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Seismic force reduction factor for steel moment resisting frames with supplemental viscous dampers

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Abstract. Damping is one of the parameters that control the performance of structures when they are subjected to seismic, wind, blast or other transient shock and vibration disturbances. By adding supplemental viscous dampers, the energy input from a transient deformation is absorbed, not only by the structure itself, but also by the supplemental dampers. The aim of this study is to evaluate the values of both damping and ductility reduction factors for steel moment resisting frames with supplemental linear viscous dampers. Two-dimensional finite element models have been established for a range of low to mid rise buildings with different parameters: number of floors; number of bays; and number of dampers with different supplemental damping ratios (from 5% to 30%). A parametric study has been performed using time history analyses and a well-documented research method (N2-method). In addition, an equation has been proposed for each reduction factor based on regression analysis for the obtained results. The results of the Time history analyses are compared with those of a modified N2-method. Moreover, a comparison with values specified in the European code EC8 and the Egyptian code ECP-201 has been performed.

Keywords: seismic force reduction factor, steel frame; viscous damper; damping; ductility

1. Introduction

This paper discusses the seismic force reduction factor for steel moment resisting frames with supplemental linear viscous dampers. Under favorable conditions, energy dissipation devices reduce drift of the structure by a factor of about two to three - if no stiffness is added - and by larger factors if the devices also added stiffness to the structure (McNamara *et al*. 2000). For most applications, energy dissipation provides an alternative approach to conventional stiffening and strengthening schemes, and is expected to achieve comparable performance levels.

FEMA 273 (1997) shows that energy dissipation devices reduce the internal forces in the structure that is responding elastically, but would not be expected to reduce the forces in structure that is responding beyond yield. In general, these devices are expected to be good candidates for projects that have target building performance level of life safety or perhaps immediate occupancy, but would be expected to have only limited applicability to projects with target building

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performance level of collapse prevention.

The velocity-dependent device represents a technology which was developed for military applications and has been shown to be effective in applications of seismic hazard mitigation. The civil engineering profession has readily accepted this technology, resulting in a number of applications as discussed by McNamara *et al*. (2000), Sophocleous (2001), Taylor and Constantinou (2007), Parulekar (2012), Basu *et al*. (2014), Landi (2014a), Landi (2014b), and Rama (2014). The response of a fluid viscous damper is considered to be out-of-phase with the building seismic response, which is considered a desirable feature in seismic designs.

Design methods for supplemental dampers for civil structures often leave unanswered the following question: starting from the structural inherent damping by the adoption of supplemental dampers, which level of damping can be achieved?

Gluck *et al*. (1996), Takewaki (1997), Seleemah and Constantinou (1997), Takewaki (2000), Sadek *et al*. (2000), Uriz and Whittaker (2001), Cimellaro and Retamales (2007), Takewaki (2009), and Occhiuzzi (2009), reported an implicit suggestion of optimal first modal damping ratio (inherent plus supplemental) in the range of 20% to 25%. Further observations have been reported: (1) higher values of the damping ratio associated with the fundamental period of vibration seem to trade off a minor reduction of inter-storey drifts with a significant increase of absolute accelerations; (2) higher vibration modes play a minor role in the design of supplemental damping systems; and (3) with a given fixed damping cost, expressed as the sum of the supplemental dampers' constants, a target value of the first modal damping ratio in the range of 20% to 25% can be obtained with a limited number of devices placed at the lower floors.

The damping reduction factor (R_B) is used in a few building codes (such as the European code EC8 and the American code ASCE-7) in order to estimate the elastic response spectrum with high damping ratios based on its 5% damped counterpart. Several expressions of the damping reduction factor have been proposed. All of them are based on SDOF systems subjected to earthquake excitation. Results derived by Newmark and Hall (1982) were implemented in the ATC-40 and FEMA 273 for the displacement-based evaluation of existing buildings and in the UBC 97, NEHRP-97 and IBC 2000 for the design of buildings with passive energy dissipation systems. Meanwhile, results obtained by Ashour (1987) were adopted in the UBC 94 and NEHRP-94. Moreover, results from Ramirez *et al*. (2000) and (2002) were used in the NEHRP-2000. Lin *et al*. (2005) proposed a period dependent formula for the damping reduction factor of SDOF systems with damping ratio up to 50% and period of vibration up to 10 Sec. Hatzigeorgiou (2010) proposed a simple, unique and accurate empirical expression to estimate the damping reduction factor for SDOF systems with damping ratios up to 50%. Moreover, direct assessment methods and frameworks have been reported by Palermo *et al*. (2013), Landi (2014a), and Landi (2014b) for nonlinear structures equipped with nonlinear fluid viscous dampers.

The aim of this study is to evaluate the damping reduction factor and the ductility reduction factor for steel moment resisting frames with supplemental viscous dampers.

2. Method of study

This section describes the parametric study including the finite element modeling and the methodology used to derive the damping and ductility reduction factors for steel moment resisting frames with and without supplemental viscous dampers.

2.1 Finite element modeling

The investigated models of steel moment resisting frames (SMRF) are assumed for residential purpose and are divided into four groups. Each group has been designed according to different damping ratios (inherent plus supplemental). Fig. 1 shows four groups of structure models. The models of group-1 (SMRF 2/7) are 7-storey frames with two bays. The models of group-2 (SMRF 2/12) are 12-storey frames with two bays. The models of group-3 (SMRF 3/7) are 7-storey frames with three bays. The models of group-4 (SMRF 3/12) are 12-storey frames with three bays. For all models: the bay width equals 6m, the horizontal spacing between frames equals 4m, and the storey height equals 4m for the ground floor, and 3m for the typical floors.

Groups 1-4 are used to evaluate the force reduction factor due to both ductility and damping. Each group consists of three structure models, namely: Str-1, Str-2 and Str-3; meanwhile, six different damping ratios (ζ%) have been considered including the inherent damping: 5%, 10%, 15%, 20%, 25%, and 30%. The model Str-1 is used to study the 5% and 10% damping; Str-2 is used to study the 15% and 20% damping; and Str-3 is used to study the 25% and 30% damping.

