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Seismic Collapse Evaluation of Steel Intermediate Moment Frames

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***Abstract*—A reduced beam section with a bolted web connection (RBS-B) is permitted for use only in Intermediate Moment Frames (IMF). In order to investigate whether IMFs with RBS-B connections provide a satisfactory seismic performance, this study evaluates the seismic performance of 3-, 6- 9- and 20-story IMFs with pre-qualified RBS-B connections following the FEAM-P695 procedure. Analysis results indicate that the probability of collapses is sensitive to the design parameters of IMF buildings with RBS-B connections.**

Keywords-performance evaluation; intermediate moment frames; reduced beam section; connection fracture

I. INTRODUCTION

The RBS moment connection limits the amount of moment demand transferred to the column face by selectively trimming portions of the beam flanges. This promotes the formation of a plastic hinge at the location where the beam section is reduced thereby protecting the beam-column connection, and increasing the moment frame ductility and energy dissipation capacity.

FEMA350 [1] reported RBS-W connection can develop a total of 0.04 radian with high confidence, sufficiently for the Special Moment Frames to repeatedly achieve a 4% story drift angle during a design-level earthquake without connection fracture. And ANSI/AISC 358-05 [3] allows the RBS-B connection to be used as a prequalified moment connection for IMF systems. Han et al. [6], however, investigated the cause of fracture in RBS-B connections designed according to FEMA 350 [1] and found that the connection moment strength equation specified in FEMA 350 [1] overestimates the actual moment strength of RBS-B connections, leading to connection brittle failure before the plastic moment capacity is reached at reduced beam sections.

When a moment frame experiences brittle connection fracture, the seismic response demands on the remaining frame element increase substantially, increasing the probability of undesirable seismic response. Therefore, it is necessary to evaluate the seismic performance of IMFs buildings with RBS-B connections designed in accordance with current design code requirements [3].

In order to investigate whether IMF systems with RBS-B connections provide satisfactory seismic performance, a seismic performance evaluation based on the procedure specified in FEAM-P695 [2] is conducted in this study. Four 3-, 6-, 9-, and 20-story model buildings with RBS-B connections are designed in accordance with current design code requirements considering different bay widths, and seismic design categories.

II. ANALYTICAL MODEL FOR IMF SYSTEMS WITH RBS-B MOMENT CONNECTIONS

A. Hysteretic Behavior of RBS-B Moment Connections

The moment strength of fracture-resistant RBS-B connections gradually deteriorates due to the local buckling after the peak moment strength is attained as shown in Fig. 1a, whereas the fracture-prone RBS-B connections Fig.1b [6] experiences connection fracture prior to gradual deterioration of moment strength, causing a sharp decrease in moment strength [6]. The fracture-resistant RBS-B connections, the connection moment strength, M_n, is larger than the moment demand at the column face, $M_{f\text{-}pr}$ corresponding to the probable maximum moment at the center of the reduced beam section. The connections, therefore, behaves in a ductile manner when they experience significant strength degradation beyond $M_{f\text{-}pr}$ due to local buckling. In contrast, the moment demand on the fracture-prone RBS-B connections exceeds the connection moment strength capacity, which leads to connection fracture. In order to predict the incidence of connection fracture, we use (1) with (2) as proposed by Han et al. [6].

$$M_{f\text{-}pr} \leq M_n \tag{1}$$

$$M_n = M_{n\text{-}flange} + M_{n\text{-}bolt} \tag{2}$$

B. Analytical Model for RBS-B Moment Connections

An analytical model for RBS-B connections is proposed based on the M2 model which can reflect clear lengths for beams and columns with explicit modeling of the panel zone as shown in Fig.2.

Columns are modeled using a fiber section model consisting of steel material with a strain hardening ratio of 3% (Fig.2a). To represent the hysteretic behavior of the panel zone, two spring elements are placed to simulate the tri-linear force-deformation relationship as shown in Fig.2b, including the contribution of post-yield stiffness and strength of column flanges after the column web yields.

