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Effect of connection rotation capacities on seismic performance of IMF systems

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Abstract. The seismic performance of moment frames could vary according to the rotation capacity of their connections. The minimum rotation capacity of moment connections for steel intermediate moment frames (IMF) was defined as 0.02 radian in AISC 341-10. This study evaluated the seismic performance of IMF frames with connections having a rotation capacity of 0.02 radian. For this purpose, thirty IMFs were designed according to current seismic design provisions considering different design parameters such as the number of stories, span length, and seismic design categories. The procedure specified in FEMA P695 was used for conducting seismic performance evaluation. It was observed that the rotation capacity of 0.02 radian could not guarantee the satisfactory seismic performance of IMFs. This study also conducted seismic performance evaluation for IMFs with connections having the rotation capacity of 3% and ductile connections for proposing the minimum rotation capacity of IMF connections.

Keywords: rotation capacity; connections; intermediate moment frame; seismic performance; seismic design

1. Introduction

The rotation capacity of moment connections may significantly affect the seismic performance of moment frames. AISC 358-10 (2010) classifies moment frames into special, intermediate, and ordinary moment frames (SMF, IMF, OMF) according to their inelastic rotation capacities. AISC 341-10 (2010) requires minimum rotation capacities for SMFs and IMFs as 0.04 and 0.02 radian, respectively, whereas only minimal level of inelastic rotation is required for OMFs. SMFs are expected to produce deformation and energy dissipation capacities larger than IMFs and OMFs. For this reason, ASCE 7-10 (2010) assigns a response modification factor (R) as 8 for SMF, whereas factors for IMF and OMF are assigned as 4.5 and 3.5, respectively.

After the 1994 Northridge earthquake, numerous studies were conducted for evaluating the rotation capacities of moment connections (Luco and Cornell 1998a, Malley 1998, Stojadinovic *et*

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al. 2000, Sotirov *et al*. 2000). Those studies identified the cause of premature brittle fracture occurred in pre-Northridge moment connections, and developed new connections that avoid brittle connection fracture. In particular, many experimental researches were conducted for developing pre-qualified SMF connections (FEMA 350 2000, FEMA 355D 2000). The effect of ductile and brittle connections on the seismic response of moment frames was investigated by Maison and Kasai (1997), Luco and Cornell (1998b, 2000), and Gupta and Krawinkler (1999). Most research has focused on evaluating the rotation capacity of SMF connections and the seismic performance of SMFs (Lee and Foutch, 2002, Jeong *et al*. 2015). Regarding the IMFs and OMFs, only limited research was conducted (Han *et al*. 2007, Han *et al*. 2015). Han *et al*. (2015) reported that the seismic performance of 20 story IMFs with reduced beam section-bolted shear tab (RBS-B) moment connections failed to meet the performance criteria specified in FEMA P695 (2009), which is a provision for collapse risk evaluation (Elkady and Lignos 2015).

AISC 341-10 states that the rotation capacity of 0.02 radian has been established based on engineering judgment applied to available tests and analytical studies, primarily those in FEMA 355C (2000) and 355F (2000). There was no study found so far, which evaluated the seismic performance of IMFs with respect to the rotation capacity of their connections.

In this study, the seismic performance evaluation was conducted according to FEMA P695 for IMFs with moment connections having different rotation capacities. For this purpose, IMFs with various numbers of stories and span lengths were designed for seismic design category (SDC) using design seismic forces determined according to ASCE 7-05 (2005) and ASCE 7-10. From the results of this study, the minimum rotation capacity for IMF connections was proposed.

2. Connections for intermediate steel moment frames

The In AISC 358-10, six prequalified moment connections are listed, which can be used as moment connections for SMFs and IMFs: (1) reduced beam section (RBS), (2) bolted unstiffened extended end plate (BUEEP), (3) bolted stiffened extended end plate (BSEEP), (4) bolted flange plate (BFP), (5) welded unreinforced flange-welded web (WUF-W), and (6) Kaiser bolted bracket (KBB). Among the pre-qualified moment connections, the RBS connections have been commonly used for seismic application. Many experimental researches were conducted for RBS connections, so that sufficient test results for the rotation capacity of RBS connections have been available.

The RBS connection can be classified into RBS-welded web connection and RBS-bolted web (RBS-B) connection. In this study, the rotation capacity of RBS-B connections, which are only valid for IMF connections (AISC 358-10), were estimated using test results (Tremblay *et al*. 1997, Engelhardt *et al*. 2000, Iwankiw and Carter 1996, Lee *et al*. 2005, Chen *et al*. 1996, Anderson and Duan 1998, Tsai and Chen 1996, Han *et al*. 2009). Fig. 1 shows backbone curves for fracture-resistant and fracture-prone RBS-B connections, representing the relationship between the moment normalized by beam plastic moment (M/M_p) and total rotation (θ_t) measured at the connection. The fracture-resistant connection was defined as the connection that did not experience connection fracture until the completion of the test, whereas the fracture-prone connection experienced connection fracture during tests. As shown in Figs. 1(a)-(b), some connections behaved in a ductile manner, whereas other connections experienced premature fracture, resulting in a small rotation capacity. When premature brittle fracture occurs at the connection, the seismic performance of IMF systems could be significantly reduced.

No study can be found, which evaluated the seismic performance of code-complied IMFs with

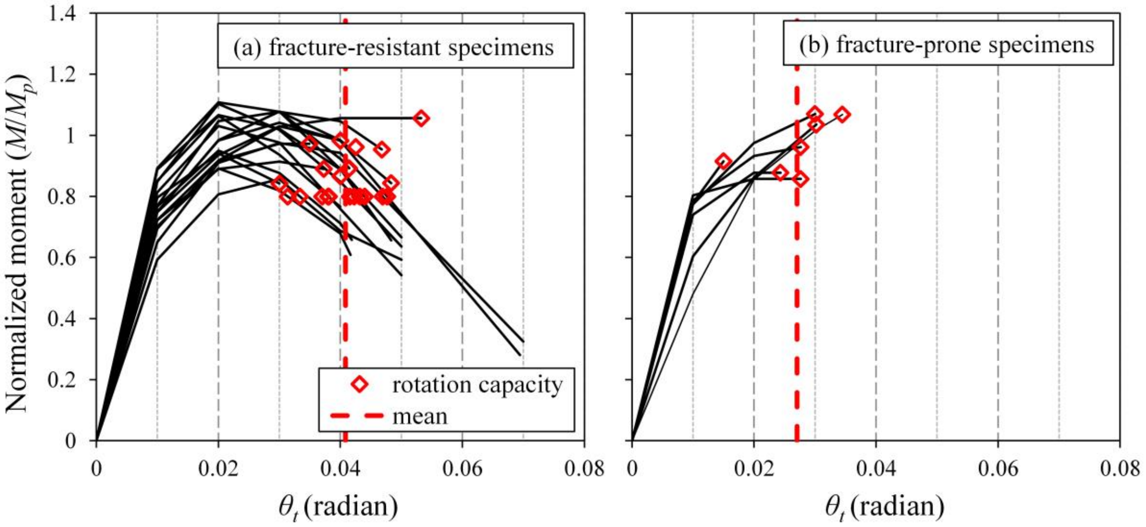

Fig. 1 Backbone curves of RBS-B connections

connection having the minimum rotation capacity of 0.02 radian that is required for IMF connections by AISC 341-10 as the minimum rotation capacity. This study evaluated the seismic performance of IMFs with their connections having the minimum rotation capacity (0.02 radian). Moreover, this study also considered connections with a rotation capacity of 0.03 radian as well as ductile connections (fracture resistant connection) because the rotation capacity of IMF connections varied as shown in Fig. 1. The seismic performance evaluation was conducted for IMFs with connections having different rotation capacities.

