Probabilistic Assessment of Seismic Force Demands in Biaxially Loaded Columns in Chevron-Configured Special Concentrically Braced Frames

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ABSTRACT

Special concentrically braced frame (SCBF) columns are designed as force-controlled elements and are intended to respond elastically during moderate-to-high return-period events. When placed at the intersection of orthogonal chevron-configured braced frames with fixed beam-column connections, SCBF columns are subjected to biaxial loading, including flexural demands developed in the beams due to unbalanced tension-compression brace forces. A probabilistic assessment of the force demands in biaxially and uniaxially loaded columns in chevron-configured SCBF is presented herein. Nonlinear response history analyses are performed on three-dimensional models of 3-, 9- and 20-story SCBF, and statistical descriptions of the results are used to investigate (1) the force demands relative to the capacity-design-based and elastic designs suggested by the American Institute of Steel Construction (AISC) *Seismic Provisions,* (2) the implications of the flexural demands transmitted to columns (via braced frame beams), and (3) the combinatorial effects of demands in biaxially loaded columns generated by orthogonal ground-motion components. At the maximum considered earthquake (MCE) hazard level, the median axial force demands in the biaxially loaded first-story columns of the three-story building are approximately at the level corresponding to the expected brace strength and exceed the design forces amplified by the overstrength factor. Axial flexure interaction is especially significant in the biaxially loaded columns of all three building cases. The results also show that the combinatorial effect of axial forces transmitted to the biaxially loaded columns via the orthogonal braces is generally lower in taller buildings and also depends on the demand level.

Keywords: special concentrically braced frames, probabilistic assessment, biaxially loaded columns, orthogonal effects, chevron braces.

INTRODUCTION

Special concentrically brace frames (SCBF) are commonly used as the seismic lateral force-resisting system (LFRS) in commercial, educational and other types of buildings. This is largely due to their cost-effectiveness in providing the strength and stiffness needed for building structures located in high seismic regions. SCBF braces are the deformation-controlled elements and are designed and detailed to sustain inelastic deformations while serving as the primary source of energy dissipation for the system. The remaining frame elements (beams, columns, and brace connections) are force-controlled and intended to respond elastically during moderate-to-high return-period events. SCBF beams, columns, and brace connection elements are therefore designed utilizing capacity design principles so that, ideally, their required strength exceeds the maximum force demands that can be delivered by the deformation-controlled elements (braces). The chevron configuration is frequently used in SCBF designs because it can provide open spaces and flexible architectural layouts However, due to the unsymmetrical cyclic axial force-deformation response of the brace in tension and compression, significant moments can be placed in the connecting beam, which are also transmitted to the columns when flexurally restrained beam-column connections are used. Improper consideration of these moments, especially in SCBF columns subjected to high axial loads, can lead to inelastic response and undesirable performance of the system.

Because of building architectural or programmatic constraints, SCBF are sometimes configured with columns located in two intersecting, orthogonal braced frames. During earthquake shaking, these columns are subjected to biaxial loading due to the simultaneous action of horizontal ground-motion components. For the case of chevron SCBF with flexurally restrained beam-column connections, the orthogonal braced frames will place axial and flexural demands in the intersecting column. As noted earlier, these columns are designed as forced-controlled components and are intended to respond elastically during moderate-to-severe earthquake shaking. Estimation of the axial and flexural demands in SCBF columns is therefore an important

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part of the design process. In a real earthquake, these demands are affected by the extent and pattern of yielding in the brace elements.

Prior studies have used nonlinear response history analyses of two-dimensional structural models to investigate the seismic force demands in braced frame columns. Tremblay and Robert (2001) analyzed a set of chevron steel braced frames ranging in height from 2 to 12 stories. From the results of these analyses, the authors suggested that the brace columns be designed using a "full capacity design" approach where the axial forces are computed assuming simultaneous buckling of braces. Richards (2009) investigated the column seismic demands in SCBF with X-bracing, buckling restrained braced frames (BRBF), and eccentrically braced frames (EBF) with different heights and strength levels. The structural models were analyzed using ground motions that were scaled to be at or above the design spectra. The results showed that for low-rise SCBF, the column axial force demands exceeded the overstrength factor ($\Omega_0 = 2.0$) used to determine the upper limit on the demands for design. The author suggested using the full tensile capacity of the braces as the basis for computing column axial demands. For taller braced frames, the column axial demands in the upper stories were more than twice the design demands. However, it was also noted that this observation has limited practical implications because top-story columns are typically overdesigned. The maximum axial force demands in the base columns of taller buildings ranged from 55 to 75% of the design demands. Based on this finding, it was noted that using more realistic (less conservative) demands for braced frame column design in taller buildings could lead to significant cost savings.

To demonstrate a newly developed reliability-based methodology for establishing force demands in capacity-designed components of LFRS, Victorsson (2011) evaluated the force demands in SCBF columns. Structural models of 6- and 16-story SCBF were analyzed using incremental dynamic analyses to investigate the effect of height and number of deformation-controlled elements on brace connection and column force demands. The base column axial forces at the maximum considered earthquake (MCE) hazard level were found to be 90% and 50% of the capacity-design-based demands suggested by the 2010 AISC *Seismic Provisions* (AISC, 2010) for the 6- and 16-story frames, respectively. However, while the axial forces in the strength-controlled 6-story frame did not exceed the maximum required demands (elastic demands times the overstrength factor), this limit significantly underestimated the demands in the upper stories of the drift-controlled 16-story frame.

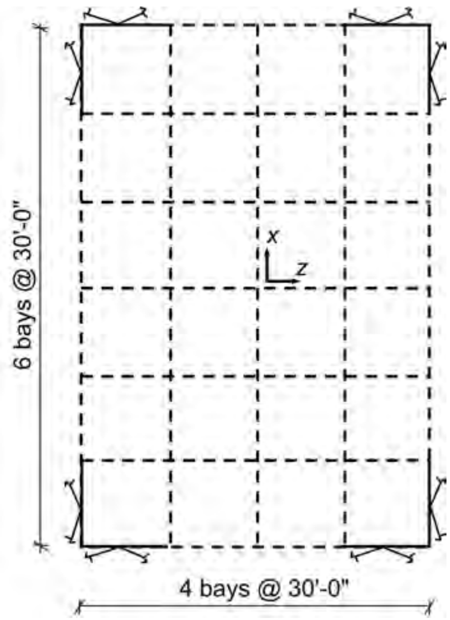
The objective of this study is to conduct a probabilistic assessment of biaxially and uniaxially loaded chevron SCBF columns to evaluate (1) the force demands relative to the capacity-design-based and elastic designs suggested by the 2010 AISC *Seismic Provisions*, (2) the implications of the flexural demands transmitted to columns (via braced frame beams) as a result of unbalanced tension-compression forces in chevron braces, and (3) the combinatorial effects of demands in biaxially loaded columns generated by orthogonal ground-motion components. These issues have direct implications to the design and performance of columns located in intersecting chevron-configured SCBF. Nonlinear response history analyses are conducted on three-dimensional SCBF structural models with varying heights using bi-directional loading. Force demands are described in a probabilistic manner to facilitate establishing reliability-based performance objectives for SCBF columns, which can then be linked to the prescribed design demands adopted by codes and standards.

