Permissible Heights Proposed for Steel Ordinary Moment Frames

Sang Whan Han

***Abstract*—Steel ordinary moment frames (OMF) are often used in building construction as a seismic force-resisting system. Among three types of steel moment frames specified in seismic design provisions such as ordinary (OMF), intermediate (IMF), and special moment frames (SMF), design and detailing requirements specified for steel OMFs are the least stringent. To evaluate the seismic performance of steel OMFs, OMFs were designed according to current seismic design and detailing provisions considering different combinations of gravity, seismic, wind loads, as well as wind drift limits. FEMA-P695 was used for seismic performance evaluation. Then, permissible heights for steel OMFs are proposed based on the results of seismic performance evaluation.**

***Index Terms*—Steel moment frames, seismic performance, design provision, seismic force-resisting system.**

I. INTRODUCTION

Steel ordinary moment frames (OMFs) can be used for multi-story buildings classified as seismic design categories (SDC) A, B, and C without limiting their heights (h_n). However, for SDCs D-, E-, and F-OMFs are not permitted. The design and detailing requirements for OMFs are less stringent than those for intermediate (IMFs) and special moment frames (SMFs). Therefore, it is expected that OMFs have a lower inelastic deformation capacity than IMFs and OMFs.

According to ANSI/AISC 341-10 [1], for OMF connections, only minimal inelastic deformation capacity is required. Inelastic responses could occur in any frame elements including columns in OMFs during earthquakes, because the strong-column weak-beam (SC/WB) requirement is not enforced in OMF connections.

To compensate for the small deformation capacity of OMFs, these structures should be designed to provide a lateral strength larger than those of IMFs and SMFs. For this purpose, the lowest response modification factor ($R = 3.5$) is required for steel OMFs among the three types of moment frames. The values of R-factor assigned for steel SMFs and IMFs are 8.0 and 4.5, respectively. More stringent limitations on structural heights are also specified in AISC 341-10 [1], ASCE 7-10 [2] and AISC 358-10 [3] for OMFs than for SMFs and IMFs (Fig. 1).

According to ANSI/AISC 341-10, welded unreinforced flange-bolted (WUF-B) connections can be used for steel OMFs. SAC post-Northridge WUF-B connections using notch-tough weld metal and improved welding practices could achieve a mean plastic rotation of 1.5% radian, averaged over a wide range of connection parameters [4]. It was also reported that with a high toughness weld metal and modified detailing, a welded unreinforced flange moment connection can achieve an inelastic rotation of 0.03 rad or more prior to failure [5]. Based on experiments utilizing code-compliant WUF-B connections with different panel zone strength ratios, Han et al. [6] showed that these connections had moderate defamation capacity exceeding 0.02 radian prior to connection failure. Since only limited experimental and analytical research has been conducted for OMFs compared to ductile frames such as SMFs, the design and detailing requirements for OMFs are based mainly on judgement rather than on research (Commentary E1 of ANSI/AISC 341-10).

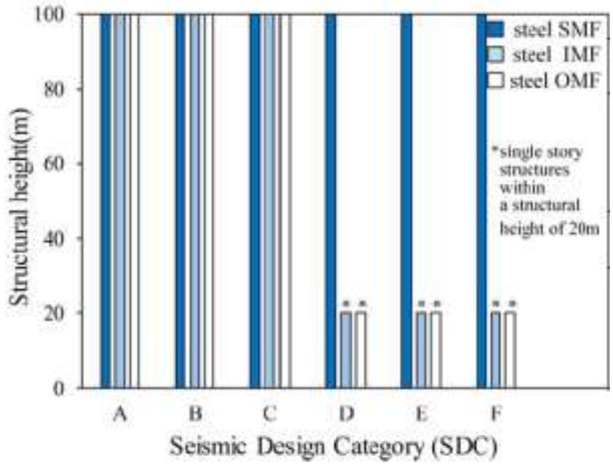

Fig. 1. Height limits for steel SMFs, IMFs and OMFs

In this study, the seismic performance of steel OMFs is evaluated. For this purpose, an analytical model is developed to simulate the seismic behavior of WUF-B connections based on test results. Steel OMFs with various structural heights ranging from three to 18 stories assigned to different SDC levels are considered as model structures.

Nonlinear static and dynamic analyses are conducted for the model structures. Seismic performance evaluation is conducted for steel OMFs according to FEMA P-695 [7]. Based on the results of seismic performance evaluation, permissible structural heights for steel OMFs are proposed according to the SDC level, which guarantees acceptable seismic performance.

