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Investigation on the ability of intensity measures for reliable seismic collapse assessment of steel SMRFs with linear viscous dampers

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Abstract

Nowadays, passive energy dissipation systems are used in the seismic design of new structures and retrofit of existing structures. Fluid viscous dampers are categorized as one of the important types of passive energy dissipation systems, which convert the kinetic energy caused by seismic excitation to heat. Using viscous dampers can considerably decrease the seismic response of structures. In this paper, seismic collapse behavior of steel Special Moment Resisting Frames (SMRFs) equipped with Fluid Viscous Dampers (FVDs) is investigated. Incremental Dynamic Analysis (IDA) is applied to obtain the collapse capacity values for three low- to mid-rise steel SMRFs equipped with FVDs considering different ground motion Intensity Measures (IMs). After obtaining the collapse capacity, IM_{col}, values by using different IMs, the ability of the considered IMs to reliably predict the seismic collapse capacity of these structures is investigated. For this purpose, the efficiency and sufficiency of the IMs, as the main desirable features of an optimal IM for collapse capacity prediction, are considered to classify the IMs.

Keywords: Fluid viscous damper, Collapse capacity prediction, Intensity Measure

1. Introduction

Using passive energy dissipation systems, including fluid viscous dampers (FVDs), hysteretic dampers, viscoelastic dampers and friction dampers, is one of the effective ways to mitigate excitations due to strong ground motions [1, 2]. FVDs are a type of passive energy dissipation systems that are extensively used for the seismic design of new structures and retrofit of existing structures [3, 4] because they reduce both displacements and accelerations simultaneously [5, 6]. FVDs provide a velocity-dependent force and can behave as linear or nonlinear elements. The force developed by a nonlinear FVD is as follows:

$$F_d = C \times |v|^{\alpha} \times \mathrm{sgn}(v) \tag{1}$$

where C is the damping coefficient, v is the relative velocity between the two ends of the damper, α is the velocity exponent, and sgn is the signum function. In seismic applications, the exponent α is in the range of 0.2 to 1.0 [7]. When α is equal to one, the damper is called "linear FVD", and values of α lower than one correspond nonlinear FVDs.

Several researchers have investigated the seismic response and the design criteria of structures equipped with FVDs [8-11]. Although a number of procedures have been developed for the design of these structures [9, 12-14], seismic collapse of these structures has not been extensively investigated. The collapse of structural systems due to strong ground motions is the primary source of casualties and loss of life during earthquakes. Seismic collapse is defined as the inability of a structural system to withstand gravity loads under ground motion. In recent years, due to significant advancements in the computational ability of computers and methods of nonlinear analysis, assessing the seismic collapse of structures has become an interesting field of study for researchers. Thus, several researches have been performed to assess the seismic collapse of structures [15-17] and to develop engineering approaches for seismic collapse assessment. The ATC-63 document [18] presents a new methodology for seismic collapse assessment of structures, to assess design criteria and seismic performance factors existing in seismic codes. Recently, some studies have been performed to assess the collapse of structures equipped with FVDs. Seo et al. [19] investigated the seismic resistance of steel moment resisting frames (MRFs) with supplemental FVDs against collapse. Hamidia et al. [20] proposed a simplified approach to assess the seismic collapse of structures equipped with FVDs.

