Available online at www.eccomasproceedia.org
Eccomas Proceedia COMPDYN (2017) 4066-4075

ECCOMAS
Proceedia

COMPDYN 2017
6th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis (eds.)
Rhodes Island, Greece, 15–17 June 2017

Collapse capacity prediction of SDOF systems Equipped with Fluid Viscous Dampers

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Keywords: Collapse capacity, Single-Degree-of-Freedom system, Viscous damper, Incremental dynamic analysis

ABSTRACT

There are various types of passive energy dissipation systems that are used in the seismic design of new structures and the seismic rehabilitation of existing structures. One of the most important kinds of passive energy dissipation systems, which reduce both displacements and accelerations simultaneously, are fluid viscous dampers (FVDs). Using FVDs can reduce the seismic response and improve the performance of structures during earthquake excitations and has a significant effect on the damping of structure. In this paper, Single-Degree-of-Freedom (SDOF) systems with different periods and ductility values, equipped with FVDs, are considered. In order to evaluate the collapse capacity of structures with higher precision, P-Δ effects and strength degradation are taken into account in modeling. Incremental dynamic analysis (IDA) is applied to obtain the collapse capacity of the SDOF systems under far-field ground motion records, and finally the results are investigated. It should be mentioned that the results of this study can be extended to Multi-Degree-of-Freedom (MDOF) structures.

doi: 10.7712/120117.5705.18225

1 INTRODUCTION

Life safety and collapse prevention have always been the most important goals in seismic design. Advances in computing power and the development of models that can more accurately simulate the behavior of structural components at the time of collapse have made probabilistic assessment of seismic collapse possible. Seismic collapse is the final limit at which dynamic instability occurs. Preventing the collapse of structures with a good margin of safety, is one of most important goals of seismic design. Vamvatsikos and Cornell [1], introduced a method called Incremental Dynamic Analysis (IDA) to evaluate the seismic performance of structures. The IDA method is a tool for investigating the performance a structures form elastic range to collapse under the set of ground motion records.

Conventional lateral force resisting systems (e.g., moment resisting frames) dissipate the energy due to seismic excitations by inelastic deformations in the components of the system and resist destructive forces such as earthquake by using ductility. But the amount of damping in these structures is not so much. After 1989 Loma Prieta and 1994 Northridge earthquakes, using passive energy dissipation systems for seismic design of structures was developed increasingly [2]. The principle role of a passive energy dissipation system is to reduce the inelastic deformation demands on the framing system of a structure [3]. These systems absorb a significant amount of the input energy from earthquake to the structure. Nowadays, there are various types of passive energy dissipation systems, such as Fluid Viscous Dampers (FVDs), hysteretic dampers, viscoelastic dampers and friction dampers [4, 5]. These dampers improve seismic performance by absorbing earthquake force and reducing structural response. Among these passive energy dissipation systems, FVDs are nowadays extensively used for the seismic design of new structures and seismic rehabilitation of existing structures [3, 6]. They can enhance the viscous damping of structures to more than 20-30% of critical damping (in addition to the inherent damping of structures) and decrease stresses and strains caused by the earthquake. Therefore, a number of procedures have been developed for the design of structures with FVDs [7-10]. These dampers can behave as linear or nonlinear elements. The velocity-dependent force developed by FVDs 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. The velocity exponent α indicates the degree of nonlinearity of FVD. α=1 corresponds to a linear FVD and the values of α lower than one correspond to nonlinear FVDs. It should be mentioned that the value of α, can vary in the range of 0.2 to 1.0 [11].

Due to the existence of the IDA method as a powerful tool for analysis, seismic collapse of structures, has been studied by many researchers [12, 13]. As a result, documents such as FEMA P695 [13] and FEMA P-58 [14] have been published, which present guides for assessing seismic collapse of structures. Collapse capacity is defined as the maximum earthquake intensity level at which the structure still keeps its dynamic stability [15]. Ibarra and Krawinkler [16] investigated the uncertainty in the deterioration parameters of SDOF systems on the variance of collapse capacity. In their study, dynamic analyses were performed on deteriorating hysteretic models. The results of the analyses indicated that uncertainties in the displacement at the peak strength and the post-capping stiffness significantly contribute to the variance of collapse capacity. Several researchers evaluated the collapse capacity of structures by use of simplified SDOF models, because of their efficacy in terms of calculation time. Certainly, the results can be extended to the Multi-

