Seismic Performance of Chevron-Configured Special Concentrically Braced Frames with Yielding Beams

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Summary

Current seismic design requirements for special concentrically braced frames (SCBFs) in chevron configurations require that the beams supporting the braces be designed to resist the demands resulting from the simultaneous yielding of the tension brace and degraded, post-buckling strength of the compression brace. Recent research, including large-scale experiments and detailed finite-element analyses, has demonstrated that limited beam yielding is not detrimental to chevron braced frame behavior and actually increases the story drift at which the braces fracture. These findings have resulted in new expressions for computing beam demands in chevron SCBFs that reduce the demand in the tension brace to be equal to the expected compressive capacity at buckling of the compression brace. In turn, the resultant force on the beam is reduced as is the required size of the beam. Further study was undertaken to investigate the seismic performance of buildings with SCBFs, including chevron SCBFs with and without yielding beams and X-braced frames. Prototype three- and nine-story braced frames were designed using all three framing systems, that is, chevron, chevron with yielding beams, and X SCBFs, resulting in six building frames. The nonlinear dynamic response was studied for ground motions simulating two different seismic hazard levels. The results were used to characterize the seismic performance in terms of the probability of salient damage states including brace fracture, beam vertical deformation, and collapse. The results demonstrate that the seismic performance of chevron SCBFs with limited beam yielding performs as well as or better than the conventionally designed chevron and X SCBFs.

1 INTRODUCTION

Special concentrically braced frames (SCBFs) have high stiffness, lateral strength, and ductility. SCBFs utilize paired braces to resist lateral loads, and these braces are often configured as multistory X-bracing and chevron bracing (sometimes referred to as V- or inverted V-bracing). Chevron SCBFs are appealing because their configuration accommodates architectural openings, including corridors, doorways, and elevators, which are regular along the building height. However, current code requirements have resulted in limited use of chevron SCBFs because of the substantial beam size required.

SCBFs conform to the American Institute of Steel Construction (AISC) Seismic Provisions for Structural Steel Buildings, denoted the Seismic Provisions herein. The braces are designed such that their factored nominal resistance exceeds the factored seismic design forces calculated according to ASCE/SEI 7-16. These seismic design forces are based on a response modification factor ($R$) of 6 in SCBFs, which implies significant ductility. SCBFs achieve this ductility in part through a capacity-based design, where all nonbrace members in the lateral force-resisting system (beams, columns, and connections) are designed to resist the forces resulting from brace buckling (including incipient and post-buckling states) and tensile yielding based on the expected yield stress ($R_y F_y$). Specifically, two load states must be analyzed. Load State 1 is an incipient buckling and yielding state where the tension and compression braces are assumed to be at their expected tensile and compressive strengths ($P_{ye}$ and $P_{cre}$, respectively). Load State 2 is a post-buckling state where the tension brace is assumed to be at its expected strength ($P_{ye}$) and the compressive brace has degraded to 30% of its expected compressive resistance (i.e., $0.3P_{cre}$). Load State 2 is critical in the design of chevron CBFs, as the resistance of the tension brace is often much larger than that of the compression brace, which imposes large flexural and axial demands on the chevron beam (see Figure 1 which shows the brace tension and compression forces, $T$ and $C$, and the resultant horizontal and vertical forces on the beam, $H_d$ and $V_d$). As such, chevron beams in SCBFs are typically deep, heavy wide-flange sections. These large beams significantly reduce the economy of chevron SCBFs, and this has reduced the use of chevron braced frames in seismic design practice. It is noted that these provisions for chevron SCBFs have existed since the 1994 Uniform Building Code, though they apply more generally to the SCBF in current design (i.e., without regard to configuration or component).
The capacity-based design loads on chevron beams are described here for clarity. The *Seismic Provisions* define the expected brace strengths as

\[ P_{ye} = R_y F_y A_g, \]

(1a)

\[ P_{cre} = 1.14 F_{cre} A_g \leq P_{ye}, \]

(1b)

where \( A_g \) is the gross cross-sectional area and \( F_{cre} \) is the expected critical buckling stress determined by Section E3 of *AISC 360-16* with the expected yield stress, \( R_y F_y \). Note that the nominal strengths, \( P_{yn} \) and \( P_{crn} \), would be computed as in Equation (1) but using \( F_y \) instead of \( R_y F_y \) and without the 1.14 factor on the compressive strength. The compressive and tensile forces in Figure 1 (\( C \) and \( T \)) are related to the expected forces as noted earlier. The resulting beam design forces (\( V_d \) and \( H_d \)) for the two load states are determined by equilibrium as in Equations (2) and (3), where \( \theta \) is the brace angle with respect to the horizontal axis.