Intensity Measure (IM) is a parameter that describes the strength of a ground motion and quantifies its effect on structures. In fact, IM is the output of the ground motion hazard analysis, which links the hazard to the seismic response of structure. An optimal IM should meet the requirements of efficiency and sufficiency [21]. In other words, efficiency and sufficiency are the main desirable features of an optimal IM. Efficiency is the ability of an IM to predict the response or capacity of a structure subjected to ground motion with small dispersion, whereas sufficiency is ability of an IM to predict the response or capacity of a structure conditionally independent of other ground motion properties. In fact, using an efficient IM leads to smaller variability in the structural response or capacity prediction, which permits use of a lower number of ground motion records in seismic analyses. In addition, using a sufficient IM reduces the complexity of record selection procedure because no other ground motion information (i.e., magnitude, distance, etc.) is required to predict the structural response or capacity. In general, IMs are classified into two groups of scalar and vector. Common scalar IMs are spectral acceleration at the fundamental period of structure, $Sa(T_1)$, Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV) and Peak Ground Displacement (PGD). Shome et al. [22] showed that $Sa(T_1)$ is more efficient and sufficient than PGA. Thus, nowadays, seismic codes throughout the world use $Sa(T_1)$ as the most common IM. Recently, several vector IMs have been proposed in the technical literature [23-27]. Most of these vector-IMs consist of $Sa(T_1)$ as the first component and a spectral shape indicator as the second component. Most of the studies in the field of ground motion IMs are focused on investigating the efficiency and sufficiency of IMs to predict the structural response. Due the importance of assessing the seismic collapse of structures, some studies have focused on investigating the efficiency and sufficiency of IMs for collapse capacity prediction [28-30]. The basic tool for assessing the seismic collapse of structures is the collapse fragility curve, which expresses the probability of collapse as a function of a scalar IM. When using a vector IM, the collapse fragility surface can be used instead of the collapse fragility curve. Using an optimal IM for collapse capacity prediction leads to a more reliable collapse fragility curve or surface.

The aim of this study is to investigate the efficiency and sufficiency of scalar and vector IMs to predict the collapse capacity of structures equipped with FVDs. For this purpose, three low- to medium-rise structures are considered, and different levels of supplemental viscous damping are added to each of the structures. Then, the collapse capacities of the structures are obtained using six scalar and vector IMs, and the efficiency and sufficiency of the IMs for collapse capacity prediction are compared.

2. Selected IMs

In this study, four common scalar IMs and two vector IMs were considered. The scalar IMs are $Sa(T_1)$, PGA, PGV and PGD, whereas the vector IMs are $(Sa(T_1),N_P)$ [24] and $(Sa(T_1),R_{T1,T2})$, [26, 27]. The first component of the vector IMs is $Sa(T_1)$ and their second components are un-scalable spectral shape indicators. The parameter N_P is defined as follows:

$$N_P = \frac{Sa_{ave}(T_1 ... T_N)}{Sa(T_1)} \tag{2}$$

where $Sa_{ave}(T_1 \ldots T_N)$ is the geometric mean of spectral accelerations over the period range of T_1-T_N, and T_N is equal to $2T_1$. The parameter $R_{T1,T2}$ is defined as follows:

$$R_{T1,T2} = \frac{Sa(T_2)}{Sa(T_1)} \tag{3}$$

where $Sa(T_2)$ is the spectral acceleration at T_2=2T_1. It should be noted that both N_P and $R_{T1,T2}$ indicate the shape of pseudo-acceleration response spectrum in periods greater than the fundamental period of structure. Thus, when the fundamental period of structure elongates due to nonlinear deformations, the vectors ($Sa(T_1)$,N_P) and ($Sa(T_1)$,$R_{T1,T2}$) can imply the severity of ground motion more realistically, compared with $Sa(T_1)$.

3. Structural modeling and analysis

The structures selected for this study are 3- 6- and 9-story steel Special Moment Resisting Fames (SMRFs) that were used by Hamidia et al. [20]. These structures were designed for the SAC steel project [31], and their detailed information can be found in FEMA 355C [32] and the study by Hall [33]. OpenSees [34] was used to create the 2-D numerical models of the structures. Distributed plasticity force-based beam-column elements consisting of five integration points, each using a fiber section, along the element length were used to model the columns. Steel02 material in the OpenSees, assuming E=200 GPa and a hardening ratio of 0.002, was applied to model the uniaxial behavior of each fiber. Thus, cyclic deterioration in the column elements was neglected. The behavior of the beams was modeled using a concentrated plasticity approach (Ibarra and Krawinkler [16]; Haselton [17]). Therefore, each beam was modeled using two zero-length rotational springs at its both ends, representing plastic hinges, and an elastic beam-column element. The modified Ibarra-Medina-Krawinkler model [35] was used to model the moment-rotation relationship of the rotational springs. The parameters of this model were determined based on the relationships proposed by Lignos and Krawinkler [35]. In order to consider the rigid end offsets of the beams and columns, rigid elements were used at both ends of the beams and columns. The lengths of rigid elements in the beams and columns, in a beam-column joint, were assumed to be equal to the half of the column section depth and beam section depth, respectively. A leaning column was used to model the P-Δ effects of gravity columns. This leaning column was modeled by using elastic beam-column elements, which have moments of inertia and areas about two orders of magnitude larger than the frame columns. These beam-column elements were connected to the joints in the floor levels by zero-length rotational spring elements with very small stiffness values. Then, these joints were connected to the SMRF by axially rigid truss elements.

