The 2020 World Congress on
The 2020 Structures Congress (Structures20)
25-28, August, 2020, GECE, Seoul, Korea

Seismic performance of steel moment frames designed using modal response spectrum analysis method

*Taeo Kim[1], Sang Whan Han[2], and Soo-Ik Cho[1]

[1], [2] Department of Architectural Engineering, Hanyang University, Seoul, Korea
[2] swhan82@gmail.com

ABSTRACT

New building structures should meet target seismic performance specified in seismic codes against big seismic events. ASCE 7 (2017) states that new ordinary buildings should have a collapse probability against maximum considered earthquake ground motions less than 10%. The objective of this study is to estimate the collapse probabilities of steel special moment frames (SMFs) designed using the modal response spectrum analysis (MRSA) method provided in in ASCE 7. For this purpose, 4-, 8-, and 16-story steel SMFs are designed using MRSA method. The collapse probability of these structure is estimated according to the procedure specified in FEMA P695 (2009). It is observed that four-story SMF has a collapse probability less than 10% whereas eight- and sixteen-story SMFs have collapse probabilities exceeding 10%.

1. INTRODUCTION

The current seismic design criteria, ASCE 7 (2017), aims to achieve a seismic performance level according to the risk category for design seismic loads applied to the target structure, and it is required to secure the safety and usability of structures against ground motions in order to achieve the seismic performance. For this purpose, ASCE 7 (2017) proposes method for calculating design spectral acceleration, analysis procedure, and design procedures according to the location and site condition to ensure the seismic performance objective. In ASCE 7 (2017), the seismic performance objective required for structures are basically to maintain the functionality for low-risk seismic loads and to prevent collapse for high-risk seismic loads. In addition, it aims to reduce the social impact and damage caused by seismic loads or to ensure the safety of life by varying the performance objective required for structures according to their risk category.

ASCE 7 (2017) proposes an equivalent lateral force (ELF) procedure, modal response spectrum analysis (MRSA), and response history analysis (RHA) procedure to predict the response to a structure for structural member due to the ground motion applied to the structure. Among the proposed analysis procedures, the MRSA is relatively simple because no ground motion selection procedure is required, and has the advantage of being widely used in practice due to the fact that there are no

---

[1] Graduate Student
[2] Professor

restrictions due to seismic design category or structural heights.

However, the vulnerabilities have been mentioned through several studies on the seismic performance of structures designed using the MRSA (NIST 2010; NIST 2012; Harris 2018). In the NIST (2010), the design cross-section of the structure where the seismic design was performed using the MRSA has an economical advantage because the size of cross-section is relatively small compared to the structure designed using the ELF procedure, but it is relatively vulnerable compared to the structures designed by the ELF procedure. Based on these studies, ASCE 7 (2017), which is the current seismic design criteria, revises the criteria for MRSA so that the design is performed on the same level of base shear force as ELF using scale factor according to MRSA.

This study intends to evaluate the validity of the standard revision through the seismic performance evaluation of the steel special moment frame (SMF) designed using the revised MRSA. To this end, steel SMF of low-, med-, and high-stories was designed and seismic performance evaluation according to FEMA P695 (2009) was performed. Also, based on the collapse mechanism of steel SMFs for static and dynamic analysis, a direction was proposed to secure seismic performance.

2. SEISMIC DESIGN

In this study, in order to evaluate the seismic performance of steel SMF designed using MRSA, 4-, 8-, and 16-story steel SMF are designed. The target frame assumed to be a office building of risk category I/II, and the importance factor (I_e) is assigned to 1.0. The plan, elevation and designed cross-section of the steel SMF are shown in Fig. 1. The load and other design condition except deflection amplification factor (C_d) used for target building are defined according to NIST (2010), which includes examples for general office building. All structures have the same plan (Fig. 1(a)), and it is assumed that the moment frame has 3 spans on the perimeter of building. The span of the moment frame was defined as 6.1 m each, and the floor height was defined as 4.6 m on the first floor and 4.0 m on the other floors. The response modification factor (R), system overstrength factor (Ω_0), and deflection amplification factor (C_d) for steel SMF are 8.0, 3.0, and 5.5 proposed by ASCE 7 (2017), respectively. The design of structural members is performed according to AISC 360 (2016) and AISC 341 (2016).