Load State 1:

\[ V_d = (P_{ye} - P_{cre}) \sin \theta \]

(2a)

\[ H_d = (P_{ye} + P_{cre}) \cos \theta \]

(2b)

Load State 2:

\[ V_d = (P_{ye} - 0.3 P_{cre}) \sin \theta \]

(3a)

\[ H_d = (P_{ye} + 0.3 P_{cre}) \cos \theta \]

(3b)

Despite the large force demands required in design of chevron beams in SCBFs, experiments have shown that the expected brace forces may not necessarily be achieved in practice. Khatib and Mahin\(^3\) demonstrated analytically that both beam strength and stiffness determine if a chevron CBF will develop full tension yielding of...
braces under lateral loading, but there are no beam stiffness requirements for SCBFs. In concordance with these findings, experiments have shown that elastic deflections of chevron beams could prevent the brace from developing its full tensile resistance (e.g., Uriz et al.⁴ and Okazaki et al.⁵). Moreover, tensile yielding of the brace is not required to sustain large deformation demands without significant strength loss. Studies investigating older CBFs that did not have special chevron beam strength requirements, including works by Fukuta et al.⁶ and Sen et al.,⁷ demonstrated that acceptable system ductility can be achieved with a plastic mechanism consisting of buckling of the compression braces and flexural yielding of the chevron beams. Studies investigating low-ductility modern chevron braced frames found similar results.⁸-¹⁰ To keep the beams elastic, expensive high-strength steel has been proposed,¹¹ but again, elastic deformation of the beam is only increased when the beam material strength is increased.

Recent large-scale experiments and detailed finite-element analyses have demonstrated that reducing the design unbalanced load, and thus the beam size, provides nonlinear behavior that is comparable with or better than chevron SCBFs designed to current requirements.¹², ¹³ This design philosophy engages the beam as an additional yielding component and is consistent with prior research on SCBFs that has demonstrated that additional yielding mechanisms (i.e., beyond the brace) are beneficial to seismic response.¹⁴

The first phase of the research studied one-story, single-bay chevron SCBFs.¹² Six full-scale quasi-static cyclic tests were conducted. The specimens investigated different beam strengths, beam stiffness, and brace specification. Beam strengths were selected to range between approximately 25% to 100% of the current AISC required strengths. The results demonstrated that a frame with a beam designed to approximately 40% of the current requirements developed a maximum lateral strength equal to the expected brace lateral resistance, 2P_{cre}\cos\theta, with less than 20% strength deterioration relative to the nominal brace lateral resistance, 2P_{crn} \cos \theta, at 2% story drift. Note that when braced frames are designed, it is common practice to assume that the brace forces are equal and the compression brace governs the brace design. Thus, the lateral resistance is cast in terms of 2P_{crn} \cos \theta. Additionally, the nominal brace lateral resistance multiplied by the resistance factor for compression, 0.9, is the upper bound of the required lateral resistance in design. The frames retained this resistance through story drifts in excess of 3%. Further, the frames had no brace fracture prior to 3.5% story drift. As expected, beam yielding occurred in frames with beams designed to lower strength than currently required by AISC Seismic Provisions, but those frames also reached considerably larger story drift prior to brace fracture. Additional finite-element analyses that expanded the parameter set beyond the experiments demonstrated that the beam must be able to develop the axial demand imposed in Load State 1 to prevent beam axial yielding and/or instability. These tests and subsequent finite-element analyses are summarized by Roeder et al.¹³

Following the single-story tests and analyses, a three-story chevron braced frame with beams on the first two stories that were approximately 34% as strong as currently required by the Seismic Provisions was tested at the National Center for Research on Earthquake Engineering in Taipei, Taiwan. The test study parameters that were not possible to explore in the single-story tests include the impact of composite slabs and fully restrained beam-to-column connections with adjacent (corner) gusset plates. The test setup, test results, and an additional finite-element parametric study are detailed by Roeder et al.¹³ and Ibarra.¹⁵ This investigation showed that

- beam yielding increases the drift capacity of chevron braced frames prior to brace fracture relative to chevron CBFs with stronger beams or CBFs with multistory X-bracing;
- composite action should be neglected in computing beam strength due to deterioration at large deformations; and
- beam stiffness, strength, and plastic hinge locations are significantly affected by restraint of the beam-to-column connection and adjacent gusset plates.
Based on the collective prior research, alternative load states for design of the chevron beam to promote the yielding beam mechanism without resulting in excessive strength deterioration were proposed. In this alternative method, the design brace force in tension is limited to its expected compressive strength, \( P_{cre} \). Equations (4) and (5) show the resultant vertical and horizontal demands on the chevron beam, \( V_d \) and \( H_d \), in this method.

**Proposed Alternative Beam Load State 1:**

\[
V_d = (P_{cre} - P_{cre}) \sin \theta = 0
\]

(4a)

\[
H_d = (P_{cre} + P_{cre}) \cos \theta = 2P_{cre} \cos \theta
\]

(4b)

**Proposed Alternative Beam Load State 2:**

\[
V_d = (P_{cre} - 0.3P_{cre}) \sin \theta = 0.7P_{cre} \sin \theta
\]

(5a)

\[
H_d = (P_{cre} + 0.3P_{cre}) \cos \theta = 1.3P_{cre} \cos \theta
\]

(5b)

Although the proposed design recommendations have been derived and verified using large-scale testing and detailed finite-element analyses, quantification of the impact of this new alternative design method on the seismic performance of chevron SCBF buildings is needed. This paper presents the approach and results of a numerical study undertaken to investigate the impact of chevron beam yielding on the dynamic system performance of SCBFs in terms of story drift demand, likelihood of brace fracture, and collapse prevention for different seismic hazard levels. Three- and nine-story buildings were analyzed using state-of-the-art line-element models in the finite-element analysis package OpenSees. The performance of chevron SCBFs with yielding beams was also benchmarked against chevron SCBFs designed using the current requirements (i.e., with yielding braces) and multistory X-braced SCBFs. Results are presented for both a highlighted, single ground motion, to illustrate the differences in the seismic performance of the frames and for sets of earthquake ground motions representing different hazard levels to compare the likelihood of various damage states including collapse.