Rayleigh viscous damping was used to model the inherent viscous damping of the structures. Thus, a five percent damping ratio was assigned to the first and third mode (i.e., the mode at which the cumulative mass participation ratio exceeds 0.95) periods of the structures. In addition to the SMRF structures, assuming three levels of supplemental viscous damping ratio (i.e., ξ_v = 0.05, 0.1, and 0.15), linear FVDs were added to the SMRFs to improve their performance under seismic excitations. In other words, three SMRFs without supplemental viscous damping and nine SMRFs with supplemental viscous damping were considered. Fig. 1 indicates the dimensions of the SMRFs, and the configuration of linear FVDs in the SMRFs with supplemental viscous damping. Table 1 presents the first mode periods of the SMRFs.

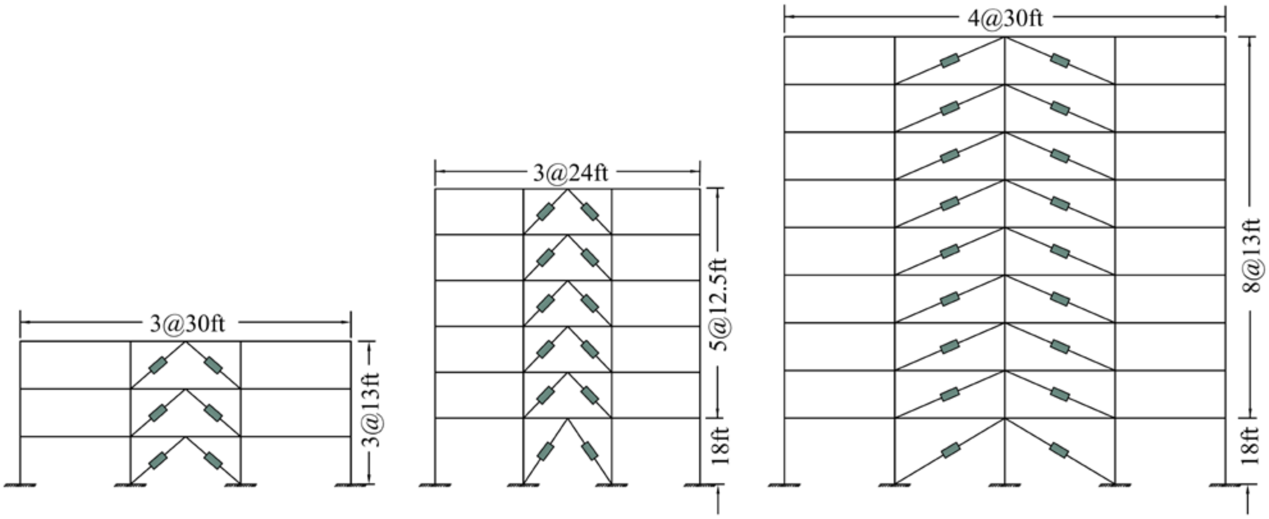

Fig. 1– Dimensions of the SMRFs and the configuration of linear FVDs in the SMRFs with linear FVDs