The design spectral acceleration was used to calculate the acceleration with the largest size within the designable range and applied it as a seismic loads. Accordingly, it was assumed that the site class corresponds to C, and the design spectral acceleration at short (S_{DS}) and 1 second (S_{D1}) are assigned 1.0 g and 0.56 g, respectively. Accordingly, the seismic design category corresponds to D. ASCE 7 (2017) requires the use of site-specific ground motion procedures depending on the spectral acceleration and site class. In this study, the effect of the site-specific ground motion procedure was excluded from consideration in this study. Seismic design of target structures is performed using ETABS software (2018), and the design results are presented in Fig. 1(b), (c), and (d).

The periods of 4-, 8-, and 16-story steel SMF are evaluated to be 1.6, 2.7, and 4.2 second, respectively. Steel SMF, which was designed using MRSA, was subjected to force-controlled design regardless of structural height, and the cross-section of the beam was determined purely by strength. Unlike the case of the beam, the cross sections of interior column are determined according to the beam-to-column moment ratio criterion, and cross-sections of exterior column are determined to resist the axial force caused by the overstrength seismic load.

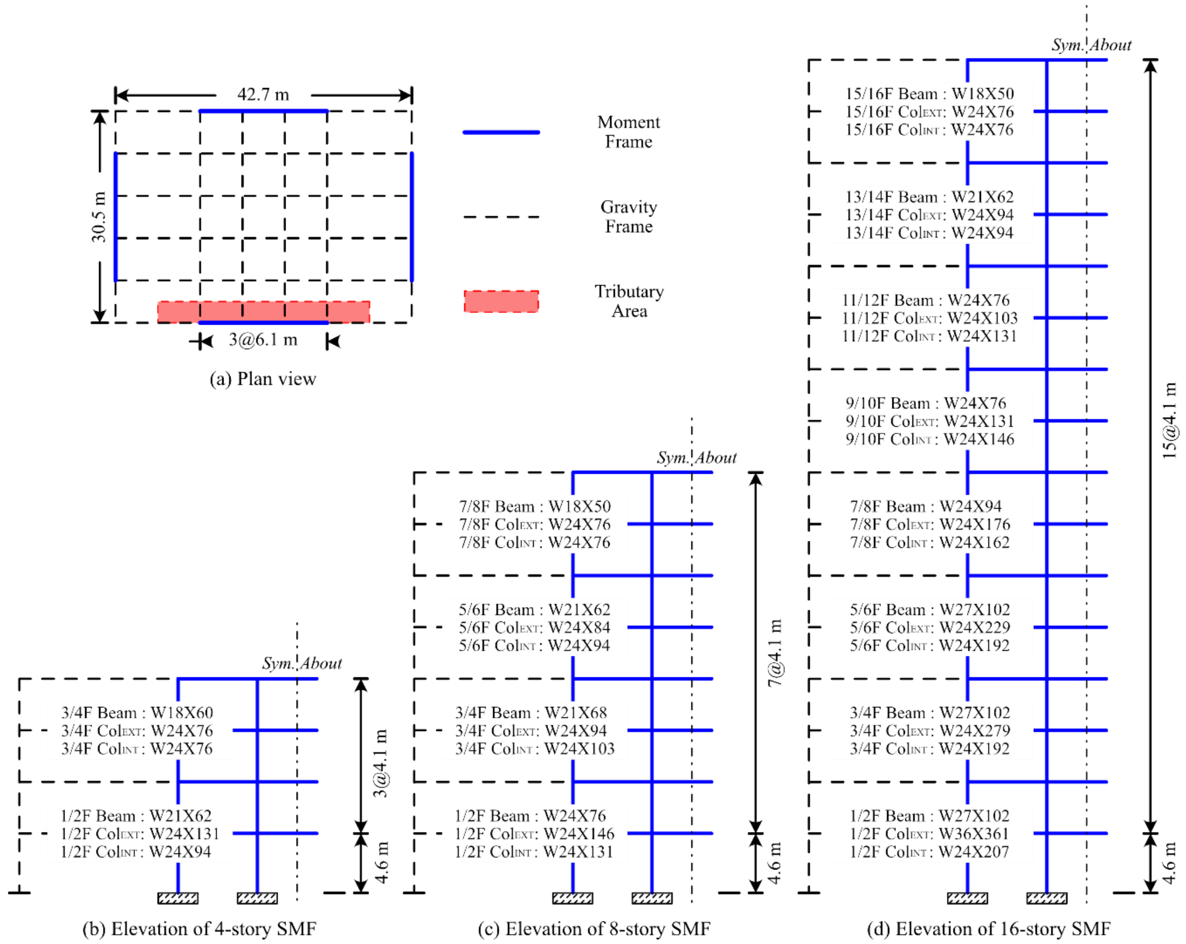

Fig. 1 Plan, elevation, and designed section of 4-, 8-, and 16-story steel SMFs.

3. Numerical model of steel SMF

In order to evaluate the seismic performance of target structure for ground motion, a two-dimensional numerical model of steel special moment frame was constructed using OpenSees software (McKenna 2011). As shown in Fig. 2, a numerical model was constructed to predict the nonlinear response of the steel SMFs. The numerical model