DESCRIPTION OF BUILDING CASES

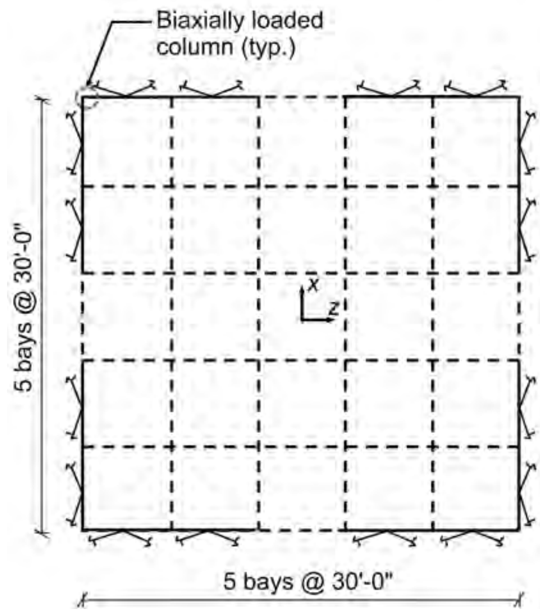
The three design cases used for the current study include 3- (SCBF-3S), 9- (SCBF-9S), and 20-story (SCBF-20S) buildings with chevron-configured SCBF. The plan dimensions, story heights, gravity loads, and framing layout of the three buildings are the same as the moment frame buildings used by Gupta and Krawinkler (1999) as shown in Figure 1. All bays are 30 ft wide and the typical story height is 13 ft. All three buildings have symmetric SCBF plan configurations and corner columns that are part of intersecting orthogonal braced frames. The braced frames in all three cases are all located on the perimeter of the building. All biaxially loaded (corner columns) are oriented with the web-direction in the z-direction (Figure 1).

The braced frames are designed in accordance with ASCE/SEI 7–10 (ASCE, 2010) and the 2010 AISC *Seismic Provisions*. (Note: All references to the AISC *Seismic Provisions* in this paper are references to the 2010 version unless noted otherwise.) The seismicity parameters ($S_S = 2.17g$ and $S_1 = 0.75g$) are based on a location in Los Angeles (–118.162, 33.996) with Site Class D. The designs are based on Risk Category II and Seismic Design Category E with a response modification factor $R = 6$, overstrength factor $\Omega_0 = 2$, drift amplification factor $C_D = 5$, and importance factor, $I = 1.0$. The seismic design loads are obtained from response spectrum analyses performed using *RAM Steel* (Bentley). SCBF beam-column connections are assumed to be flexurally rigid, and column bases are pinned. Key seismic design parameters are summarized in Table 1.

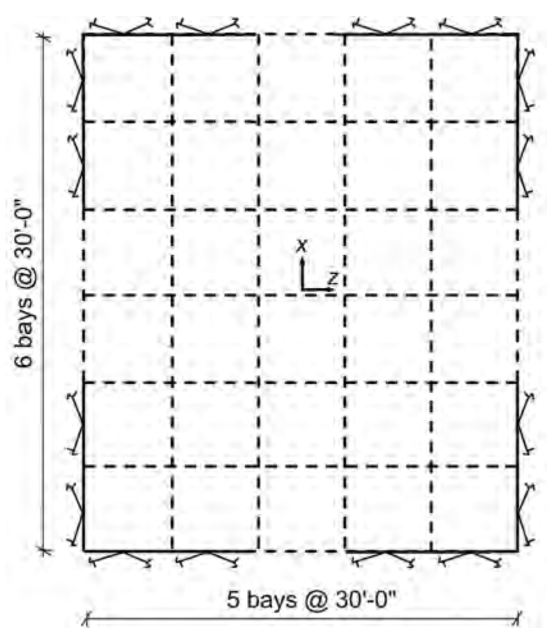
The braces are designed in accordance with AISC *Seismic Provisions* Section F2 to meet the required strength, slenderness and compactness requirements. Based on the size range of the SCBF beams and columns, the effective brace length is taken to be 3'-11" (or approximately 12%) less than that of the workpoint-to-workpoint length. The design forces in the force-controlled components (beams, columns and connections) are determined in accordance with AISC *Seismic Provisions* Section F.2.3. In all cases, the design-level



(a) SCBF-3S



(b) SCBF-9S



(c) SCBF-20S

Fig. 1. Floor plans showing the layout of the braced frames.

Table 1. Summary of Key Building Design Parameters						
Building	Number of Stories	Seismic Weight (kips)	Approximate Period[1], T_a(s)	Seismic Response Coefficient, C_s	Design Base Shear, V (kips)	Design Drift[2] (%)
SCBF-3S	3	5,247	0.44	0.24	1259	0.68
SCBF-9S-A	9	17,106	0.93	0.14	2395	0.70
SCBF-9S-B	9	17,106	0.93	0.14	2395	0.70
SCBF-20-S	20	47,117	1.81	0.07	3298	0.96

[1] ASCE/SEI 7-10 Equation 12.8-7
[2] Includes drift amplification factor, $C_D = 5$

demands from response spectrum analyses amplified by the overstrength factor was lower than the expected brace-capacity-based demands, therefore, the former was used in the design.

In addition to gravity, the beams are designed for the unbalanced brace compression and tension forces. The strength of the columns is determined based on gravity loads plus the seismic demands corresponding to the amplified response spectrum analysis forces for the case where the compression braces are removed as required by AISC *Seismic Provisions* Section F2.3, Exception 2. Note that there is no explicit requirement for the columns to be designed for the moments transmitted from the SCBF beams, which are generated by the unbalanced brace force (e.g., if the beam-to-column connection were fully restrained). Because some designers might choose to treat the beam boundary condition as pinned, regardless of the actual connection, the columns were designed for axial load only in this study to highlight the implications of underestimating the design loads. It is worth noting that the 2016 AISC *Seismic Provisions* (AISC, 2016) have been released, and the provision that limits the design forces in force-controlled components to elastic demands amplified by the overstrength factor has been removed. Going forward, all force-controlled components are required to be designed for the expected brace-capacity-based demand. In the current study, the demands from nonlinear response history analyses are evaluated against both the design-level demands amplified by the overstrength factor and the expected brace-capacity-based demands.

The SCBF member sizes (braces, chevron beams, and columns) and demand-to-capacity ratios are summarized in Tables 2 and 3, respectively. The demand-to-capacity ratios in the chevron beams are generally high across the three building cases, ranging from 0.82 to 0.95. The braces at the lower stories have the highest demand-to-capacity ratios for all building cases, ranging from 0.78 to 0.95. The upper-story braces are generally overdesigned and have much lower demand-to-capacity ratios (0.23 to 0.70), especially in the 9- and 20-story buildings. Like the braces, the upper-story columns are conservatively designed with the lowest demand-to-capacity ratios being less than 0.05. Because the beam-column connections are assumed flexurally rigid (welded connection), the columns are sized such that the flange width matches that of the beam. This explains why the demand-to-capacity ratios in the upper-story columns are very low (less than 0.1) compared to the other SCBF elements. The demand-to-capacity ratios in the lower-story columns range from 0.52 in the 3-story building to 0.82 in the 9- and 20-story buildings, respectively. The varying extent to which conservatism is incorporated in the SCBF columns is especially relevant to the probabilistic demand assessment presented later in the paper.