II. MODEL STRUCTURES AND SEISMIC DESIGN

In the present study, 75 office buildings are considered as model structures and classified as risk category II. The

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importance factor (I_e) for these structures is assigned as 1.0. The model structures are assumed to be located at a site classified as class D. The plan and elevation of the model buildings are presented in Fig. 2.

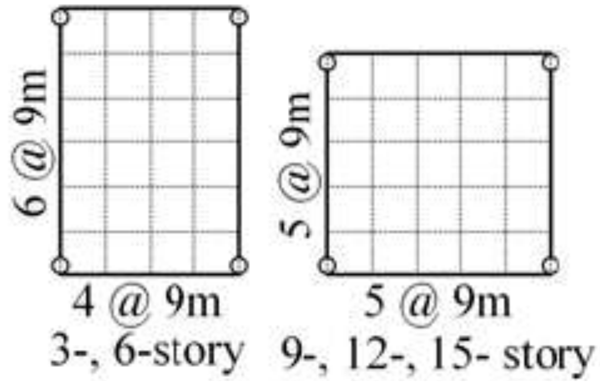

Fig. 2. Plans of the 3-, 6-, 9-, 12-, and 15-story steel OMFs.

Steel OMFs are used to resist seismic forces for model structures, which are placed along the perimeters of the model structures. The span length and story height of the buildings are 9.0 m and 4.0 m, respectively, except for buildings with nine or more stories; the height of the first story of these structures is 5.5m, whereas the height of the other stories is 4m.

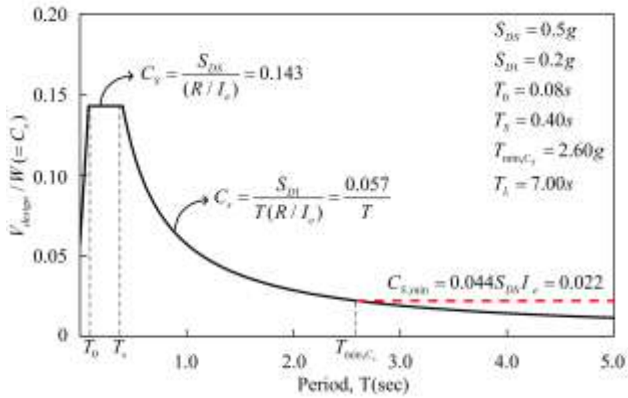

Fig. 3. Normalized design base shear of steel OMFs.

The perimeter OMFs in the east-west direction are designed according to AISC 360-10 [8], AISC 341-10 [1], and ASCE 7-10 [2], considering gravity and seismic loads. The dead and live loads for typical floors are 4.59 $\mathrm{kN/m^2}$ and 2.40 $\mathrm{kN/m^2}$, respectively, whereas the dead and live loads for roofs are 3.97 $\mathrm{kN/m^2}$ and 0.96 $\mathrm{kN/m^2}$ [9]. ASTM A572 Gr.50 steel is used for the design of member sections, which has a specified minimum yield strength (F_y) of 345 MPa.

The equivalent lateral force analysis (ELF) procedure is used to estimate seismic design force and drift demands, which is permitted for all structures assigned as SDC A, B, or C (Table 12.6-1 of ASCE/SEI 7-10). Fig. 3 shows design base shear forces normalized by weight for steel OMFs.

For calculating the seismic base shear force, the upper limit of calculated fundamental periods is considered. The base shear force should be larger than the minimum base shear force, which is the larger value of $0.044S_{DS}I_eW$ or $0.01W$, where S_{DS} is the design earthquake spectral acceleration parameter for short periods. Member forces are then estimated using design base shear forces for designing member sections. It is noted that there is no specific requirement to use the upper limit (= C_uT_a) of calculated periods and minimum base shear forces for assessing story drifts in ASCE/SEI 7-10 [2].

Beam and column sections are selected to resist member forces and to satisfy allowable story drifts specified in ASCE 7-10; story drifts should be less than 2% of the story height. Column sections are also analyzed to determine whether they satisfy the requirement specified in Section D1.4a of ANSI/AISC 341-10 [1]; the ratio of axial force demand on a column to its available axial strength should be less than $1/\Omega_o$ (=1/3).

