8th International Conference on Behavior of Steel Structures in Seismic Areas
Shanghai, China, July 1-3, 2015

Seismic Design Criteria for Steel Moment Resisting Frames for Collapse Risk Mitigation

Ahmed Elkady* and Dimitrios G. Lignos*

* Dept. of Civil Eng. & Applied Mechanics, McGill University, Canada
e-mails: ahmed.elkady@mail.mcgill.ca, dimitrios.lignos@mcgill.ca

Keywords: Dynamic overstrength, Collapse risk, steel SMF, Strong-column weak-beam ratio.

Abstract. *This paper quantifies the lateral overstrength and the collapse risk of steel buildings with perimeter special moment frames (SMFs) designed in highly seismic regions in North America. State-of-the-art analytical models that consider the contributions of the composite concrete slab and the interior gravity framing to the lateral resistance and strength of steel frame buildings are employed. The findings demonstrate that the quantification of system overstrength based on dynamic analysis is more appropriate to nonlinear static analysis, since dynamic amplification of story shear forces due to higher mode effects is considered. Collapse risk is quantified using the mean annual frequency of collapse. It is found that low to mid-rise SMFs designed with a strong column weak-beam (SCWB) ratio larger than 1.0, achieve a probability of collapse larger than 1 percent in 50 years. A tolerable probability of collapse is achieved when a SCWB > 1.5 is implemented into the seismic design of steel SMFs.*

1 INTRODUCTION

A number of analytical and experimental studies related to the seismic behaviour of steel frame buildings demonstrated the need to re-assess the lateral system overstrength factor that is currently used in the seismic design of frame buildings in highly seismic regions in North America [1-3]. Quantification of seismic performance factors, such as lateral overstrength, is typically concerned with the proper simulation of the nonlinear response of the respective lateral load resisting system (LFRS). Prior analytical studies on steel frame buildings with perimeter special moment frames (SMFs) [1] showed that the overstrength factor calculated based on nonlinear static procedures can considerably vary from the one specified by ASCE/SEI 7-10 [4] (i.e., Ω_o=3.0). Moreover, it is becoming increasingly important to quantify the collapse risk of steel frame buildings when subjected to extreme earthquakes beyond the design level consideration [5, 6].

To properly quantify the system overstrength and collapse risk of steel SMFs, the lateral stiffness and strength contributions from structural elements other than the main LFRS should be considered. In particular, the composite concrete slab and the interior gravity framing system of steel frame buildings are main contributors to the steel frame building seismic performance [7]. These contributions have been typically ignored in past analytical studies as well as in the current seismic design practice in North America [1]. Based on past experimental studies [8, 9], the composite concrete slab can significantly increase the steel beam plastic flexural strength by 50% when the composite slab is in compression. Similarly, for simple shear connections of the interior gravity framing, past experimental studies [10, 11] showed that such connections can develop a flexural resistance up to 50% of the plastic flexural strength of the gravity beam. These aforementioned contributions could lead to a significant increase in the lateral overstrength of steel frame buildings [12].

The flexural stiffness and strength contributions provided by the composite slab and the gravity framing system can either benefit or impair the overall seismic performance of a steel frame building. Prior analytical studies [2, 13] showed that considering the gravity framing in the analytical model of a frame building can increase both its lateral stiffness and base shear strength as well as mitigate the lateral drifts of the LFRS. On the other hand, the amplified flexural strength of the steel beams due to the composite action may cause plastic hinging in the steel columns and consequently local story collapse mechanisms may develop. This was demonstrated recently in a full-scale collapse test of a 4-story steel building with steel SMFs [14].

To this end, a comprehensive analytical study is conducted on five archetype steel frame buildings with perimeter SMFs with heights ranging from 2 to 20 stories. State-of-the-art analytical models are developed considering both the effects of the composite action and the interior gravity framing system on the lateral resistance and flexural stiffness of the archetypes. Nonlinear static and response history analysis through collapse are employed to quantify realistic overstrength values as well as the collapse risk of the considered archetype steel frame buildings. The effect of the strong-column-weak-beam (SCWB) ratio on the overstrength and collapse risk of the archetype steel frame buildings with perimeter SMFs is quantified.