STRUCTURAL MODELING, GROUND MOTIONS, AND NONLINEAR RESPONSE HISTORY ANALYSES

Structural Modeling

Three-dimensional nonlinear structural models of the three building cases are developed in *OpenSees* (UC Berkeley) using expected gravity loads ($1.05D + 0.25L$). Only the SCBF frames are included in the structural model with a P-Δ column placed at the center-of-mass (geometric center) to account for the destabilizing effect of the gravity loads that are not explicitly considered. A schematic illustration of a single SCBF frame for the three-story building is shown in Figure 2. Beams and columns are modeled with fiber elements that incorporate the *Steel02* material model with expected strengths of R_yF_y ($R_y = 1.1$ and $F_y = 50$ ksi). For beams, the fiber element properties are used for axial force and strong-axis bending (in the vertical plane). Fiber elements in columns account for bending about both axes (i.e., P-M-M interaction). Beam-column connections are modeled as flexurally rigid. The SCBF braces are modeled using force-based nonlinear beam-column elements with the *Steel02* material (UC Berkeley) also using expected strengths ($R_y = 1.4$ and $F_y = 46$ ksi) and strain hardening of 0.3%. Initial imperfections and co-rotational transformations are used to simulate out-of-plane buckling. The discretization of

Table 2a. Summary of SCBF Brace Sizes		
Building	**Story**	**Brace Size**
SCBF-3S	1	HSS7.5×0.5
	2	HSS7.5×0.375
	3	HSS6.625×0.312
SCBF-9S	1 to 3	HSS7.5×0.5
	4 to 5	HSS7.625×0.375
	6 to 7	HSS6.625×0.5
	8 to 9	HSS6.625×0.312
SCBF-20S	1 to 10	HSS8.625×0.5
	11 to 16	HSS7.5×0.5
	17 to 18	HSS7.5×0.375
	19 to 20	HSS6.625×0.312

Table 2b. Summary of SCBF Column Sizes		
Building	**Story**	**Braced Frame Column Size**
SCBF-3S	1 to 3	W14x132
SCBF-9S	1	W14×257
	2 to 3	W14×211
	4 to 9	W14×132
SCBF-20S	1 to 2	W14×665
	3 to 4	W14×550
	5 to 6	W14×455
	7 to 8	W14×370
	9 to 10	W14×311
	11 to 12	W14×257
	13 to 14	W14×193
	15 to 20	W14×132

Table 2c. Summary of SCBF Beam Sizes		
Building	**Level**	**Chevron Beam Size**
SCBF-3S	2	W27×281
	3	W27×217
	4	W24×192
SCBF-9S	2 to 4	W30×261
	5 to 6	W27×217
	7 to 8	W27×258
	9 to 10	W24×192
SCBF-20S	2 to 11	W30×292
	12 to 17	W30×261
	18 to 19	W27×217
	20 to 21	W24×192

Table 3. Summary of Demand-to-Capacity Ratios for Braces and SCBF Beams and Columns			
	Demand-to-Capacity Ratios		
Building	Braces	Beams	Columns
SCBF-3S	0.70 to 0.86	0.82 to 0.92	0.04 to 0.52
SCBF-9S	0.23 to 0.78	0.84 to 0.89	0.04 to 0.82
SCBF-20S	0.54 to 0.95	0.85 to 0.95	0.02 to 0.82

Table 4. Modal Periods from Eigenvalue Analyses			
	Modal Periods, (s)		
Building	1st Mode	2nd Mode	3rd Mode
SCBF-3S	0.38	0.37	0.15
SCBF-9S	0.81	0.81	0.38
SCBF-20-S	2.00	2.00	0.82

the brace elements along the length, number of integration points, and number of fibers are determined based on the recommendations provided in Uriz et al. (2008). The ends of the braces are modeled as pinned, and rigid elastic elements are placed at the ends of beams, columns and braces in the region of the gusset plate. Rayleigh damping corresponding to 3% of critical damping in the first and third modes is applied. The first three modal periods for the three building cases obtained from eigenvalue analyses of the *OpenSees* models, are summarized in Table 4.

Ground Motions

Nonlinear response history analyses are conducted on the structural models of the three building cases using a single set of 26 ground motions, each with two orthogonal horizontal components. The set includes records from events with moment magnitudes ranging from 6.4 to 7.6 and rupture distances between 6.84 and 27.3 miles. The ground-motion spectra are shown in Figure 3 with the ASCE/SEI 7–10 estimated periods corresponding to the three building cases identified. The median spectral acceleration level corresponding to the code-based period for the 3-story, (T = 0.44s), 9-story (T = 0.93s), and 20-story (T = 1.83s) buildings are 0.60*g*, 0.40*g* and 0.19*g*, respectively.

Nonlinear Response History Analyses

The force demands in the biaxially loaded (corner) SCBF columns (Figure 4) is the response parameter of primary

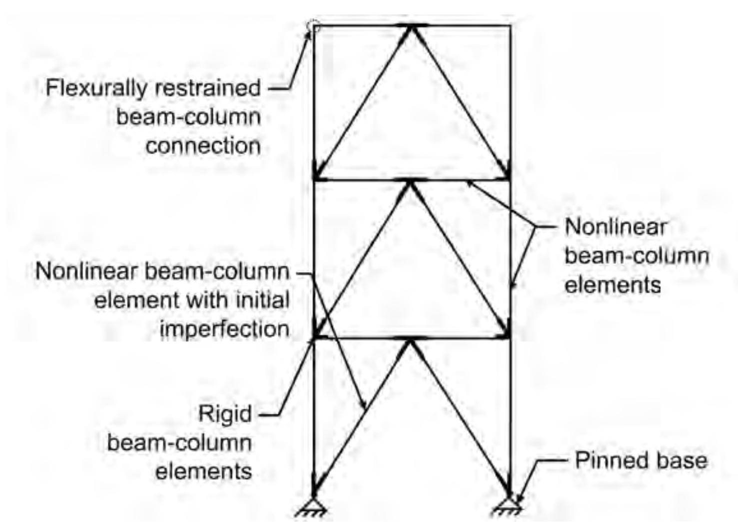


Fig. 2. Schematic illustration of OpenSees *model for a typical three-story SCBF.*

interest. However, as described later, the demands in the uniaxially loaded columns (Figure 4) are used as a benchmark to evaluate the combinatorial effect of orthogonal loading. Demands from the SCBF braces in the two principal directions of the LFRS transmit axial forces to the biaxially loaded columns. Flexural demands in the SCBF beams, which develop because of the unbalanced tension-compression response of the chevron braces, are also transmitted to these columns in the form of biaxial bending moments. Incremental dynamic analyses (IDA) are performed using bi-directional loading at ground-motion hazard levels ranging from 10 to 100% of the MCE at 10% increments. This range of ground-motion intensities is used to evaluate the effect of the extent of inelastic response on the combinatorial effects of demands from orthogonal ground motions. In the analyses, each ground-motion pair is scaled such that the geometric mean spectra matches the target intensity at the building's fundamental period.

