STRONG FRAME SPECIAL MOMENT FRAME CONNECTIONS TO REDUCE RESIDUAL DRIFTS IN STEEL FRAME BUILDINGS

J. Judd(1), S. Pryor(2), B. Chi(2)

(1) *Assistant Professor, University of Wyoming, jjudd5@uwyo.edu*
(2) *Director of Innovation, Simpson Strong-Tie, spryor@strongtie.com*
(2) *Engineering Manager for Lateral Systems, Simpson Strong-Tie, bchi@strongtie.com*

Abstract

Steel moment frames provide excellent seismic performance in terms of collapse prevention, but conventional beam-to-column connections in the moment frames achieve this performance through energy dissipation (damage) that occurs in the beam and column during an earthquake. Damage to the beam or column is difficult to repair and can lead to permanent (residual) story drifts. The Simpson Strong-Tie Strong Frame moment connection was developed to prevent damage to the beam or the column. The Strong Frame moment connection is a partially-restrained connection that uses a single plate shear connection to transfer shear and axial forces and a pair of Yield-Links®, one on the top of the beam top flange, one on the bottom of the beam bottom flange, to dissipate seismic energy.

In this paper, the seismic performance of buildings using Strong Frame special moment frame connections is compared to buildings using reduced beam section (RBS) connections. Four building heights are considered: 4-story, 8-story, and 12-story buildings designed for large spectral accelerations, and 20-story buildings designed for moderate spectral accelerations. The buildings were designed using the Equivalent Lateral Force (ELF) procedure in ASCE 7-16 and two deflection amplification factors, C_d were considered (C_d equal to 5.5, and C_d equal to 8.0). An analytical model of the moment-rotation behavior of the Strong Frame connections was calibrated using experimental data from large-scale tests. Nonlinear response history analyses were conducted using OpenSees finite element software. The analyses used the methodology described in FEMA P-695 *Quantification of Building Seismic Performance Factors*, and the drift was correlated to damage using the method described in FEMA P-58 *Seismic Performance Assessment of Buildings*. The results predict that Strong Frame buildings will be safer and will have less residual drift compared to buildings using RBS connections. The improved performance of buildings using Strong Frame special moment frame connections is achieved because the column panel zone is designed for a larger force in the Strong Frame connection design procedure, compared to the force that is used to design the RBS column panel zone, and because the effect of bending moment from seismic over-strength load combinations is not neglected.

Keywords: moment frame; residual drift; FEMA P-695

1. Introduction

Damage in buildings after an earthquake can be costly to repair, can exacerbate business downtime, and can hamper community recovery. Conventional beam-to-column connections in steel moment frame buildings provide adequate life safety, but do so through the formation of plastic hinges. The formation of plastic hinges in beams and columns of conventional connections can cause significant structural damage. Furthermore, although conventional connections are considered rigid (because they maintain the angle between and beam and column), moment frames are flexible compared to shear walls and conventional moment frame beam-to-column connections can lead to buildings that are susceptible to permanent (residual) story drifts during a major earthquake.

The Simpson Strong-Tie Strong Frame moment connection is a partially-restrained connection. The connection is described in AISC 358-16 *Prequalified connections for Special and Intermediate steel moment frames for seismic applications* [1]. The connection consists of single plate shear connections designed to transfer shear and axial forces and a pair of Yield-Links® (one at the beam top flange, one at the beam bottom flange) to transfer flexure and to act as a structural fuse. The structural fuse concentrates damage into components that can easily be replaced, while preventing damage to the structure (beam or column).

In a previous study, the predicted response of 2-story, 4-story, and 6-story residential buildings using Strong Frame connections was shown to be comparable to using RBS connections [2]. A subsequent study compared the response of mid-rise (4-story, 8-story) steel frame buildings and tall (12-story and 20-story) steel frame buildings using Strong Frame connections with buildings using RBS connections [3]. In that study, the moment frames were designed to meet ASCE 7-05 Minimum design loads for buildings and other structures [4] requirements, except that the frames were designed using a deflection amplification factor, C_d equal to the response modification factor, R. The results showed that the predicted seismic performance of the Strong Frame buildings was superior to the performance of the RBS buildings.