Table 1 – First mode periods of the SMRF structures

Structure	First mode period (s)
3-story	0.95
6-story	1.32
9-story	2.08

The supplemental viscous damping ratio for the first mode of a structure with linear FVDs can be obtained as:

$$\xi_v = \frac{T_1 \sum_{j=1}^{N} C_j f_j^{\,2} (\phi_{1j} - \phi_{1(j-1)})^2}{4\pi \sum_{j=1}^{N} m_j \phi_{1j}^{\,2}} \tag{4}$$

where T_1 is the first mode period of the structure, C_j is the damping coefficient of story j, f_j is a displacement magnification factor that depends on the geometrical configuration of the dampers at story j (for a diagonal damper with an angle of inclination θ_j, $f_j = cos\theta_j$), ϕ_{1j} is the first mode shape value at the top of story j, normalized to have a unit component at the roof, m_j is the mass of story j, and N is the number of stories. Given a supplemental viscous damping ratio for the first mode a structure, and assuming $C_j f_j^{\,2}$ to be proportional to the interstory drift obtained on the basis of the first mode shape, $\phi_{1j} - \phi_{1(j-1)}$, the following equation can be obtained, based on Eq. (4), to determine the damping coefficient of story k:

$$C_k = \frac{4\pi \xi_v (\phi_{1k} - \phi_{1(k-1)}) \sum_{j=1}^{N} m_j \phi_{1j}^{\,2}}{T_1 f_k^{\,2} \sum_{j=1}^{N} (\phi_{1j} - \phi_{1(j-1)})^3} \tag{5}$$

Eq. (5) was used to determine the damping coefficients corresponding to the different stories of the structures with supplemental viscous damping. Table 2 presents the values of story damping coefficients, C_k, for the three SMRFs with the supplemental viscous damping ratio of 0.05. It should be mentioned that the C_k, values for supplemental viscous damping ratios of 0.1 and 0.15 can be obtained from multiplying the values presented in this table by 2 and 3, respectively. To model the linear FVDs, it was assumed that the supporting brace member is rigid, and the dampers do not reach their stroke limits during seismic loading.

Table 2 – Values of story damping coefficients, C_k, for the three SMRFs with the supplemental viscous damping ratio of 0.05

	Story Damping Coefficient, C_k [kips.s/in]		
	3-story	**6-story**	**9-story**
1st Story	0.343	0.364	0.373
2nd Story	0.569	0.595	0.598
3rd Story	0.324	0.343	0.340
4th Story	-	0.550	0.541
5th Story	-	0.270	0.282
6th Story	-	0.264	0.275
7th Story	-	-	0.523
8th Story	-	-	0.679
9th Story	-	-	0.420

To obtain the collapse capacities of the structures using Incremental Dynamic analysis (IDA) method [15], 67 ground motion records used by Yakhchalian et al. [28, 36] were considered. The collapse was assumed to occur when the maximum interstory drift ratio in the structure reaches 0.15. Considering $Sa(T_1)$ as the IM for performing IDAs, the intensity of each ground motion record, $Sa(T_1)$, was incrementally increased until the collapse occurs. Thus, the value of $Sa(T_1)$ corresponding to collapse, Sa_{col}, was computed for each of the ground motion records. Fig. 2 illustrates the IDA curves of the 3-story structure with the supplemental viscous damping ratio of 0.1. When the values of Sa_{col} for a structure were computed by using the selected ground motion records, it is easy to obtain the corresponding IM_{col} values of the other scalar IMs, (i.e., PGA_{col}, PGV_{col} and PGD_{col}). For instance, the value of PGA_{col} for each record can be calculated by multiplying the un-scaled value of PGA for that record by the ratio of Sa_{col} to the un-scaled value of $Sa(T_1)$. Thus, the values of Sa_{col}, PGA_{col}, PGV_{col} and PGD_{col} corresponding to the selected ground motion records were obtained for all of the structures.