3. Analytical model for moment connections

For obtaining the reliable results from the seismic performance evaluation, the use of an accurate analytical model is very important. In this study, the IMF connection was modeled using OpenSees (Mazzoni *et al*. 2007) as shown in Fig. 2. The connection model was mainly based on 'M2' model developed by Gupta and Krawinkler (1999). As shown in Fig. 2, the actual dimension of the panel zone was precisely reflected in this model.

The fiber section model was used for simulating the behavior of columns. In this model, the inelastic behavior of a member is predicted using the stress-strain relationship assigned for fiber sections. The interaction between axial force and moment can be accurately reflected using this model. For panel zones, rigid link elements were used along the perimeter of the panel zone. To simulate the tri-linear inelastic behavior of the panel zone, two rotational springs were installed at one corner of the panel zone (Fig. 2(b)), whereas pins were placed at the other three corners to connect the rigid link elements. The strength and stiffness of the two springs were determined based on Gupta and Krawinkler (1999).

The beam member was modeled using rigid element and inelastic rotational spring element that was placed at the ends of the beam (Fig. 2(c)). For fracture prone connections having rotation

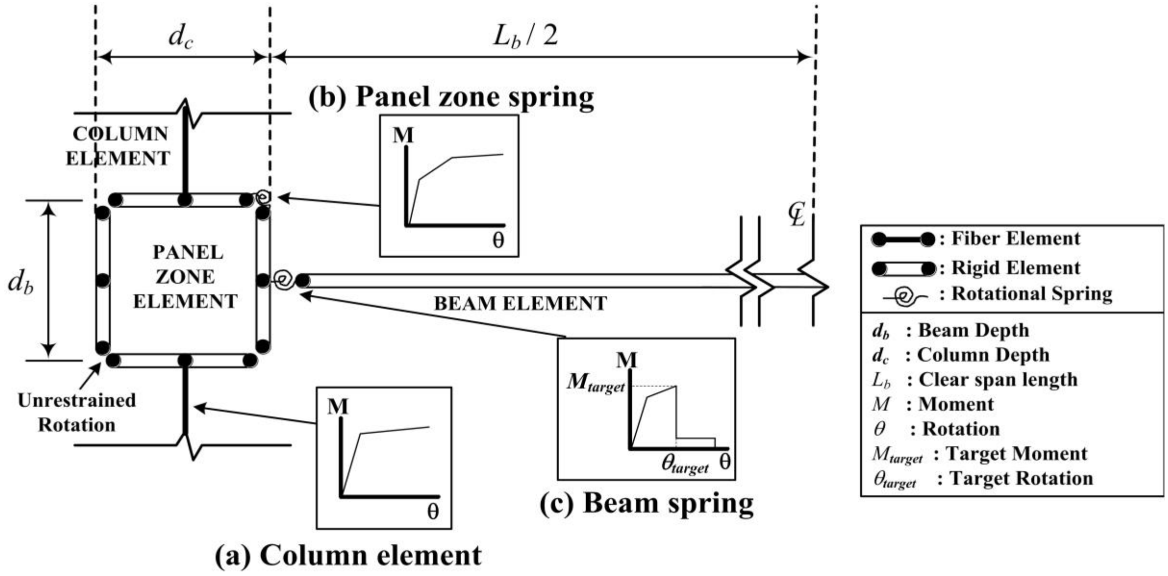

Fig. 2 Analytical model for IMF connections used in this study

capacities of 0.02 and 0.03 radian, the rotational spring was modeled to drop its strength suddenly when the rotation of the spring reaches its rotation capacity. However, for ductile connections without connection fracture, the rotation spring was modeled based on Lignos (2014), which behaved in a ductile manner without a sudden strength drop.

4. Model frames

For conducting seismic performance evaluation for IMFs, model frames were designed considering various important design parameters such as the number of stories, span length, seismic design category, and seismic response coefficient. To investigate the effect of the design parameter on the seismic performance of the IMF system, the model buildings with the same design parameters are grouped, which is named 'performance group' (FEMA P695).

Since the higher mode contribution varies according to the number of stories, this study considered four different numbers of stories (3, 6, 9, and 20 stories). To account for the effect of gravity loads on beams, three different span lengths were included: 6.1 m (20 ft), 7.6 m (25 ft), and 9.1 m (30 ft).

Seismic design category (SDC) *C* was used for determining the design spectral acceleration parameters at short period (S_{DS}) and at 1-s period (S_{D1}) for model frames considering possible maximum and minimum seismic criteria of SDC: SDC $C_{\min}$ (S_{DS}=0.33 g, S_{D1}=0.133 g) and SDC $C_{\max}$ (S_{DS}=0.50 g, S_{D1}=0.20 g) where g is the gravitational acceleration. Since there is no height limitation on IMFs for SDC *C* (ASCE 7-10), high-rise buildings, which are strongly affected by higher mode contributions, can be designed.

The seismic response coefficient (C_S) was determined according to ASCE 7-05 and ASCE 7-10. The seismic base shear can be determined as C_S times the weight of the building (*W*). Both standards produced the same C_S in entire period range; however, they specify different minimum values of C_S, which may govern for long-period structures. ASCE 7-05 requires that seismic response coefficient C_S should be 0.01 or larger, whereas in ASCE 7-10, C_S should not be less than the smaller of $0.044S_{DS}I_e$ and 0.01 g, where I_e is the importance factor. The values determined according to ASCE 7-05 and ASCE 7-10 were the same for 3-, 6- and 9-story model frames, whereas they were different for 20 story model frames because minimum values of C_S controlled for the 20 story frames (Fig. 3). Therefore, 3-, 6- and 9-story frames were designed using ASCE 7-10 design seismic forces, whereas 20-story frames were designed for design seismic forces determined according to ASCE 7-05 and ASCE 7-10.

Considering all design parameters as mentioned above, twenty four IMFs were designed, which were summarized in Table 1. The frames were classified into 6 performance groups according to span length (6.1, 7.6, and 9.1 m) and seismic design categories (SDC $C_{\min}$ and SDC $C_{\max}$). To investigate the effect of different design seismic forces determined from ASCE 7-05 and ASCE 7-10 on the seismic performance of IMFs, six 20-story model frames were designed, which were grouped as an additional performance group. Thus, this study considered a total of thirty IMFs and seven performance groups.

To investigate the effect of connection rotation capacity on the seismic performance of IMFs, moment connections were modeled using rotational spring element with three different rotation capacities: (1) 2%, (2) 3%, and (3) ductile connection. For connections with rotation capacities of 2% and 3%, the rotational spring element was forced to fracture at rotations of 2% and 3%, respectively. The ductile connection was modeled with a ductile spring element without fracture. Since three different connection models for each model frame were used, the number of model frames used for performance evaluation were 90 (=30×3). In Table 1, frame ID 'MIN6-10-2%' represents the IMF with a span length of 6.1 m and a connection rotation capacity of 2%, which were designed for seismic loads for SDC $C_{\min}$ according to ASCE 7-10 design seismic forces.