For model OMFs, WUF-B connections are used. According to Chapter 3 of ANSI/AISC 358-10 [3], WUF-B connections should be constructed using high toughness weld metal, using improved welding and modified detailing practices that satisfy the detailing and welding requirements. Panel zones in connections are designed to have a shear strength greater than the shear force demand estimated using load combinations specified in ASCE/SEI 7-10 [2]. If required, doubler plates (A572 Gr.50 steel) are used to increase the shear strength of panel zones.

III. NUMERICAL MODEL

A two dimensional analytical model is constructed using OpenSees software [10] for steel OMFs with different structural heights and SDC levels, which can accurately simulate the cyclic behavior of WUF-B connections.

Fig. 4 shows idealized analytical models for frames and connections. To consider the effect due to gravity load, a leaning column is placed as shown in Fig. 4a, which is connected to the OMF using axially rigid elements. Damping matrices are constructed using Rayleigh damping with a damping ratio of 2% assumed for the first and fifth modes. Gravity frames and slabs are not considered in the analytical model.

Fig. 4b shows the configurations of constituent elements of connections. Beams are modeled with elastic beam elements and plastic rotational spring elements lumped at the both ends of the beam (Fig. 4b-1) in order to represent inelastic cyclic behavior. The deformation parameters (θ_p, θ_g) in Fig. 4b-1 are calculated using Eqs. (1) and (2) according to ASCE/SEI 41-13 [11], where is the plastic rotation capacity, and is the maximum rotation for supporting gravity load.

$$\theta_p = 0.021 - 0.0003d_{beam} \quad (1)$$

$$\theta_g = 0.05 - 0.0006d_{beam} \quad (2)$$

Panel zones are modeled using four rigid elements, as shown in Fig. 4b-2, which are connected with three pins and one spring element that follows a trilinear backbone curve with a strain hardening ratio of 6% (Fig. 4b-3) [9].

Cyclic deterioration is not considered in panel zone spring elements [12]. For simulating the cyclic behavior of columns, elastic beam-column elements are used with plastic springs placed at the both ends of the column (Fig. 4b-4).

The deterioration parameters proposed by Lignos and Krawinkler [13] are used for these plastic springs. [14].

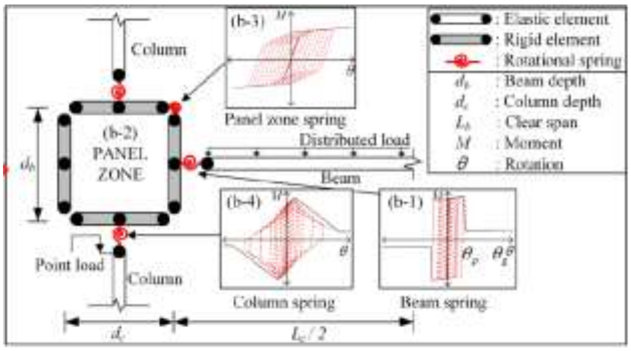

Fig. 4. Numerical model for steel OMFs and beam-to-column connections.

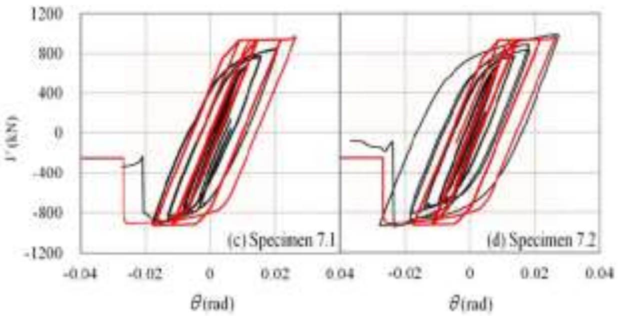

Fig. 5. Analytical model for steel OMFs and beam-to-column connections.

To verify the accuracy of the proposed model, cyclic curves are generated using the proposed analytical model for four WUF-B connections (7.1, 7.2) tested by Stojadinovic et al. [4], and then compared to those obtained from experimental tests.

Fig. 5 shows the cyclic curves of four different WUF-B connection specimens obtained from tests (Stojadinovic et al., 2000) and analyses using parameters provided in ASCE/SEI 41-13 [11].

The cyclic curves for specimens 7.1 and 7.2 generated using the proposed analytical model with strength, deformation and stiffness parameters calculated according to ASCE/SEI 41-13 [11] fairly accurately match those obtained from tests.