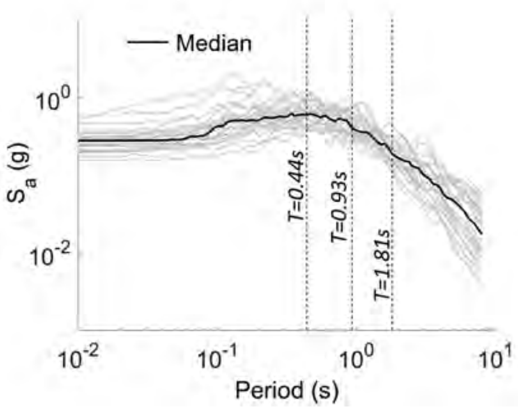


Fig. 3. Response spectra for ground motions used in nonlinear structural analysis.

PROBABILISTIC ASSESSMENT OF FORCE DEMANDS IN SCBF COLUMNS

Brace Force Demands at MCE Hazard Level

The primary goal of this section is to probabilistically assess the force demands in the SCBF columns relative to (1) the demands based on expected brace strengths, (2) the design demands obtained from response spectrum analysis (before amplifying by the overstrength factor), and (3) the nominal strengths. To facilitate interpreting those demands, median brace compressive forces at the MCE hazard level, C_{max}, in each direction, normalized by the expected brace strengths C_{exp}, are shown in Figure 5. The median C_{max}/C_{exp} ratio is approximately 0.95 for both (x and z directions) orthogonal first-story braces of all three buildings. The C_{max}/C_{exp}

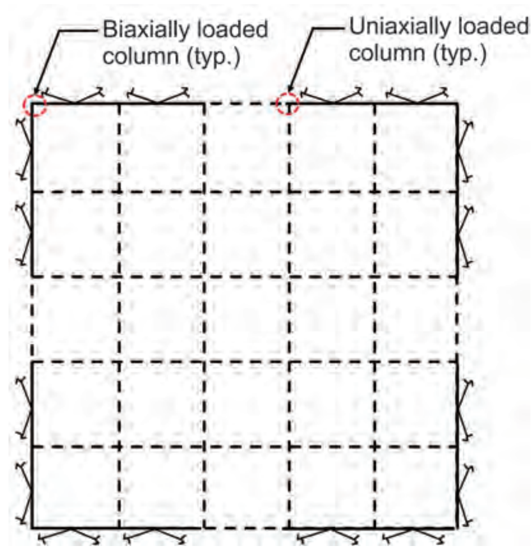
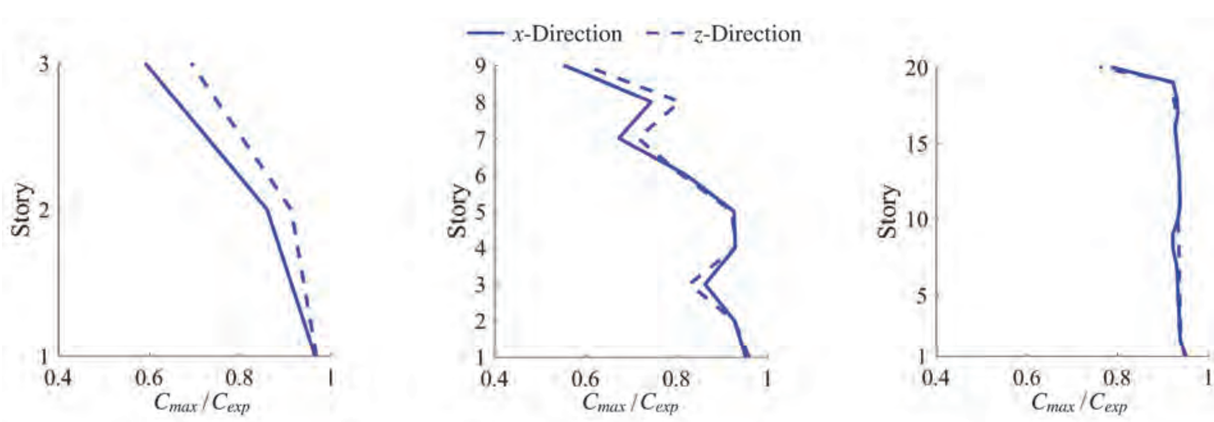


Fig. 4. Identifying biaxially and uniaxially loaded columns.



(a) SCBF-3S *(b) SCBF-9S* *(c) SCBF 20-S*

Fig. 5. Brace force demands normalized by expected strength.

ratio is generally much lower in the uppermost stories of SCBF-3S (0.6 to 0.7) and SCBF-9S (0.55 to 0.6). The C_{max}/C_{exp} profile in SCBF-20 is uniform along the height of the building between the 1st and 18th stories because the brace compressive demands for many (more than half) of the ground motions are at or near the expected strength

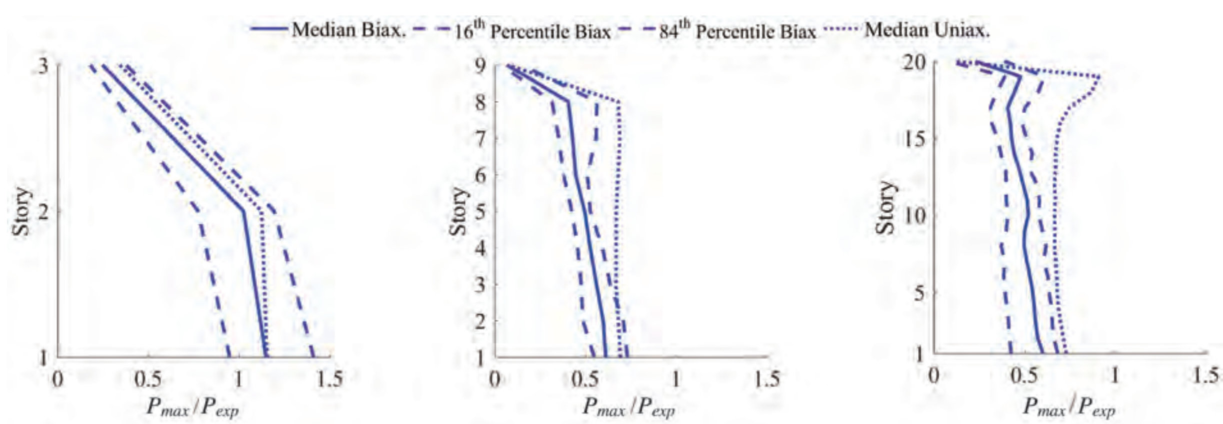
SCBF Column Axial Compressive Force Demands at MCE Level

Figure 6 shows the full profile of the maximum compression force in the biaxially (median, 16th and 84th percentile) and uniaxially (median) loaded columns, P_{max}, normalized by the demands based on the expected strength of the braces, P_{exp}. The median P_{max}/P_{exp} ratio for the biaxially loaded columns in the first story of SCBF-3S, SCBF-9S and SCBF-20S is 1.16, 0.61 and 0.6, respectively. The reduction in the first-story column P_{max}/P_{exp} ratio in SCBF-3S and SCBF-9S with building height is consistent with the profile trend of brace demands shown in Figures 5(a) and 5(b). Note that, for biaxially loaded columns, P_{exp} is based on 100% of expected compressive strength of the braces in one direction and 30% in the orthogonal direction—that is, the 100–30 rule is applied to obtain P_{exp}. The overall trend is that P_{max}/P_{exp} decreases up the height of the building. For example, in the uppermost story, the median P_{max}/P_{exp} ratio is between 0.06 and 0.25 for the biaxially loaded columns in all three structures. The dispersion in P_{max}/P_{exp} also decreases up the height of the building as evidenced by the reduction in the difference between the 84th and 16th percentile values, which ranges from 0.48 in the first story of SCBF-3S to 0.2 in the uppermost story. In SCBF-9S, the range is 0.18 in the first story to 0.11 in the uppermost story.