Degree-of-Freedom (MDOF) structures [17,18]. The P-Δ effects are moments that are created in the structure by gravity loads due to displacement by the lateral force. Although hysteretic models have been considered in studies have positive post-yield stiffness, but considering P-Δ effects creates negative post-yield stiffness and finally leads to the collapse of structure. Adam et al. [17] investigated the P-Δ effects on collapse mechanism of non-deteriorating MDOF structures by use of equivalent SDOF systems and pushover analysis. They showed that the collapse capacities obtained from simplified SDOF models were very close, with good accuracy, to the results obtained from the corresponding actual MDOF structures by use of IDAs. Sun et al. [19] investigated the effect of gravity on the dynamic behavior of SDOF systems and its impacts on the change of periods of systems. They showed that the maximum displacement, the system can bear without collapse is directly related to the stability coefficients and yield displacement. Jennings and Husid. [20] evaluated the time required to occur collapse in SDOF systems. The systems had positive post-yield stiffness but because of assuming P-Δ effects, negative post-yield stiffness had been created in structures. The results showed that the time to collapse was related to the yield strength, and the post-yield stiffness. Adam and Jäger [21] investigated the effect of both near-field ground motions and a finite ductility on the collapse capacity of inelastic non-deteriorating SDOF systems by use IDAs. The P-Δ effect was taken into account in modeling. The results showed that for short period structures the median "instability collapse capacity" based on a near-field ground motion set is smaller compared to a corresponding one based on a far-field ground motion set; however, for structures with a period larger than 1.5 s the opposite was true. Miranda and Akkar [22] investigated the effects of the negative post-yield stiffness and structural vibration period of bilinear SDOF systems on lateral resistance required to avoid dynamic instability. They proposed an empirical equation to estimate the minimum lateral resistance to prevent the collapse of SDOF systems with P-Δ. The results showed that the lateral strength required to prevent collapse increased as the post-yield descending branch of the force-deformation relationship was steeper. Many researchers investigated the seismic behavior of structures equipped with FVDs [23-27]. Hamidia et al. [28] proposed a simplified procedure to assess the seismic collapse of steel moment resisting frames equipped with linear and nonlinear FVDs. The proposed procedure is based on the seismic response of nonlinear equivalent SDOF systems equipped with FVDs, and using pushover analysis without the need for nonlinear time history dynamic analysis. However, this method is only a fast approximate procedure for assessing the collapse capacity with acceptable accuracy, and cannot completely replace very detailed nonlinear time history dynamic analyses. Due to the increasing use of FVDs to improve the seismic performance of structures, in this study the collapse capacity of SDOF systems equipped with FVDs is investigated. For this aim, 21 SDOF systems without supplemental viscous damping and 420 SDOF systems with FVDs, having different values of ductility, damping ratio and velocity exponent, are considered. The collapse capacities of these SDOF systems are obtained by use of IDAs.

2 STRUCTURAL MODELING AND ANALYSIS

In this study, the seismic collapse capacity of SDOF systems equipped with FVDs was investigated. OpenSees software [29] was used to create the structural models of the SDOF systems. The elastic periods of the considered SDOF systems (T_{el}) are 0.4, 0.6, 0.8, 1, 1.5, 2 and 3 s. For each period, three ductility, $\mu=\delta_c/\delta_y$, values of 2, 4 and 8, which correspond respectively to brittle, moderately-ductile and ductile structures were assumed. To model the inherent damping of the SDOF systems, a mass proportional damping ratio of 0.05 was considered. To improve the seismic performance of the systems, in addition to the inherent damping, linear and nonlinear FVDs were added to the SDOF systems. Five values for velocity exponent of FVD (i.e., α=0.2, 0.4, 0.6, 0.8 and 1) and four values for supplemental viscous damping ratio (i.e., ξ_{sup}=0.05, 0.1, 0.15 and 0.2) corresponding to each value of α were selected. Thus, a total number of 420 SDOF systems with FVDs having different values of ξ_{sup} and α, and 21 SDOF systems without FVDs were considered. The IDA method was used to compute the collapse capacity of SDOF systems under the 44 far-field ground motion records in FEMA P695 [13]. The P-Δ effect was taken into account in modeling and cyclic deterioration was neglected. The modified Ibarra-Medina-Krawinkler model (Bilin) [30] was applied to model the nonlinear behavior of the systems. Figure 1 illustrates the moment-rotation relationship of the structural models.