SAP2000 (2014), a well known finite element analysis software, has been employed in this study to perform the nonlinear static and dynamic analyses. Planar prismatic Frame elements with four degrees of freedom at each node are used to model the beams and the beam-columns of different models. The geometric nonlinearity has not been considered in this study since the structures have intermediate range of periods (Borzi and Elnashai, 2000).

The steel grade DIN-17100 has been employed for all structural elements with a yield stress (F_y) equals 2.4 t/cm^2 (240MPa), ultimate strength (F_u) equals 3.6 t/cm^2 (360MPa), modulus of elasticity (E_s) equals 2100 t/cm^2 (210000MPa), strain hardening parameter (υ) equals 0.01, and specific weight (γ) equals 7.85 t/m^3. Material nonlinearity has been considered in the analyses through employment of plastic fiber hinges, according to FEMA-356 (1997), at both ends of beams and columns.

The dead and live loads have been considered equal 5kN/m2 and 2.5kN/m2, respectively. The design has been performed in accordance with the Egyptian code for loads (ECP-201), which has a response spectrum equivalent to that of the European code EC8, and the Egyptian code of practice for steel construction, ECP-205 ASD. Compact and non-compact steel profiles are used for both beams and columns. Tables 1-4 show the steel profiles and damping properties of structure models in Groups 1-4, respectively. For the listed steel profiles, I-sections have been adopted where: h_w is the web height, b_f is the flange width, t_w is the web thickness, t_f is the flange thickness. The damping coefficient (C) of the supplemental viscous dampers has been listed in Tables 1-4, where it has been obtained using the modified formula proposed by Hwang *et al*. (2008) to account for combined shear and flexural deformations. For group-1, the supplemental damping has been provided through incorporating two viscous dampers at the first storey. For group-2, the supplemental damping has been provided through incorporating four viscous dampers at the first and the second stories. For group-3, the supplemental damping has been provided through incorporating three viscous dampers at the first storey. For group-4, the supplemental damping has been provided through incorporating six viscous dampers at the first and the second stories.

Figs. 1a-1d show the arrangement of supplemental damping devices in models of groups 1-4, respectively.

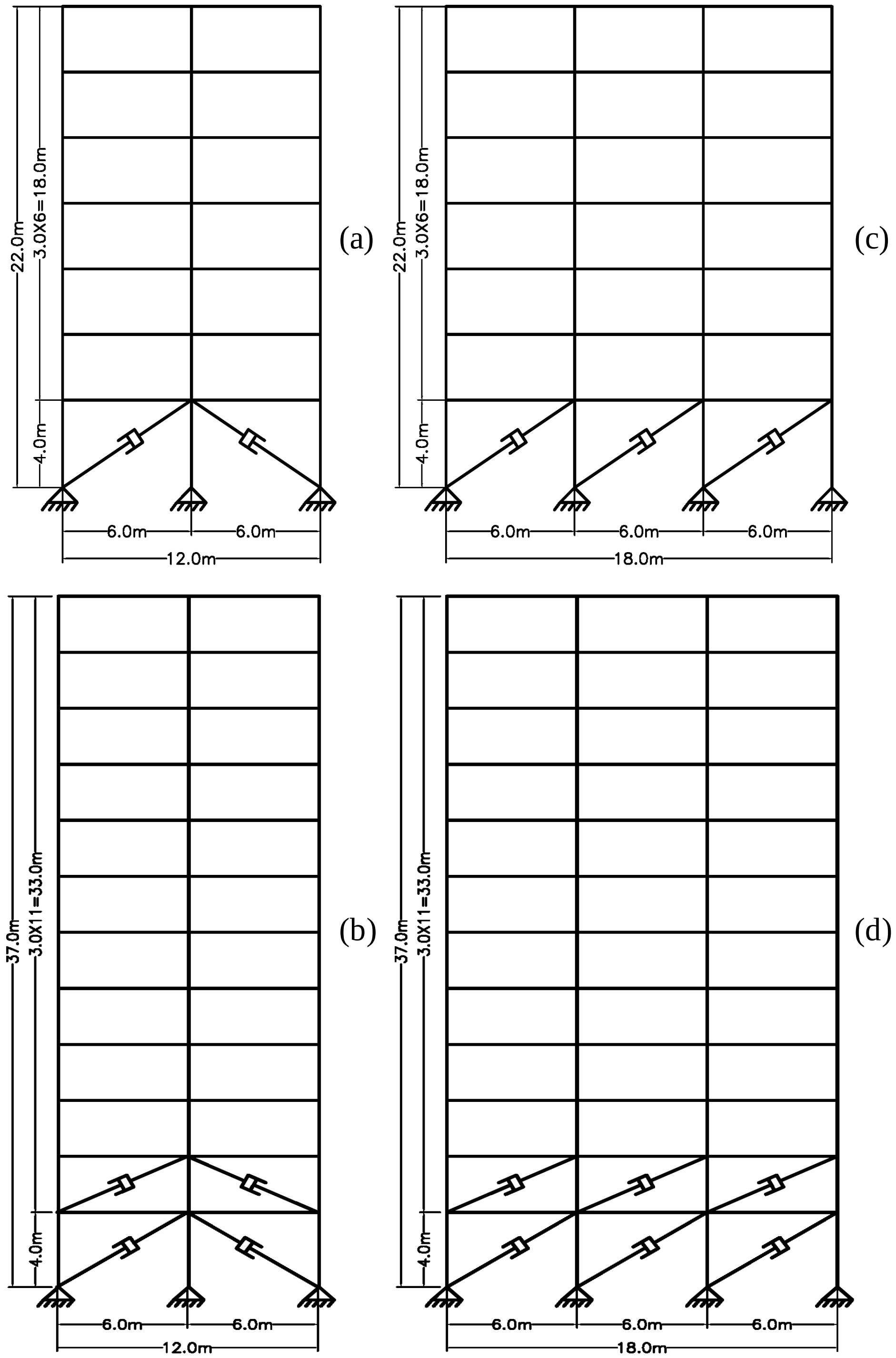


Fig. 1 Models of Study: (a) Group-1 SMRF-2/7 with 2 dampers, (b) Group-2 SMRF-2/12 with 4 dampers, (c) Group-3 SMRF-3/7 with 3 dampers, and (d) Group-4 SMRF-3/12 with 6 dampers