Figure 6(a) shows that P_{max}/P_{exp} in the uniaxially loaded columns is comparable to that of the biaxially loaded columns for SCBF-3S. For example, the median P_{max}/P_{exp} = 1.16 in the first story of the uniaxially loaded columns in SCBF-3S. However, for SCBF-9S and SCBF-20S, P_{max}/P_{exp} is higher for the uniaxially loaded columns (P_{max}/P_{exp} ranges from 0.69 to 0.73 in the first story). As expected, P_{max} is generally higher in the biaxially loaded columns. However, because P_{exp} for the uniaxially loaded columns is based on the compressive strength of braces in one direction, it is smaller than that of the biaxially loaded columns, which, as noted earlier, are based on 100% of compressive strength of braces in one direction and 30% in the other. The P_{max}/P_{exp} being higher for the uniaxially loaded columns in the 9- and 20-story structures but comparable to that of the biaxially loaded columns for the 3-story structure suggests that the combinatorial effects of loading from orthogonal braced frames is more significant in the latter. This finding is further explored later in the paper when a more direct approach to evaluating "orthogonal effect" is implemented.

Figure 7 shows that the median of the ratio P_{max}/P_{rsa} is 2.6, 1.7 and 1.8 in the biaxially loaded columns at the first story of SCBF-3S, SCBF-9S and SCBF-20S, respectively. Note that P_{max}/P_{rsa} is greater than the Ω_0 = 2.0 for the biaxially loaded column in SCBF-3S, which means that the median demand at the MCE level exceeds the upper limit on the design axial force set by the AISC *Seismic Provisions*. However, as noted earlier, the upper limit corresponding to the design level demands amplified by the overstrength factor has been removed in the 2016 *Provisions*. For SCBF-3S, P_{max}/P_{rsa} = 1.9 for the uniaxially loaded first-story columns, which is 37% smaller than its biaxially loaded counterpart, where P_{max}/P_{rsa} = 2.6 [Figure 7(a)]. However, P_{max}/P_{rsa} for uniaxially and biaxially loaded first-story columns are approximately equal for SCBF-9S and SCBF-20S. Again, this observation is consistent with the P_{max}/P_{exp} ratios discussed earlier and will be explored further later in the paper.

The maximum compression force in biaxially (median, 16th and 84th percentile) and uniaxially (median) loaded



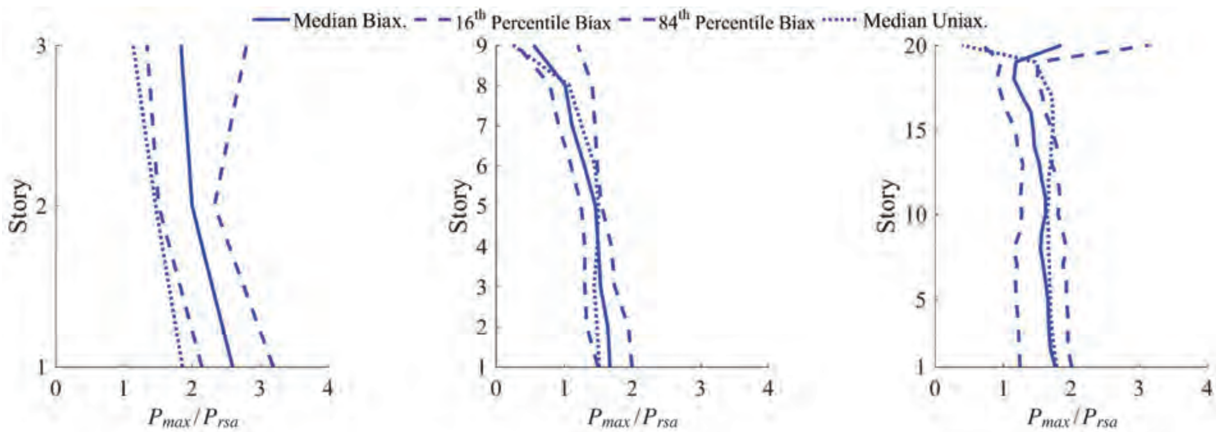
(a) SCBF-3S (b) SCBF-9S (c) SCBF 20-S

Fig. 6. Compression force demands in SCBF columns at MCE level normalized by demands based on expected strength of braces.

columns normalized by the nominal strength, P_{max}/P_n is shown in Figure 8. For SCBF-3S, the peak demand ratio occurs in the first-story columns and is about 0.5 for the biaxially loaded columns [Figure 8(a)]. For SCBF-9S and SCBF-20S, the maximum P_{max}/P_n also occurs at the first story, and the median values are 0.67 and 0.76, respectively for the biaxially loaded columns. These relative ratios are somewhat consistent with the demand-to-capacity ratios used in the design (reported in Table 3), which were 0.52, 0.82 and 0.82 in SCBF-3S, SCBF-9S and SCBF-20S, respectively. P_{max}/P_n drops off to less than 0.05 in the uppermost columns of all three buildings for the biaxially loaded columns. P_{max}/P_n in the first-story uniaxially loaded columns of SCBF-3S is approximately 20% less than its biaxially loaded counterpart. For SCBF-9S and SCBF-20S, P_{max}/P_n in the first-story uniaxially loaded columns is approximately 14% and 7% less, respectively, when compared to the biaxially loaded ones. The reduction in the difference between the demands in the uniaxially and biaxially loaded columns as the building height increases is consistent with earlier observations. The higher dispersion in the lower-story demands on the taller buildings is also consistent with earlier observations.

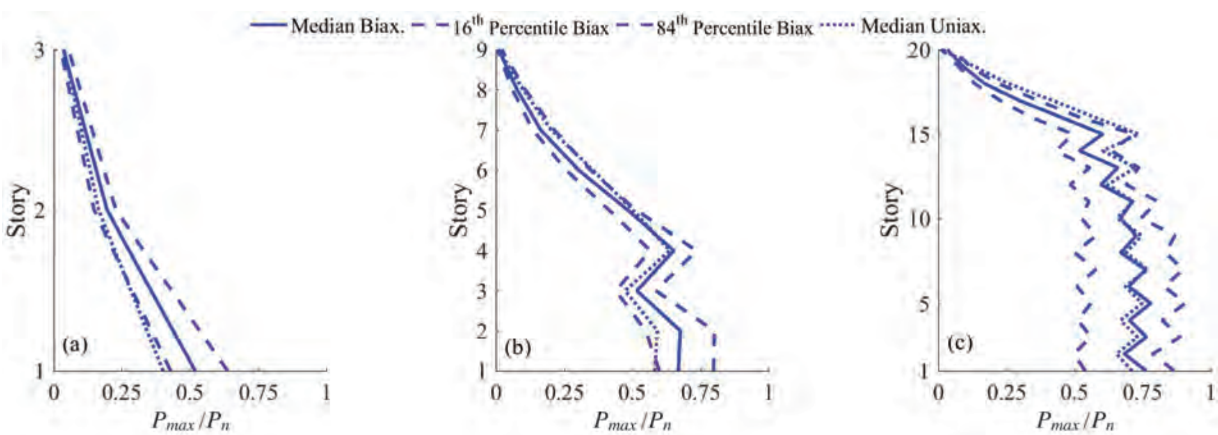
SCBF Column Flexural and Axial-Flexure Interaction Demands at MCE Hazard Level