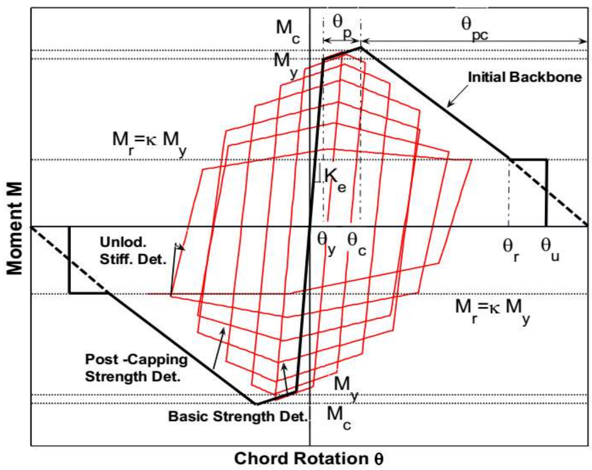
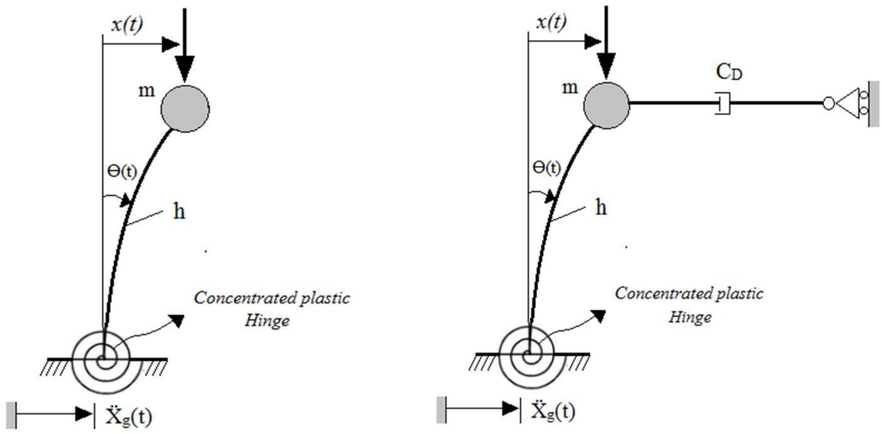


Figure 1: Modified Ibarra-Medina-Krawinkler model [30]

Concentrated plasticity approach was used to model the nonlinear behavior of the systems. Each system was modeled by an elastic beam-column element, and a zero-length rotational spring at its end. To model FVD, a horizontal two-node-link element connected to the mass of SDOF system was used, and a ViscousDamper material [31] was assigned to it. This material uses the Maxwell model [32], which consists of a dashpot and an elastic spring element in series. The input parameters for the ViscousDamper material are the damping coefficient C, velocity exponent, α, and an equivalent damper stiffness. In this study, a very large value was used as the damper stiffness. It should be mentioned that residual strength was assumed equal to zero in this study (i.e., κ=0). It was assumed that the collapse occurs when the drift ratio of the system reaches the collapse drift limit, which corresponds to the point of zero strength at the end of the softening branch of the back-bone curve. Figures 2(a) and 2(b) respectively illustrate SDOF systems without and with supplemental viscous damping.