The objective of this study is to predict the seismic performance of buildings that utilize Strong Frame connections designed using ASCE 7-16 [5]. Seismic performance of archetypical steel frame buildings with Strong Frame connections is compared to the performance of buildings with RBS connections using the FEMA P-695 *Quantification of building seismic performance factors methodology* [6]. Nonlinear static pushover analyses and incremental dynamic nonlinear response history analyses are then used to predict structural response and building damage).

2. Approach

For purposes of comparison, this study uses a set of building archetypes that matches the set of building archetypes previously developed by the National Institute of Standards and Technology (NIST) for conventional steel moment frame buildings with RBS connections [6]. The building plan (Figure 1) is 42.7 m (140 ft) by 30.5 m (100 ft) with a 6.10 m (20 ft) bay size. The first story height is equal to 4.57 m (15 ft). The upper story height is equal to 3.96 m (13 ft). Each side has a three-bay perimeter moment frame.

In this study, 4-story, 8-story, 12-story, and 20-story buildings are evaluated. The 4-story, 8-story, and 12-story buildings were designed for the maximum spectral accelerations in ASCE 7-16 Seismic Design Category D (“SDC Dmax”), and the 20-story building was designed for the minimum spectral accelerations in ASCE 7-16 SDC D (“SDC Dmin”). Buildings were designed for a stiff soil site (ASCE 7-16 Site Class D). The moment frames were designed using the Equivalent Lateral Force (ELF) procedure in ASCE 7-16. The base shear coefficient, C_s was based on the maximum permitted period of vibration, $T=C_uT_a$. For drift calculations, the base shear coefficient was determined per Section 12.8.6.1 of ASCE 7-16 and the period used to compute drift was based on ASCE 7-16 Section 12.8.6.2.

For RBS connections, the moment at the column used to determine the column web panel zone shear was calculated using AISC 358-16 Equation 5.8-5. By contrast, for the Strong Frame, the moment is calculated using $F_u R_t Z$, where F_u is the minimum specified tensile stress, R_t is the ratio of expected to specified tensile stress, and Z is the plastic section modulus of the beam. Thus, for equivalent sections, the Strong Frame is designed for approximately a 23% larger force compared to the RBS frame. Furthermore, for the RBS frame the resistance factor, F is equal to 1.0 when determining available panel zone shear resistance according to AISC 360-16 *Specification for Structural Steel Buildings* [7] Equation J10-11. By contrast, for the Strong Frame F is equal to 0.9. Furthermore, for the RBS frame, the required axial forces are required to be determined from overstrength load combinations, but column moments are permitted to be neglected (refer to AISC 341-16 *Seismic provisions for structural steel buildings* [8] Section E3.3 User Note and Section D1.4a). By contrast, for the Strong Frame this exception is not permitted. The strong frame column is required to be designed for overstrength axial force and moment. AISC 358-16 Section 12.9 Step 13.2 requires that column strength satisfy the minimum of either the maximum load the system can deliver or the overstrength seismic loads. Taken together, these differences in design generally lead to larger column sections in the Strong Frame compared to the RBS frame.

The moment frame beam and column sizes are given in Table 1. For both Strong Frame and RBS frame designs, column sizes were increased instead of using doubler plates to produce an economical design. The resulting increase in column weight did not exceed 100 plf for the buildings considered in this study, which meets commonly accepted design guidelines [9,10]. For taller buildings where frame instability (P-Δ) controlled design, the maximum stability coefficient, θ_{max} in ASCE 7-16 Section 12.8.7 was conservatively taken equal to 0.10. Beams met the compactness requirements in AISC 341-16 and the beam slenderness ratio (L/r) was assumed to satisfied by beam bracing.