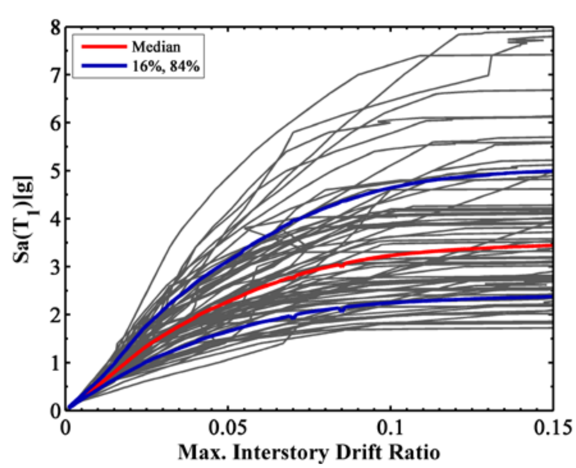

Fig. 2 – IDA curves of the 3-story structure with the supplemental viscous damping ratio of 0.1

4. Investigating the efficiency of the IMs for collapse capacity prediction

Efficiency of an IM for collapse capacity prediction is the ability of that IM to predict the collapse capacity of a structure subjected to ground motion with small dispersion. Thus, an efficient IM can predict the collapse capacity of structures with a lower record-to-record variability in which causes more accuracy in obtaining collapse fragility curves or surfaces. Hence, by using an efficient IM the mean annual frequency of collapse can be estimated more reliably. The logarithmic standard deviation of IM_{col} values, $\sigma_{\ln IMcol}$, is an index for the efficiency of scalar IMs for collapse capacity prediction. In other words, a lower value for $\sigma_{\ln IMcol}$ represents a more efficient IM for collapse capacity prediction. When using the vector IMs ($Sa(T_1)$,N_P) and ($Sa(T_1)$,$R_{T1,T2}$), the conditional logarithmic mean of collapse capacity, $\mu_{\ln Sacol|IM2}$, can be estimated, by performing a linear regression, as follows:

$$\mu_{\ln Sa_{col}|IM_2} = a + b \ln IM_2 \tag{6}$$

where a and b are the regression coefficients, and IM_2 is the second component of the considered vector IM. When using the vector IMs ($Sa(T_1)$,N_P) and ($Sa(T_1)$,$R_{T1,T2}$), the standard deviation of the regression residuals, $\sigma_{\ln Sacol|IM2}$ is an index for the efficiency of the IMs for collapse capacity prediction. Table 3 presents the values of $\sigma_{\ln IMcol}$ and $\sigma_{\ln Sacol|IM2}$ obtained for the scalar and vector IMs. It can be seen that, in the most cases, the vector IMs ($Sa(T_1)$,N_P) and ($Sa(T_1)$,$R_{T1,T2}$) are more efficient than the scalar IMs to predict the collapse capacity of the structures. Furthermore, PGV is more efficient than the other scalar IMs. The results also show that when the supplemental viscous damping ratio of a structure increases the efficiency of the IMs, except in some cases for PGV and PGD, to predict the collapse capacity of the structures decreases correspondingly.

Table 3 – Values of $\sigma_{\ln IMcol}$ and $\sigma_{\ln Sacol|IM2}$ obtained for the scalar and vector IMs

Structure	IM	Supplemental viscous damping ratio			
		-	0.05	0.1	0.15
3-Story	$Sa(T_1)$	0.343	0.364	0.373	0.395
	PGA	0.569	0.595	0.598	0.623
	PGV	0.324	0.343	0.340	0.361
	PGD	0.549	0.550	0.541	0.538
	($Sa(T_1)$,$R_{T1,T2}$)	0.259	0.270	0.282	0.315
	($Sa(T_1)$,N_P)	0.254	0.264	0.275	0.310
6-Story	$Sa(T_1)$	0.452	0.500	0.523	0.533
	PGA	0.602	0.656	0.679	0.685
	PGV	0.344	0.405	0.420	0.421
	PGD	0.519	0.546	0.559	0.541
	($Sa(T_1)$,$R_{T1,T2}$)	0.278	0.319	0.364	0.385
	($Sa(T_1)$,N_P)	0.308	0.374	0.420	0.434
9-Story	$Sa(T_1)$	0.409	0.435	0.458	0.491
	PGA	0.676	0.707	0.717	0.731
	PGV	0.391	0.412	0.429	0.454
	PGD	0.470	0.473	0.472	0.506
	($Sa(T_1)$,$R_{T1,T2}$)	0.295	0.301	0.316	0.360
	($Sa(T_1)$,N_P)	0.278	0.289	0.309	0.351