2 ANALYTICAL MODELING OF ARCHETYPE STEEL FRAME BUILDINGS

The analytical study discussed herein consists of five archetype steel buildings with perimeter SMFs and heights ranging from 2, 4, 8, 12, and 20 stories. The buildings are located in urban California and designed according to regional seismic provisions [15]. The perimeter SMFs are designed with reduced beam section moment connections (RBS) as discussed in [16]. Figures 1(a) and 1(b) show a typical plan view of the archetype buildings and the elevation view of the three-bay perimeter SMF of the 4-story building, respectively. The connections of the interior gravity framing system shown in figure 1(a) are designed as conventional single-plate shear-tab connections as per [17]. Extensive design details of the archetype buildings can be found in [18, 19].

Two-dimensional models of the perimeter SMFs in the EW direction are developed in OpenSEES [20] using a concentrated plasticity approach. The nonlinear springs simulating the hysteretic behavior of the fully restrained RBS connections utilize the modified Ibarra-Medina-Krawinkler (IMK) hysteretic model [21, 22]. This model is able to simulate the cyclic deterioration in strength and stiffness as well as the asymmetric hysteretic behavior resulting from the presence of a composite concrete slab. The input parameters of the backbone curve bounding the hysteretic response of composite beams with RBS are based on [18]. Figure 2(a) shows a sample calibration of the modified IMK model with past experimental data of a composite beam with RBS [9]. This figure indicates the significance of simulating deterioration in structural components for predicting the collapse potential of frame buildings during earthquakes [23].

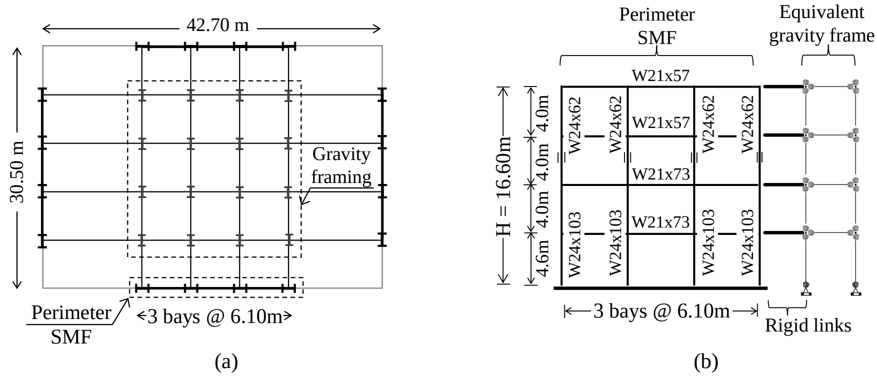


Figure 1. (a) Plan view of a typical archetype steel frame building; (b) elevation view of the four-story perimeter SMF showing the gravity framing representation.

The equivalent gravity frame [2] is used to model the interior gravity framing system as shown in figure 1(b). The one bay equivalent gravity frame is connected to the main SMF model through axially rigid links. This frame has a lateral stiffness and strength properties equivalent to those of the interior gravity framing system. The beam-to-column connections of the equivalent gravity frame simulate the hysteretic behaviour of the simple shear-tab connections of the interior gravity framing system. The Pinching4 hysteretic model [24] is used to simulate the pinched force-deformation hysteretic response of typical shear-tab connection. The input parameters of the Pinching4 model are obtained as proposed by [19]. Figure 2(b) shows a sample calibration of the Pinching4 model with past experimental data of a composite shear-tab connection [10] including beam binding to the gravity column.

The perimeter SMF in the EW direction of each archetype building is modeled using four different analytical configurations: (a) considering only the bare steel properties of the SMF (i.e., B model); (b) considering the composite slab effect when modelling the SMF (i.e., C model) (c) considering the bare gravity frame in the analytical model (i.e., BG-model) (d) considering both composite slab and the gravity framing in the analytical model (i.e., CG model).