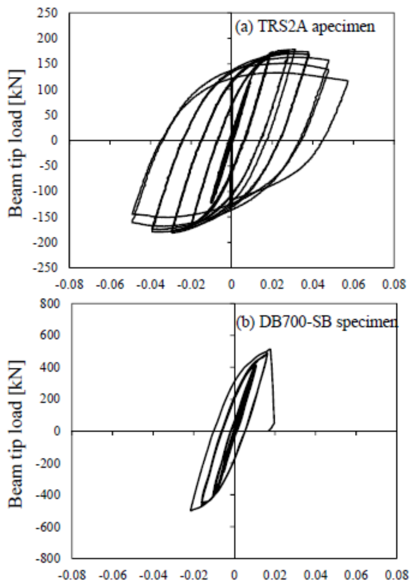

Figure 1. Cyclic behavior of RBS-B connection specimens: (a) fracture-resistant RBS-B specimen, (b) fracture-prone RBS-B specimen

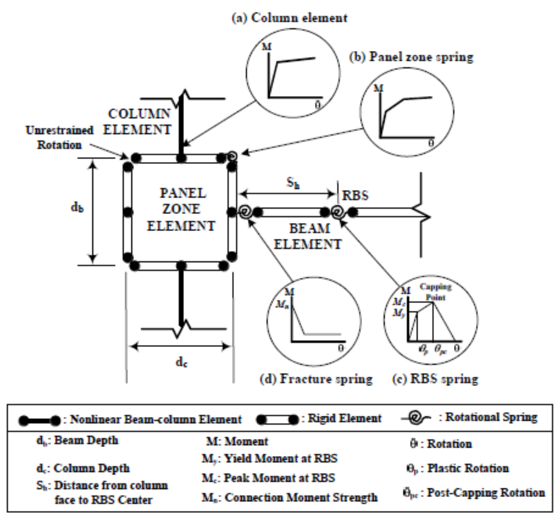

Figure 2. Analytic model for the RBS-B connections

For the beam-column connection, a fracture spring is installed (Fig.2d) to fail when the moment demand at the connection reaches the connection moment strength [6]. The RBS is modeled with an inelastic rotational spring element, placed at the center of the RBS (Fig.2c) to simulate the strength and the stiffness deterioration using a tri-linear backbone curve.

In Fig.2c, $M_y(=S_xF_y)$ and $M_c(=C_{pr}Z_{RBS}F_y)$ denote the yield and maximum moment strengths, respectively, and θ_p and θ_{pc} are the plastic rotation and post-capping rotation capacities at the reduced beam section in the beam, respectively. Here, S_x is the elastic section modulus of a whole beam section, Z_{RBS} is the effective plastic section modulus of the reduced beam section, and C_{pr} is the peak connection strength coefficient.

III. Summary of the FEMA-P695 Procedure for Seismic Performance Evaluation

FEAM-P695 [2] provided a methodology for quantifying system performance and response parameters for use in seismic design. The two performance objectives used in FEAM-P695 are:

(1) The probability of collapse for the maximum considered earthquake (MCE) ground motions is recommended to be 10% or less on average across a performance group that contains model frames having a specific seismic force resisting system with different configurations.

(2) For an individual model frame, the probability of collapse is 20%, or less.

A step-by-step procedure to determine the average value ($\overline{ACMR}$) and individual value ($ACMR_i$) of the adjusted collapse margin ratio is as follows:

- Select the model frames having a specific seismic force resisting system of interest which reflects the range of design variables such as number of stories s, bay lengths, and seismic design categories (SDC), and design the selected frames according to current seismic design provisions. Divide the model frames into performance groups that share a common set of features and behavioral characteristics.
- Idealize the model frame using a proper analytical model.
- Conduct nonlinear static analysis to determine the overstrength factor, Ω, and period-based ductility, $\mu_T(=\delta_u / \delta_{yeff})$ using (3), and (4) with (5), respectively.

$$\Omega = \frac{V_{max}}{V_d} \tag{3}$$

$$\mu_T = \frac{\delta_u}{\delta_{y,eff}} \tag{4}$$

$$\delta_{y,eff} = C_0 \frac{V_{max}}{W}\left[\frac{g}{4\pi^2}\right]\left(max\left(T_n, T_1\right)\right)^2 \tag{5}$$

Where V_{max} is the maximum base shear resistance, V_d is the design base shear, δ_u is the roof drift displacement at the point of 20% strength loss, $\delta_{y,eff}$ is the effective yield roof displacement.