5. Investigating the sufficiency of the IMs for collapse capacity prediction

Sufficiency of an IM for collapse capacity prediction is the ability of that IM to predict the collapse capacity of a structure conditionally independent of other ground motion properties such as earthquake magnitude (M), source-to-site distance (R), etc. In fact, when using a sufficient IM to predict the collapse capacity of structures, there is no need to use complex ground motion record selection procedures, because the IM represents all other ground motion properties. In order to test the sufficiency of a scalar IM with respect to a ground motion parameter (i.e., M, R, etc.) for predicting the collapse capacity of structures, a linear regression can be applied as follows:

$$E[\ln IM_{col}] = c + d\,(X) \tag{7}$$

where $E[\ln IM_{col}]$ is the expected value of $\ln IM_{col}$ values; c and d are the regression coefficients; and X is the earthquake magnitude, M, or the natural logarithm of the source-to-site distance, $\ln R$. The coefficient d in Eq. (7) is estimated using a finite number of observations; thus, statistical tests such as F-test [37] can be used to examine the statistical significance of the coefficient d. The result of the F-test is a p-value, which indicates the sufficiency or insufficiency of the IM with respect to the considered ground motion parameter. A p-value of less than 0.05, which implies the statistical significance of the coefficient d, indicates the insufficiency of the considered IM, whereas a p-value of greater than 0.05 indicates the sufficiency of the considered IM. In order to test the sufficiency of the vector IMs ($Sa(T_1)$,N_P) and ($Sa(T_1)$,$R_{T1,T2}$), the residuals of the regression performed to obtain Eq. (6) should be used in Eq. (7) instead of $\ln IM_{col}$ values.

Table 4 presents the p-values obtained from testing the sufficiency of the IMs, with respect to R, for collapse capacity prediction of the structures. It can be seen that all the p-values are greater than 0.05, and thus all the considered IMs are sufficient with respect to R. Table 5 presents the p-values obtained from testing the sufficiency of the IMs, with respect to M, for collapse capacity prediction of the structures. It can be seen that PGV is sufficient with respect to M for all the structures, whereas none of the vector IMs is sufficient with respect to M for all the structures. But the vector IM ($Sa(T_1)$,$R_{T1,T2}$) is sufficient with respect to M for most of the structures. Fig. 3 compares the sufficiency of the vector IMs ($Sa(T_1)$,N_P) and ($Sa(T_1)$,$R_{T1,T2}$) with respect to R to predict the collapse capacity of the 3-story structure with the supplemental viscous damping ratio of 0.1. It can be seen that both the vector IMs are sufficient with respect to R, but ($Sa(T_1)$,N_P) is more sufficient than ($Sa(T_1)$,$R_{T1,T2}$). Fig. 4 compares the sufficiency of the vector IMs ($Sa(T_1)$,N_P) and ($Sa(T_1)$,$R_{T1,T2}$) with respect to M to predict the collapse capacity of the 3-story structure with the supplemental viscous damping ratio of 0.1. It can be seen that ($Sa(T_1)$,$R_{T1,T2}$) is sufficient with respect to M, whereas ($Sa(T_1)$,N_P) is insufficient with respect to M, because the p-value obtained from testing its sufficiency is less than 0.05. In other words, the vector IM ($Sa(T_1)$,N_P) cannot predict the collapse capacity of the selected structure independently from M.

