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Seismic performance of steel intermediate moment frames according to building heights

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ABSTRACT

Steel moment frames have been widely used as a seismic force resisting system in buildings due to their superior ductility and energy dissipation capacities. In ASCE 7-10, moment frames are classified into special moment frame (SMF), intermediate moment frame (IMF), and ordinary moment frame according to their inelastic deformation capacity. The IMF is intended to have limited levels of inelastic deformation capacity. The IMF connection must be capable of sustaining a story drift angle of 0.02 rad. In this study, the seismic performance of IMFs with different heights is evaluated, from which the effect of the building height on the seismic performance of the IMFs is investigated. Based on the results of this study, the validity of the height limit for steel IMFs specified in ASCE 7-10 is evaluated.

INTRODUCTION

Steel moment frames have been widely used as a seismic force resisting system in buildings due to their superior ductility and energy dissipation capacities. In ASCE 7-10 (2010), moment frames are classified into special, intermediate, and ordinary moment frames (SMF, IMF, OMF) according to their inelastic deformation capacity. The IMF are intended to have limited inelastic deformation capacity. The IMF connection must be capable of sustaining a story drift angle of 0.02 rad. The IMFs are typically used in low and moderate seismic regions.

With an increase in the level of SDC, a more stringent height limit is required (ASCE 7-10, Table 12.2-1). The objective of this study is to evaluate the seismic performance of IMFs with different heights. For this purpose, five steel IMFs are designed according to ASCE 7-10 and AISC 360-10 (2010). The seismic performance of the IMFs is evaluated according to FEMA P-695 (2009).

SEISMIC DESIGN OF STEEL IMFS ACCORDING TO ASCE 7-10

To evaluate the seismic performance of steel IMFs according to their heights or the number of stories, five steel IMFs are designed according to ASCE 7-10 and AISC

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360-10 (2010). The response modification coefficient (R), overstrength factor (Ω), and deflection amplification factor (C_d) for steel IMFs are 4.5, 3, and 4, respectively (Table 12.2-1 of ASCE 7-10).

Figure 1 shows the plan and elevation of the 3-, 6-, 9-, 12- and 15-story model buildings with design spectral response acceleration for SDC $C_{\max}$. The dead and live loads for typical floors are assumed as 4.12 kN/m^2 and 0.957 kN/m^2, respectively, whereas a dead load of 3.97 kN/m^2, and a live load of 0.957 kN/m^2 are used for roofs (Gupta and Krawinkler, 1999). The model buildings are assumed as standard office buildings, classified as risk category I. The importance factor (I_e) for risk category I is 1 (Table 1.5-2 of ASCE 7-10). This study uses the modal response spectrum analysis for the seismic design of IMFs.

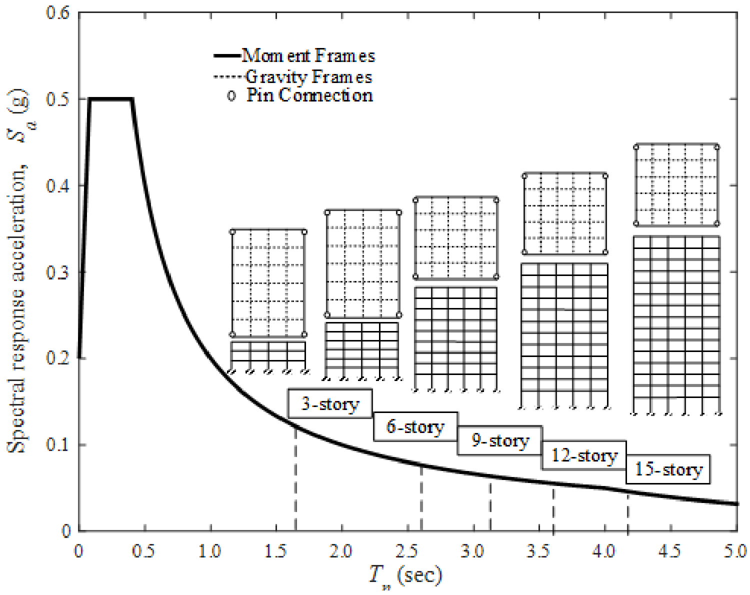

Figure 1. Design spectral response acceleration of IMFs assigned to SDC $C_{\max}$

In this study, nonlinear static pushover analyses and response history analyses are conducted using software OpenSees (Mazzoni et al., 2007). Figure 2 illustrated the analytical model for a two dimensional model frame. To account for $P-\Delta$ effect, a leaning column is placed as shown in Fig. 2a.

As shown in Fig. 2b, the connection is modeled using the 'M2' model developed by Gupta and Krawinkler (1999). To simulate the tri-linear force-deformation relationship of the panel zone, two spring elements are placed at the one corner of the M2 model (Fig. 2c). Pins are installed at the other three corners of the 'M2' model and rigid link elements were used for the boundary components of the 'M2' model.

Columns and modeled using fiber elements with a strain hardening ratio of 3% (Fig. 2d). To simulate the hysteretic behavior of beams including fracture, a rotation spring element developed by Ibarra et al. (2005) is placed at the ends of the beam (Fig. 2e). At a drift of 0.02 that is the drift capacity of IMF connections required by AISC 341-10, the strength of the spring suddenly drops by 80% of its maximum strength (ASCE 41, 2013) as shown in Fig. 2e.