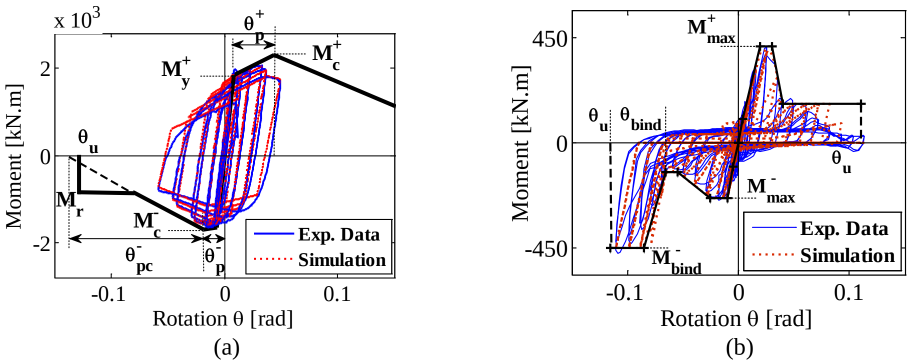


Figure 2. (a) Modified IMK model calibrated with composite beam with RBS (experimental data from [9]); (b) Pinching4 model calibrated with composite shear-tab connection (experimental data from [10]).

3 RESULTS AND DISCUSSION

3.1 Evaluation of Static and Dynamic Lateral Overstrength

Modal analysis is first conducted for the four analytical models of the five archetype steel frame buildings. Table 1 summarizes the predominant period of each model. The lower period values of the C and BG models compared to the B model demonstrate the additional flexural stiffness provided to the bare SMFs when the composite action and/or the gravity framing effects are considered in the analytical model of the steel frame building. Obviously, the CG model has the smallest lowest period between the other analytical models due to the combined composite and the interior gravity framing actions.

Next, nonlinear static analysis is conducted using the first mode shape of each analytical model as a lateral load pattern. Figure 3(a) shows a comparison of the base shear force-roof displacement relation of the four analytical models of the 4-story archetype building. The base shear force V_1 is normalized with respect to the seismic weight of the building W. The roof displacement δ_r is normalized with respect to the total height of the building H. The nonlinear static curves show that, in average, the composite action and the gravity framing increases the lateral strength of a steel frame building with perimeter SMFs by 20% and 15%, respectively. When both the composite and the gravity framing actions (i.e., CG models) are considered, these result in a 50% increase in the lateral strength compared to the B models.

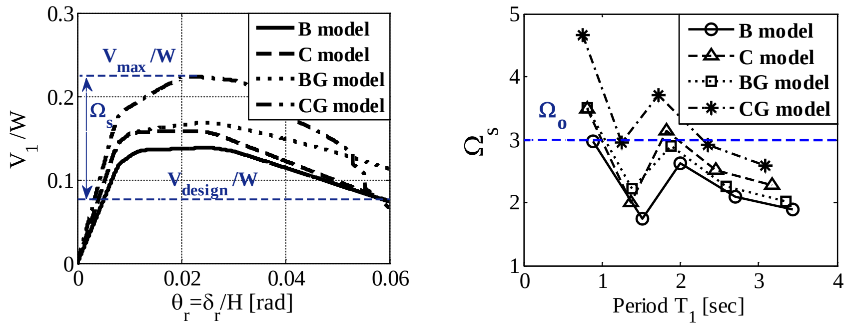
The lateral strength increase is further evaluated through the quantification of the static overstrength factor, Ω_s. The static overstrength factor Ω_s is calculated as the ratio of the maximum base shear force

V_{max} to the design base shear force V_{design} in accordance with [4]. The static overstrength factor, for the CG model of the 4-story archetype building, is illustrated in figure 3(a). The values of the static overstrength factor for all the archetype buildings are summarized in table 1. For steel SMFs, the current seismic code in the US, ASCE-7-10 [4], specifies an Ω_o=3.0. However, all the bare SMFs (i.e., B models) have an Ω_s < 3.0. Except for the 20-story building, an overstrength factor ≥ 3.0 is only achieved when both the composite action and the gravity framing are considered in the analytical model (see figure 3b). Figure 3(b) also shows that, considering the composite action and the gravity framing contributions, the current overstrength factor of 3.0 specified by [4] might be over conservative for low-rise steel frame buildings. Note that for the 2-story archetype steel building, an Ω_s > 4.0 is achieved.

Furthermore, figure 3(b) demonstrates the dependency of the static overstrength factor on the structure's predominant period. Taller buildings achieve lower overstrength values compared to low-rise ones. It is understood that the adequacy of the currently employed static overstrength factors for estimating the force demands in force-controlled elements during earthquake shaking becomes questionable [1]. Consequently, nonlinear response history analysis is conducted for the archetype steel buildings in order to further evaluate their overstrength.

Table 1. First-mode period, static, and dynamic overstrength factors for all archetype steel frame buildings.