The model frames were assumed to be used in office buildings located at a site with soil condition D (ASCE 7-10). Fig. 4 shows floor plans and elevations for the model frames. As seen in Fig. 4, the seismic resistance for buildings was provided by perimeter frames. Steel material ASTM A992/A992M was used for beams and columns. According to ASCE 7-10, for IMFs, the response modification factor (R), overstrength factor (Ω), and deflection amplification factor (C_d) are 4.5, 3.0, and 4.0, respectively. Members were designed according to AISC 341-10 and

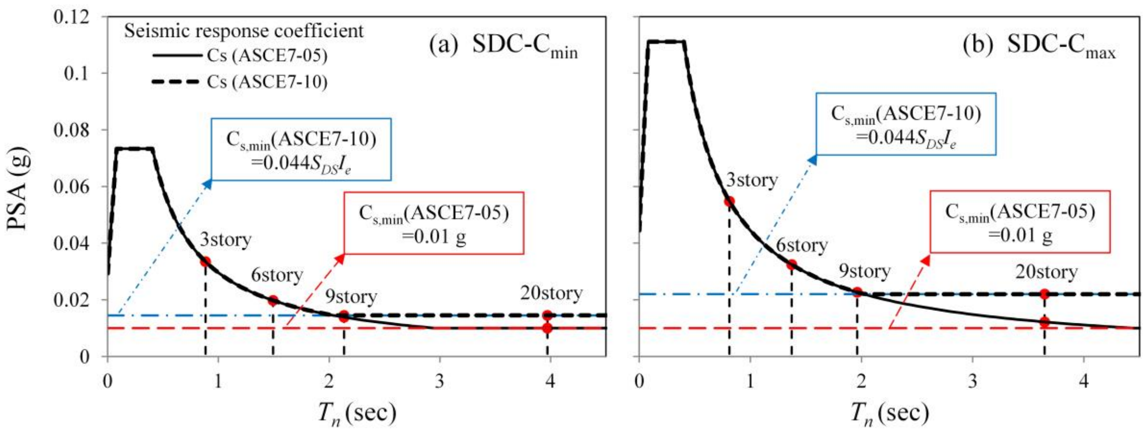

Fig. 3 Seismic response coefficient of model frames for (a) SDC $C_{\min}$ and (b) SDC $C_{\max}$

Table 1 Summary of model frames and performance groups

No.	Performance Group	No.	Arch.ID	Design Load (ASCE 7)	Bay Size	No. Of Stories	Seismic Design Criteria: SDC	Mass (ton)	T_n^* (sec)	T_1^{**} (sec)	C_S^+ (g)
1	Min6-10-x^{++}%	1	3CMIN10-6.1	2005/2010	6.1 m (20 ft)	3	Cmin	582	0.858	1.479	0.0345
		2	6CMIN10-6.1	2005/2010		6		1,188	1.493	2.084	0.0198
		3	9CMIN10-6.1	2005/2010		9		1,902	2.135	2.893	0.0138
		4	20CMIN10-6.1	2010		20		5,223	3.972	3.804	0.0145
2	Max6-10-x%	5	3CMAX10-6.1	2005/2010		3	Cmax	586	0.787	1.160	0.0565
		6	6CMAX10-6.1	2005/2010		6		1,191	1.371	1.914	0.0324
		7	9CMAX10-6.1	2005/2010		9		1,902	1.960	2.658	0.0227
		8	20CMAX10-6.1	2010		20		5,236	3.646	3.577	0.0220
3	Min7-10-x%	9	3CMIN10-7.6	2005/2010	7.6 m (25 ft)	3	Cmin	910	0.858	1.522	0.0345
		10	6CMIN10-7.6	2005/2010		6		1,849	1.493	2.125	0.0198
		11	9CMIN10-7.6	2005/2010		9		2,944	2.135	2.658	0.0138
		12	20CMIN10-7.6	2010		20		8,171	3.972	3.540	0.0145
4	Max7-10-x%	13	3CMAX10-7.6	2005/2010		3	Cmax	918	0.787	1.155	0.0565
		14	6CMAX10-7.6	2005/2010		6		1,853	1.371	1.920	0.0324
		15	9CMAX10-7.6	2005/2010		9		2,951	1.960	2.625	0.0227
		16	20CMAX10-7.6	2010		20		8,194	3.646	3.406	0.0220
5	Min9-10-X%	17	3CMIN10-9.1	2005/2010	9.1 m (30 ft)	3	Cmin	1,318	0.858	1.524	0.0345
		18	6CMIN10-9.1	2005/2010		6		2,680	1.493	2.159	0.0198
		19	9CMIN10-9.1	2005/2010		9		4,293	2.135	2.587	0.0138
		20	20CMIN10-9.1	2010		20		11,933	3.972	3.570	0.0145
6	Max9-10-X%	21	3CMAX10-9.1	2005/2010		3	Cmax	1,329	0.787	1.162	0.0565
		22	6CMAX10-9.1	2005/2010		6		2,686	1.371	1.916	0.0324
		23	9CMAX10-9.1	2005/2010		9		4,297	1.960	2.539	0.0227
		24	20CMAX10-9.1	2010		20		11,981	3.646	3.570	0.0220
7	20F-05-x%	25	20CMIN05-6.1	2005	6.1 m (20 ft)	20	Cmin	5,284	3.972	3.600	0.0100
		26	20CMAX05-6.1			20	Cmax	5,182	3.646	3.742	0.0122
		27	20CMIN05-7.6		7.6 m (25 ft)	20	Cmin	8,168	3.972	3.547	0.0100
		28	20CMAX05-7.6			20	Cmax	8,172	3.646	3.526	0.0122
		29	20CMIN05-9.1		9.1 m (30 ft)	20	Cmin	11,930	3.972	3.576	0.0100
		30	20CMAX05-9.1			20	Cmax	11,934	3.646	3.672	0.0122

* T_n^* : Fundamental period(=$C_u T_a$), T_1^{**} : 1st mode period, C_S^+ : Seismic response coefficient, x^{++}%: connection rotation capacity (2%, 3% radian, and ductile)

AISC 360-10 (2010). The dead load and live load for member deign were assumed as 4.12 kPa (86 lb/ft2) and 0.96 kPa (20 lb/ft2), respectively. The basic wind speed of 51 m/sec was used, and surface roughness was assumed as B for urban office buildings [Chapters 26 and 27 in ASCE