consists of a structural member composed of panel zones, beams, and columns, and a leaning column to simulate the effect of the load applied to the gravity frame. Each beam and column simulated the nonlinear behavior using an elastic element representing the length of the member and a plastic hinge located at the ends of both ends. To simulate the nonlinear behavior of beam and column, the concentrated plastic hinge is constructed using modified IMK model (Ibarra 2005) (Fig. 2(b)). In addition, the strength, deformation, and deterioration parameter presented in Lignos (2011) are used for construct plastic hinge. The PM reduction in the columns was performed using the formula presented in AISC 360 (2016) according to the procedure proposed in Zareian (2010). In addition, shear deformation due to shear force generated in the panel zone of the steel SMF is simulated using eight rigid elements and a tri-linear nonlinear speing according to the procedure proposed in ATC 72-1 (2009) (Fig. 2(c)).

In order to simulate the lateral force generated by ground motion, the lumped masses are assigned corresponding to $1.05D + 0.25L$ at each connection, where D and L mean the dead and live load, respectively. In addition, the damping matrix of the target structure is assigned to lumped mass and elastic element using Rayleigh damping having a damping ration of 2% for the 1st and 5th modes (Zareian 2010). To simulate the effect of gravity load applied to the gravity frame out side the tributary area of moment frame, a leaning column connected to the moment frame is constructed with a rigid truss element, and the vertical load applied to the leaning column in the form of a concentrated load as shown in Fig. 2(a).

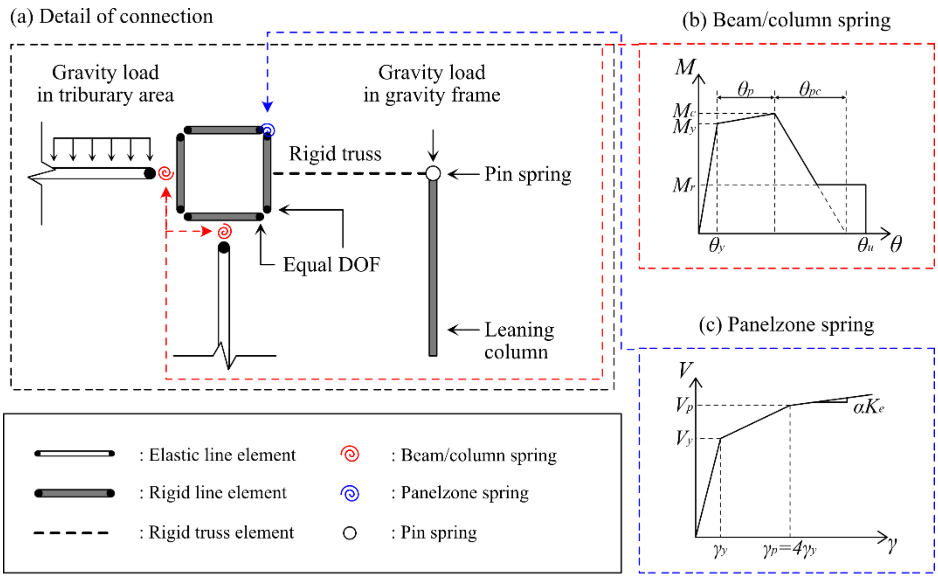

Fig. 2 Detail of numerical model and nonlinear spring of steel SMFs.