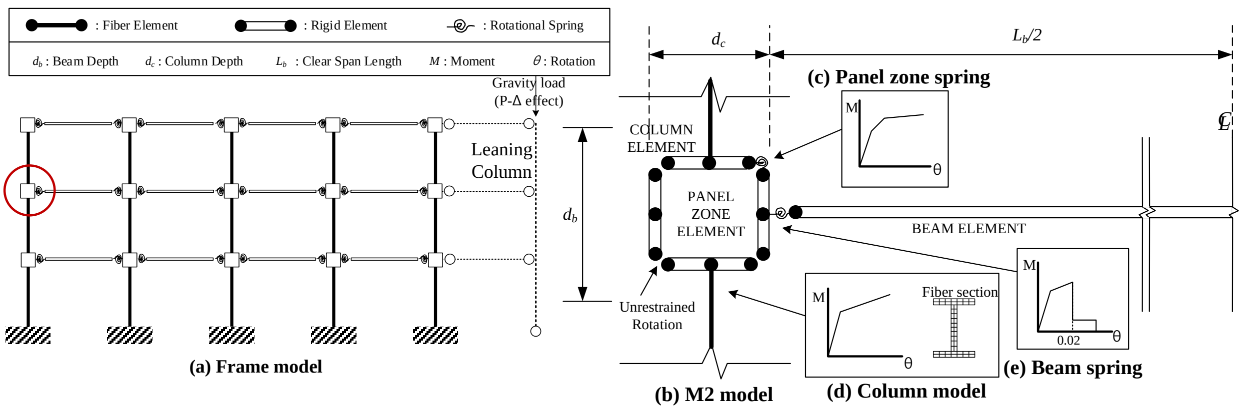

Figure 2. Analytic model used in this study

SEISMIC PERFORMANCE OF SDC $C_{\max}$ STEEL IMFS WITH DIFFFERENT HEIGHTS

In this study, the seismic performance of steel IMFs designed according to ASCE 7-10 is evaluated according to the methodology in FEMA P-695, which was developed to quantify system performance and response parameters for use in seismic design.

FEMA P-695 specifies two limiting values (P_a) of probability of collapse: (1) for a performance group, P_c is 0.1, or less on average across a performance group, and (2) for an individual model frame, P_c is 0.2, or less.

The probability of collapse (P_c) for the MCE ground motions can be calculated using Eq. (1)

$$P_c = P(Collapse \mid S_{MT}) = \Phi\left(\frac{\ln(S_{MT}) - \ln(\hat{S}_{CT} \times SSF)}{\beta_{TOT}}\right) \qquad (1)$$

where $\hat{S}_{CT}$ is the median collapse intensity obtained from the results of IDA. The collapse intensity (S_{CT}) is the ground motion intensity causing global dynamic instability that occurs when deformation increases without bound according to the slight increase in ground motion intensity (Vamvatsikos and Cornell, 2002; Han and Chopra, 2006).

The probability of collapse is calculated for 3-, 6-, 9-, 12-, and 15-story SDC $C_{\max}$ IMFs according to FEMA P-695. Before evaluating the seismic performance of the steel IMFs, pushover analyses and incremental dynamic analyses are conducted. The results are summarized in Table 1.

Table 1. The probability of collapse (P_c) for SDC $C_{\max}$ IMFs

Number of stories	Height (m)	Computed overstrength and parameters						Acceptance check		
		S_{MT}	Ω	μ_T	SSF	$\hat{S}_{CT}$	β_{TOT}	$P(C \mid S_{MT})$	$p_c \le 0.2$	$p_c \le 0.1$
3	12	0.38	3.40	1.77	1.07	0.900	0.67	0.080	Pass	Pass
6	24	0.22	3.99	2.69	1.18	0.500	0.71	0.080	Pass	Pass
9	37.5	0.16	4.86	2.08	1.16	0.300	0.68	0.113	Pass	Fail
12	49.5	0.12	4.73	1.54	1.11	0.220	0.66	0.144	Pass	Fail
15	61.5	0.10	5.36	2.15	1.16	0.135	0.68	0.267	Fail	Fail

COUNCLUSIONS

This study conducted seismic performance evaluation for steel IFMs with different heights according to FEMA P-695. The conclusions of this study are as follows:

Five steel IMFs with various heights assigned to SDC C_{max} were designed according to ASCE 7-10 and AISC 360-10. It was observed that with an increase in the height of steel IMFs, the probability of collapse increased. Even though no height limit is required SDC C_{max} steel IMFs in ASCE 7-10, the 15 story steel IMF had the probability of collapse larger than 0.2, whereas the collapse probabilities of the 9-, and 12-story IMFs exceeded 0.1.

With the acceptable collapse probability of 0.2, it is recommended that the height limits for steel IMFs assigned to SDC C should be 49.5m (12story).

If the acceptable collapse probability is 0.1, the height limit of steel IMFs assigned to SDC C is recommended to be 24m (6story).

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