Conduct the incremental dynamic analysis (IDA) for computing median collapse capacity, and calculate the collapse margin ratio, $\hat{S}_{CT}$, for each model frame. Collapse capacity is represented by pseudo spectral acceleration $PSA(T_n,5\%)$ at the fundamental period (T_n) of a 5% damped SDF system. The CMR is calculated using (6), where S_{MT} is the pseudo spectral acceleration at T_n of a model frame corresponding to the maximum considered earthquake at a site of interest:

$$CMR = \frac{\hat{S}_{CT}}{S_{MT}} \tag{6}$$

Calculate the spectral shape factor (*SSF*) that accounts for the spectral shape of rare ground motion and adjust the *CMR* where $\bar{\varepsilon}_0$ is 1.0 for SDC B and C

$$SSF = exp\left[\beta_1\left(\bar{\varepsilon}_0(T) - \bar{\varepsilon}(T)_{record}\right)\right] \tag{7}$$

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

The *ACMR* is the calculated using (9)

$$ACMR = CMR \times SSF \tag{9}$$

Determine whether individual model frames and the performance group satisfy the acceptable performance criteria specified in (1) and (2).

IV. FEAM-P695 SEISMIC COLLAPSE EVALUATION OF THE MODEL BUILDINGS

A. Model Performance Group and Ground Motions

This study adopts the seismic performance evaluation procedure prescribed by FEAM-P695 [2]. Four performance groups comprising frame model with varying number of stories, bay widths, and different seismic design categories (SDC) are considered. The seismic force resisting system for the model frame is the IMF with RBS-B connections. Frames with 3, 6, 9 and 20 stories are considered. The bay widths considered are 6 and 9 m. Two seismic design categories are considered, the lower and upper bounds of Seismic Design Category C (SDC C_{min} and SDC C_{max}). Note that the SDC C building height is not limited by ASCE/SEI 7 [4] for IMF systems. Table 1 summarizes the 16 different models considered in this study, grouped into four FEAM-P695 performance groups.

The buildings are assumed to be standard office buildings (occupancy category II) located on sites classified as site class D. For IMF systems, the response modification coefficient (R) is 4.5, system overstrength factor (Ω_0) is 3, and deflection amplification factor (C_d) is 4 using specified value in ASCE/SEI 7-10 [4]. The gravity dead and live loads for design are 4.12 kPa and 0.96 kPa, respectively. The IMFs are designed according to ASCE/SEI 7 [4] along with the IMF system design requirements specified in ANSI/AISC-341 and the design procedures for the RBS connection specified in ANSI/AISC 358 [3]. To determine the dimensions for the RBS section, the $M_{f\text{-}pr}/M_n$ ratio is set to 0.95.

To conduct the incremental dynamic analysis, 44 far-field ground motions, obtained by FEAM-P695 [2] from the PEER NGA database considering earthquake magnitudes ranging from 6.5 to 7.6 and site classes C and D are used.

B. FEAM-P695 Seismic Collapse Evaluation Result

Seismic performance evaluation is conducted for the 16 model frames shown in Table 1 using the procedure prescribed in FEAM-P695 [2]. Table 2 summarizes the evaluation results for each model building. Five of the 16 model frames and one of the four performance groups fail to meet the FEAM-P695 acceptance criteria. Fig. 10 shows the probability of collapse for MCE earthquake with respect variables.

The failed five model frames are 6C_{max}-6, 9C_{max}-6, and 20C_{max}-6 with a bay width of 6 m designed for SDC C_{max}, 20C_{min}-6 with a bay width of 6 m designed for SDC C_{min} and 20C_{max}-9 with a bay width of 9 m designed for SDC C_{max}. The failed performance group is Group 1 that contains model frames with bay length of 6 m designed for SDC C_{min}.

TABLE I. THE PROPERTIES OF IMFS AND THE FEAM-P695 PERFORMANCE GROUP

Performance Group	*Type ID*	*SDC*	*Bay size*	*No. of Stories*
Group1	3C_{min}-6	C_{min}	6m	3
	6C_{min}-6			6
	9C_{min}-6			9
	20C_{min}-6			20
Group2	3C_{max}-6	C_{max}		3
	6C_{max}-6			6
	9C_{max}-6			9
	20C_{max}-6			20
Group3	3C_{min}-9	C_{min}	9m	3
	6C_{min}-9			6
	9C_{min}-9			9
	20C_{min}-9			20
Group4	3C_{max}-9	C_{max}		3
	6C_{max}-9			6
	9C_{max}-9			9
	20C_{max}-9			20