No. of Stories	T_1 [sec]				Ω_s				Ω_d			
	B	C	BG	CG	B	C	BG	CG	B	C	BG	CG
2	0.88	0.81	0.82	0.76	2.98	3.48	3.51	4.66	3.45	3.50	3.71	4.22
4	1.51	1.37	1.38	1.25	1.75	2.00	2.21	2.95	3.02	3.21	3.30	3.60
8	2.00	1.82	1.89	1.72	2.63	3.13	2.89	3.71	3.93	4.32	4.20	4.70
12	2.70	2.46	2.58	2.35	2.09	2.52	2.25	2.92	3.40	3.75	4.00	4.13
20	3.44	3.17	3.35	3.08	1.89	2.27	2.02	2.59	3.69	4.34	3.84	4.73



(a) (b)

Figure 3. (a) Comparison of the pushover curves of the analytical models of the 4-story archetype building; (b) comparison of the static overstrength factor versus the first mode period for all analytical model configurations.

Incremental dynamic analysis (IDA) [25] is conducted using the 44 records of the Far-Field ground motion set described in [5]. For each analytical model of a given archetype building, each ground motion is scaled with respect to the first mode, 5% damped, spectral acceleration of the associated B model until collapse occurs. A steel SMF collapses when a number of its stories displaces sufficiently and the corresponding story shear capacity reaches zero due to P-Delta effects accelerated by component deterioration in flexural strength and/or stiffness. The IDA results are used to quantify the dynamic overstrength factor and the collapse risk as discussed in the next section.

The dynamic overstrength factor, Ω_d, is computed as the ratio of the maximum dynamic base shear force to the design base shear force. The dynamic overstrength factor is illustrated in figure 4(a) in which the normalized dynamic base force is plotted versus the first story drift ratio SDR_1 for the CG model of the 4-story building when subjected to a ground motion record scaled to the collapse intensity. The pushover curve of the same CG model based on its first mode lateral load pattern is also superimposed in figure 4(a) to demonstrate the difference between the dynamic and static overstrength factors. The ratio of the dynamic-to-static overstrength factors is plotted versus the first-mode period of the various analytical models of the archetype steel frame buildings in figure 4(b). This figure shows that the ratio of dynamic-to-static overstrength is lowest at shorter periods and highest at larger periods (i.e., taller buildings). This is due to the dynamic amplification of story shear forces due to higher-mode effects [26]. Furthermore, the dynamic overstrength values, which are summarized in table 1, show no period dependency. An average dynamic overstrength value of 4.0 is achieved for the CG models of all the archetype steel buildings. Consequently, the dynamic overstrength can be potentially employed as part of the seismic design process of steel frame buildings subjected to earthquake shaking.

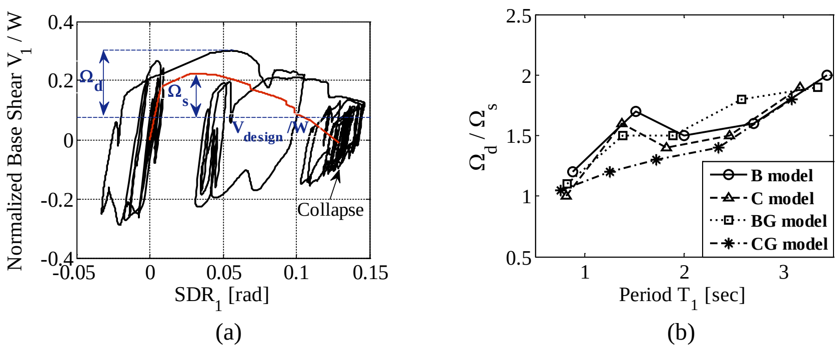


Figure 4. (a) Normalized base shear force versus first story drift ratio at collapse intensity for the CG model of the 4-story frame building; (b) ratio of dynamic to static overstrength factor versus first-mode period of different analytical models of the archetype steel buildings.

3.2 Collapse Risk Assessment

The collapse risk of the archetype steel frame buildings is quantified using the mean annual frequency of collapse λ_c as discussed in [27]. The mean annual frequency of collapse is a reliable collapse metric because it considers all the spectral intensities contributing to the collapse risk of the respective frame building. The collapse fragility curve, of each analytical model configuration, is constructed from the 44 collapse intensities obtained by IDA. The seismic hazard curves are obtained from the USGS website (2008 update of the US national seismic hazard maps).