Figure 9 shows that the median of the maximum flexural demands, M_{max}, in the biaxially loaded columns at the MCE hazard level are as high as approximately 100% of the nominal flexural strength, M_n, for all three buildings. The lower flexural demands in upper stories is consistent with the brace demand pattern observed in Figure 5. For all three buildings, the flexural demand ratios are comparable about the two axes. Except for the first two stories of each building, the flexural demand ratios in the uniaxially loaded columns are higher about the strong axis. The exceptions



(a) SCBF-3S (b) SCBF-9S (c) SCBF 20-S

Fig. 7. Compression force demands in SCBF columns at MCE level normalized by demands from response spectrum analysis.



(a) SCBF-3S (b) SCBF-9S (c) SCBF 20-S

Fig. 8. Compression force demands in SCBF columns at MCE level normalized by nominal strength.

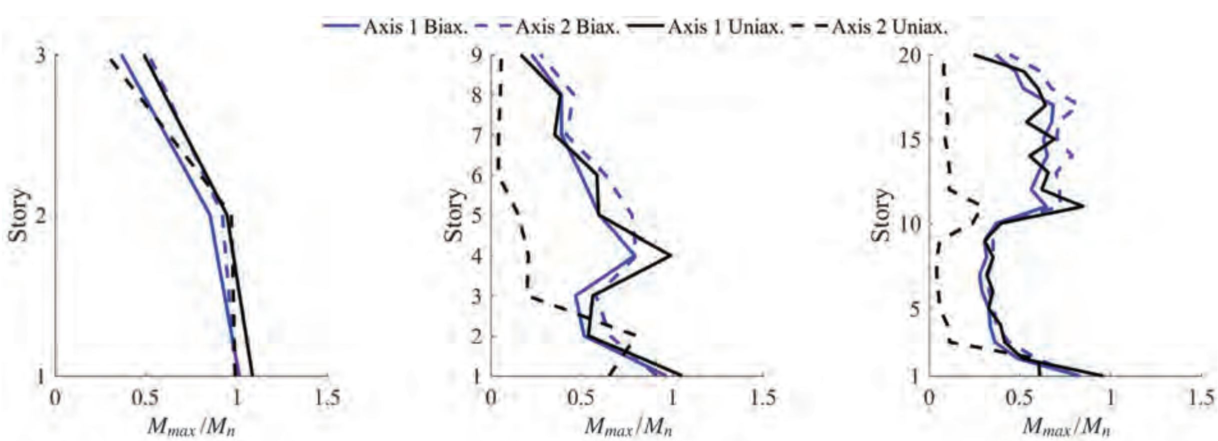
are due to high drift concentrations in the first-story weak-axis direction, which results in high flexural demands at the top of the first-story and bottom of second-story columns. Recall that these flexural demands originate from the unbalanced tension-compression response of the chevron braces, which results in significant flexural demands in the SCBF beam. If the SCBF beam column connections are flexurally restrained, as was assumed in this study, these moments are also transmitted to the columns. The AISC *Seismic Provisions* require the chevron beams to be designed for the moments caused by the brace force imbalance. However, as noted earlier, there is no explicit language requiring that the moment demands in the column be considered in the design.

It is well understood that axial force and flexural demands in beam-column elements interact to produce axial stresses. For the most part, very little attention is given to axial-flexure interaction in SCBF systems because it is generally assumed that SCBF columns are not subjected to significant moment demands. However, as presented earlier, using chevron-configured SCBF with flexurally rigid beam-column connections resulted in moment demands as high as 100% of the nominal flexural strength for the buildings considered in this study.

Figure 10 shows the profile of maximum axial-flexure interaction demands at the MCE hazard level for the biaxially and uniaxially loaded columns. The *P-M* interaction demands are described in terms of the sum of the axial and flexural demands (both axes) normalized by their respective nominal strengths ($P_{max}/P_n + M_{max,1}/M_{n,1} + M_{max,2}/M_{n,2}$). The median interaction ratio is greater than 1.0 for the biaxially loaded first- and second-story columns of SCBF-3S and the first, second, fourth and fifth stories of SCBF-9S. In SCBF-20S, the median ratio is approximately 1.17 in the first-story biaxially loaded column. The interaction ratio is generally higher for the biaxially loaded columns compared to the uniaxially loaded ones. The maximum ratio (first-story) is 5%, 8% and 14% higher for the biaxially loaded columns of SCBF-3S, SCBF-9S and SCBF-20S, respectively. The median interaction ratio drops off to approximately 40% in the uppermost biaxially loaded columns for SCBF-9S and SCBF-20S. For SCBF-3S, the median interaction ratio is approximately 65% in the uppermost story.

PROBABILISTIC EVALUATION OF COMBINATORIAL EFFECTS FOR ORTHOGONAL RESPONSE DEMANDS IN BIAXIALLY LOADED SCBF COLUMNS

When designing LFRS elements, the structure is typically analyzed independently for each horizontal translational component of earthquake loading, and the demands are combined accordingly. The rules used to combine the demands from orthogonal loads are intended to account for the simultaneous actions of ground-motion components. In ASCE/SEI 7–10, which was used to design the building cases for the current study, the 100–30 rule (Rosenblueth and Contreras, 1977) is adopted, which uses the larger of the responses obtained from combining 100% of the demand from loading in one direction with 30% of the demands associated with loading in the orthogonal direction. Other approaches to combining the demands from orthogonal earthquake loads include the 100–40 rule Newmark (1975), the square-root-sum-of-squares (SRSS) and the CQC3 rule (Smeby and Der Kiureghian, 1985). Several researchers have investigated the efficacy of these combination rules (e.g., Menun and Der Kiureghian, 1998; Heredia-Zavoni and Machiacao-Barrionuevo, 2004; Lopez et al., 2001). However, most of these studies did not consider nonlinear response in their evaluations, and none have focused on the specific issue of biaxially loaded columns in SCBF.