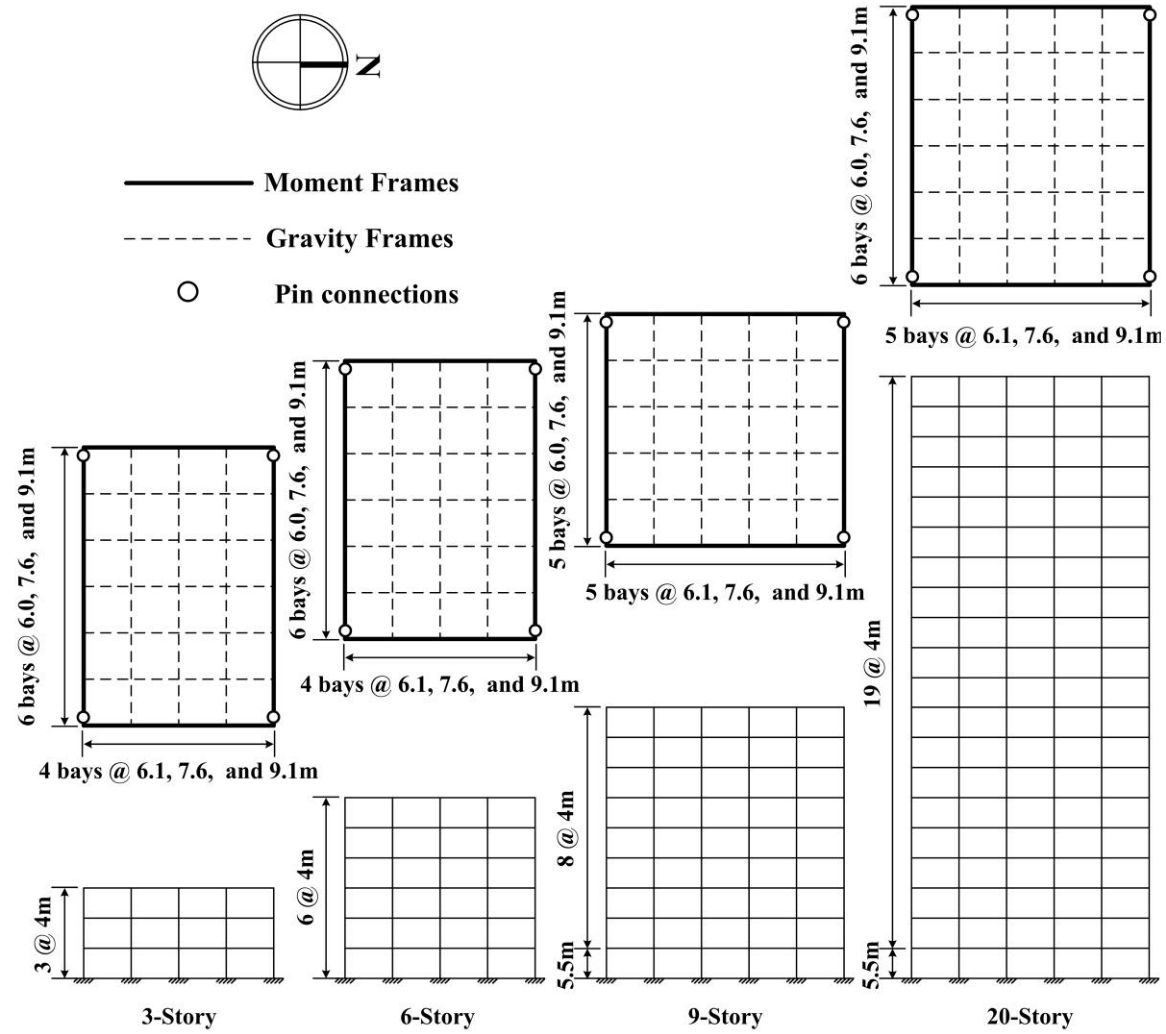

Fig. 4 Floor plans and elevations of model buildings

7-10]. Fig. 3 shows the seismic design coefficient for SDC C_{min} and SDC C_{max} determined according to ASCE 7-05 and ASCE 7-10. As mentioned earlier, two dimensional analytical model was used for analysis, which can account for P-Δ effect using an additional dummy column.

5. Seismic performance evaluation

Seismic performance evaluation was conducted according to FEMA P695, which is a provision for evaluating collapse risk. The collapse is defined as global dynamic instability (Adam and Jäger 2012, Qi *et al*. 2012, Eads *et al*. 2013, Hamidia *et al*. 2014). Acceptable performance is defined by two collapse prevention objectives: (1) the probability of collapse for maximum considered earthquake (MCE) ground motions is 10% or less on average across a performance group, and (2) the probability of collapse for MCE ground motions is 20% or less for each frame within a performance group. Acceptable performance is achieved when adjusted collapse margin rations, *ACMR*, for each model frame and performance group, meet the following two criteria: (1) The average value ($\overline{ACMR_i}$) of *ACMR* for each performance group exceeds $ACMR_{10\%}$, and (2) individual values ($ACMR_i$) of *ACMR* for each model frame within a performance group exceeds $ACMR_{20\%}$, where $ACMR_{10\%}$ and $ACMR_{20\%}$ are the limiting values of *ACMR* corresponding to the probabilities of collapse of 10% and 20% for MCE ground motions, respectively. The procedure for estimating $ACMR_i$ and $\overline{ACMR_i}$ is summarized as follows. Fig. 5 illustrates the procedure.

(1) Conduct a static pushover analysis, and determine the overstrength factor (Ω) and period dependent ductility factor (μ_T) using Eqs. (1)-(2). (Fig. 5(a))

$$\Omega = V_{\max} / V_d \tag{1}$$

$$\mu_T = \delta_u / \delta_{y,eff} \tag{2}$$

where $V_{\max}$ is the maximum base shear force, V_d is the design base shear force, δ_u is the ultimate roof displacement defined in Fig. 5(a), and $\delta_{y,eff}$ is the effective yield roof displacement.

(2) Conduct incremental dynamic analyses (IDA), and determine the median collapse strength ($\hat{S}_{CT}$) and collapse margin ratio (*CMR*) using Eq. (3) (Fig. 5(b))

$$CMR = \hat{S}_{CT} / S_{MT} \tag{3}$$

where S_{MT} is the 5% damped pseudo spectral acceleration at T_n for the MCE ground motions.

(3) To account for the effect of the spectral shape of ground motions used for performance

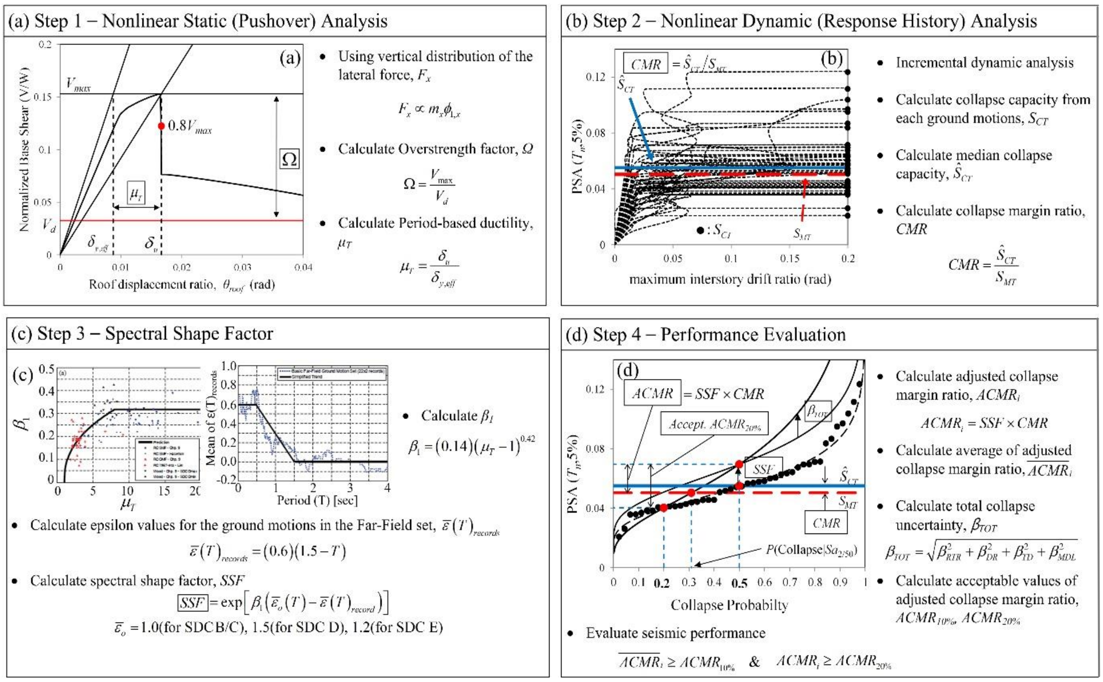

Fig. 5 The summarized procedure for seismic performance evaluation in FEMA P695

evaluation, the spectral shape factor (*SSF*) is calculated (Fig. 5(c)), which is the function of μ_T, T_n and the level of SDC.