2 REQUIRED STRENGTHS OF BEAMS AND COLUMNS IN CHEVRON SCBFBS

This section discusses the required strength of beams and columns in chevron SCBFs. The required strengths are determined from a plastic mechanism analysis using the above resultant forces. This method is appropriate to design beams under the current required load states (Equations 2 and 3) and the proposed alternative load states, which allow controlled beam yielding (Equations 4 and 5) and are used later in this paper to design archetype buildings for numerical analysis. In practice, engineers will make various, largely conservative assumptions about the strength of the beam. However, as demonstrated in other systems, larger components can inhibit rather than enhance the seismic performance if multiple yield mechanisms are expected. In the proposed system, yielding is expected in the braces and beams. Therefore, it is necessary to design chevron beams to resist the demands, but not excessively so.

The demands on the beam result in combined flexural and axial loading. Only the brace demands on the beam are considered here for simplicity, whereas chevron beams in practice must also be designed to resist gravity loading using appropriate load cases from ASCE 7-16. As such, the AISC axial-flexural interaction equations (eq.
H1-1a and H1-1b in AISC 2017c) are used with appropriate resistance factors. The required axial and flexural strengths of the beam are denoted as \( P_r \) and \( M_r \), respectively. \( P_n \) is the nominal axial strength of the beam, and \( M_n \) is the nominal flexural strength of the beam, neglecting any contribution from a composite slab.

Current Seismic Provisions require that the beams used in chevron SCBF must be laterally braced to meet the requirements for moderate ductile members, which ensures that they can develop the plastic moment capacity. In addition to this requirement, as observed in the previous test,\(^{13}\) the composite slab carries a large amount of axial force and braces the beam against the lateral torsional buckling and minor axis flexural buckling. Thus, the axial yield strength and plastic moment were used for \( P_y \) and \( M_y \), respectively. Note that the major axis flexural buckling could occur, but the slenderness ratio of the beam is small and the critical buckling stress is very close to the yield stress in the frame design to be described later. In fact, no major axis flexural buckling of the beam has been observed in previous experiments using even very weak beams.\(^{12}\)

The maximum value of these equations is 1.0. The design philosophy is to select a beam section that results in a capacity close to, but not exceeding, this limit. The following provides a methodology to determine the required strength of the beam.

1. Use Equation (1) with either Equations (2) and (3) or Equations (4) and (5) to determine \( V_d \) and \( H_d \), the vertical and horizontal loads on the beam resulting from the load state pair of interest. Both load states in the pair must be used for design.
2. The horizontal resultant force \( H_d \) is assumed to be distributed equally to each side of the beam, as shown in Equation 6.
   \[
   P_r = \pm \frac{H_d}{2} \tag{6}
   \]
3. The required flexural strength depends on the beam end restraint. Research shows that the presence of a gusset plate at the corner beam-to-column connections rotationally restrains the beam such that the plastic moment capacity of the beam is developed at its ends.\(^{13}\) Note that this is not the case at the top story if chevron braces are used at every story where either simple connections or fully restrained beam-to-column connections are used. In the case of fully restrained connections such as those where gusset plates are present, \( M_r \) is computed using the beam plastic mechanism shown in Figure 2A. For beams with beam-to-column connections that may be considered simple, \( M_r \) is based on the plastic mechanism in Figure 2B or 2C.
4. Using the appropriate mechanism in Figure 2, the moment demand is determined using either Equation 7a for chevron beams restrained by corner gusset plate or Equation 7b for chevron beams with simple connections.
   \[
   M_r = \frac{V_d}{8} (L - L_{mg} - 2L_{cg} - d_c) = \frac{V_d l_1}{4}, \tag{7a}
   \]
   \[
   M_r = \frac{V_d}{4} (L - L_{mg} - d_c) = \frac{V_d l_2}{2}, \tag{7b}
   \]
   where \( l_1 \) and \( l_2 \) are the lengths between rigid links.
1. Under the proposed alternative load states only, the column must be strong enough to develop the plastic moment capacity of the beam. For fully restrained connections, the columns should then, at least, satisfy the strong column–weak beam requirements for special moment frames:

$$\frac{\sum M_{pc}^*}{\sum M_{pb}^*} > 1.0,$$

where $\sum M_{pb}^*$ is the sum of the projections of the expected flexural strengths of the beams at the plastic hinge locations to the column centerline and $\sum M_{pc}^*$ is the sum of the projections of the nominal flexural strengths of the columns at the plastic hinge locations to the beam centerline. Both the flexural strengths of the beams and columns may be reduced for the presence of axial load. When evaluating the resistance of the beam, research on chevron beams has shown that the increased resistance due to composite action with the slab should be neglected because this added resistance sustains significant deterioration during inelastic deformation.

Note that some design engineers consider the moment resistance provided by the gusset plate connections when designing beams in chevron braced frames and some do not. Here, that restraint is considered in the beam capacity to result in the smallest possible beams for the beam demands from the proposed load states.