4. Seismic performance evaluation

ASCE 7 (2017) proposes a design procedure to ensure uniform risk regardless of the system, site, and site condition under which the design is performed. Additionally, the target seismic performance in performance-based design is suggested as form of a conditional collapse probability (P_c) of collapse given maximum considered earthquake (MCE). In this study, seismic performance evaluation for steel SMFs is performed using FEMA P695 (2009) methodology to calculate the P_c of structures. FEMA P695 (2009) requires nonlinear static (pushover) analysis and incremental dynamic analysis (IDA; Vamvatsikos 2002) as a procedure for performing seismic performance evaluation.

Pushover analysis is performed using the lateral load according to distribution of 1st mode shape. The parameters overstrength factor (Ω) and period-based ductility (μ_T)

can be calculated using Eqs. 1 and 2 as a result of pushover analysis, and the results are shown in Fig. 3. As shown in the figure, the Ω and μ_T of steel SMFs tended to decrease with increasing number of stories. In addition, the Ω is evaluated to have a value lower than 3.0, which is the system overstrength factor (Ω_0) proposed for steel SMF in ASCE 7 (2017).This means that the steel SMFs that was designed using MRSA does not exhibit the intended maximum base shear strength regardless of height. In addition, the decrease in Ω and μ_T as the height increases indicates that the collapse rapidly occurs before the wide range of damages intended by the seismic design criteria occur.

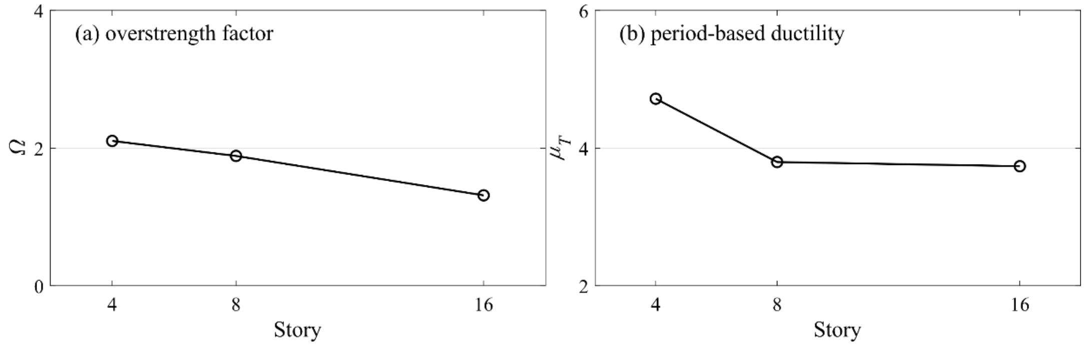

Fig. 3 Overstrength factor (Ω) and period-based ductility (μ_T) based on pushover.

Fig. 4 shows the median IDA curve resulting from IDA according to the procedure proposed in FEMA P695 (2009), IDA curve for specific ground motion, and P_c of steel SMFs. Fig. 4a shows the median IDA curve as a result of IDA. In the figure, the horizontal and vertical axes represent the maximum inter-story drift ratio (θ_{max}) and intensity of ground motion at upper limit period, T. Where, T is C_uT_a as the upper limit of the period according to ASCE 7 (2017). According to the median IDA curve, it was evaluated that it showed an elastic response regardless of the structural height until the θ_{max} reached 0.04 rad. A sudden collapse occurred after yielding in all steel SMFs, and this phenomenon was severe as the structural height increased. It is thought that the sway mechanism occurred as the P-Delta effect increased due to the increase in the lateral displacement of the structure. This tendency also occurred in the Ida curve for the specific ground motion shown in Fig. 4b.

As a result of seismic performance evaluation of steel SMF shown in Fig. 4c, it was evaluated that the P_c increase as the structural height increased. Due to this tendency, the 4-story steel SMF which is a low-rise building was evaluated as having a target seismic performance, but, 8- and 16-story steel SMF which mid- and high-rise building did not achieve the target seismic performance required by ASCE 7 (2017).