The building archetypes were modelled in the longitudinal direction with two-dimensional structural frames and concentrated plasticity using OpenSees finite element software [11]. Beam and columns were represented using an assembly of beam-column and spring elements to represent plastic hinge behavior. Panel zones were modeled explicitly using a parallelogram of rigid elements connected by a spring element to represent shear distortion behavior [12, 13].

RBS beam and column plastic hinge behavior was idealized using the **`Bilin`** moment-rotation model in *OpenSees*, and the Strong Frame plastic hinge behavior was idealized using the **`SteelMPF`** and **`MinMax`** uniaxial materials in *OpenSees*. The assumed location of the plastic hinge depended on the application (column versus beam) and type of beam-to-column connection. For Strong Frame connections, the plastic hinge was assumed to occur at the face of the column. For RBS connections, the plastic hinge was assumed to occur at the centerline of the radius-cut reduced section. Although it is recognized that first yielding of the beam occurs at the face of the column or at the link, and that for RBS connections, the center of rotation then eventually migrates into the beam span with a "final" center of rotation located at approximately half the beam depth from the column, the global building response is relatively insensitive to the precise location of the plastic beam hinge, regardless of the actual distribution of plasticity, because column plastic hinges tend to control global collapse [14]. In the RBS model, the ultimate rotation is equal to 0.20 rad, matching the value used by NIST. In the Strong Frame, parameters were calibrated to experimental data from large-scale tests [15,16]. Ultimate rotation is equal to 0.06 rad for monotonic loading, and 0.05 rad for cyclic loading.

A sequence of analyses was conducted in OpenSees for each archetypical building. First, a gravity pre-load analysis was used to simulate the expected gravity loads. Then, an eigenvalue analysis was used to calculate the periods of vibration (e.g. T_1). Next, a nonlinear static "pushover" analysis was used to determine system overstrength, Ω and system ductility, μ_T and to incorporate the effect of combined axial and flexure loads in columns. Finally, incremental nonlinear dynamic response history analyses were conducted using the FEMA P-695 "Far-Field" ground motion set to determine collapse safety. Building damage was predicted based on residual story drift in the response history analyses. Ground motions that caused collapse (if it occurred) were removed when computing the average peak and residual story drifts. Damage due to residual drift is based on damage states defined in Appendix C of FEMA P-58 *Seismic performance assessment of buildings* [17].

Table 1. Moment frame yield link, beam, and column sizes.