(4) Calculate $ACMR_i$ for each model frame using Eq. (4). Then, the average value of $ACMR_i$ for model frames within each performance group is calculated ($\overline{ACMR_i}$)

$$ACMR_i = SSF \times CMR \tag{4}$$

(5) Find the acceptable values $ACMR_{10\%}$ and $ACMR_{20\%}$ from the Table provided in FEMA P695 (Tables 7-3).

(6) Check whether acceptance criteria are met using Eqs. (5)-(6).(Fig. 5(d))

$$\overline{ACMR_i} \geq ACMR_{10\%} \tag{5}$$

$$ACMR_i \geq ACMR_{20\%} \tag{6}$$

6. Seismic performance of IMFs with connections having a rotation capacity of 2%

Seismic performance evaluation was conducted for thirty IMFs. The rotation capacity of IMFs was assumed as 0.02 radian, which is defined as the minimum rotation capacity for IMF in AISC 341-10. Table 2 summarizes the results of the performance evaluation conducted according to FEMA P695 procedure. As seen in this table, twelve frames and one performance group did not meet the criteria specified in FEMA P695.

To investigate the vulnerability of IMFs, the probability of collapse for MCE ground motions (P(Collapse|S_{MT})) was calculated. Fig. 6 shows the collapse probabilities according to design parameters.

The collapse probability varied significantly according to the number of stories as well as design seismic forces determined from ASCE 7-05 and ASCE 7-10. With an increase in the number of stories, the collapse probability becomes larger (Fig. 6(a)). The average values for the collapse probabilities of 3-, 6-, 9-, and 20-story frames were 0.008, 0.024, 0.1423, and 0.2972, respectively. All 3- and 6-story model frames had the probability of collapse less than 20%, which satisfied the FEMA P695 criteria, whereas some of 9- and 20-story model frames did not satisfy the criteria. In particular, ten 20-story frames did not meet the criteria among twelve 20-story model frames (Table 2).

Fig. 6(d) shows that the numbers of 20-story frames, which did not meet the criteria, were the same, regardless of ASCE 7-05 and ASCE 7-10 design seismic forces. However, the average collapse probability (=0.223) of the 20-story frames designed using the ASCE 7-10 design seismic force is smaller than that (=0.370) of the frames designed using the ASCE 7-05 design seismic force.

As shown in Fig. 6(b)-(c), the collapse probability did not vary significantly according to span length and the level of SDC. The average values of the collapse probabilities of IMFs having span lengths 6.1, 7.6 and 9.1 m were 0.114, 0.177, and 0.170, respectively.

Regarding the level of SDC, the number of the frames with unsatisfactory seismic performance became larger with increasing the level of SDC from SDC $C_{\min}$ to SDC $C_{\max}$ (Fig. 6(c)). The average collapse probabilities of the frames designed for SDC $C_{\min}$ and $C_{\max}$ were 0.120 and 0.187, respectively.

Table 2 Summary of collapse margin parameters and acceptance check for IMF systems with connections having a rotation capacity of 0.02 radian

Performance group	Arch.ID	Computed overstrength and collapse margin parameters							Acceptance check		
		S_{MT}	Ω	μ_T	S_{CT}	*CMR*	*SSF*	*ACMR*	β_{TOT}	Accept. *ACMR*	Pass /Fail
Min6-10-2%	3CMIN10-6.1	0.233	4.99	2.00	1.158	4.96	1.09	5.41	0.68	1.77	Pass
	6CMIN10-6.1	0.134	6.11	1.97	0.382	2.85	1.15	3.27	0.68	1.76	Pass
	9CMIN10-6.1	0.094	8.98	1.55	0.243	2.60	1.11	2.89	0.66	1.74	Pass
	20CMIN10-6.1	0.050	4.34	3.15	0.137	2.73	1.21	3.31	0.73	1.84	Pass
	Average		**6.11**	**2.17**		**3.28**		**3.72**	**0.68**	**2.40**	**Pass**
Max6-10-2%	3CMAX10-6.1	0.381	4.57	2.20	1.624	4.26	1.09	4.65	0.69	1.78	Pass
	6CMAX10-6.1	0.219	4.71	2.37	0.841	3.84	1.16	4.46	0.69	1.79	Pass
	9CMAX10-6.1	0.153	5.55	1.95	0.265	1.73	1.15	1.99	0.67	1.76	Pass
	20CMAX10-6.1	0.082	5.32	2.14	0.113	1.38	1.16	1.60	0.68	1.78	Fail
	Average		**5.04**	**2.17**		**2.80**		**3.17**	**0.68**	**2.40**	**Pass**
Min7-10-2%	3CMIN10-7.6	0.233	4.87	2.05	1.196	5.13	1.09	5.60	0.68	1.77	Pass
	6CMIN10-7.6	0.134	6.12	2.14	0.387	2.89	1.16	3.35	0.68	1.78	Pass
	9CMIN10-7.6	0.094	8.61	1.61	0.178	1.90	1.12	2.13	0.66	1.74	Pass
	20CMIN10-7.6	0.050	6.28	1.46	0.069	1.36	1.11	1.51	0.65	1.73	Fail
	Average		**6.47**	**1.82**		**2.82**		**3.15**	**0.67**	**2.36**	**Pass**
Max7-10-2%	3CMAX10-7.6	0.381	4.88	2.11	1.941	5.09	1.09	5.54	0.68	1.77	Pass
	6CMAX10-7.6	0.219	4.99	2.40	0.921	4.21	1.16	4.88	0.70	1.80	Pass
	9CMAX10-7.6	0.153	5.53	1.78	0.236	1.54	1.13	1.75	0.67	1.75	Fail
	20CMAX10-7.6	0.082	5.38	1.82	0.108	1.31	1.14	1.49	0.67	1.76	Fail
	Average		**5.20**	**2.03**		**3.04**		**3.42**	**0.68**	**2.38**	**Pass**
Min9-10-2%	3CMIN10-9.1	0.233	4.85	2.03	1.186	5.08	1.09	5.55	0.68	1.77	Pass
	6CMIN10-9.1	0.134	6.25	1.93	0.426	3.18	1.14	3.64	0.67	1.76	Pass
	9CMIN10-9.1	0.094	8.97	2.15	0.194	2.07	1.16	2.40	0.68	1.78	Pass
	20CMIN10-9.1	0.050	7.36	1.46	0.067	1.32	1.11	1.46	0.65	1.73	Fail
	Average		**6.86**	**1.89**		**2.91**		**3.26**	**0.67**	**2.37**	**Pass**
Max9-10-2%	3CMAX10-9.1	0.381	4.82	2.09	1.653	4.34	1.09	4.71	0.68	1.77	Pass
	6CMAX10-9.1	0.219	5.08	2.26	0.923	4.22	1.15	4.87	0.69	1.79	Pass
	9CMAX10-9.1	0.153	6.32	1.98	0.225	1.47	1.15	1.69	0.68	1.77	Fail
	20CMAX10-9.1	0.082	4.76	1.81	0.121	1.48	1.14	1.68	0.67	1.75	Fail
	Average		**5.25**	**2.04**		**2.87**		**3.24**	**0.68**	**2.38**	**Pass**
20F-05-2%	20CMIN05-6.1	0.050	9.10	1.94	0.096	1.90	1.15	2.17	0.67	1.76	Pass
	20CMAX05-6.1	0.082	8.40	1.66	0.082	0.99	1.12	1.12	0.66	1.75	Fail
	20CMIN05-7.6	0.050	8.82	1.44	0.061	1.20	1.10	1.33	0.65	1.73	Fail
	20CMAX05-7.6	0.082	7.26	1.68	0.073	0.88	1.13	0.99	0.66	1.75	Fail
	20CMIN05-9.1	0.050	10.38	1.43	0.059	1.17	1.10	1.29	0.65	1.73	Fail
	20CMAX05-9.1	0.082	7.73	2.07	0.074	0.90	1.15	1.04	0.68	1.77	Fail
	Average		**8.61**	**1.71**		**1.17**		**1.32**	**0.66**	**2.34**	**Fail**