Table 4 – P-values obtained from testing the sufficiency of the IMs with respect to *R*

Structure	IM	Supplemental viscous damping ratio -	0.05	0.1	0.15
3-Story	$Sa(T_1)$	0.276	0.252	0.128	0.155
	PGA	0.985	0.928	0.738	0.753
	PGV	0.678	0.790	0.870	0.887
	PGD	0.302	0.342	0.488	0.481
	$(Sa(T_1),R_{T1,T2})$	0.466	0.422	0.193	0.246
	$(Sa(T_1),N_P)$	0.949	0.911	0.523	0.561
6-Story	$Sa(T_1)$	0.750	0.921	0.949	0.841
	PGA	0.625	0.878	0.863	0.949
	PGV	0.665	0.912	0.947	0.810
	PGD	0.589	0.384	0.411	0.324
	$(Sa(T_1),R_{T1,T2})$	0.248	0.665	0.680	0.850
	$(Sa(T_1),N_P)$	0.504	0.977	0.954	0.908
9-Story	$Sa(T_1)$	0.591	0.635	0.613	0.727
	PGA	0.702	0.729	0.706	0.774
	PGV	0.772	0.808	0.770	0.886
	PGD	0.500	0.485	0.518	0.470
	$(Sa(T_1),R_{T1,T2})$	0.351	0.375	0.349	0.514
	$(Sa(T_1),N_P)$	0.180	0.201	0.193	0.328

Table 5 – P-values obtained from testing the sufficiency of the IMs with respect to *M*

Structure	IM	Supplemental viscous damping ratio -	0.05	0.1	0.15
3-Story	$Sa(T_1)$	0.238	0.240	0.155	0.071
	PGA	0.047	0.053	0.036	0.021
	PGV	0.792	0.855	0.912	0.887
	PGD	0.000	0.000	0.000	0.000
	$(Sa(T_1),R_{T1,T2})$	0.142	0.139	0.074	0.028
	$(Sa(T_1),N_P)$	0.006	0.005	0.002	0.001
6-Story	$Sa(T_1)$	0.009	0.031	0.021	0.011
	PGA	0.023	0.052	0.038	0.023
	PGV	0.665	0.880	0.652	0.438
	PGD	0.000	0.000	0.000	0.001
	$(Sa(T_1),R_{T1,T2})$	0.046	0.198	0.126	0.065
	$(Sa(T_1),N_P)$	0.016	0.088	0.058	0.030
9-Story	$Sa(T_1)$	0.057	0.037	0.037	0.142
	PGA	0.014	0.011	0.010	0.029
	PGV	0.280	0.182	0.162	0.415
	PGD	0.000	0.001	0.001	0.000
	$(Sa(T_1),R_{T1,T2})$	0.796	0.900	0.889	0.438
	$(Sa(T_1),N_P)$	0.821	0.642	0.627	0.794

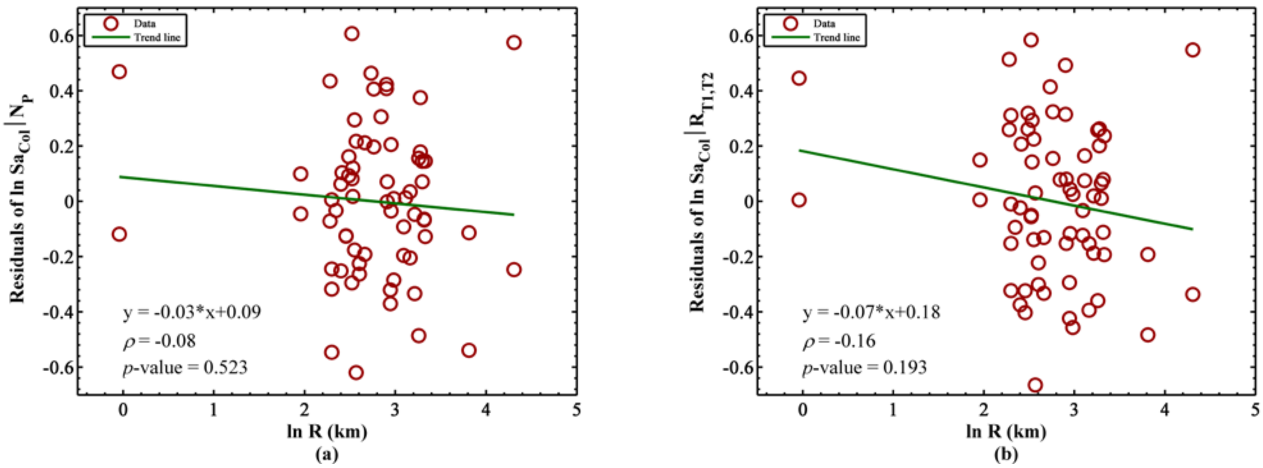

Fig. 3 – Comparison between the sufficiency of the vector IMs with respect to R for collapse capacity prediction of the 3-story structure with the supplemental viscous damping ratio of 0.1: (a) ($Sa(T_1)$,N_P); (b) ($Sa(T_1)$,$R_{T1,T2}$)