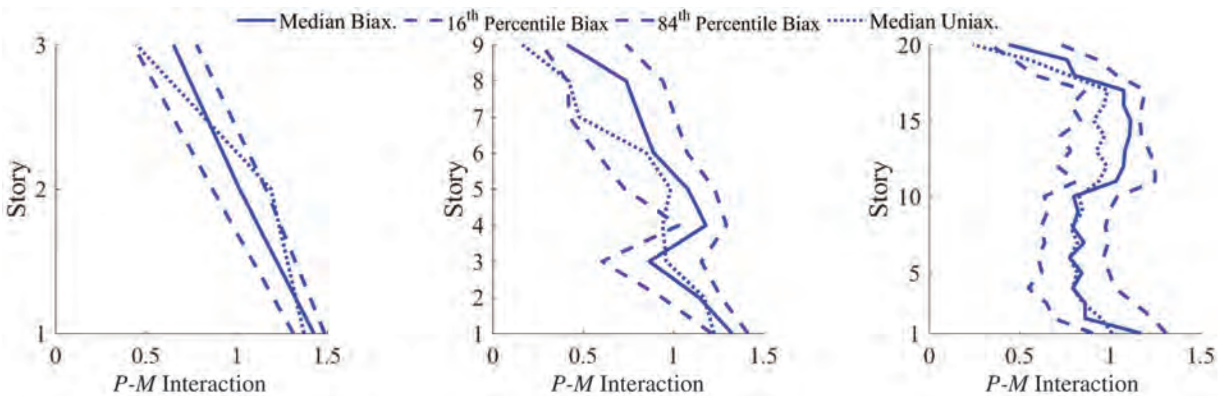
(a) SCBF-3S (b) SCBF-9S (c) SCBF 20-S

Fig. 9. Flexural demands in SCBF columns at MCE level normalized by nominal strength.

The "orthogonal effects" combination rules are used to amplify the force demands from uni-directional loading so that they are representative of the demands from bi-directional loading. As noted earlier, the corner SCBF columns are loaded biaxially by the two orthogonal ground-motion components. However, because of the symmetry of the LFRS used in the current study, the axial demands in the "non-corner" or uniaxially loaded columns are affected by a single ground-motion component. As such, the ratio of the axial compression demands in the biaxially and uniaxially loaded columns is used as the basis of evaluating the combinatorial effects of the response demands in the former. The maximum axial compressive force in the biaxially and uniaxially loaded columns are denoted as $P_{max,bi}$ and $P_{max,uni}$, respectively. For bi-directional nonlinear response history analysis performed using a single ground-motion pair, the ratio $P_{max,bi}/P_{max,uni}$ is obtained. By using a set of ground motions for each of the load cases, a full probability distribution of $P_{max,bi}/P_{max,uni}$ is determined.

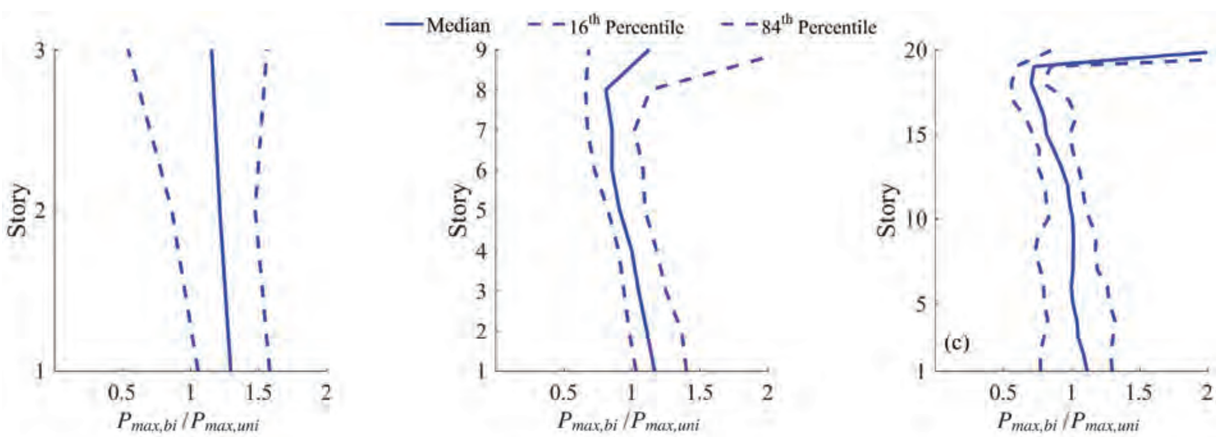
Figure 11 shows the full height profile of the median, 16th and 84th percentile of $P_{max,bi}/P_{max,uni}$ corresponding to the MCE hazard level. In SCBF-3S, the median ratio is 1.3 in the first story and reduces to 1.16 in the uppermost story. It can also be observed that $P_{max,bi}/P_{max,uni}$ generally decreases as building height increases. In the first-story columns of SCBF-9S and SCBF-20S, the median $P_{max,bi}/P_{max,uni}$ is 1.17 and 1.12, respectively, which serves as further evidence that the combinatorial effects of orthogonal loading is lower for taller buildings.

Figure 12 shows the effect of ground-motion intensity on the $P_{max,bi}/P_{max,uni}$ ratios in the first-story columns. Figure 12(a) shows the median, 16th and 84th percentile ratios for SCBF-3S, which are obtained from an IDA performed at intensities ranging from 10 to 100% of the MCE hazard level. An overall increase in $P_{max,bi}/P_{max,uni}$ with the ground-motion intensity is observed. For example, the 84th percentile value ranges from 1.39 at the lowest intensity level to 1.58



(a) SCBF-3S (b) SCBF-9S (c) SCBF 20-S

Fig. 10. Axial-flexure (P-M) interaction demands ($P_{max}/P_n + M_{max,1}/M_{n,1} + M_{max,2}/M_{n,2}$) *in SCBF columns at MCE level.*



(a) SCBF-3S (b) SCBF-9S (c) SCBF 20-S

Fig. 11. Full-height profile for the ratio of compression force demands in biaxially and uniaxially loaded columns at MCE level.

at the highest intensity level. This observation highlights the need to consider both inelastic response and ground-motion intensity level when evaluating rules for combining response demands from orthogonal ground-motion components.

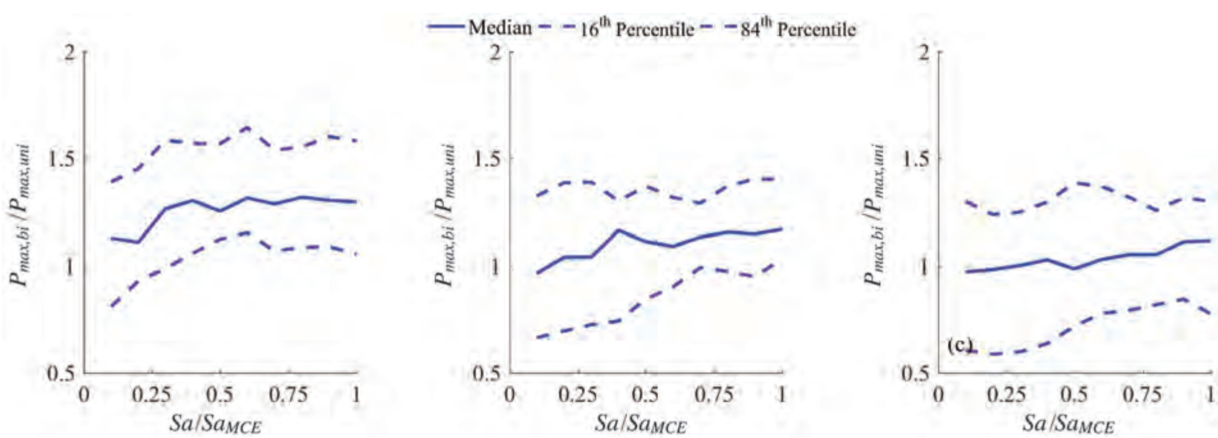
Figures 12(b) and 12(c) show that in addition to $P_{max,bi}/P_{max,uni}$ being generally lower, the effect of ground-motion intensity is also less significant for taller buildings. For instance, the difference between the 84th percentile $P_{max,bi}/P_{max,uni}$ at 10% and 100% of the MCE intensity level is only 0.07 in SCBF-9S compared to 0.19 in SCBF-3S. For the SCBF-20S building, the 84th percentile $P_{max,bi}/P_{max,uni}$ is approximately the same at the 10% and 100% MCE intensity levels.