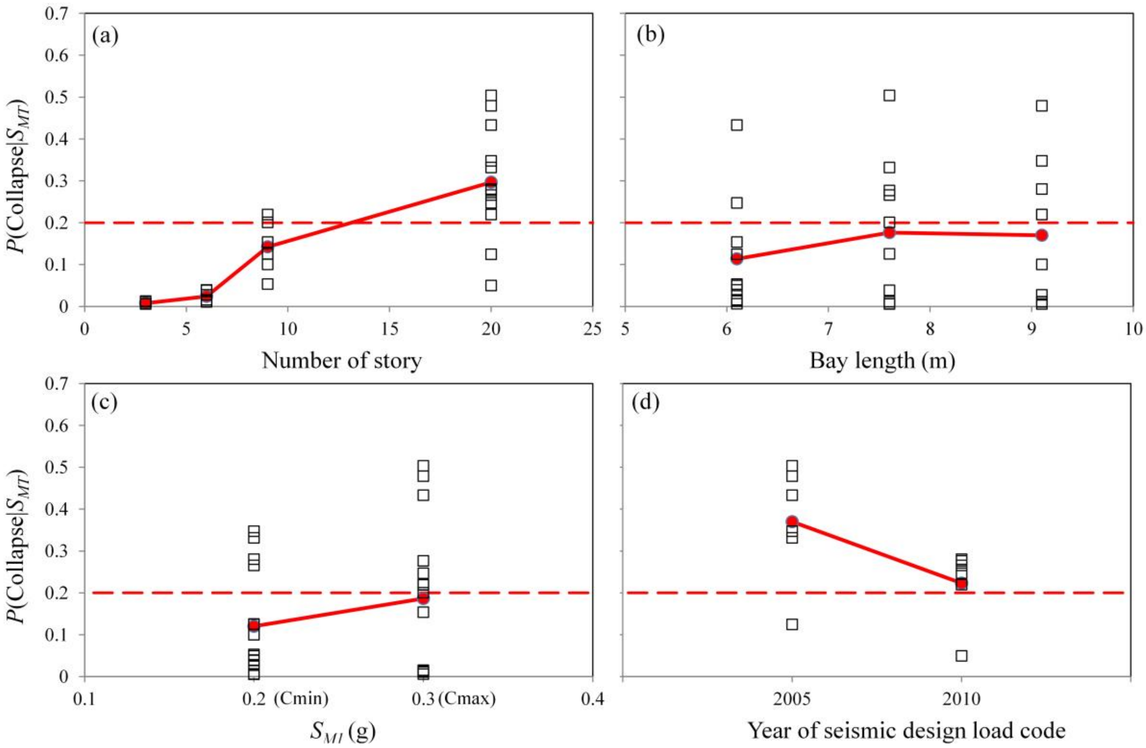

Fig. 6 Seismic response coefficient of model frames for (a) SDC C_{min} and (b) SDC C_{max}

7. Seismic performance of IMFs according to different rotation capacities of connections

This study evaluated the effect of connection rotation capacities on the seismic performance of IMFs. This study considered IMF connections fractured at rotations of 0.02 and 0.03 radian (connection rotation capacity) as well as ductile connection without fracture. Figs. 7-8 shows the collapse probabilities of IMFs for MCE ground motions with respect to different rotation capacities of their connections. Since 3- and 6- story frames with connections having a rotation capacity of 0.02 radian, satisfied the FEMA P695 criteria, those frames were not included in Figs. 7-8. Tables 3-4 summarize the results of seismic performance evaluation for IMFs with connections having a rotation capacity of 0.03 radian and ductile capacity, respectively.

As shown in Fig. 7(a), all 9 story IMFs designed for SDC C_{min} using ASCE 7-10 design seismic forces satisfied the FEMA P695 criteria irrespective of their rotation capacities. The IMFs designed for SDC C_{max} with a span length of 6.1 m satisfied the seismic performance criteria when their connections had a rotation capacity of 0.02 radian or larger, whereas the frames with span lengths of 7.6 and 9.1 m met the criteria only when their connections had a rotation capacity of 0.03 radian or larger (Fig. 7(b)).

Fig. 7(c) shows that 20 story frames with a span length of 6.1 m designed for SDC C_{min} satisfied the FEMA P695 criteria when their connections had a rotation capacity of 0.02 radian or larger. However, the frames with span lengths of 7.6 and 9.1 m met the criteria only when their connections had a rotation capacity as large as 0.03 radian. As shown in Fig. 7(d), 20 story frames designed for SDC C_{max} did not satisfied the criteria when their connections had a rotation capacity of 0.02 radian.

Fig. 8 shows the collapse probability of 20-story frames designed using ASCE 7-05 design seismic forces rather than using ASCE 7-10 design seismic forces. IMFs with a span length of 6.1 m designed for SDC C_{min} satisfied the criteria irrespective of the rotation capacity of their connections (Fig. 8(a)). However, the frames with span lengths of 7.6 m and 9.1 m satisfied the criteria only when they had ductile connections. It is noted that 20 story frames designed using the ASCE 7-10 design seismic forces satisfied the criteria when their connections had a rotation capacity of 0.03 radian or larger.

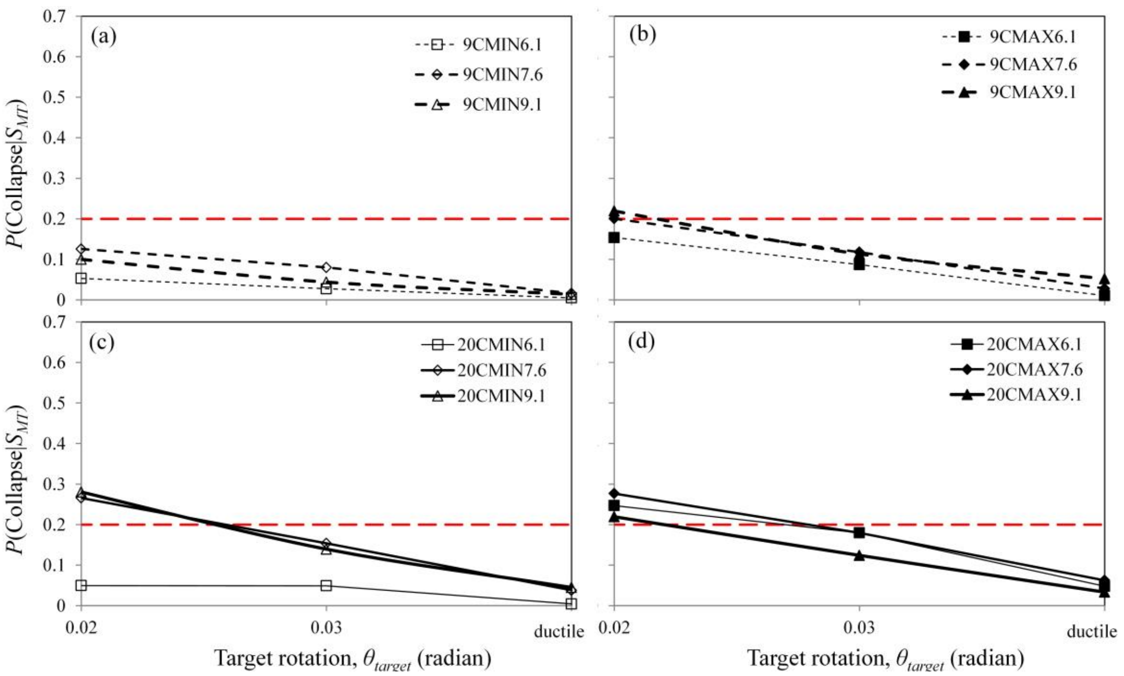

Fig. 7 Probability of collapse for MCE earthquake with response to rotation capacity for each building designed by ASCE7-10 seismic load