	Strong Frame				RBS Frame		
Story	Yield Link t_{stem} x b_{yield}	Beam	Interior Column	Exterior Column	Beam	Interior Column	Exterior Column
4-Story Building							
4	1.0x2.0 in.	W24x68	W24x162	W24x68	W18x35	W24x131	W24x84
3	1.0x3.125 in.	W24x76	W24x162	W24x68	W21x57	W24x131	W24x84
2	1.0x4.25 in.	W24x94	W24x250	W24x207	W24x68	W24x146	W24x94
1	1.0x3.5 in.	W24x94	W24x250	W24x207	W24x62	W24x146	W24x94
8-Story Building							
8	0.75x2.5 in.	W21x55	W27x114	W24x62	W18x35	W24x131	W24x94
7	0.75x3.75 in.	W24x84	W27x114	W24x62	W24x62	W24x131	W24x94
6	0.75x5.0 in.	W27x84	W27x235	W27x102	W24x62	W24x192	W24x131
5	1.0x4.5 in.	W27x94	W27x235	W27x102	W24x84	W24x192	W24x131
4	1.0x4.5 in.	W30x99	W27x235	W27x146	W24x84	W24x192	W24x131
3	1.0x4.625 in.	W30x108	W27x235	W27x146	W27x94	W24x192	W24x131
2	1.0x5.0 in.	W30x108	W27x258	W27x336	W27x94	W24x192	W24x131
1	1.0x5.0 in.	W30x108	W27x258	W27x336	W27x76	W24x192	W24x131
12-Story Building							
12	0.75x2.375 in.	W24x68	W27x129	W21x62	W18x35	W24x103	W24x94
11	0.75x3.875 in.	W27x84	W27x129	W21x62	W24x55	W24x103	W24x94
10	0.75x5.25 in.	W27x94	W36x182	W30x108	W24x68	W24x192	W24x131
9	1.0x4.625 in.	W30x108	W36x182	W30x108	W24x94	W24x192	W24x131
8	1.0x5.0 in.	W33x118	W36x194	W33x169	W27x94	W24x229	W24x131
7	1.0x5.0 in.	W36x118	W36x194	W33x169	W27x102	W24x229	W24x131
6	1.0x5.625 in.	W36x135	W36x262	W36x210	W27x114	W24x229	W24x176
5	1.0x5.5 in.	W36x135	W36x262	W36x210	W30x108	W24x229	W24x176
4	1.0x5.75 in.	W36x135	W36x282	W36x282	W30x108	W24x229	W24x229
3	1.0x5.625 in.	W36x135	W36x282	W36x282	W30x108	W24x229	W24x229
2	1.0x5.125 in.	W36x135	W36x330	W36x529	W30x116	W24x229	W24x279
1	1.0x6.0 in.	W36x150	W36x330	W36x529	W27x84	W24x229	W24x279
20-Story Building							
20	0.75x2.125 in.	W21x62	W30x99	W24x55	W21x44	W36x182	W36x160
19	0.75x3.25 in.	W21x62	W30x99	W24x55	W24x62	W36x182	W36x160
18	0.75x4.375 in.	W27x84	W36x150	W27x84	W24x84	W36x231	W36x160
17	0.75x5.0 in.	W30x99	W36x150	W27x84	W27x94	W36x231	W36x160
16	0.75x5.25 in.	W30x108	W36x210	W33x118	W30x116	W36x302	W36x182
15	1.0x5.5 in.	W33x118	W36x210	W33x118	W30x116	W36x302	W36x182
14	1.0x5.5 in.	W33x118	W36x210	W36x170	W30x116	W36x361	W36x231
13	1.0x5.5 in.	W33x118	W36x210	W36x170	W30x116	W36x361	W36x231
12	1.0x6.0 in.	W36x135	W36x302	W36x210	W33x130	W36x361	W36x262
11	1.0x6.0 in.	W36x135	W36x302	W36x210	W33x130	W36x361	W36x262
10	1.0x6.0 in.	W36x135	W36x361	W36x256	W33x141	W36x361	W36x302
9	1.0x6.0 in.	W36x160	W36x361	W36x256	W33x141	W36x361	W36x302
8	1.0x6.0 in.	W36x210	W36x441	W36x395	W36x170	W36x361	W36x361
7	1.0x6.0 in.	W36x330	W36x441	W36x395	W36x170	W36x361	W36x361
6	1.0x6.0 in.	W36x330	W36x441	W36x395	W36x170	W36x361	W36x361
5	1.0x6.0 in.	W36x330	W36x441	W36x395	W36x170	W36x361	W36x361
4	1.0x6.0 in.	W36x330	W36x441	W36x441	W36x182	W36x361	W36x361
3	1.0x6.0 in.	W36x330	W36x441	W36x441	W36x182	W36x361	W36x361
2	1.0x6.0 in.	W36x256	W36x441	W36x652	W36x170	W36x361	W36x361
1	1.0x6.0 in.	W36x170	W36x441	W36x652	W30x108	W36x361	W36x361

3. Results

Table 2 summarizes the results from the static pushover and incremental dynamic analyses. The Strong Frame buildings exhibit higher system overstrength compared to the RBS buildings, except for the 20-story RBS building. The Strong Frame buildings also exhibit greater period-based ductility compared to RBS buildings, except for the 4-story building. Both the RBS and Strong Frame buildings passed the P-695 criteria of no more than a 10% probability of collapse. However, the Strong Frame buildings demonstrate a higher collapse margin ration (CMR) compared to RBS buildings and the spectral shape factor (SSF) is greater for Strong Frame buildings compared to the RBS buildings. Consequently, the Strong Frame buildings have a higher adjusted collapse margin ratio (ACMR) compared to the RBS buildings.

