-Degree-of-Freedom Systems with Nonlinear Viscoelastic Dampers. Journal of Engineering Mechanics 2003; 129(6):597-606.
23. Whittaker A, Constantinou M, Ramirez O, Johnson M, Chrysostomou C. Equivalent Lateral Force and Modal Analysis Procedures of the 2000 NEHRP Provisions for Buildings with Damping Systems. Earthquake Spectra 2003; 19(4):959–980.

24. Whittle J, Williams MS, Karavasilis TL, Blakeborough A. A comparison of viscous damper placement methods for improving seismic building design. Journal of Earthquake Engineering 2012; 16:540-560.
25. OpenSees. Open system for earthquake engineering simulation. Pacific Earthquake Engineering Research Center, University of California at Berkeley, Berkeley, CA, 2013.
26. Newell J, Uang C-M. Cyclic behaviour of steel columns with combined high axial load and drift demand. Report No. SSRP-06/22. Department of Structural Engineering, University of California, San Diego, La Jolla, 2006.
27. Lignos DG, Krawinkler H. Deterioration modeling of steel components in support of collapse prediction of steel moment frames under earthquake loading. Journal of Structural Engineering, ASCE 2011; 137(11):1291–1302.
28. Krawinkler H. Shear design of steel frame joints. Engineering Journal AISC 1978; 15(2):82-91.
29. Miyamoto HK, Gilani AS, Wada A, Ariyaratana C. Limit states and failure mechanisms of viscous dampers and the implications for large earthquakes. Earthquake Engineering and Structural Dynamics 2010; 39(11):1279–1297.
30. Taylor Devices inc. Accessible to http://www.taylordevices.com.
31. FEMA. Quantification of building seismic performance factors, FEMA P695 Report. Federal Emergency Management Agency, 2009.
32. Freddi F, Padgett JE, Dall'Asta A. Probabilistic seismic demand modeling of local level response parameters of an RC frame. Bulletin of Earthquake Engineering 2016; DOI 10.1007/s10518-016-9948-x
33. Freddi F, Tubaldi E, Ragni L, Dall'Asta A. Probabilistic performance assessment of low-ductility reinforced concrete frames retrofitted with dissipative braces. Earthquake Engineering and Structural Dynamics 2013; 42(7): 993-1011.