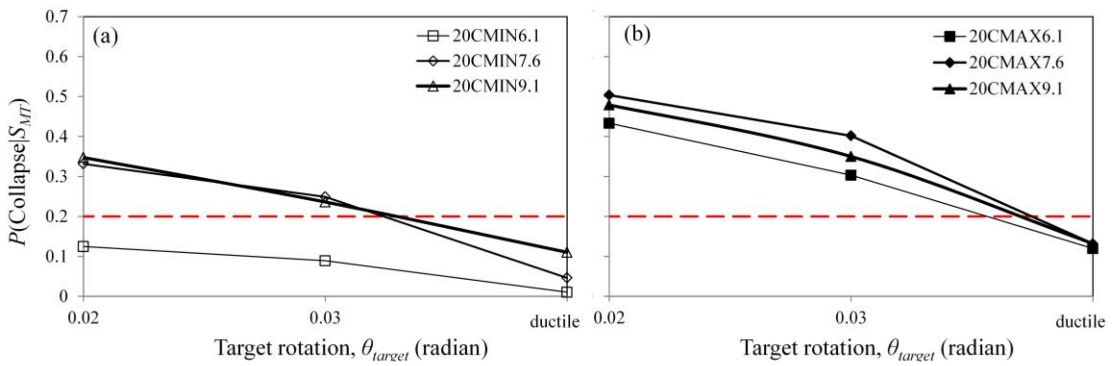

Fig. 8 Probability of collapse for MCE earthquake with response to rotation capacity for 20-story building designed by ASCE7-05 seismic load

Table 3 Summary of collapse margin parameters and acceptance check for IMF systems with connections having a rotation capacity of 0.03 radian

Performance group	Arch.ID	Computed overstrength and collapse margin parameters							Acceptance check		
		S_{MT}	Ω	μ_T	S_{CT}	*CMR*	*SSF*	*ACMR*	β_{TOT}	Accept. *ACMR*	Pass /Fail
Min6-10-3%	3CMIN10-6.1	0.233	5.06	2.76	1.394	5.98	1.12	6.66	0.71	1.82	Pass
	6CMIN10-6.1	0.134	6.27	2.60	0.434	3.24	1.19	3.84	0.70	1.81	Pass
	9CMIN10-6.1	0.094	9.13	1.99	0.298	3.18	1.15	3.65	0.68	1.77	Pass
	20CMIN10-6.1	0.050	4.56	3.50	0.136	2.71	1.23	3.32	0.73	1.84	Pass
	Average		**6.26**	**2.71**		**3.77**		**4.37**	**0.71**	**2.47**	**Pass**
Max6-10-3%	3CMAX10-6.1	0.381	4.69	3.05	1.919	5.04	1.11	5.61	0.73	1.84	Pass
	6CMAX10-6.1	0.219	4.75	3.11	0.964	4.40	1.19	5.25	0.73	1.84	Pass
	9CMAX10-6.1	0.153	5.64	2.64	0.337	2.20	1.19	2.62	0.71	1.81	Pass
	20CMAX10-6.1	0.082	5.34	2.49	0.132	1.61	1.18	1.90	0.70	1.80	Pass
	Average		**5.11**	**2.82**		**3.31**		**3.84**	**0.71**	**2.50**	**Pass**
Min7-10-3%	3CMIN10-7.6	0.233	4.90	2.86	1.403	6.01	1.12	6.73	0.72	1.83	Pass
	6CMIN10-7.6	0.134	6.35	2.78	0.446	3.33	1.19	3.98	0.71	1.82	Pass
	9CMIN10-7.6	0.094	8.82	1.96	0.210	2.25	1.15	2.58	0.67	1.76	Pass
	20CMIN10-7.6	0.050	6.42	1.81	0.088	1.74	1.14	1.98	0.67	1.75	Pass
	Average		**6.62**	**2.35**		**3.33**		**3.81**	**0.69**	**2.43**	**Pass**
Max7-10-3%	3CMAX10-7.6	0.381	4.94	2.98	2.455	6.44	1.11	7.17	0.73	1.84	Pass
	6CMAX10-7.6	0.219	5.04	3.12	1.061	4.85	1.19	5.79	0.73	1.84	Pass
	9CMAX10-7.6	0.153	5.70	2.37	0.297	1.94	1.17	2.28	0.69	1.79	Pass
	20CMAX10-7.6	0.082	5.47	2.20	0.133	1.61	1.16	1.88	0.69	1.78	Pass
	Average		**5.29**	**2.67**		**3.71**		**4.28**	**0.71**	**2.48**	**Pass**
Min9-10-3%	3CMIN10-9.1	0.233	4.92	2.82	1.459	6.25	1.12	6.99	0.72	1.83	Pass
	6CMIN10-9.1	0.134	6.47	2.55	0.492	3.67	1.18	4.34	0.70	1.81	Pass
	9CMIN10-9.1	0.094	9.23	1.88	0.258	2.75	1.14	3.14	0.67	1.76	Pass
	20CMIN10-9.1	0.050	7.54	1.78	0.091	1.81	1.13	2.06	0.67	1.75	Pass
	Average		**7.04**	**2.26**		**3.62**		**4.13**	**0.69**	**2.42**	**Pass**
Max9-10-3%	3CMAX10-9.1	0.381	4.94	2.78	2.049	5.38	1.11	5.95	0.71	1.82	Pass
	6CMAX10-9.1	0.219	5.12	2.82	1.011	4.62	1.18	5.45	0.72	1.83	Pass
	9CMAX10-9.1	0.153	6.43	2.51	0.302	1.98	1.18	2.33	0.70	1.80	Pass
	20CMAX10-9.1	0.082	4.85	2.22	0.156	1.90	1.16	2.21	0.69	1.78	Pass
	Average		**5.34**	**2.58**		**3.47**		**3.99**	**0.70**	**2.47**	**Pass**
20F-05-3%	20CMIN05-6.1	0.050	9.22	2.49	0.110	2.18	1.18	2.57	0.70	1.80	Pass
	20CMAX05-6.1	0.082	8.72	1.87	0.102	1.24	1.14	1.41	0.67	1.76	Fail
	20CMIN05-7.6	0.050	9.02	1.77	0.070	1.39	1.13	1.57	0.67	1.75	Fail
	20CMAX05-7.6	0.082	7.43	2.04	0.084	1.03	1.15	1.18	0.68	1.77	Fail
	20CMIN05-9.1	0.050	10.64	1.71	0.072	1.43	1.13	1.61	0.66	1.75	Fail
	20CMAX05-9.1	0.082	7.85	2.55	0.091	1.11	1.18	1.31	0.70	1.81	Fail
	Average		**8.81**	**2.07**		**1.39**		**1.61**	**0.68**	**2.39**	**Fail**