IV. Incremental Dynamic Analyses

To estimate ground motion intensities causing OMFs to collapse, incremental dynamic analyses (IDA) are conducted. A 5%- damped pseudo spectral acceleration (A) at a period of   is used to represent the ground motion intensity, where is the upper limit of fundamental periods calculated according to Section 12.8.2 of ASCE/SEI 7-10 [2]. The ordinate and abscissa of the IDA curves represent and maximum story drift ratio ($\theta_{\max}$), respectively.

A set of 44 far field ground motions provided in FEMA P-695[7] is used as input ground motions for IDA. Fig. 6a shows IDA curves for a six-story steel OMF. In this figure, solid circles denote a state of global dynamic instability, in which a drift increases excessively with a slight increase in ground motion intensity. The ground motion intensity corresponding to this state is named collapse intensity (S_{CT}). To expedite IDA, the hunt and fill algorithm is used [15]. Median collapse intensity ($\hat{S}_{CT}$) is calculated using 44 plotted in Fig. 6a.

Fig. 6b shows according to structural height for a given SDC C. With an increase in structural height, a smaller $\hat{S}_{CT}$ is generally obtained. OMF-GEWDs designed considering gravity, earthquake, wind and wind drift limit have $\hat{S}_{CT}$ values greater than those of corresponding OMF-GEs and OMF-GEWs, which may result from the large overstrength and ductility capacity inherent in OMF-GEWDs. Although OMF-GEWs are designed considering not only gravity and seismic loads but also wind loads, these structures sometimes have smaller $\hat{S}_{CT}$ values than corresponding OMF-GEs.

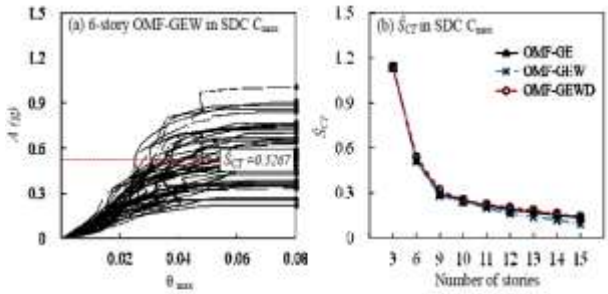

Fig. 6. IDA curve and median collapse intensity ($\hat{S}_{CT}$) of steel OMFs

V. Caculation of Probability of Collapse

In this study, seismic performance evaluation is conducted for steel OMFs designed according to ASCE/SEI 7-10 [2] according to FEMA P-695 [7]. The performance objective of the methodology provided in FEMA P-695 [7] is life safety by requiring a low probability of collapse (P_c) against maximum considered earthquake (MCE) ground motions. For a performance group of model buildings, the average value of P_c should be 0.1 or less, whereas for an individual model structures, should be 0.2 or less. Collapse probability (P_c) is calculated using Eq. (5).

$$P_c = P(Collapse \mid S_{MT}) = \Phi\left(\frac{\ln(S_{MT}) - \ln(\hat{S}_{CT} \times SSF)}{\beta_{TOT}}\right) \quad (5)$$

where S_{MT} is the MCE level spectral acceleration, and SSF is the spectral shape factor calculated using Eq. (6).

$$SSF = exp\left\lfloor \beta_1\left(\bar{\varepsilon}_0(T) - \bar{\varepsilon}(T)_{record}\right)\right\rfloor \quad (6)$$

$$\beta_1 = 0.14(\mu_T - 1)^{0.42} \quad (7)$$

$$\bar{\varepsilon}(T)_{record} = 0.6(1.5 - T) \quad (8)$$

where $\bar{\varepsilon}_0$ is taken as 1.0 for SDC A, B and C, and 1.5 for

SDC D. Total system collapse uncertainty (β_{TOT}) is calculated using Eq. (9) with β_{RTR}, β_{DR}, β_{TD}, and β_{MLD}, which are the record-to-record-, design requirement-related-, test data-related-, and modeling-uncertainties, respectively. The record-to-record uncertainty (β_{RTR}) is directly estimated from IDA.

$$\beta_{TOT} = \sqrt{\beta^2{}_{RTR} + \beta^2{}_{DR} + \beta^2{}_{TD} + \beta^2{}_{MDL}} \tag{9}$$

$$\beta_{RTR} = 0.1 + 0.1\mu_T \leq 0.4 \tag{10}$$

Fig 7 shows P_c for OMF-GEs, OMF-GEWs, and OMF-GEWDs according to structural height and SDC level. With an increase in SDC level for a steel OMF, P_c increases.