(a) (b)

Figure 2: a) SDOF system without FVD and b) SDOF system with FVD

The supplemental viscous damping ratio for a SDOF system equipped with FVD can be obtained as:

$$\xi_{\text{sup}} = \frac{\lambda}{\pi} \frac{1}{2m\omega} \frac{C}{(\Omega u_0)^{1-\alpha}} \tag{2}$$

where u_0 is the displacement of the SDOF system, λ is a constant that is a function of the velocity exponent α, C is the damping coefficient of the SDOF system, m is the mass of the SDOF system, and Ω is the excitation frequency, which is assumed equal to the natural frequency of the SDOF system, ω. Given a supplemental viscous damping ratio, the damping coefficient of the SDOF system can be obtained as follows:

$$C = \frac{2\pi m \xi_{\text{sup}} \omega^{2-\alpha}}{\lambda u_0^{\alpha-1}} \tag{3}$$

To determine the damping coefficient of a SDOF system with nonlinear FVD, u_0 was assumed equal to the yield displacement of the system, δy. The yield displacement of the SDOF system was obtained by using pushover analysis. Five percent damped pseudo-spectral acceleration at the elastic period of each system, $Sa(T_{el})$, was used as the IM for performing IDAs. Ground motion records were scaled to several intensity levels until the collapse occurs for each SDOF system. Therefore, the intensities of ground motion records corresponding to the collapse, (i.e., 44 Sa_{col} values) were computed for each of the SDOF systems. Figure 3 indicates the IDA curves of the SDOF system with T=0.8, μ=4 and α=0.6 having a supplemental viscous damping ratio of 0.2.

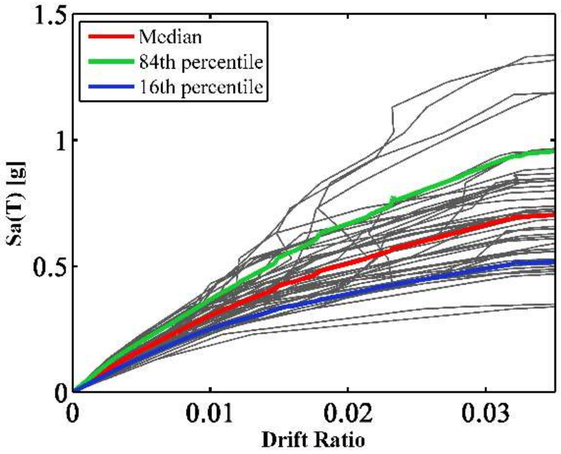


Figure 3: IDA curves of the SDOF system with T=0.8, μ=4 and α=0.6 having a supplemental viscous damping ratio of 0.2

3 INVESTIGATING THE EFFECT OF DUCTILITY AND SUPPLEMENTAL VISCOUS DAMPING RATIO ON THE COLLAPSE CAPACITY

In this section, the median collapse capacities of SDOF systems for different values of ductility and supplemental viscous damping ratio with respect to the elastic periods of the considered SDOF systems are presented. Each chart in Figure 4 corresponds to a specified supplemental viscous damping ratio and α=1. Ductility, $\mu=\delta_c/\delta_y$, varies between three values of 2, 4 and 8 in each chart, which corresponds to brittle, moderately-ductile and ductile structures, respectively. For a better comparison between the charts, the median collapse capacity of each SDOF system equipped with FVD was normalized to the median collapse capacity of its corresponding SDOF system without supplemental viscous damping, with only the 5% inherent viscous damping. It can be seen that, for a specified supplemental viscous damping ratio, the median collapse capacity of the system equipped with FVD normalized to the median collapse capacity of its corresponding SDOF system without supplemental viscous damping is not a function of ductility as the elastic period of SDOF system increases. Because the P-Δ effects control the collapse capacity of long period structures with and without supplemental viscous damping. According to the charts, by increasing the supplemental viscous damping ratio in structures, the median collapse capacities of SDOF systems increase.