The mean annual frequency of collapse is then used to calculate the corresponding probability of collapse in a return period of 50 years P_c(50 years). In figures 5(a and b), both λ_c and the corresponding P_c(50 years) for the C and CG models of all the archetype buildings, respectively, are plotted against the SCWB ratio implemented in the design of the steel SMFs. Figures 5(a and b) show that when a SCWB ratio $\geq$ 1.0 is implemented as part of the SMF design, as per [15], mid-rise steel frame buildings achieve a probability of collapse larger than 1% in 50 years (i.e., acceptable limit specified in [4]). More recently, the authors [18] demonstrated that when the composite action is considered as part of the analytical model representation of a steel frame building with perimeter SMFs, when a SCWB ratio $\geq$ 1.0 is employed, the respective steel SMFs experience weak story collapse mechanisms and excessive panel zone shear distortion, which may potentially lead to weld fractures in fully restrained beam-to-column connections. The same study illustrated that the aforementioned issues can be avoided if the steel SMFs are designed using a SCWB ratio $\geq$ 1.5. In addition, when a SCWB ratio $\geq$ 1.5 is employed as part of the design process of the same steel SMFs, they achieve a probability of collapse less than 1.0% in a return period of 50 years as shown in figure 5(b). For SMFs designed with SCWB ratio $\geq$ 2.0, a uniform probability of collapse of about 0.25% is achieved in 50 years. Note that when the effect of the interior gravity framing is considered on the lateral strength and stiffness of the analytical model representation of the archetype steel frame buildings with perimeter SMFs, only when a SCWB > 1.5 is employed as part of the design process of the steel SMFs, a probability of collapse less than 1% over 50 years can be achieved (see figure 5b). Further details regarding these findings can be retrieved from [19].

Implementing a higher SCWB ratio as part of the design process of steel SMFs would typically result in heavier column sections. For SCWB ratios larger than 1.5 and 2.0, the column steel weight increases by 149 to 298 kg/m (100 to 200 lbs/ft), respectively [18]. However, using heavier column sections, with thicker webs, would result in a reduction in the thickness and the number of welded doubler plates required to control the panel zone strength as per [17]. The number of doubler plates is reduced by an average of 29% and 58% when a SCWB ratio larger than 1.5 and 2.0 is implemented in the design, respectively. This reduction in the number of doubler plates translates into a fabrication cost reduction associated with welding of the doubler plates to the column web. In addition, a stockier column will potentially prevent or at least delay deteriorating mechanisms such as column axial shortening that is currently not considered as part of the seismic design process for steel columns [28, 29].

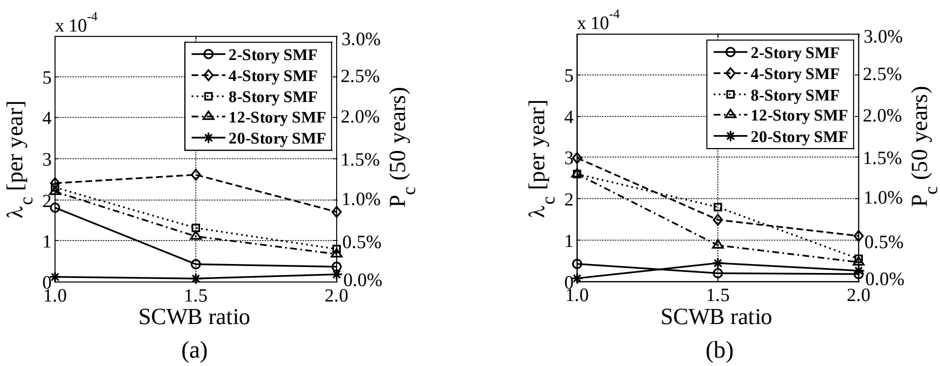


Figure 5. Mean annual frequency of collapse and the corresponding probability of collapse in 50 years versus SCWB ratio for the (a) C models and (b) CG models of all archetype frame building in the EW loading direction.