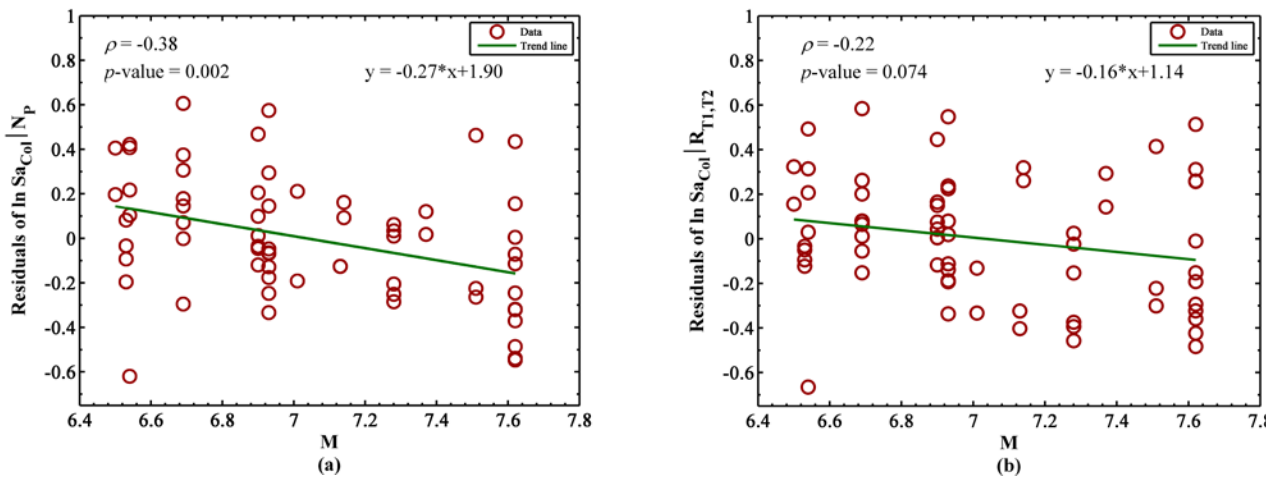

Fig. 4 – Comparison between the sufficiency of the vector IMs with respect to M for collapse capacity prediction of the 3-story structure with the supplemental viscous damping ratio of 0.1: (a) ($Sa(T_1)$,N_P); (b) ($Sa(T_1)$,$R_{T1,T2}$)

6. Conclusions

In this study, the efficiency and sufficiency of four common scalar and two vector IMs for collapse capacity prediction of SMRFs with linear FVDs were investigated. The results indicate that, in the most cases, the vector IMs ($Sa(T_1)$,N_P) and ($Sa(T_1)$,$R_{T1,T2}$) are more efficient than the scalar IMs. Moreover, PGV is more efficient than the other scalar IMs. It should be mentioned that when the supplemental viscous damping ratio of a structure increases, the efficiency of the IMs, except in some cases for PGV and PGD, to predict the collapse capacity of the structures decreases correspondingly. Investigating the sufficiency of the IMs with respect to R, for collapse capacity prediction of the structures, shows that all the IMs are sufficient with respect to R. Furthermore, investigating the sufficiency of the IMs with respect to M, for collapse capacity prediction of the structures, shows that among the scalar IMs only PGV is sufficient with respect to M for all the structures, whereas none of the vector IMs is sufficient with respect to M for all the structures. But the vector IM ($Sa(T_1)$,$R_{T1,T2}$) is sufficient with respect to M for most of the structures.

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