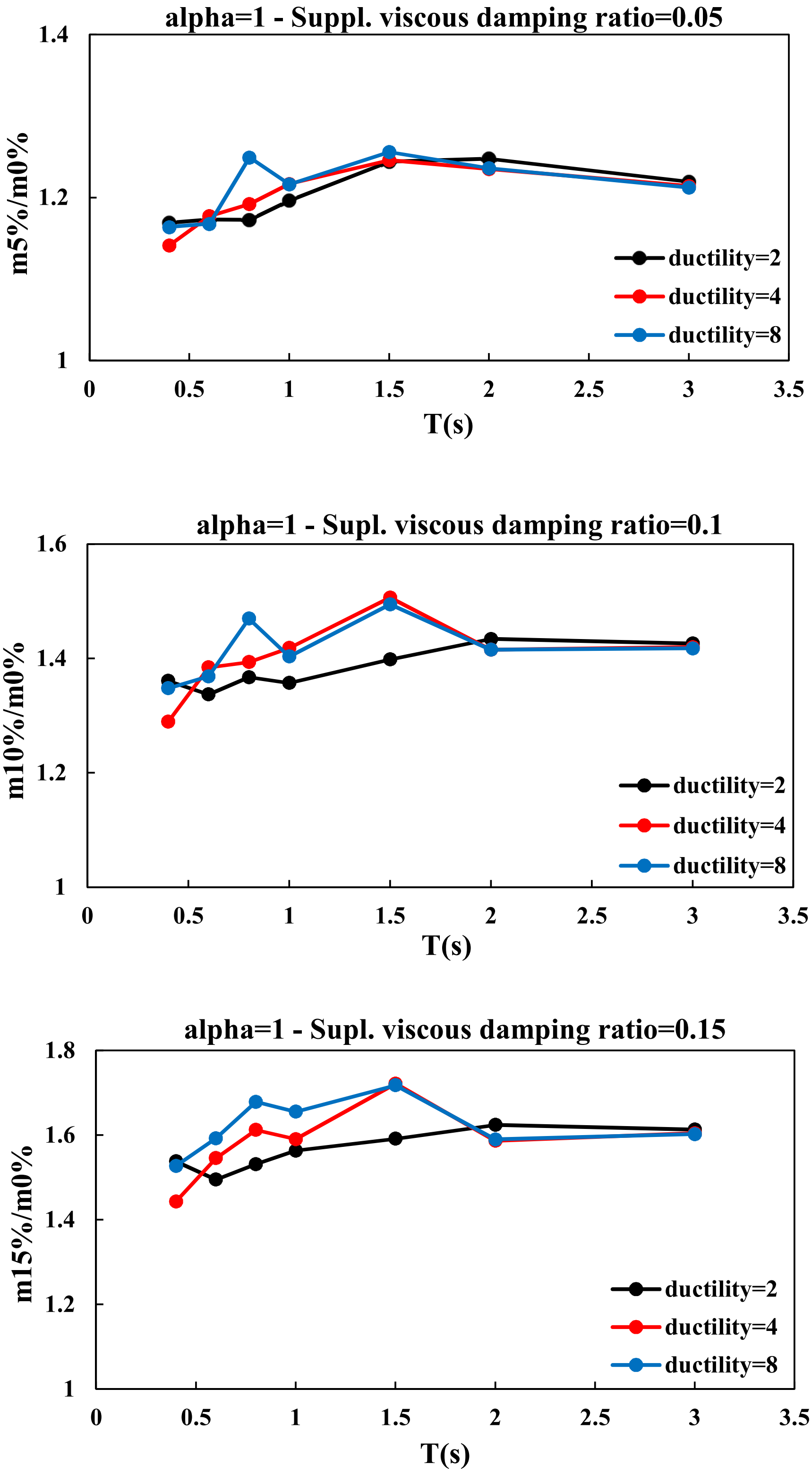


Figure 4. Median collapse capacity of SDOF systems equipped with FVDs normalized to the median collapse capacity of their corresponding SDOF systems without supplemental viscous damping, considering different ductility values, $\alpha = 1$