Figs 4d, e, and f show the distribution of plastic hinges near collapse due to a specific ground motion in Fig. 4b. as shown at the top of the center in Fig. 4, black, blue, and red marks indicate yield strength, maximum strength, and fracture, respectively, and show the damage of the nonlinear spring located in all structural members. As shown in the figure, the structure that was designed using MRSA was evaluated to have a distribution of damage to the columns and beams throughout the structure. In addition, it was evaluated that the damage to the beam was wider than that of the column, which is the intended result by the moment ratio criterion in the design of the steel SMF. Because of the wide distribution of damage to the structure, it was evaluated that it exhibited the same ductile behavior as intended in the steel SMF. However, the distribution of the fracture hinge is evaluated to be relatively concentrated in the lower part of the structure, and collapse occurred due to the sway mechanism at the location where the damage is concentrated. Therefore, in the case of mid- and high-rise steel SMF, which was designed using MRSA according to the current seismic criteria, despite the ductile behavior as intended, collapse is occurred caused by the rapid lateral displacement due to the P-Delta effect at an early time. In order to improve the vulnerability, it is necessary to delay the collapse due to the P-Delta effect, and it is considered that additional control of lateral displacement is necessary.

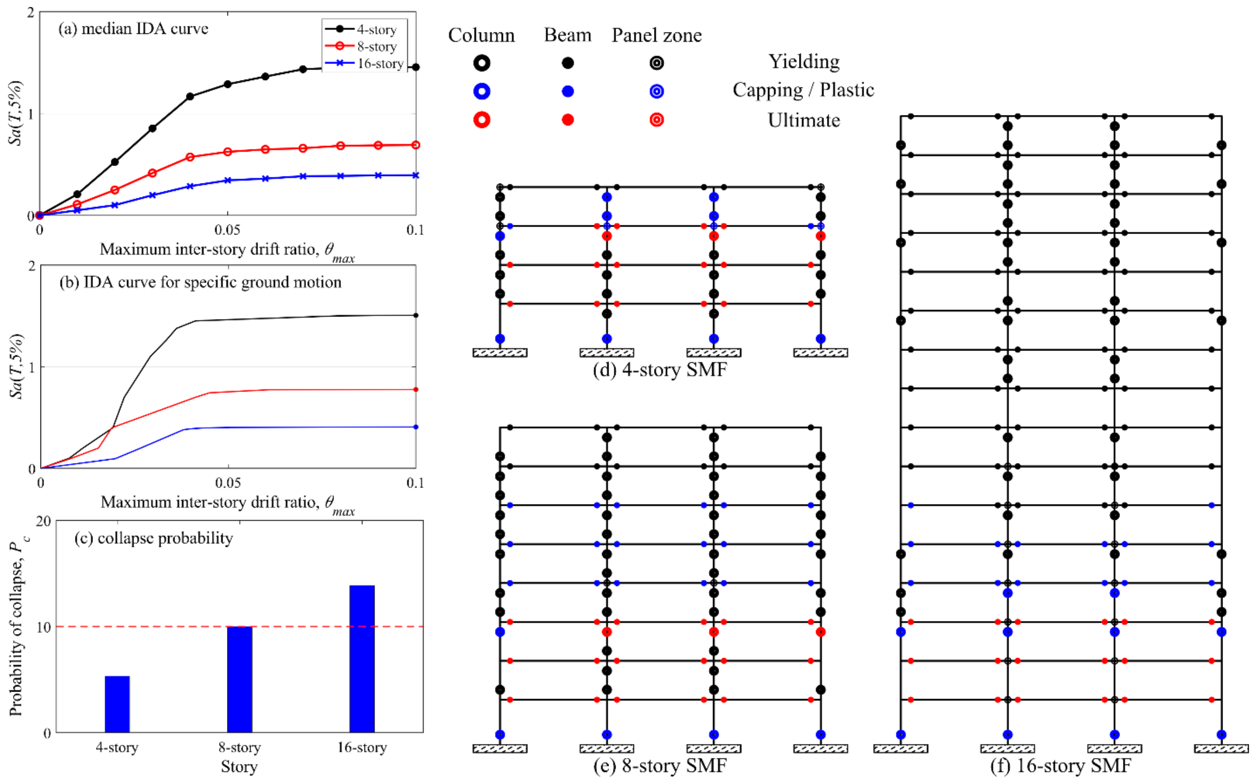

Fig. 4 Median IDA curve and plastic hinge distribution near collapse at dynamic analysis.