3 BUILDING FRAME DESIGNS

Three- and nine-story building geometries were used as the basis of the study. The three-story frames had a typical story height of 3.96 m, whereas the nine-story frame had a first-story height of 5.50 and 3.96 m for all other stories. For each frame geometry, three different SCBF designs were developed and analyzed:

1. chevron bracing with yielding beams using the proposed design philosophy (‘Proposed Chevron’),
2. chevron bracing with beams meeting current Seismic Provisions (‘AISC Chevron’), and
3. Multistory X-bracing (‘X-Braced’).

For all of the frames (six in total), the seismic forces were calculated with the equivalent lateral force (ELF) procedure in ASCE 7-16 for a location in Seattle, Washington. Figure 3 shows typical elevations and floor plans. The braced bays were placed at the perimeter of the building in both directions, as illustrated in Figure 3.
The design spectrum defined by ASCE 7-16\textsuperscript{2} had an $S_{DS}$ of 0.94 g and an $S_{D1}$ of 0.48 g for the Seattle location on Site Class C soil. The seismic response modification coefficient, $R$, considered for all designs was equal to 6. An occupancy importance factor of 1.0 was used. The factored gravity loads (dead load plus 0.5 times the live load) for typical levels and the roof level are listed in Table 1. Table 1 also provides the approximate periods ($T_a$), the computed natural periods ($T_1$), and the computed seismic response coefficient ($C_s$) per ASCE 7-16. $C_s$ was determined using $T_a$, because the use of $T_a$ results in seismic demands that are larger than those computed by using $T_1$.

### TABLE 1. Seismic design parameters

<table>
<thead>
<tr>
<th>Archetype</th>
<th>Gravity load (kPa)</th>
<th>Computed natural period, $T_1$ (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Roof</td>
<td>Floor</td>
</tr>
<tr>
<td>Three-story</td>
<td>4.71</td>
<td>4.52</td>
</tr>
<tr>
<td>Nine-story</td>
<td>4.69</td>
<td>4.49</td>
</tr>
</tbody>
</table>

The braces were A500 Gr. C rectangular hollow structural sections. All beams and columns used A992 Gr. 50 wide-flange sections. All satisfied the AISC width-to-thickness criteria for highly ductile members (AISC 2017\textsuperscript{1}). The braces were assumed to be welded to gusset plates with $8t_p$ elliptical and $6t_p$ vertical clearances for corner and middle gusset plates, respectively, where $t_p$ is the thickness of gusset plate.\textsuperscript{14}

For each building height, the three frames only differed in beam size. The Proposed Chevron buildings had beams designed using the proposed design load cases in Equations (4) and (5) and the design method presented in Equations (6) and (7). The AISC Chevron and X-Braced buildings were designed based on the load cases specified in the Seismic Provisions (Equations 2 and 3)\textsuperscript{1}. The same columns were used for all frames to avoid any bias in the performance provided by the column contribution to lateral strength. The column sizes were designed to develop the accumulated brace forces along the height and strengthened as required to meet the strong column–weak beam requirements for the beam sizes in the Proposed Chevron designs, as specified in Equation 8. Thus, the columns in the AISC Chevron and X-Braced frames are larger than would be necessary in practice. The gusset plate connections were designed using the clearances specified above and the balanced design procedure to permit secondary yielding in the gusset plate.\textsuperscript{14}

The first natural periods ($T_1$) of the structures were computed using the numerical models and are listed in Table 1. Tables 2 and 3 list all the brace, beam, and column sections for each three-story and nine-story frame, respectively. Table 4 shows the total weight of one bay of braced frame for the three- and nine-story buildings.
The alternative load states used in the Proposed Chevron building design result in reduced weights for chevron SCBFs by 20%–30% relative to the current AISC requirements. The weights for the Proposed Chevron designs are similar to those for the X-Braced designs. This represents a considerable savings in the weight and will reduce the cost of chevron SCBFs. Additionally, the reduced beam size reduces the size of the beam-to-column connections, which are labor intensive, and thus reduces the connections costs.

**TABLE 2.** Three-story archetype member sizes

<table>
<thead>
<tr>
<th>Story</th>
<th>Column</th>
<th>Brace</th>
<th>(L_c/r)</th>
<th>(b/t)</th>
<th>Proposed Chevron</th>
<th>AISC Chevron</th>
<th>X-Braced</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>W14 × 82</td>
<td>HSS6 × 6 × 1/2</td>
<td>85.2</td>
<td>9.90</td>
<td>W18 × 65</td>
<td>W24 × 103</td>
<td>W14 × 38</td>
</tr>
<tr>
<td>2</td>
<td>W14 × 82</td>
<td>HSS5 × 5 × 1/2</td>
<td>104</td>
<td>7.75</td>
<td>W14 × 53</td>
<td>W24 × 94</td>
<td>W14 × 38</td>
</tr>
<tr>
<td>3</td>
<td>W14 × 82</td>
<td>HSS4-1/2 × 4-1/2 × 3/8</td>
<td>114</td>
<td>9.89</td>
<td>W18 × 65</td>
<td>W24 × 146</td>
<td>W24 × 146</td>
</tr>
</tbody>
</table>