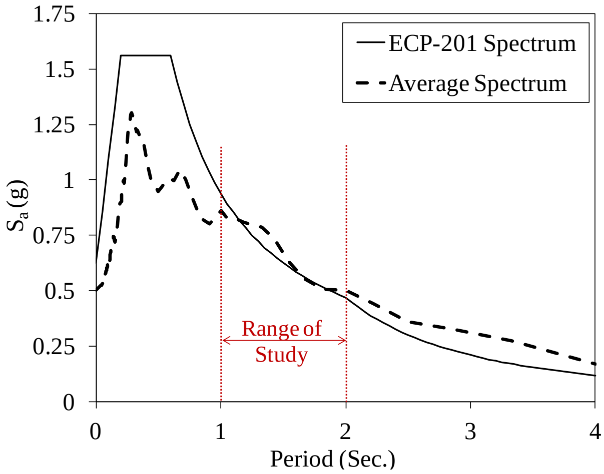


Fig. 2 Response Spectra for the Average of Seven Earthquake Records Compared with the Egyptian Code Spectra Type-I (ECP-201, 2012), PGA=0.5g

Table 1 Steel Profiles and Damping Properties of Group-1 SMRF 2/7

Model	Steel Profiles (h_w x b_f / t_w x t_f)				Damping	
	Floor	Exterior Columns	Interior Columns	Beams	ζ (%)	C (kN.Sec./m)
Str-1	1-3	500×250/10×16	550×250/10×18	500×250/10×16	5	No dampers
	4-7	400×250/8×14	400×250/8×14	400×250/8×14	10	200
Str-2	1-3	500×250/10×12	550×250/10×14	500×250/10×12	15	395
	4-7	400×200/6×10	400×200/6×10	400×200/6×10	20	590
Str-3	1-3	400×250/8×12	500×250/8×12	400×250/8×12	25	595
	4-7	400×200/6×8	400×200/6×8	400×200/6×8	30	745

Table 2 Steel Profiles and Damping Properties of Group-2 SMRF 2/12

Model	Steel Profiles (h_w x b_f / t_w x t_f)				Damping	
	Floor	Exterior Columns	Interior Columns	Beams	ζ (%)	C (kN.Sec./m)
Str-1	1-4	700×330/14×20	700×330/14×24	600×330/10×12	5	No dampers
	5-8	600×250/10×14	600×250/10×14	500×250/6×12		
	9-12	500×200/8×12	500×200/8×12	400×200/6×10	10	1410
Str-2	1-4	700×330/10×18	700×330/10×18	500×250/8×14	15	2270
	5-8	600×250/8×14	600×250/8×14	400×250/8×10		
	9-12	400×200/6×12	400×200/6×12	400×200/5×10	20	3400
Str-3	1-4	700×330/10×14	700×330/10×14	500×250/8×10	25	3750
	5-8	600×250/8×12	600×250/8×12	400×250/6×10		
	9-12	400×200/6×10	400×200/6×10	400×200/5×8	30	4700

Table 3 Steel Profiles and Damping Properties of Group-3 SMRF 3/7

Model	Steel Profiles (h_w x b_f / t_w x t_f)				Damping	
	Floor	Exterior Columns	Interior Columns	Beams	ζ (%)	C (kN.Sec./m)
Str-1	1-3	500×250/10×16	550×250/12×20	400×250/10×14	5	No dampers
	4-7	400×250/8×14	400×250/8×14	400×250/6×10	10	221
Str-2	1-3	500×250/8×14	550×250/10×18	400×250/8×12	15	424
	4-7	400×250/6×10	400×250/6×10	400×250/5×8	20	636
Str-3	1-3	500×250/8×10	550×250/10×14	400×250/6×8	25	735
	4-7	400×250/6×8	400×250/6×8	400×200/5×6	30	919

Table 4 Steel Profiles and Damping Properties of Group-4 SMRF 3/12

Model	Steel Profiles (h_w x b_f / t_w x t_f)				Damping	
	Floor	Exterior Columns	Interior Columns	Beams	ζ (%)	C (kN.Sec./m)
Str-1	1-4	700×330/14×20	700×330/14×24	600×330/10×12	5	No dampers
	5-8	600×250/12×14	600×250/12×14	500×250/6×10		
	9-12	500×200/10×12	500×200/10×12	400×200/6×10	10	1070
Str-2	1-4	700×330/12×18	700×330/14×20	600×330/8×12	15	2145
	5-8	600×250/10×14	600×250/10×14	500×200/6×10		
	9-12	500×200/8×12	500×200/8×12	400×200/6×8	20	3220
Str-3	1-4	700×330/10×14	700×330/10×18	600×200/8×10	25	3570
	5-8	600×250/8×14	600×250/8×14	400×200/5×10		
	9-12	500×200/6×10	500×200/6×10	400×200/5×8	30	4460

Table 5 The Selected Seven Earthquake Records

No.	Earthquake	Date	Magnitude	PGA(g)	Duration (Sec.)	Station/ Component
1	Izmit	08/17/1999	7.40	0.31	27.17	SKR/E-W
2	Dinar	10/01/1995	6.40	0.32	27.95	Dinar/E-W
3	Banja Luka	08/13/1981	5.70	0.36	28.30	Banja Luka-4/N-S
4	Faial	07/09/1998	6.10	0.30	135.78	HORTA/ E-W
5	Northridge	01/17/1994	6.70	0.57	59.98	Castaic Old Ridge (24278)/ Chn-1: 90DEG
6	Christchurch	02/21/2011	6.30	0.71	67.435	Christchurch Resthaven (REHS) /N10W
7	Off S Niigata Prefecture	07/16/2007	6.30	0.68	75.48	Kashiwazaki (NIG018)/N-S