It is noted that MCE level spectral acceleration (S_{MT}) increases with an increase in the level of SDC. It is also observed that P_c generally increases with an increase in structural height. OMF-GEWDs always have a lower P_c than corresponding OMF-GEs and OMF-GEWs. However, OMF-GEWs generally have a P_c value greater than corresponding OMF-GEs.

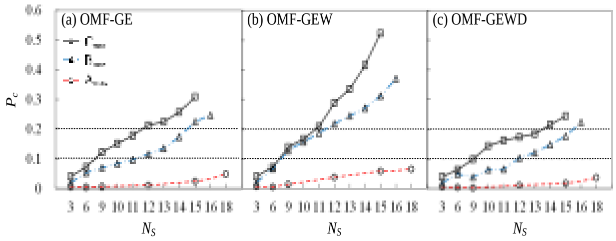

Fig. 7. Collapse probability (P_c) of steel OMFs with different SDCs according to the number of stories (N_s).

For SDC A_{max} OMFs, the values of P_c for all model structures is less than 0.1 irrespective of structural height, SDC level, and load combinations of gravity, seismic and wind loads, as well as wind drift limits.

Three- and six-story OMFs also demonstrates values less than 0.1 irrespective of their SDC level. However, for OMFs having a story number (N_s) greater than six, P_c is sometimes greater than 0.1.

VI. Structural Heights Proposed for OMFS

Structural heights are proposed for steel OMFs that satisfy the permissible probability of collapse (P_c = 0.1). Since P_c varies according to the SDC level and load combinations considered in the design, these variables are considered in proposing building heights for steel OMFs. Fig. 8 shows permissible heights for steel OMF-GEs, OMF-GEWs, and OMF-GEWDs assigned to different SDCs for a given permissible probability of collapse.

As addressed earlier, no height limit is required for steel OMFs assigned to SDC A, B or C in ASCE/SEI 7-10 (2010). This indicated that height limits for steel OMFs specified in ASCE/SEI 7-10 [2] may not guarantee a satisfactory seismic performance.

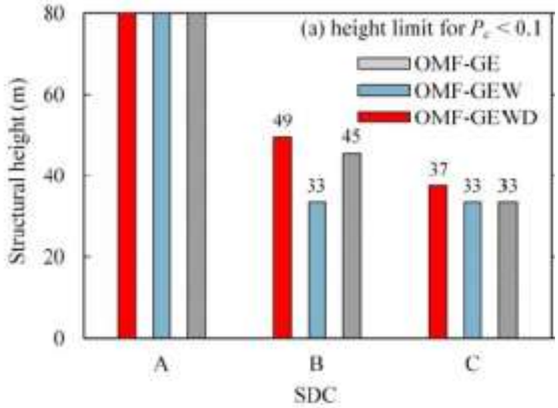

Fig. 8. Height limits of steel OMFs according to the level of SDC.

VII. Conclusions

The seismic performance was evaluated for 75 model steel OMFs designed considering different structural heights, SDC levels, and load combinations of gravity, seismic and wind loads, as well as wind drift limits according to the methodology specified in FEMA-P-695 [7]. To satisfy the allowable collapse probabilities (0.1) specified in FEMA P-695 [7], height limits were newly proposed for steel OMFs. The following are our specific conclusions obtained based on the results of this study.

(1) OMF-GEWDs had a median collapse intensity, which was larger than that of corresponding OMF-GEs and OMF-GEWs. The large $\hat{S}_{CT}$ value of steel OMF-GEWDs was attributed to large overstrength and ductility capacity inherent in OMF-GEWDs. It was observed that generally decreased with increasing the structural height of the frames. The difference in between OMF-GEs and OMF-GEWs was not noticeable.

(2) The probability of collapse (P_c) generally became higher with an increase in the structural height of steel OMFs. SDC -OMFs had a value lower than the limiting value of 0.1 specified for a performance group [7]. Many SDC B_{max} - and SDC C_{max} -OMFs had values greater than 0.2, which is more prominently observed for steel OMFs having greater number of stories.

(3) In the present study, OMF height limits were proposed according to the SDC level that satisfy allowable collapse probability (0.1. The proposed height limits for steel OMFs were more stringent than those specified in ASCE/SEI 7-10 [2]. This indicates that height limits for steel OMFs specified in ASCE/SEI 7-10 [2] cannot guarantee satisfactory seismic performance.

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