**TABLE 3.** Nine-story archetype member sizes

<table>
<thead>
<tr>
<th>Story</th>
<th>Column</th>
<th>Brace</th>
<th>(L_c/r)</th>
<th>(b/t)</th>
<th>Proposed Chevron</th>
<th>AISC Chevron</th>
<th>X-Braced</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>W14 × 283</td>
<td>HSS7 × 7 × 5/8</td>
<td>87.1</td>
<td>9.05</td>
<td>W24 × 84</td>
<td>W24 × 146</td>
<td>W24 × 62</td>
</tr>
<tr>
<td>2</td>
<td>W14 × 283</td>
<td>HSS6 × 6 × 5/8</td>
<td>87.6</td>
<td>7.33</td>
<td>W21 × 68</td>
<td>W24 × 131</td>
<td>W18 × 55</td>
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<tr>
<td>3</td>
<td>W14 × 283</td>
<td>HSS6 × 6 × 5/8</td>
<td>87.6</td>
<td>7.33</td>
<td>W21 × 68</td>
<td>W24 × 131</td>
<td>W18 × 55</td>
</tr>
<tr>
<td>4</td>
<td>W14 × 193</td>
<td>HSS6 × 6 × 5/8</td>
<td>87.6</td>
<td>7.33</td>
<td>W21 × 68</td>
<td>W24 × 131</td>
<td>W18 × 55</td>
</tr>
<tr>
<td>5</td>
<td>W14 × 193</td>
<td>HSS6 × 6 × 1/2</td>
<td>85.2</td>
<td>9.90</td>
<td>W18 × 65</td>
<td>W24 × 94</td>
<td>W18 × 55</td>
</tr>
<tr>
<td>6</td>
<td>W14 × 193</td>
<td>HSS6 × 6 × 1/2</td>
<td>85.2</td>
<td>9.90</td>
<td>W18 × 65</td>
<td>W24 × 94</td>
<td>W18 × 55</td>
</tr>
<tr>
<td>7</td>
<td>W14 × 193</td>
<td>HSS5 × 5 × 3/8</td>
<td>102</td>
<td>11.3</td>
<td>W14 × 48</td>
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<tr>
<td>8</td>
<td>W14 × 193</td>
<td>HSS5 × 5 × 3/8</td>
<td>102</td>
<td>11.3</td>
<td>W21 × 68</td>
<td>W24 × 146</td>
<td>W24 × 146</td>
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</tbody>
</table>

**TABLE 4.** Weight comparison of one-bay braced frames

<table>
<thead>
<tr>
<th>Archetype</th>
<th>Weight (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Proposed Chevron</td>
</tr>
<tr>
<td>Three-story</td>
<td>5380</td>
</tr>
<tr>
<td>Nine-story</td>
<td>28 900</td>
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</tbody>
</table>
### TABLE 5. Beam strength ratios and strong column–weak beam moment ratios for three-story designs

<table>
<thead>
<tr>
<th>Story</th>
<th>Proposed Chevron</th>
<th>$\frac{M_r}{\phi M_n}$</th>
<th>$\frac{P_r}{\phi P_n}$</th>
<th>AISC eq. H1-1 $\frac{\sum M_{pc}^*}{\phi M_n}$</th>
<th>$\frac{M_r}{\phi M_n}$</th>
<th>$\frac{P_r}{\phi P_n}$</th>
<th>AISC eq. H1-1 $\frac{\sum M_{pc}^*}{\phi M_n}$</th>
<th>$\frac{M_r}{\phi M_n}$</th>
<th>$\frac{P_r}{\phi P_n}$</th>
<th>AISC eq. H1-1 $\frac{\sum M_{pc}^*}{\phi M_n}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.89</td>
<td>0.19</td>
<td>0.98</td>
<td>1.14</td>
<td>0.79</td>
<td>0.21</td>
<td>0.91</td>
<td>0.53</td>
<td>0.86</td>
<td>0.13</td>
</tr>
<tr>
<td>2</td>
<td>0.91</td>
<td>0.13</td>
<td>0.97</td>
<td>2.30</td>
<td>0.80</td>
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<td>0.89</td>
<td>0.78</td>
<td>0.17</td>
<td>0.14</td>
</tr>
<tr>
<td>3</td>
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<td>0.06</td>
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<td>-</td>
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<td>0.90</td>
<td>-</td>
<td>0.86</td>
<td>0.08</td>
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</table>