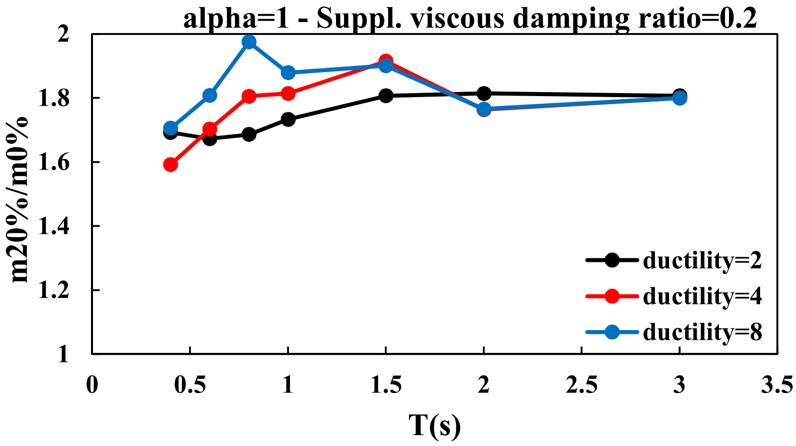


Figure 4. continued

4 CONCLUSIONS

In this study, the collapse capacity of SDOF systems equipped with FVDs under 44 far-field ground motion records were investigated. The results indicate that for a given level of supplemental viscous damping ratio and velocity exponent, by increasing the elastic period of SDOF system the median collapse capacity of the system equipped with FVD normalized to the median collapse capacity of its corresponding SDOF system without supplemental viscous damping is not a function of ductility. This is because of the fact that the P-Δ effects control the collapse capacity of long period structures with and without supplemental viscous damping.

References

[1] Vamvatsikos, D. and Cornell, C.A. (2002) "Incremental dynamic analysis. *Earthquake*" *Engineering & Structural Dynamics*, *31*(3), pp.491-514.

[2] Ramirez, O.M., et al. (2000) "Development and Evaluation of Simplified Procedures for Analysis and Design of Buildings with Passive Energy Dissipation Systems", MCEER Report 00-0010, Revision, 1, Multidisciplinary Center for Earthquake Engineering Research, University at Buffalo, State University of New York, Buffalo, NY, 470 pp. Multidisciplinary Center for Earthquake Engineering Research, University at Buffalo, State University of New York, Buffalo, NY.

[3] Symans, M.D. Charney, F.A. Whittaker, A.S. Constantinou, M.C. Kircher, C.A. Johnson, M.W. and McNamara, R.J. (2008) "Energy dissipation systems for seismic applications: current practice and recent developments",*Journal of Structural Engineering* 2008; 134(1): 3–21. DOI: 10.1061/ (ASCE) 0733-9445, 134:1(3).

[4] Soong, T.T. and Dargush, G.F. (1997) "Passive Energy Dissipation Systems in Structural Engineering", John Wiley & Sons: Chichester.

[5] Constantinou, M.C., Soong, T.T. and Dargush, G.F. (1998) "Passive energy dissipation systems for structural design and retrofit", Monograph Series, MCEER, University of Buffalo, NY.

[6] Dargush, G.F. and Sant, R.S. (2005) "Evolutionary aseismic design and retrofit of structures with passive energy dissipation", *Earthquake Engineering and Structural Dynamics* 2005; 34(13): 1601–1626. DOI: 10.1002/eqe.497.

[7] Hwang, J.S., Huang, Y.N., Yi, S.L. and Ho, S.Y. (2008) "Design formulations for supplemental viscous dampers to building structures", *Journal of Structural Engineering* 2008; 134(1): 22–31. DOI: 10.1061/ (ASCE) 0733-9445, 134:1(22).