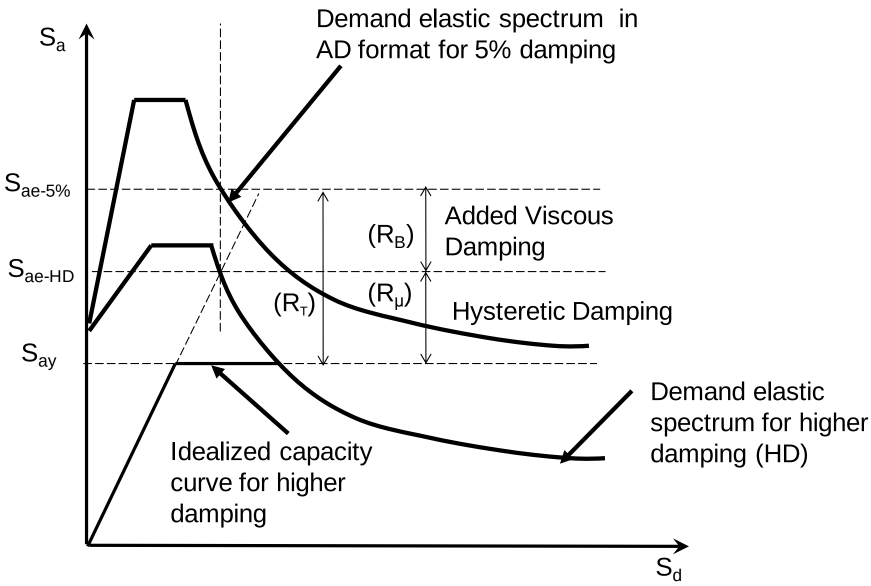


Fig. 3 Elastic Demand Spectra versus Capacity Diagram

Seven earthquake records have been selected from a strong ground motion database (Iervolino *et al*., 2014) and listed in Table 5, based on the consistency of their average spectrum with ECP-201 spectrum. This consistency is well satisfied in the range of periods of the selected steel frames (T=1.0 ~ 2.0 Sec.), as shown in Fig. 2.

2.2 Ductility and damping reduction factors

The ductility capacity has been evaluated using the N2-method proposed by Fajfar and Fischinger (1989). However, the method has been modified using Hatzigeorgiou (2010) formula to account for higher damping ratios beyond the inherent damping. The modification has been performed through employing damping reduction factor (R_B) to obtain the spectral acceleration (S_a) and displacement (S_d) responses to plot the demand elastic spectrum for higher damping (HD) ratios, as shown in Fig. 3. The ductility reduction factor (R_μ) has been calculated using both Newmark and Hall (1982) formula and Park and Ang (1985) formula. The damping reduction factor (R_B) has been calculated as the ratio between the elastic 5% damped base shear ($S_{ae\text{-}5\%}$) and the elastic higher damped base shear ($S_{ae\text{-}HD}$): $R_B = (S_{ae\text{-}5\%}) / (S_{ae\text{-}HD})$.

The values of R_μ and R_B resulted from the modified N2-method have been validated against nonlinear time history analysis results using the average of the selected seven earthquake records. The details behind step-by-step calculations of R_μ and R_B can be found in the reference (Ramy A Diab, 2013). It is worth noting that the total force reduction factor (R) is taken equal to 5 for ordinary moment resisting steel frames, according to ECP-201. Meanwhile, Uang (1991) recommended an over-strength factor (Rs) equal to 2.3, and a redundancy factor (R_R) equal to 1.4. Hence, R_μ should equal to 1.55 according to ECP-201.

3. Results and discussion

3.1 Numerical model verification

The numerical modeling of steel moment resisting frame in presence of velocity dependent dampers has been verified through comparison with the numerical model of Hwang *et al*. (2008). It is a twenty storey model with forty viscous dampers, each damper has a damping coefficient (C) equals 8429 kN Sec./m. The model profiles and storey weight have been listed in Table 6. The verification model, developed in this study, has a first modal period and damping ratio of 2.02 Sec. and 8.64%, respectively, against 1.919 Sec. and 9% of Hwang *et al*. model. Moreover, Table 6 shows verification for the obtained horizontal first modal shape vector (Φ_{h1}) against Hwang *et al*. model results. The results are in good fitting where a maximum error of 6.1% is resulted.

3.2 Ductility and damping reduction factors

Fig. 4 shows the demand curve in the well known acceleration-displacement (A-D) format: spectral acceleration (S_a) versus spectral displacement (S_d). On the same figure, the capacity curve has been plotted for the equivalent SDOF system. The figure shows four sample snapshots for the four groups 1-4 comparing the response of different models with different: number of floors, number of bays, and supplemental damping ratio. Figs. 5 and 6 show the damping reduction factor R_B, the ductility reduction factor Rμ, the combined reduction factor ($R_B \times R\mu$), and the damped inelastic base shear for structure models of groups 1-4, respectively. The figures show the results for: the modified N2-Method, nonlinear time history analysis, and European code EC8 equation (same as of ECP-201) at different damping ratios up to 30%.

For the damping reduction factors (R_B), it is evident that the modified N2-method and the time history analysis exhibit close values; whereas, the EC8 equation yields underestimated values. Based on a linear regression analysis for the obtained results, an equation has been proposed for R_B as a function of the target first modal damping ratio ζ: [$R_B = 0.86 + 4.37\,\zeta$]. This equation has a coefficient of determination (R-squared) of 97%.

For the ductility reduction factors (Rμ), the modified N2-method and the time history analysis do not exhibit close results. This is attributed to the fact that Rμ is calculated based on Park and Ang (1985) formula where the hysteretic ductility (μ_h) increases the structure ductility (μ) by ratio of 30% and 40% for low-rise and mid-rise steel frames, respectively, as being evident in Table 7 in comparison with Newmark and Hall (1982). The EC8 code equation still yields underestimated ductility reduction factor. Based on a linear regression analysis for the obtained results, an equation has been proposed for Rμ as a function of the target first modal damping ratio ζ and R_B: [$R_{\mu\text{-}\zeta} = R_{\mu\text{-}5\%}\,(1.9 + 5\,\zeta - 1.16\,R_B)$]. This equation has a regression (R-squared) of 73% which is attributed to the unstable pattern of the ductility reduction factor that has been observed for damping ratios greater than 20%, as being evident in Fig. 7b. If the equation has been limited to damping ratios less than or equal to 20%, the R-squared is obtained as 90% and the equation can be modified as: [$R_{\mu\text{-}\zeta} = R_{\mu\text{-}5\%}\,(1.9 - 12\,\zeta + 1.4\,R_B)$], where $\zeta \leq 20\%$. The observed unstable patter for Rμ has been also reported in the literature (Sadek *et al*. 2000; Takewaki 2009) for damping ratios greater than 20%.