Table 4 Summary of collapse margin parameters and acceptance check for IMFs with ductile connections

Performance group	Arch.ID	Computed overstrength and collapse margin parameters							Acceptance check		
		S_{MT}	Ω	μ_T	S_{CT}	*CMR*	*SSF*	*ACMR*	β_{TOT}	Accept. *ACMR*	Pass /Fail
Min6-10-ductile	3CMIN10-6.1	0.233	5.09	5.31	1.908	8.18	1.17	9.59	0.73	1.84	Pass
	6CMIN10-6.1	0.134	6.41	5.26	0.683	5.10	1.29	6.59	0.73	1.84	Pass
	9CMIN10-6.1	0.094	9.19	2.95	0.498	5.31	1.20	6.39	0.72	1.84	Pass
	20CMIN10-6.1	0.050	4.75	6.17	0.255	5.07	1.32	6.70	0.73	1.84	Pass
	Average		**6.36**	**4.92**		**5.92**		**7.32**	**0.73**	**2.53**	**Pass**
Max6-10-ductile	3CMAX10-6.1	0.381	4.69	5.92	2.755	7.23	1.17	8.45	0.73	1.84	Pass
	6CMAX10-6.1	0.219	4.75	4.42	1.406	6.42	1.24	7.98	0.73	1.84	Pass
	9CMAX10-6.1	0.153	5.64	3.67	0.660	4.31	1.24	5.32	0.73	1.84	Pass
	20CMAX10-6.1	0.082	5.34	2.94	0.228	2.77	1.20	3.33	0.72	1.84	Pass
	Average		**5.11**	**4.24**		**5.18**		**6.27**	**0.73**	**2.53**	**Pass**
Min7-10-ductile	3CMIN10-7.6	0.233	4.90	4.90	1.911	8.19	1.16	9.54	0.73	1.84	Pass
	6CMIN10-7.6	0.134	6.46	4.16	0.752	5.61	1.25	7.04	0.73	1.84	Pass
	9CMIN10-7.6	0.094	9.02	3.05	0.367	3.92	1.21	4.73	0.73	1.84	Pass
	20CMIN10-7.6	0.050	6.46	2.83	0.150	2.98	1.20	3.56	0.72	1.83	Pass
	Average		**6.71**	**3.73**		**5.18**		**6.22**	**0.72**	**2.53**	**Pass**
Max7-10-ductile	3CMAX10-7.6	0.381	4.94	4.99	3.118	8.18	1.15	9.44	0.73	1.84	Pass
	6CMAX10-7.6	0.219	5.04	4.07	1.360	6.21	1.23	7.64	0.73	1.84	Pass
	9CMAX10-7.6	0.153	5.70	3.80	0.493	3.22	1.24	4.00	0.73	1.84	Pass
	20CMAX10-7.6	0.082	5.47	2.70	0.205	2.49	1.19	2.97	0.71	1.82	Pass
	Average		**5.29**	**3.89**		**5.03**		**6.01**	**0.72**	**2.52**	**Pass**
Min9-10-ductile	3CMIN10-9.1	0.233	4.92	5.17	1.941	8.32	1.17	9.73	0.73	1.84	Pass
	6CMIN10-9.1	0.134	6.51	4.51	0.809	6.04	1.27	7.65	0.73	1.84	Pass
	9CMIN10-9.1	0.094	9.49	3.06	0.383	4.09	1.21	4.95	0.73	1.84	Pass
	20CMIN10-9.1	0.050	7.54	2.10	0.138	2.74	1.16	3.17	0.68	1.77	Pass
	Average		**7.12**	**3.71**		**5.30**		**6.38**	**0.71**	**2.50**	**Pass**
Max9-10-ductile	3CMAX10-9.1	0.381	4.94	4.94	2.706	7.10	1.15	8.19	0.73	1.84	Pass
	6CMAX10-9.1	0.219	5.12	4.11	1.485	6.79	1.23	8.35	0.73	1.84	Pass
	9CMAX10-9.1	0.153	6.43	3.09	0.411	2.68	1.21	3.25	0.73	1.84	Pass
	20CMAX10-9.1	0.082	4.85	3.24	0.256	3.11	1.22	3.78	0.73	1.84	Pass
	Average		**5.34**	**3.85**		**4.92**		**5.89**	**0.73**	**2.54**	**Pass**
20F-05-ductile	20CMIN05-6.1	0.050	9.22	3.14	0.223	4.43	1.21	5.38	0.73	1.84	Pass
	20CMAX05-6.1	0.082	8.86	2.38	0.159	1.93	1.17	2.26	0.69	1.79	Pass
	20CMIN05-7.6	0.050	9.10	2.77	0.140	2.78	1.19	3.32	0.71	1.82	Pass
	20CMAX05-7.6	0.082	7.50	3.00	0.154	1.87	1.21	2.25	0.73	1.84	Pass
	20CMIN05-9.1	0.050	10.68	2.07	0.100	1.99	1.15	2.30	0.68	1.77	Pass
	20CMAX05-9.1	0.082	7.85	3.33	0.152	1.84	1.22	2.25	0.73	1.84	Pass
	Average		**8.87**	**2.78**		**2.47**		**2.96**	**0.71**	**2.49**	**Pass**

As shown in Fig. 8(b), the collapse probabilities of 20 story IMF frames designed for SDC $C_{\max}$ using ASCE 7-05 design seismic forces is much larger than those for SDC $C_{\min}$. Regardless of span length, the frames designed SDC C_{max} failed to satisfy the criteria even though the rotation capacity of their connections was 0.03 radian. The frames were only satisfactory only when they had ductile connections. Note that, regardless of the level of SDC and connection rotation capacity, 20-story frames designed for ASCE 7-10 design seismic forces (Fig. 7(c)) satisfied the criteria when their connections had a rotation capacity of 0.03 radian.

8. Conclusions

This study evaluated the seismic performance of steel intermediate moment frames designed considering different design parameters and seismic design forces. To conduct the seismic performance evaluation according to FEMA P695, three different rotation capacities of IMF connections were considered. The conclusions are summarized as follows:

• Even though IMF connections had a rotation capacity of 2%, which is the minimum rotation capacity required by AISC 341-10, twelve IMFs failed the acceptance criteria among 30 model frames, and one performance group did not satisfy the acceptance criteria among 7 performance groups.

• The number of stories significantly affected the probability of collapse for MCE ground motions. With an increase in the number of stories, the collapse probability became larger. The frames designed using ASCE 7-05 design seismic forces have larger collapse probability than that of the frames designed using ASCE 7-10. The probability of collapse was not significantly affected by different span lengths. The frames designed for SDC C_{max} had a larger collapse probability than corresponding frames designed for SDC C_{min}.

• To satisfy the FEMA P695 criteria, it is recommended that nine-story (9-story) and twenty (20-story) IMFs designed using ASCE 7-10 design seismic forces should have a rotation capacity of 0.03 radian or larger.

• The collapse probability of the 20-story frames designed using ASCE 7-05 seismic force was larger than that of the frames designed using ASCE 7-10 design seismic forces because ASCE 7-05 design seismic forces for the 20-story frame was lower than the ASCE 7-10 design seismic forces. Twenty story (20-story) IMFs designed using the ASCE 7-05 design seismic forces satisfied the acceptance criteria only when they had ductile connections.

Acknowledgments

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