[8] Silvestri, S., Gasparini, G. and Trombetti, T. (2010) "A five-step procedure for the dimensioning of viscous dampers to be inserted in building structures", *Journal of Earthquake Engineering* 2010; 14(3): 417–447. DOI: 10.1080/13632460903093891.
[9] ASCE (2010). *Minimum design loads for buildings and other structures (ASCE/SEI 7-10)*, American Society of Civil Engineers (ASCE), Reston, VA.
[10] Guo, J.W.W., and Christopoulos, C. (2013) "Performance spectra based method for the seismic design of structures equipped with passive supplemental damping systems", *Earthquake Engineering and Structural Dynamics* 2013; 42(6): 935–952. DOI:10.1002/eqe.2261.
[11] Christopoulos, C. and Filiatrault, A. (2006) "Principles of Passive Supplemental Damping and Seismic Isolation", IUSS Press: Pavia, Italy, 2006.
[12] Haselton, C.B.(2006) "Assessing collapse safety of modern reinforced concrete moment frame buidlings", *Ph.D. Thesis*, Department of Civil and Environmental Engineering, Stanford University, 2006.
[13] ATC. Quantification of building seismic performance factors, FEMA P-695.(2009) *Technical Report*, Applied Technology Council: Redwood City, California.
[14] FEMA. (2012) "FEMA P-58-1: Seismic Performance Assessment of Buildings. Volume 1–Methodology."
[15] Krawinkler, H., Zareian, F., Lignos, D.G. and Ibarra, L.F. (2009) "Prediction of collapse of structures underearthquake excitations", 2nd International Conference on Computational Methods in Structural Dynamics and Earthquake Engineering. (Papadrakakis M, Lagaros ND, Fragiadakis M. eds). June 22-24, 2009, Rhodes, Greece, CD-ROM paper, paper no. CD449.
[16] Ibarra, L. and Krawinkler, H. (2011) "Variance of collapse capacity of SDOF systems under earthquake excitations", *Earthquake Engineering and Structural Dynamics* 40:12, 1299-1314.
[17] Adam, C., Ibarra, L. F., and Krawinkler H. (2004) "Evaluation of P-Delta Effects in Nondeteriorating MDOF Structures from Equivalent SDOF Systems", *Proceedings of 13th World Conference on Earthquake Engineering,* Vancouver, B.C., Canada.
[18] Vamvatsikos, D., and Cornell, C. A. (2005) "Direct Estimation of Seismic Demand and Capacity of Multidegree-of-Freedom Systems Through Incremental Dynamic Analysis of Single Degree of Freedom Approximation", *Journal of Structural Engineering,* 131(4), 589–599.
[19] Sun, C.K. Berg, G.V. and Hanson, R.D. (1973) "Gravity effect on single-degree inelastic systems", *Journal of Engineering Mechanics, ASCE,* 99, 1, pages 183-200.
[20] Jennings, P. and Husid, R. (1968) "Collapse of yielding structures during earthquakes," *Journal of the Engineering Mechanics Division, ASCE,* 94, EM5, pages 1045-1065,Oct. 1968.
[21] Jäger, C. and Adam, C. (2013) "Influence of collapse definition and near-field effects on collapse capacity spectra", *Journal of Earthquake Engineering* 17.6: 859-878.
[22] Miranda, E.and Akkar, S.D. (2003) "Dynamic instability of simple structural systems.", Journal of Structural Engineering; 129:1722–1726.
[23] Constantinou MC, Symans MD. Experimental and analytical investigation of seismic response of structures with supplemental fluid viscous dampers. *Report No. NCEER-92-0032*, National Center for Earthquake Engineering Research, Buffalo, NY.
[24] Dong, B., Sause, R. and Ricles, J.M. (2016) "Seismic response and performance of a steel MRF building with nonlinear viscous dampers under DBE and MCE", *Journal of Structural Engineering* 2016; 142(6). DOI: 10.1061/ (ASCE) ST.1943-541X.0001482, 04016023.
[25]Syman, M.D. and Constatinou, M.C. (1998) "Passive fluid viscous passive systems for seismic energy dissipation", ISET J. Earthq. Technol. 35 (4), 185–206, Paper No. 382.
[26] Lee, D. and Taylor, D.P. (2001) "Viscous damper development and future trends", The Structural Design of Tall Buildings 2001; 10(5): 311-320
[27] Hwang, J., Tsai, C., Wang, S. and Huang, Y. (2006) "Experimental study of RC building structures with supplemental viscous dampers and lightly reinforced walls", Engineering Structures 2006; 28(13):1816–1824.
[28] Hamidia, M., Filiatrault, A. and Aref, A. (2014) "Simplified seismic collapse capacity-based evaluation and design of frame buildings with and without supplemental damping systems", *MCEER-14-0001*, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, NY.

[29] Mazzoni, Silvia, et al. "OpenSees command language manual." *Pacific Earthquake Engineering Research (PEER) Center* (2006).
[30] Lignos, D. G., H. Krawinkler, and A. S. Whittaker. (2011) "Prediction and validation of sidesway collapse of two scale models of a 4-story steel moment frame", *Earthquake Engineering & Structural Dynamics* 40, 807-825.
[31] Akcelyan, S. et al. (2016) "Evaluation of Simplified and State-of-the-Art Analysis Procedures for Steel Frame Buildings Equipped with Supplemental Damping Devices Based on E-Defense Full-Scale Shake Table Tests."*Journal of Structural Engineering* 142.6, 04016024
[32] Makris, N. and Constantinou, M.C. (1991) "Fractional-derivative Maxwell model for viscous dampers", *Journal of Structural Engineering*, *117*(9), pp.2708-2724.