Fig. 7 illustrates a sample comparison using Group-1 models where the equation results have been plotted against the results of the modified N2-Method and the time history analysis.

Table 6 Numerical model verification results against Hwang *et al*. Model

Storey	Columns	Beams	Storey Weight (kN)	Φ_{h1} *This Study*	Φ_{h1} *Hwang et al. 2008*	Errors
1				0.0188	0.0177	6.10%
2	Box 1300×25	W 800×400 /25×25	1082	0.0510	0.0492	3.56%
3				0.0912	0.0892	2.20%
4				0.1368	0.1352	1.17%
5				0.1903	0.189	0.68%
6				0.2498	0.2494	0.17%
7				0.3118	0.3122	-0.11%
8	Box 1100×25	W 700×350 /25×25	1014	0.3744	0.3753	-0.23%
9				0.4364	0.4381	-0.38%
10				0.4973	0.4996	-0.47%
11				0.5568	0.5594	-0.47%
12				0.6155	0.6184	-0.46%
13				0.6777	0.6804	-0.40%
14				0.7405	0.7433	-0.38%
15				0.7999	0.8026	-0.33%
16	Box 900×25	W 600x300 /25×25	947	0.8537	0.8559	-0.25%
17				0.9008	0.9026	-0.20%
18				0.9405	0.9418	-0.14%
19				0.9731	0.9739	-0.08%
20				1	1	0.00%

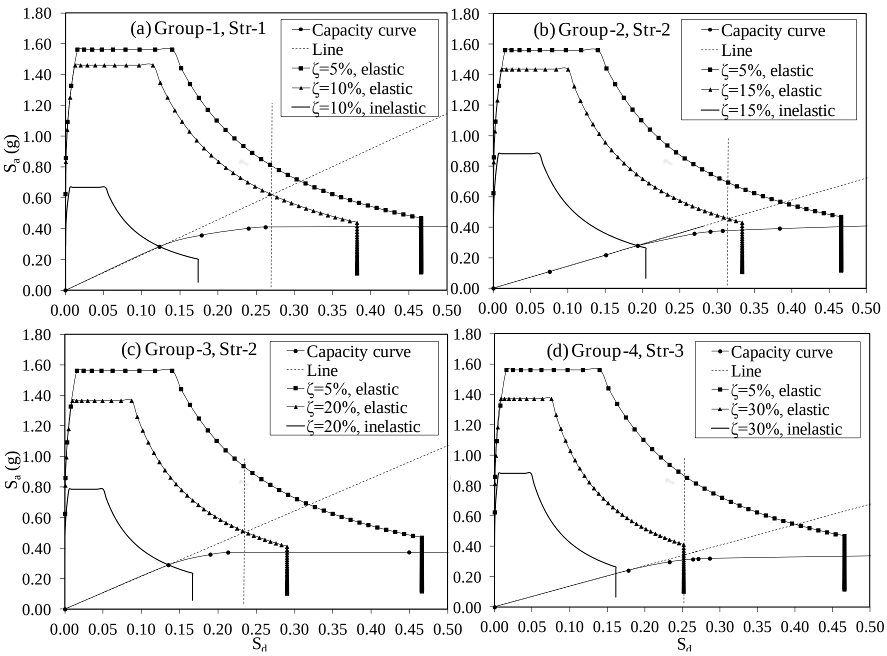
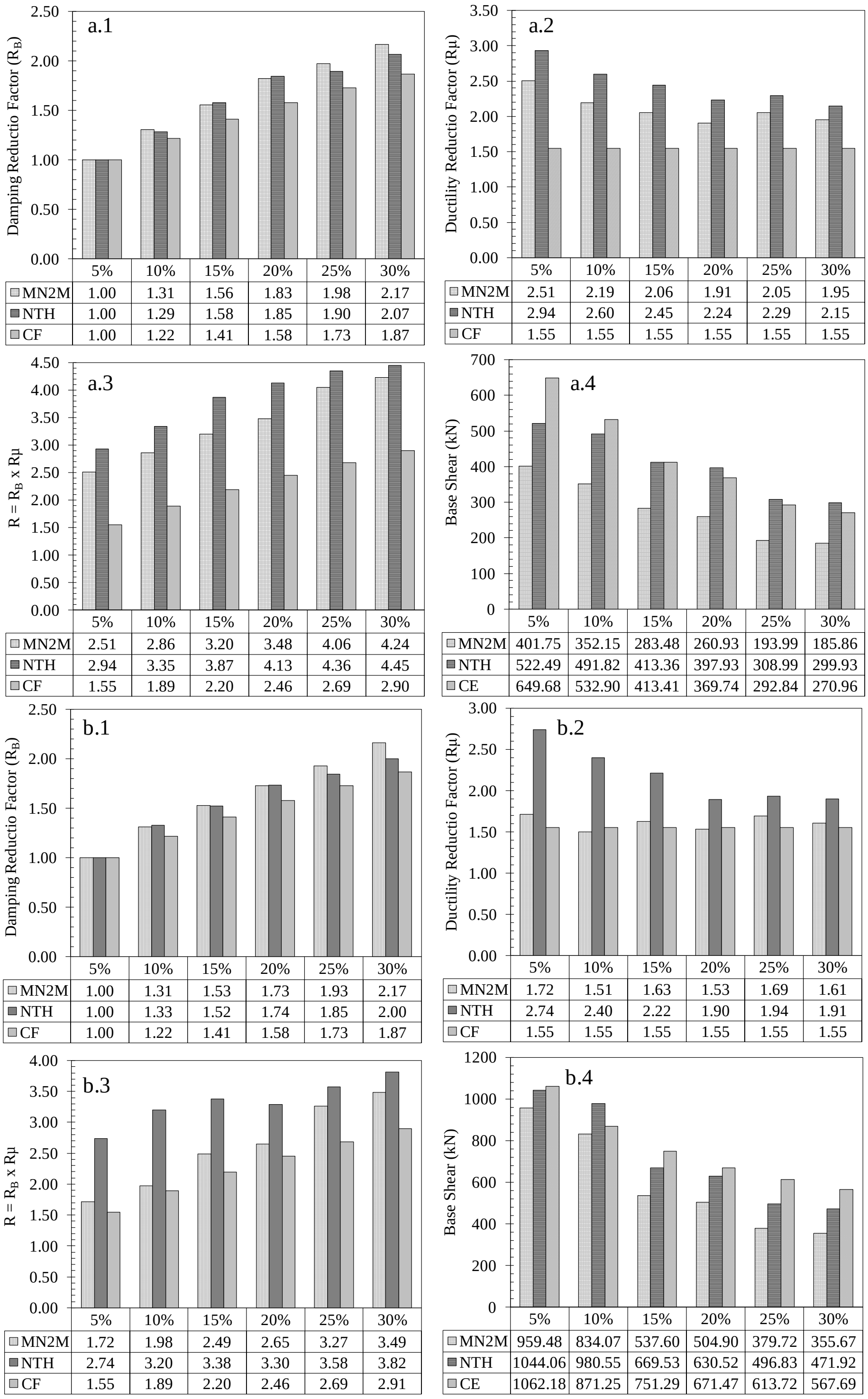