### TABLE 6. Beam strength ratio and strong column–weak beam moment ratios for nine-story designs

<table>
<thead>
<tr>
<th>Story</th>
<th>Proposed Chevron</th>
<th>$\frac{M_r}{\phi M_n}$</th>
<th>$\frac{P_r}{\phi P_n}$</th>
<th>AISC eq. H1-1 $\frac{\sum M_{pc}^*}{\phi M_n}$</th>
<th>$\frac{M_r}{\phi M_n}$</th>
<th>$\frac{P_r}{\phi P_n}$</th>
<th>AISC eq. H1-1 $\frac{\sum M_{pc}^*}{\phi M_n}$</th>
<th>$\frac{M_r}{\phi M_n}$</th>
<th>$\frac{P_r}{\phi P_n}$</th>
<th>AISC eq. H1-1 $\frac{\sum M_{pc}^*}{\phi M_n}$</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>0.80</td>
<td>0.20</td>
<td>0.90</td>
<td>1.68</td>
<td>0.90</td>
<td>0.17</td>
<td>0.99</td>
<td>0.92</td>
<td>0.95</td>
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<td>2</td>
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<td>0.23</td>
<td>0.92</td>
<td>3.41</td>
<td>0.70</td>
<td>0.19</td>
<td>0.80</td>
<td>1.51</td>
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<tr>
<td>3</td>
<td>0.78</td>
<td>0.23</td>
<td>0.92</td>
<td>4.02</td>
<td>0.70</td>
<td>0.19</td>
<td>0.80</td>
<td>1.78</td>
<td>0.12</td>
<td>0.00</td>
</tr>
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<td>4</td>
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<td>6</td>
<td>0.83</td>
<td>0.21</td>
<td>0.94</td>
<td>3.76</td>
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<td>1.00</td>
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<td>8</td>
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<td>0.13</td>
<td>0.84</td>
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<td>0.81</td>
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<td>0.88</td>
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<td>0.09</td>
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</tr>
<tr>
<td>9</td>
<td>0.94</td>
<td>0.09</td>
<td>0.99</td>
<td>-</td>
<td>0.93</td>
<td>0.09</td>
<td>0.97</td>
<td>-</td>
<td>0.93</td>
<td>0.09</td>
</tr>
</tbody>
</table>
As discussed above, the beams were designed for either of the two load state pairs (Equations 2 and 3 or Equations 4 and 5) with eq. H1-1 of the AISC Specification. Due to limitations in available beam sizes, the resulting value of eq. H1-1 is less than 1.0, reflecting an unavoidable but realistic oversize of the selected member. For clarity, the utilization ratio into two as the one from flexure and the one from axial compression, the final interaction equation values and strong column–weak beam ratios are given in Tables 5 and 6.

4 BRACED FRAME MODELING

To evaluate the seismic performance of these SCBFs, nonlinear numerical models of the braced frames presented in Figure 3 were implemented in OpenSees following the modeling methodology proposed by Hsiao et al. with some modifications, as described below. These models consisted of planar frames but were three-dimensional to simulate out-of-plane brace buckling.

The braced frame modeling methodology is illustrated in Figure 4. The brace was modeled using displacement-based fiber cross section beam–column elements divided into 16 segments with five integration points per element. The braces were given a sinusoidal initial imperfection with an amplitude of 1/500 of the brace length at the midlength. The OpenSees Steel02 uniaxial material (a Giuffré-Menegotto-Pinto Model with kinematic strain hardening) was used for each steel fiber. The steel fibers were assigned a modulus of elasticity, $E$, of 200 000 MPa; yield stress, $R_y$, of 448 MPa; and a post-yield stiffness (i.e., kinematic hardening stiffness) of 2000 MPa. The out-of-plane flexural stiffness and strength of the gusset plates were simulated using zero-length rotational springs based on the Whitmore section and average length, as proposed by Hsiao et al. and illustrated in Figure 4B. The spring constitutive behavior was also modeled using the Steel02 uniaxial material with rotational stiffness ($K_R$) and flexural strength.

![FIGURE 4](image)

FIGURE 4 Schematic of the braced frame models. (A) overall. (B) Brace–beam–column connection details

The brace fibers lost strength to simulate tearing and fracture using a maximum strain range (MSR) material wrapper, where the MSR is the maximum difference between the maximum and minimum strains throughout the deformation history. Strength of the fiber is lost once this value exceeds the MSR limit, $MSR_{f,disp}$, as given in Equation 9. Originally developed by Hsiao et al., the expression and OpenSees implementation was updated by Sen et al. to account for asymmetry in deformation demand; compression-biased asymmetric is characteristic of braces in chevron configurations, which elongates the fracture life of the brace as observed in recent tests.

$$MSR_{f,disp} = 0.554 \left( \frac{b}{t} \right)^{-0.75} \left( \frac{L_c}{R} \right)^{-0.47} \left( \frac{E}{R_y F_y} \right)^{0.21} \left( \frac{\delta_{c, max}}{\delta_{t, max}} \right)^{0.008}$$

(9)
In Equation 9, \(b/t\) is the local slenderness ratio; \(L_c/r\) is the global slenderness ratio, in which \(L_c\) is the effective length \((KL)\) and \(r\) is the moment of radius of gyration with respect to the buckling axis; \(E/R_yF_y\) is the ratio between the modulus of elasticity and the brace yield stress used in the analysis \((R_yF_y)\); and \(\delta_{c,max}/\delta_{t,max}\) is the ratio between axial compression and tension deformation of the brace. Fiber fracture occurs once its MSR reaches \(MSR_{f,disp}\). Note that whereas fracture of the braces is considered, the effects of dynamic buckling, such as those identified in Kazemzadeh et al.\(^{22}\), are neglected.

The steel beams and columns were modeled using displacement-based beam–column fiber elements using the Steel02 material model. Each column and beam was subdivided into four elements with five integration points. Cyclic deterioration of the beams and columns was not considered because (i) the sections meet the highly ductile compactness requirements of the Seismic Provisions, as required in SCBFs, (ii) the rotation demand on the members is relatively small, and (iii) comparison of the performance between the various systems depends largely on the differences in brace demand and behavior. The steel fibers for the beams and columns were assigned an initial modulus of elasticity, \(E\), of 200 000 MPa; yield stress, \(R_yF_y\), of 379 MPa (=1.1 \times 345 MPa); and a post-yield stiffness of 2000 MPa.