4 CONCLUSION

This paper summarizes a comprehensive analytical study related to the collapse risk of steel frame buildings with perimeter special moment frames (SMFs) designed in North America. Five archetype steel buildings with heights ranging from 2 to 20 stories were designed as per the current seismic code in the US. Rigorous nonlinear static and response history analyses are conducted using 2-dimensional (2D) analytical models that consider the flexural stiffness and strength contributions of the composite action and the interior gravity framing. The first goal of the paper was to propose more robust overstrength measures that may be potentially used as part of the seismic design process of steel SMFs. the second goal of the paper was to propose an effective value of the strong-column-weak-beam (SCWB) ratio that can be employed as part of the design process of steel SMFs in order to minimize their collapse risk over the archetype building life expectancy. The main conclusions of this paper are summarized as follows:

1. Bare steel SMFs develop a dynamic overstrength factor, Ω_d, larger than 3.0. When the composite action and the gravity framing are considered, the dynamic overstrength is in average equal to 4.0 without any period dependency. The dynamic overstrength captures the inelastic force redistribution due to dynamic loading in steel frame buildings; thus it can be used to reliably measure the system overstrength and to evaluate the reliability of force-sensitive components in steel frame buildings subjected to earthquake loading.

2. When both the composite and interior gravity framing action is considered as part of the analytical model representation of steel frame buildings with perimeter steel SMFs, low- to mid-rise SMFs, designed as per [15] (i.e., SCWB ratio ≥ 1.0), achieve a probability of collapse, in a return period of 50 years, larger than the 1% limit specified by the current seismic provisions [4]. A probability of collapse less than 1% in a return period of 50 years is achieved only when the SMFs are designed with a SCWB ratio ≥ 1.5. This ratio results in heavier column sections, however, the number of doubler plates required by the design provisions [17] are significantly reduced and therefore the fabrication cost associated with the welding of these plates is also significantly reduced.

ACKNOWLEDGEMENTS

This study is based on work supported by the National Science and Engineering Research Council of Canada (NSERC) under the Discovery Grant Program. Funding is also provided by the Steel Structures Education Foundation (SSEF). This financial support is gratefully acknowledged. Any opinions, findings, and conclusions or recommendations expressed in this paper are those of the authors and do not necessarily reflect the views of sponsors.

REFERENCES

[1] NIST. "Tentative framework for development of advanced seismic design criteria for new buildings". *NIST GCR 12-917-20*, NEHRP consultants Joint Venture, 2012.

[2] Gupta, A. and Krawinkler, H. "Seismic demands for the performance evaluation of steel moment resisting frame structures". *Report No. 132*, The John A. Blume Earthquake Engineering Center, Stanford University, CA, 1999.

[3] Whittaker, A., Hart, G., and Rojahn, C. "Seismic response modification factors". *Journal of Structural Engineering*; 125(4): 438-444, 1999. DOI:10.1061/(ASCE)0733-9445(1999)125:4(438).

[4] ASCE. *Minimum design loads for buildings and other structures, ASCE/SEI 7-10*. American Society of Civil Engineers: Reston, VA., 2010.

[5] FEMA. "Quantification of building seismic perofrmance factors". *Report FEMA-P695*, Federal Emergency Management Agency, Washington, DC, 2009.

[6] Ibarra, L.F. and Krawinkler, H. "Global collapse of frame structures under seismic excitations". *Report No. 152*, The John A. Blume Earthquake Eng. Center, Stanford University, CA, 2005.

[7] Lignos, D.G. and Krawinkler, H. "Sidesway collapse of deteriorating structural systems under seismic excitations". *Report No. TB 177*, The John A. Blume Earthquake Eng. Center, Stanford University, CA, 2012.

[8] Engelhardt, M.D., Fry, G., Johns, S., Venti, M., and Holliday, S. "Behavior and design of radius-cut reduced beam section connections". *Report No. SAC/BD-00/17*, SAC Joint Venture, Sacramento, CA, 2000.

[9] Ricles, J.M., Zhang, X., Fisher, J.W., and Lu, L.W. "Seismic performance of deep column-to-beam welded reduced beam section moment connections". *Proc. of The 5th International Workshop Connections in Steel Structures V: Behaviour, Strength and Design*. Amestrdam, Netherlands, 2004.

[10] Liu, J. and Astaneh-Asl, A. "Cyclic testing of simple connections including effects of slab". *Journal of Structural Engineering*; 126(1): 32-39, 2000. DOI: 10.1061/(ASCE)0733-9445(2000)126:1(32).

[11] Green, T., Leon, R., and Rassati, G. "Bidirectional tests on partially restrained, composite beam-to-column connections". *Journal of Structural Engineering*; 130(2): 320-327, 2004.