The uncertainty in the values of $P_{max,bi}/P_{max,uni}$ conditioned on the ground-motion intensity level can be described by fitting a theoretical probability distribution to the empirical data-points at that intensity. The two-sample Kolmogorov-Smirnov (KS) test (Massey, 1951) is performed to determine the appropriate distribution based on the null hypothesis that the empirical values of $P_{max,bi}/P_{max,uni}$ follow that distribution. The output of the KS test is a p-value, which corresponds to the probability that there is a match between the empirical and theoretical distributions. A threshold of 5% is used as the acceptable margin of the p-value. The difference between the theoretical and empirical distributions is deemed significant if the p-value obtained from the hypothesis test falls below this threshold. The results from the KS test showed that the log-normal distribution produces a p-value that is larger than 5% across all intensity levels. Therefore, $P_{max,bi}/P_{max,uni}$ is assumed lognormal. Probability of exceedance curves for $P_{max,bi}/P_{max,uni}$ conditioned on the MCE hazard level, which are generated from the theoretical probability distributions, are shown in Figure 13. The distribution for each building is generated using the median and log-standard deviation values from the empirical data. As noted earlier, the median $P_{max,bi}/P_{max,uni}$ is generally lower for tall buildings, which results in higher overall exceedance probabilities. For example, SCBF-20S has an exceedance probability of 0.43 at $P_{max,bi}/P_{max,uni} = 1.0$, which is almost half that of SCBF-3S. The exceedance probability corresponding to $P_{max,bi}/P_{max,uni} = 1.3$, which ranges between 0.12 for SCBF-20S and 0.48 for SCBF-3S, can be used as the basis for evaluating the 100–30 rule.

CONCLUSIONS

A probabilistic evaluation of the force demands in the biaxially loaded columns of special concentrically braced frames (SCBF) is presented with a specific focus on (1) the maximum considered earthquake (MCE) demand levels relative to the capacity-design-based and design level forces; (2) the implications of flexural demands, which are transmitted from the chevron beams; and (3) the adequacy of the combinatorial effects of loading from orthogonal ground-motion components. Nonlinear response history analyses of three-dimensional models of 3-, 9- and 20-story SCBF are used as the basis of the evaluation.

For both the biaxially and uniaxially loaded columns, the ratio of the median MCE level axial compression demands, P_{max}, normalized by (1) the demands based on the expected brace strength, P_{exp}, (2) the demands from response spectrum analysis before amplification by the overstrength factor, P_{rsa}, and (3) the nominal compressive strength, P_n, was assessed. By comparing P_{max}/P_{exp} and P_{max}/P_{rsa} for the biaxially and uniaxially loaded columns in the three building cases, the combinatorial effect of orthogonal loads was found to decrease as the building height increased. Moreover, the three-story building case was the one where, in the first-story columns, P_{max}/P_{exp} exceeded 1.0 and P_{max}/P_{rsa} exceeded the overstrength factor for an SCBF



(a) SCBF-3S (b) SCBF-9S (c) SCBF 20-S

Fig. 12. Ratio of compression force demands in biaxially and uniaxially loaded first columns from IDA.

building (2.0). However, it should be noted that the latter is no longer a concern because the 2016 AISC *Seismic Provisions* (AISC, 2016) require SCBF columns to be designed for P_{exp}. The demand dispersion was found to be highest in the lower stories and generally increased with building height.

The highest P_{max}/P_n values in the biaxially loaded columns were found to be comparable with the demand-to-capacity-ratio used in design: 0.25, 0.82 and 0.82 in the 3-, 9- and 20-story building cases, respectively. For the biaxially loaded columns, the median of the maximum MCE level flexural demands was as high as 100% of the nominal strength in all three buildings. An interaction ratio was computed by summing the axial and flexural demands (both axes) normalized by their respective nominal strengths. The median of the maximum MCE level value of this ratio was found to be greater than or equal to 1 for both the uniaxially and biaxially loaded columns of all three buildings. It is worth reiterating that the goal here was to highlight the performance implications of neglecting the seismic moment generated in the SCBF columns via the chevron beams. However, it is recognized that some engineers do account for these moments in their design, and the axial-flexural interaction demand ratios presented in this study are not representative of those cases.

The combinatorial effects of orthogonal response demands in the biaxially loaded columns was evaluated by generating full probability distributions of these demands normalized by the demands in the uniaxially loaded columns ($P_{max,bi}/P_{max,uni}$). The full profile (along building height) of the median, 16th and 84th percentile of $P_{max,bi}/P_{max,uni}$ at the MCE hazard level showed that combinatorial effects are generally higher in the lower stories of the three building cases and decreased as the building height increased. Results from incremental dynamic analyses showed that while $P_{max,bi}/P_{max,uni}$ generally increased with ground-motion intensity, the effect was smaller for taller buildings.

The 100–30 combination rule was evaluated by computing the probability of $P_{max,bi}/P_{max,uni} > 1.3$. This probability was found to range from 0.12 to 0.48 for the 20- and 3-story buildings, respectively. At least for SCBF systems, this suggests that the current 100–30 rule underestimates the axial force demand in biaxially loaded columns. Note that the 2016 AISC *Seismic Provisions* implies that a 100–100 combination rule should be used to account for the simultaneous action of orthogonal ground-motion components. However, the results of this study suggest that for the considered SCBF systems, this would be overly conservative.

This study did not attempt to identify a more appropriate combination rule, principally because its focus was to assess whether the current combination rule resulted in consistently underestimating biaxial column demands in SCBF systems. It is important to note that mere exceedance of estimated demands in individual columns does not necessarily lead to poor performance (e.g., collapse). Moreover, appropriately estimating demands on biaxially loaded columns is not a material-dependent issue. Further research is needed to evaluate the performance implications of alternative combination rules. For SCBF systems, such research must consider the impact of all relevant limit states that might dominate behavior as the combination rule is varied. Similar research investigating orthogonal load effects in other seismic-force-resisting systems is also needed to assess whether the existing combination rule should be changed or alternative approaches implemented.

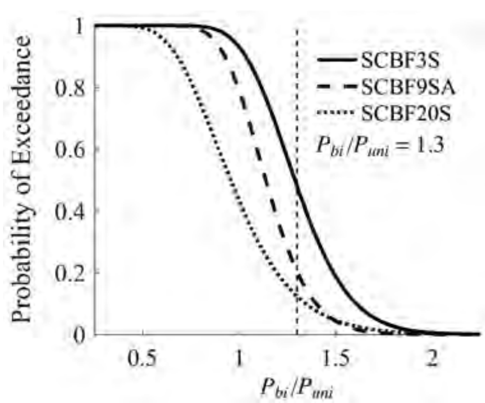


Fig. 13. Probability of exceedance curves for ratio of compression force demands in biaxially and uniaxially loaded first columns at MCE level.

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