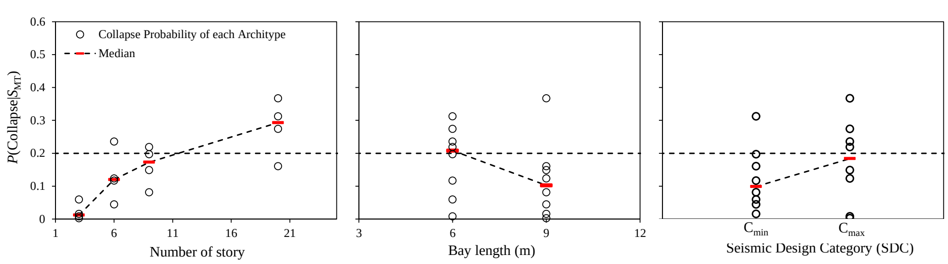

Figure 3. Probability of collapse for MCE earthquake with respect to: (a) number of story, (b) bay length and (c) seismic design category

TABLE II. SUMMARY OF COLLAPSE MARGIN PARAMETERS AND ACCEPTANCE CHECK

Performance Group	*Type ID*	*Computed overstrength and collapse margin paramerter*							*FEAM-P695 acceptance check*		
		S_{MT}	Ω	μ_T	S_{CT}	*CMR*	*SSF*	*ACMR*	β_{ToT}	Accept ACMT	Pass/Fail
Group1	$3C_{min}$-6	0.233	3.183	1.315	0.609	2.61	1.05	2.75	0.65	1.73	Pass
	$6C_{min}$-6	0.134	4.981	1.608	0.263	1.96	1.12	2.20	0.66	1.74	Pass
	$9C_{min}$-6	0.094	6.090	1.265	0.150	1.60	1.08	1.74	0.65	1.72	Pass
	$20C_{min}$-6	0.050	3.252	1.574	0.062	1.24	1.12	1.38	0.66	1.74	Fail
Average			**4.377**	**1.441**		**1.85**		**2.02**	**0.65**	**2.31**	**Fail**
Group2	$3C_{max}$-6	0.381	5.203	1.350	1.722	4.52	1.05	4.76	0.65	1.73	Pass
	$6C_{max}$-6	0.219	3.825	1.356	0.322	1.47	1.09	1.60	0.65	1.73	Fail
	$9C_{max}$-6	0.153	5.504	1.347	0.232	1.51	1.09	1.66	0.65	1.73	Fail
	$20C_{max}$-6	0.082	3.785	1.480	0.110	1.34	1.11	1.48	0.65	1.74	Fail
Average			**4.579**	**1.383**		**2.21**		**2.37**	**0.65**	**2.30**	**Pass**
Group3	$3C_{min}$-9	0.233	4.631	1.651	0.902	3.87	1.07	4.16	0.66	1.75	Pass
	$6C_{min}$-9	0.134	5.932	2.410	0.373	2.78	1.17	3.27	0.70	1.80	Pass
	$9C_{min}$-9	0.094	5.752	1.664	0.210	2.24	1.13	2.52	0.66	1.75	Pass
	$20C_{min}$-9	0.050	3.491	1.496	0.087	1.73	1.11	1.92	0.66	1.74	Pass
Average			**4.952**	**1.805**		**2.65**		**2.97**	**0.67**	**2.36**	**Pass**
Group4	$3C_{max}$-9	0.381	5.753	1.786	2.359	6.19	1.08	6.66	0.67	1.75	Pass
	$6C_{max}$-9	0.219	4.715	1.905	0.421	1.92	1.13	2.18	0.67	1.76	Pass
	$9C_{max}$-9	0.153	5.413	1.615	0.272	1.78	1.12	1.99	0.66	1.74	Pass
	$20C_{max}$-9	0.082	4.823	1.812	0.091	1.10	1.14	1.25	0.67	1.75	Fail
Average			**5.176**	**1.780**		**2.75**		**3.02**	**0.67**	**2.35**	**Pass**

V. CONCLUSION

This study evaluated the seismic performance of intermediate moment frames with RBS-B connections designed according to current seismic design provisions. The procedure for seismic performance evaluation prescribed in FEAM-P695 [2] was used. An analytical model for RBS-B connections was developed to emulate the hysteretic behaviors of fracture-resistant and fracture-prone RBS-B connection specimens.

Sixteen model frames and four performance groups were with different number of stories, bay lengths and SDC values were considered. Five of 16 model frames and one of four performance groups failed to meet acceptable performance levels against collapse according to FEAM-P695 [2].

The collapse performance of IMFs with RBS-B connections evaluated using the FEAM-P695 procedure is, in general, not satisfactory. It is recommended that RBS-B connections should not be used in IMFs higher than 6 stories. In particular, it depends on the design parameters such that an increase in story height and a decrease in bay width lead to a higher probability of system collapse.

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