[12] Ramos, M.D., Mosqueda, G., and Lignos, D.G. "Hybrid simulation of the seismic response of a steel moment frame building structure through collapse". Multidisciplinary Center for Earthquake

Research (MCEER), University at Buffalo, State University of New York, NY, 2014.

[13] Ji, X., Kato, M., Wang, T., Hitaka, T., and Nakashima, M. "Effect of gravity columns on mitigation of drift concentration for braced frames". *Journal of Constructional Steel Research*; 65(12): 2148-2156, 2009. DOI: 10.1016/j.jcsr.2009.07.003.

[14] Suita, K., Yamada, S., Tada, M., Kasai, K., Matsuoka, Y., and Shimada, Y. "Collapse experiment on 4-story steel moment frame: Part 2 detail of collapse behavior". *Proc. of The 14th World Conference on Earthquake Engineering (WCEE)*. Beijing, China, 2008.

[15] AISC. *Seismic provisions for structural steel buildings, ANSI/AISC 341-10*. American Institute for Steel Construction: Chicago, IL, 2010.

[16] AISC. *Prequalified connections for special and intermediate steel moment frames for seismic applications, ANSI/AISC 358-10*. American Institute for Steel Construction: Chicago, IL, 2010.

[17] AISC. *Specification for structural steel buildings, ANSI/AISC 360-10*. American Institute for Steel Construction: Chicago, IL, 2010.

[18] Elkady, A. and Lignos, D.G. "Modeling of the composite action in fully restrained beam-to-column connections: implications in the seismic design and collapse capacity of steel special moment frames". *Earthquake Engineering & Structural Dynamic*; 43(13): 1935-1954, 2014.

[19] Elkady, A. and Lignos, D.G. "Effect of gravity framing o the overstrength and collapse capacity of steel frame buildings with perimeter special moment frames". *Earthquake Engineering & Structural Dynamic*, 2014. DOI: 10.1002/eqe.2519.

[20] Mckenna, F.T. "Object-oriented finite element programming: Frameworks for analysis, algorithms and parallel computing". *Ph.D.* Department of Civil Engineering, University of California, 1997.

[21] Lignos, D.G. and Krawinkler, H. "Deterioration modeling of steel components in support of collapse prediction of steel moment frames under earthquake loading". *Journal of Structural Engineering*; 137(11): 1291-1302, 2011. DOI: 10.1061/(ASCE)ST.1943-541X.0000376.

[22] Ibarra, L. "Global collapse of frame structures under seismic excitations". *Ph.D. Dissertation*, Department of CEE, Stanford University, 2003.

[23] Krawinkler, H., Zareian, F., Lignos, D.G., and Ibarra, L.F., "Significance of modeling deterioration in structural components for predicting the collapse potential of structures under earthquake excitations", *Advances in performance-based earthquake engineering*. 2010. p. 173-181.

[24] Lowes, L.N., Mitra, N., and Altoontash, A. "A beam-column joint model for simulating the earthquake response of reinforced concrete frames". *Report No. 2003/10*, Pacific Earthquake Engineering Research Center (PEER), College of Engineering, University of California, 2003.

[25] Vamvatsikos, D. and Cornell, C.A. "Incremental dynamic analysis". *Earthquake Engineering & Structural Dynamics*; 31(3): 491-514, 2002. DOI: 10.1002/eqe.141.

[26] Lignos, D.G., Putman, C., and Krawinkler, H. "Application of simplified analysis procedures for performance-based earthquake evaluation of steel special moment frames". *Earthquake Spectra*, 2015. DOI: 10.1193/081413EQS230M.

[27] Eads, L., Miranda, E., Krawinkler, H., and Lignos, D.G. "An efficient method for estimating the collapse risk of structures in seismic regions". *Earthquake Engineering & Structural Dynamics*; 42(1): 25-41, 2013. DOI: 10.1002/eqe.2191.

[28] Elkady, A. and Lignos, D.G. "Analytical investigation of the cyclic behavior and plastic hinge formation in deep wide-flange steel beam-columns". *Bulletin of Earthquake Engineering*: 1-22, 2014. DOI: 10.1007/s10518-014-9640-y.

[29] Suzuki, Y. and Lignos, D.G. "Large scale collapse experiments of wide flange steel beam-columns". *Proc. of The 8th International Conference on Behavior of Steel Structures in Seismic Areas*. Shanghai, China, 2015.