Fig. 4 Demand versus Capacity using Modified N2-Method at Different Supplemental Damping: (a) Group-1, Str-1; (b) Group-2, Str-2; (c) Group-3, Str-2; and (d) Group-4, Str-3



a.1

	5%	10%	15%	20%	25%	30%
MN2M	1.00	1.31	1.56	1.83	1.98	2.17
NTH	1.00	1.29	1.58	1.85	1.90	2.07
CF	1.00	1.22	1.41	1.58	1.73	1.87

a.2

	5%	10%	15%	20%	25%	30%
MN2M	2.51	2.19	2.06	1.91	2.05	1.95
NTH	2.94	2.60	2.45	2.24	2.29	2.15
CF	1.55	1.55	1.55	1.55	1.55	1.55

a.3

	5%	10%	15%	20%	25%	30%
MN2M	2.51	2.86	3.20	3.48	4.06	4.24
NTH	2.94	3.35	3.87	4.13	4.36	4.45
CF	1.55	1.89	2.20	2.46	2.69	2.90

a.4

	5%	10%	15%	20%	25%	30%
MN2M	401.75	352.15	283.48	260.93	193.99	185.86
NTH	522.49	491.82	413.36	397.93	308.99	299.93
CE	649.68	532.90	413.41	369.74	292.84	270.96

b.1

	5%	10%	15%	20%	25%	30%
MN2M	1.00	1.31	1.53	1.73	1.93	2.17
NTH	1.00	1.33	1.52	1.74	1.85	2.00
CF	1.00	1.22	1.41	1.58	1.73	1.87

b.2

	5%	10%	15%	20%	25%	30%
MN2M	1.72	1.51	1.63	1.53	1.69	1.61
NTH	2.74	2.40	2.22	1.90	1.94	1.91
CF	1.55	1.55	1.55	1.55	1.55	1.55

b.3

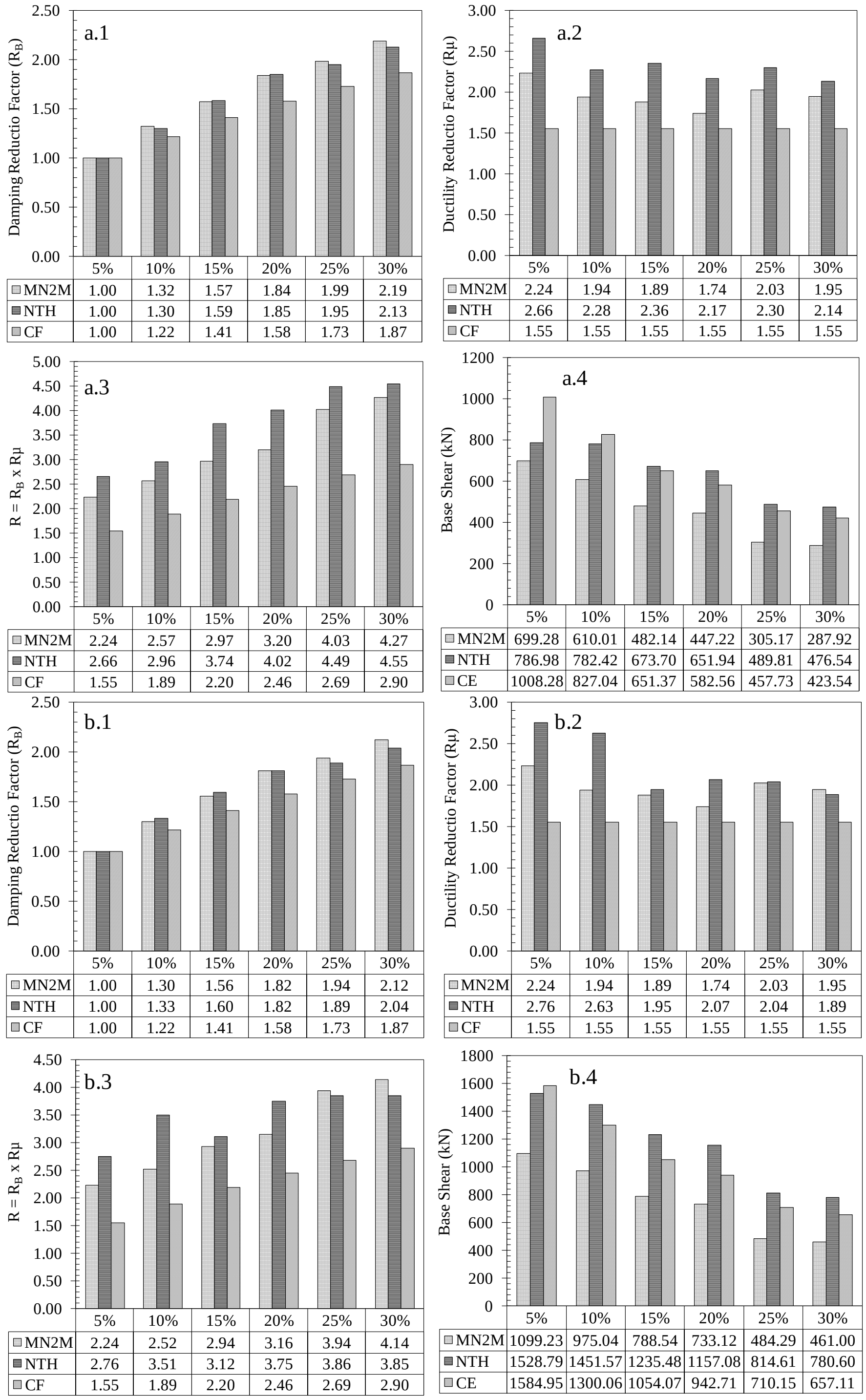
	5%	10%	15%	20%	25%	30%
MN2M	1.72	1.98	2.49	2.65	3.27	3.49
NTH	2.74	3.20	3.38	3.30	3.58	3.82
CF	1.55	1.89	2.20	2.46	2.69	2.91