The beam-to-column connections were designed as fully restrained with fully welded adjacent gusset plates; this results in inelastic action in the beams and columns outside of the gusset plate regions. The effect of the gusset plates was modeled as shown in Figure 4 with essentially rigid elastic beam–column elements extending 100% and 75% of the vertical and horizontal dimensions of the gusset plate, respectively, as proposed by Hsiao et al.\(^{19}\). Connections at the top story were designed as bolted single-plate shear connections and modeled as rotational springs using the Pinching4 constitutive model and with backbone properties based on the model proposed by Liu and Astaneh-Asl.\(^{23}\) The column bases were modeled with pinned boundary conditions.

The transverse beams were modeled to provide more realistic boundary conditions for the columns. The cross sections of transverse beams were the same as those used in CBFs and were modeled to the beam inflection point (i.e., half the total length) and connected to the column with nonlinear rotational springs representing bolted single-plate shear moment–rotation behavior as described above.

Because the tensile resistance of the slab deteriorates due to the occurrence of cracks at large drifts, the compressive resistance of the diaphragm, which is a composite deck, was modeled with a compression-only elastic truss element. The truss elements had a cross-section area equal to the effective width of the slab and a modulus of elasticity calculated according to ACI 318-14 using the expected concrete compressive strength of 41.4 MPa.\(^{24}\)

The gravity system was modeled in OpenSees using a leaning column and beam stubs to account for its contribution to the building's strength, stiffness, and second-order effects (P-Delta). The leaning column assembly was connected to the braced frame using the EqualDOF multipoint constraint (shown in Figure 5). Previous research (e.g., MacRae et al.\(^{25}\) and Hsiao et al.\(^{26}\)) has shown that the gravity frame can resist a significant proportion of the seismic loads, and therefore, the models here include nonlinear rotational springs that account for the gravity frame's stiffness and strength with bolted single-plate shear connections as in the bolted single-plate shear connections designed for the braced frames. The P-Delta columns were continuous over the entire height of the three-story frames and spliced at midheight of stories four and eight of the nine-story frames. In both cases, the P-Delta column was assigned a moment of inertia in the plane of the frame equal to the sum of the moments of inertia of all gravity columns within the tributary seismic weight region of the braced frames modeled.
The accuracy of the model was evaluated by comparing the simulated and experimental results of the three-story chevron braced frame specimen with yielding beams from Roeder et al.,\textsuperscript{13} shown in Figure 5. Recall that the specimen was designed to meet the current requirements in the Seismic Provisions for SCBFs except that the beams on the lower two stories were designed using the proposed procedure, resulting in a strength that was approximately 34\% of current design requirements in the Seismic Provisions, permitting yielding of the W14 × 30 beams due to the unbalanced load from the braces. Composite slabs were placed on the beams and connected with shear studs in a single row to achieve partial composite action. Lateral loading was applied to the top-story beam, which was designed to remain elastic to enable transfer of the loading from actuators to the specimen. Thus, a W24 × 94 beam and thicker concrete slab was used at top level. The braces were ASTM A1085 HSS-5 × 5 × 1/2 braces at all stories. The columns were continuous W12 × 106 members and fully restrained at the base. Additional details for the test specimen and experimental results can be found in the literature.\textsuperscript{15}

Figure 6 compares the experimental and numerical global responses in terms of base shear versus roof drift and base shear versus story drift. Note that load was only applied at the top of the structure, and hence, the story shears were equivalent to the base shear. The test specimen achieved very large story drifts without brace fracture. Following the cycles shown in Figure 6, the specimen sustained column fracture at the second story due to boundary condition issues forced by the very stiff and strong top story.\textsuperscript{13} The comparison here focuses on the behavior prior to that column fracture. As shown in Figure 6, the numerical model slightly underestimates the experimental maximum strength and subsequent post-buckling resistance prior to about 1\% story drift, because the numerical model neglects the contribution of the partially composite slabs to the flexural strength and stiffness of the beams. However, the numerical model simulated frame resistances at story drifts exceeding 2\% with good accuracy because the composite action in the test specimen degraded due to slab cracking and separation of the deck from the steel beams at large drifts. Additionally, the distribution of story drift is somewhat different in the analyses relative to test. As shown, the numerical model predicts larger drift on the third story but smaller drift on the first and second stories relative to the test. The smaller drift on the first story in the numerical analysis has the impact of resulting in a smaller vertical beam displacement at that level as shown below.
Figure 7 compares the numerical and experimental results for midspan deflection of the second floor beam versus first-story drift response. The beam deflection is normalized by the bay width (6000 mm). The comparison shows a similar trend as that observed for the global response. The numerical model overestimates the beam vertical displacement at story drifts less than 1% but agrees well with the experimental results at large story drifts where the composite action with the slab has deteriorated. Note that because the first-story drift in the numerical model is smaller than the test, the maximum beam deformation is smaller. However, the relationship between story drift and beam deformation is simulated well in the analysis.