Table 2. Dynamic response history analyses results.

Stories / SDC	SMF	Ω	μ_T	CMR	SSF	ACMR	Accept. ACMR
4-story	Strong Frame	2.81	7.55	1.73	1.39	2.40	1.52
SDC D_{max}	RBS	1.83	8.43	1.67	1.45	2.42	
8-story	Strong Frame	2.47	6.88	1.56	1.55	2.42	1.52
SDC D_{max}	RBS	2.06	4.64	1.50	1.44	2.15	
12-story	Strong Frame	2.43	7.36	2.05	1.58	3.24	1.52
SDC D_{max}	RBS	1.96	3.74	1.44	1.38	1.98	
20-story	Strong Frame	2.40	8.67	5.23	1.61	8.41	1.52
SDC D_{min}	RBS	4.56	2.99	3.74	1.32	4.96	

Table 3 summarizes residual story drifts. During the design basis earthquake (DBE) intensity ground motions, no Strong Frame buildings collapsed. By contrast, several RBS buildings collapsed. During the maximum considered earthquake (MCE) intensity ground motions, significantly more RBS buildings collapsed. During both the DBE and MCE intensities, the Strong Frame buildings had less residual drift compared to RBS buildings, except the residual drift is equal for the 20-story buildings during the DBE intensity. Accordingly, only non-structural damage is expected for Strong Frame buildings during the DBE intensity. During the MCE intensity, some (non-collapsed) RBS buildings are expected to be a total loss.

Table 3. Average maximum story drift for DBE-level intensity ground motions.

Stories / SDC	SMF	No. of Collapses	Residual	Expected damage
		DBE-level intensity ground motions		
4-story	Strong Frame	0	0.3%	Non-Structural Damage
SDC D_{max}	RBS	1	0.5%	Structural Damage
8-story	Strong Frame	0	0.4%	Non-Structural Damage
SDC D_{max}	RBS	1	0.6%	Structural Damage
12-story	Strong Frame	0	0.4%	Non-Structural Damage
SDC D_{max}	RBS	2	0.9%	Structural Damage
20-story	Strong Frame	0	0.3%	Non-Structural Damage
SDC D_{min}	RBS	0	0.3%	Non-Structural Damage
		MCE-level intensity ground motions		
4-story	Strong Frame	1	0.7%	Structural Damage
SDC D_{max}	RBS	3	0.9%	Structural Damage
8-story	Strong Frame	3	0.9%	Structural Damage
SDC D_{max}	RBS	4	1.2%	Total Loss
12-story	Strong Frame	1	0.7%	Structural Damage
SDC D_{max}	RBS	6	1.7%	Total Loss
20-story	Strong Frame	0	0.4%	Non-Structural Damage
SDC D_{min}	RBS	0	0.5%	Structural Damage

4. Conclusions

The seismic performance of 4-story, 8-story, 12-story, and 20-story archetypical Special steel moment frame buildings using Strong Frame connections designed using ASCE 7-16 was compared to the performance of buildings using RBS connections designed using ASCE 7-16. The results showed that Strong Frame buildings exhibit higher system overstrength, except for the 20-story building, greater period-based ductility, except for the 4-story building, and a higher CMR. Although both RBS and Strong Frame buildings passed the P-695 criteria of no more than a 10% probability of collapse, the probability of collapse was significantly lower for the 12-story and 20-story Strong Frame buildings. Under DBE level ground motions, no Strong Frame buildings collapsed. The improved seismic performance of Strong Frame buildings compared to RBS building is attributed to the larger design force for the Strong Frame column panel zone, compared to the RBS column panel zone, and to the inclusion of bending moment from seismic overstrength load combinations.

5. References

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