5. Conclusion

The purpose of this study is to evaluate the feasibility of the MRSA proposed in ASCE 7 (2017) by evaluating the seismic performance of the steel SMF, which was designed according to current seismic design criteria using MRSA. To this purpose, 4-, 8-, and 16-story steel SMF are designed, and seismic performance evaluation according to FEMA P695 (2009) is performed. As a result of the study, as the structural height increased, the probability of collapse of the steel SMF increased, and in the case of a mid- and high-rise building, the target seismic performance is not secured. The increase in the probability of collapse according to the structural height is evaluated as the cause of the sway mechanism, as the damage to the structure occurred intensively in the lower stories. although all the steel SMFs in which the design was carried out showed ductile behavior due to a wide range of damaged as intended by the seismic design criteria, the intended seismic performance is not secured by the collapse caused by the P-delta effect at an early stage. Therefore, it is judged that an additional design criterion for controlling the lateral displacement is necessary to secure the seismic performance for the mid- and high-rise steel SMFs.

ACKNOWLEDGMENTS

It is acknowledged that the research has been supported by grants from the National Research Foundation of Korea (NRF-2020R1A2C2010548). The valuable comment of two anonymous reviewers are also greatly appreciated.

REFERENCES

AISC (2016), *Seismic Provisions for Structural Steel Buildings (AISC 341-16)*, American Institute of Steel Construction, Chicago, IL.

AISC (2016), *Specification for Structural Steel Buildings (AISC 360-16)*, American Institute of Steel Construction, Chicago, IL.

ASCE (2017), *Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7-16)*, American Society of Civil Engineers, Reston, VA.

ATC (2010), *Modeling and Aceeptance Criteria for Seismic Design and Analysis of Tall Buildings (ATC 72-1)*, Applied Technology Council, Redwood, CA.

Computers and Structures Inc. ETABS v18 2.0. software, http://www.csiamerica.com/.

FEMA (2009), *Quantification of Building Seismic Performance Factors (FEMA P695)*, Federal Emergency Management Agency, Washington, DC.

Gupta, A. and Krawinkler, H. (1999), *Seismic demands for the performance evaluation of steel moment resisting frame structures*, John A. Blume Earthquake Engineering Research Center, Stanford, CA.

Harris, J. and Speicher, M. (2018), "Assessment of Performance-Based Seismic Design Methods in ASCE 41 for New Steel Buildings: Special Moment Frames." *Earthq Spectra.*, Vol. **34**(3), 977-999.

Ibarra, L.F., Medina, R.A., and Krawinkler, H. (2005), "Hysteretic models that incorporate strength and stiffness deterioration." *Earthq Eng Struct D.*, Vol. **34**(12), 1489-1511.

Lignos, D.G. and Krainkler, H. (2011), "Deterioration modeling of steel components in support of collapse prediction of steel moment frames under earthquake loading." *J. Struct. Eng.*, Vol. **137**(11), 1291-1302.

McKenna, F. (2011), "OpenSees: A Framework for Earthquake Engineering Simulation." *Comput Sci Eng.*, Vol. **13**(4), 58-66.

NIST (2010), *Evaluation of the FEMA P-695 Methodology for Quantification of Building Seismic Performance Factors (NIST GCR 10-917-8),* The National Institute of Standards and Technology, Redwood, CA.

NIST (2012), *Tentative Framework for Development of Advanced Seismic Design Criteria for New Buildings (NIST GCR 12-917-20),* The National Institute of Standards and Technology, Redwood, CA.

Vamvatsikos, D. and Cornell, C.A. (2002), "Incremental dynamic analysis." *Earthq Eng Struct D.*, Vol. **31**(3), 391-514.

Zareian, F. and Medina, R.A. (2010), "A practical method for proper modeling of structural damping in inelastic plane structural systems." *Comput Struct.*, Vol. **88**(1-2), 45-53.

Zareian, F., Lignos, D.G., and Krawinkler, H. (2010), "Evaluation of seismic collapse performance of steel special moment resisting frames using FEMA P695 (ATC-63) methodology", *Proceedings of the 2010 Structures Congress,* Orlando, FL.