b.4

	5%	10%	15%	20%	25%	30%
MN2M	959.48	834.07	537.60	504.90	379.72	355.67
NTH	1044.06	980.55	669.53	630.52	496.83	471.92
CE	1062.18	871.25	751.29	671.47	613.72	567.69

MN2M: Modified N2 Method NTH: Nonlinear Time History CF: Code Factor CE: Code Equation

Fig. 5 RB, Rμ, R and Base Shear for (a) Group-1, and (b) Group-2



a.1

	5%	10%	15%	20%	25%	30%
MN2M	1.00	1.32	1.57	1.84	1.99	2.19
NTH	1.00	1.30	1.59	1.85	1.95	2.13
CF	1.00	1.22	1.41	1.58	1.73	1.87

a.2

	5%	10%	15%	20%	25%	30%
MN2M	2.24	1.94	1.89	1.74	2.03	1.95
NTH	2.66	2.28	2.36	2.17	2.30	2.14
CF	1.55	1.55	1.55	1.55	1.55	1.55

a.3

	5%	10%	15%	20%	25%	30%
MN2M	2.24	2.57	2.97	3.20	4.03	4.27
NTH	2.66	2.96	3.74	4.02	4.49	4.55
CF	1.55	1.89	2.20	2.46	2.69	2.90

a.4

	5%	10%	15%	20%	25%	30%
MN2M	699.28	610.01	482.14	447.22	305.17	287.92
NTH	786.98	782.42	673.70	651.94	489.81	476.54
CE	1008.28	827.04	651.37	582.56	457.73	423.54

b.1

	5%	10%	15%	20%	25%	30%
MN2M	1.00	1.30	1.56	1.82	1.94	2.12
NTH	1.00	1.33	1.60	1.82	1.89	2.04
CF	1.00	1.22	1.41	1.58	1.73	1.87

b.2

	5%	10%	15%	20%	25%	30%
MN2M	2.24	1.94	1.89	1.74	2.03	1.95
NTH	2.76	2.63	1.95	2.07	2.04	1.89
CF	1.55	1.55	1.55	1.55	1.55	1.55

b.3

	5%	10%	15%	20%	25%	30%
MN2M	2.24	2.52	2.94	3.16	3.94	4.14
NTH	2.76	3.51	3.12	3.75	3.86	3.85
CF	1.55	1.89	2.20	2.46	2.69	2.90

b.4

	5%	10%	15%	20%	25%	30%
MN2M	1099.23	975.04	788.54	733.12	484.29	461.00
NTH	1528.79	1451.57	1235.48	1157.08	814.61	780.60
CE	1584.95	1300.06	1054.07	942.71	710.15	657.11

MN2M: Modified N2 Method NTH: Nonlinear Time History CF: Code Factor CE: Code Equation

Fig. 6 R_B, Rμ, R and Base Shear for (a) Group-3, and (b) Group-4

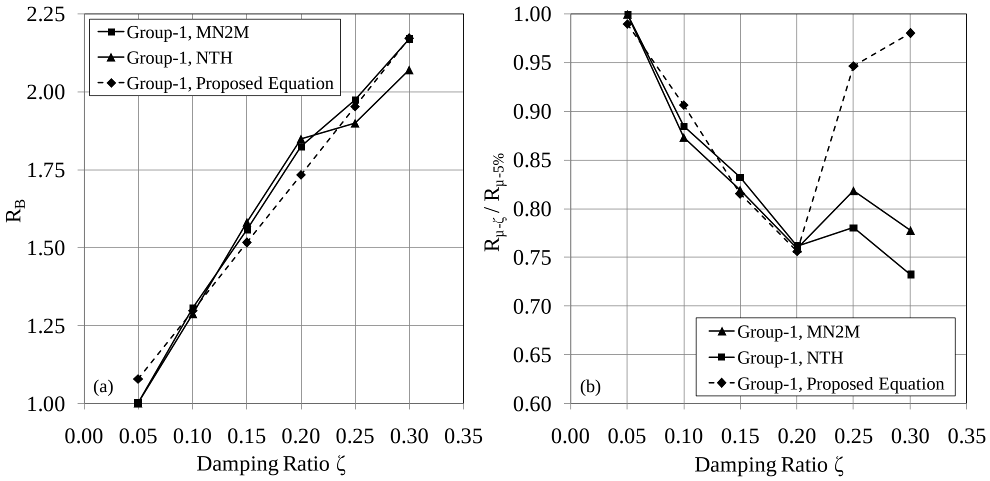


Fig. 7 Proposed Equation Comparison for: (a) R_B; and (b) $R_{\mu\text{-}\zeta}$ / $R_{\mu\text{-}5\%}$

Table 7 Results for Ductility Reduction Factor, Rμ, and Damping Reduction Factor R_B

Group	Model	ζ (%)	Ductility Reduction Factor (R_μ) Newmark and Hall (1982)	Ductility Reduction Factor (R_μ) Park and Ang (1985)	Damping Reduction Factor (R_B) $S_{ae\text{-}5\%}$ / $S_{ae\text{-}HD}$
Group-1 SMRF 2/7	Str-1	5	2.19	2.94	1.00
		10	1.96	2.60	1.29
	Str-2	15	1.87	2.45	1.58
		20	1.69	2.24	1.85
	Str-3	25	1.72	2.29	1.90
		30	1.61	2.15	2.07
Group-2 SMRF 2/12	Str-1	5	2.71	3.24	1.00
		10	1.85	2.40	1.33
	Str-2	15	1.65	2.22	1.52
		20	1.39	1.90	1.74
	Str-3	25	1.41	1.94	1.85
		30	1.38	1.91	2.00
Group-3 SMRF 3/7	Str-1	5	1.95	2.66	1.00
		10	1.72	2.28	1.30
	Str-2	15	1.76	2.36	1.59
		20	1.65	2.17	1.85
	Str-3	25	1.75	2.31	1.95
		30	1.60	2.14	2.13
Group-4 SMRF 3/12	Str-1	5	2.09	2.76	1.00
		10	2.02	2.63	1.33
	Str-2	15	1.43	1.95	1.60
		20	1.54	2.07	1.82
	Str-3	25	1.48	2.04	1.89
		30	1.35	1.89	2.04

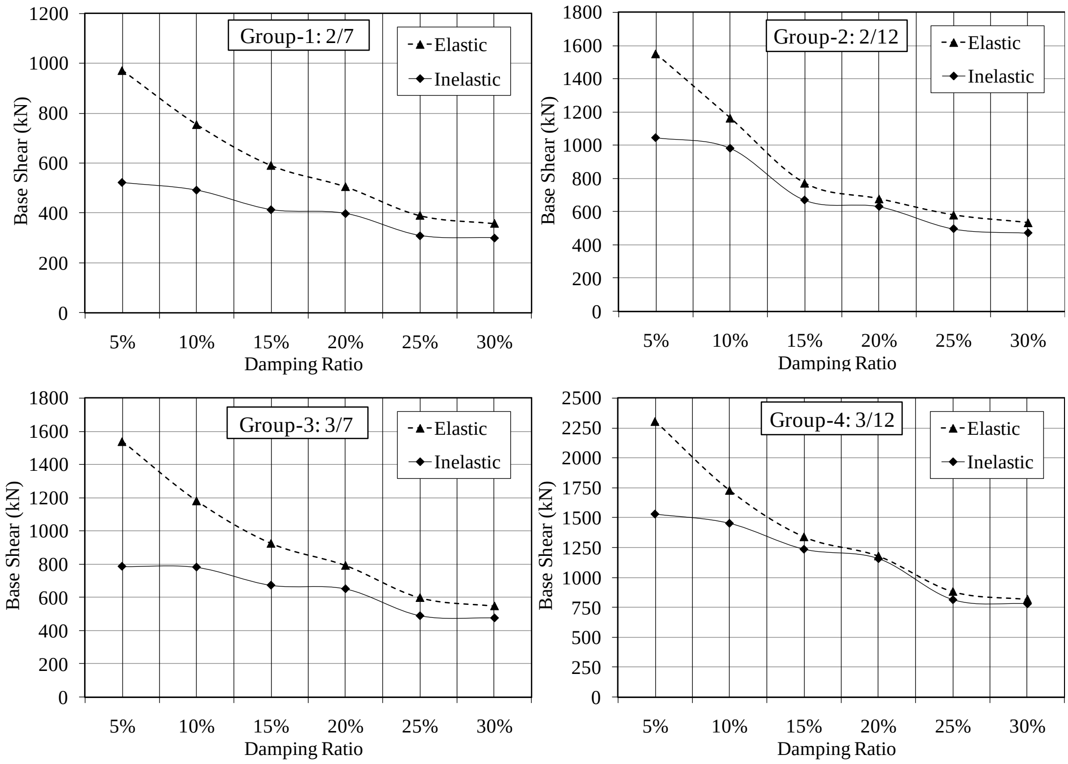


Fig. 8 Base Shear Reduction as Resulted from Elastic and Inelastic Time History Analyses for the Four Groups of Structure Models at different Damping Ratios

Fig. 8 shows the reduction induced on the base shear due to damping ratio increase for both elastic and inelastic behavior of the four groups of structure models. It is evident that the velocity dependent dampers are more effective in reducing the base shear value in elastic structures compared with inelastic ones. This observation is consistence with the results reported in literature (FEMA 273, 1997 and Sadek *et al*. 2000). Moreover, for both elastic and inelastic structures the base shear has been effectively reduced with increasing the damping ratio up to 20%. Beyond this value, the base shear is almost constant with further increase in the damping ratio. This result is in compliance with previous studies reported the optimum viscous damping ratio (Occhiuzzi 2009). It is worth noting that larger values of damping ratio (greater than approximately 40%) adversely affect the absolute acceleration response; and consequently, the value of the base shear (Sadek *et al*, 2000).

4. Conclusions

The seismic force reduction factor for steel moment resisting frames with supplemental viscous dampers has been studied. Both ductility and damping reduction factors have been evaluated. Two-dimensional finite element models have been established for a range of low to mid rise buildings with different parameters: number of floors; number of bays; and number of dampers with different supplemental damping ratios (from 5% to 30%). The main conclusions can be summarized as follows:

1. For the damping reduction factor (R_B), the modified N2-method provides results that are in good agreement with the time history analysis results; whereas, EC8 equation yields underestimated values.
2. An equation has been proposed for R_B as a function of the target first modal damping ratio ζ: [$R_B = 0.86 + 4.37\ \zeta$], with regression (R-squared) of 97%.
3. For the ductility reduction factor (Rμ), the modified N2-method and the time history analysis do not exhibit close results due to the hysteresis effect; meanwhile, EC8 ductility reduction factor is underestimated.
4. An equation has been proposed for Rμ as a function of the target first modal damping ratio ζ and R_B: [$R_{\mu\text{-}\zeta} = R_{\mu\text{-}5\%}\,(1.9 + 5\ \zeta - 1.16\ R_B)$], with regression (R-squared) of 73%. If the equation has been limited to damping ratios less than or equal to 20%, the R-squared is obtained as 90% and the equation can be modified as: [$R_{\mu\text{-}\zeta} = R_{\mu\text{-}5\%}\,(1.9 - 12\ \zeta + 1.4\ R_B)$], where $\zeta \leq 20\%$.
5. It is evident that Rμ is inversely proportional with R_B.
6. The viscous dampers are more effective for elastic structures than inelastic ones.
7. The parametric study that has been performed in this research is intended to cover a range of low to mid rise steel buildings. The obtained results revealed a trend of variation and correlation for both the damping and ductility reduction factors; meanwhile, the range of target structures may be further extended in order to generalize the obtained formulas.

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