5 NONLINEAR SEISMIC ANALYSIS

5.1 Ground motion selection and scaling

Dynamic analyses were conducted to evaluate the seismic response of the buildings. Two suites of ground motions were selected from the NGAWest2 Database. These record suites are targeted to the 10%/50-year and 2%/50-year probability of exceedance uniform hazard spectra (UHS) obtained from the USGS hazard maps for the site of the structure in Seattle, Washington. These hazard levels were selected to represent rare events (2% in 50 years) and more frequent events (10% in 50 years). Each suite contains 30 pairs of ground motions (60 horizontal components), which were selected and scaled to minimize the weighted error between the UHS and geometric mean spectral accelerations. Figure 8 shows the target UHS and geometric mean of the scaled spectra for each suite of ground motions. The error was evaluated between $0.5T_1$ and $5T_1$ with maximum weight at $T_1$ and logarithmically decaying weight to the period bounds. The period bounds were chosen to ensure that the ground motions were sufficiently intense at shorter periods corresponding to higher modes and elongated periods corresponding to post-brace buckling and post-brace fracture structural performance states. The same sets of ground motions were used for both three- and nine-story buildings. No more than two records were picked up from the same event and all scale factors were constrained to be less than 5.0.
5.2 Comparison of dynamic behavior and damage

To illustrate the differences in the seismic performance between the three braced frame designs, the behavior of the three-story structures for a single selected ground motion (RSN: 769, Earthquake: Loma Prieta, Station: Gilroy Array #6) is examined. The selected ground motion caused maximum first-story drift close to 84th percentile of the 60 analyses for the 2%/50-year hazard level. The response of each frame is shown in Figure 9. To illustrate the impact of brace fracture on the system performance, only the first-story response is shown; this story had the largest drift demand and was the only story to sustain brace fracture. Figure 9 shows the normalized brace axial force versus first-story drift hysteresis where the brace axial force, $P$, is normalized by $P_{ye}$ and $P_{cre}$ and brace axial deformation is normalized by the brace clear length. As Figure 9 shows, brace fracture occurred at the first story for the AISC Chevron and X-Braced frames while the Proposed Chevron frame did not sustain brace fracture in this ground motion. The most severe damage to the Proposed Chevron frame was the vertical deflection of the beam, which just exceeded 1% of the span length (i.e., $L/100$). However, the beam deflection mitigates fracture of the braces due to low-cycle fatigue. In the Proposed Chevron, the braces sustained large axial shortening deformation but relatively small tensile elongation, as observed in the previous experimental results (Sen et al.,7 Roeder et al., 201912). Additionally, Figure 9 shows that the peak brace forces in the Proposed Chevron design are limited to those in the proposed load case given by Equation (5) and that the peak brace forces in the AISC Chevron are limited to those used in the current load state in the Seismic Provisions, which are given by Equation (3). These results demonstrate that the demand on the beam is dependent on the beam strength and that beam yielding can provide more ductile system response.

To detail the response of the beam, Figure 10 shows the normalized axial force, $P$-bending moment, $M$ orbits at the edge of the middle and corner gusset plates, and the rotation angle at the edge of the corner gusset plate in the Proposed Chevron. In Figure 10A,B, the AISC axial-flexural interaction equations (eq. H1-1a and H1-1b in AISC 2017c17) were also plotted with a dashed line. The axial force in the beam reaches its maximum when the left brace buckles. Whereas the maximum tensile force in the beam is about 27% of $\phi P_n$ of the beam, the maximum compressive force is up to about 16% of $\phi P_n$ due to the composite slab carrying a certain level of force. The effect of such a difference in axial force on flexural strength is as high as 7% based on the AISC interaction equation. After the braces buckled, the bending moment of the beam rapidly increases and $P$–$M$ orbits were outside the interaction equation, indicating that the beam was yielded at nearly this stage.
However, the rotation angle at the edge of the corner gusset plate is within 0.02 rad as shown in Figure 10C. This range of rotation angle is much smaller than the expected rotational capacity, especially because the rotation occurs only in one direction (i.e., is not cyclic). The results show that the proposed design allows for limited yielding of the beams, but does not result in significant plastic deformation where fracture is a concern.

Figures 11, 12, 13, and 14 show the maximum drift at each story of the three-story and nine-story buildings for the 10% and 2%/50-year hazard level (i.e., Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE) levels, respectively) ground motion suites, respectively. Median (50th percentile) and 84th percentile values are highlighted in the figures. Tables 7 and 8 summarize the median and the 84th percentile value for each archetype and hazard level. Although the medians of the maximum story drift are similar for the three-story braced frame designs, the results for the Proposed Chevron show a more uniform distribution of story drift demand up the height of the building. This effect is even more significant when examining the 84th percentile results. Further, the 84th percentiles of the maximum first-story drifts for the Proposed Chevron are lower than those for both the AISC Chevron and the X-Braced configurations, which indicates that the increased story drift at which brace fracture occurs under limited chevron beam yielding actually decreases the drift demand on the frame.
FIGURE 13 Maximum drift on each story for the three-story buildings for the 2%/50-year hazard level. (A) Proposed Chevron; (B) AISC Chevron; and (C) X-Braced

FIGURE 14 Maximum drift on each story for the nine-story buildings for the 2% in 50-year hazard level. (A) Proposed Chevron; (B) AISC Chevron; and (C) X-Braced

TABLE 7. Median and 84th percentile maximum story drifts for three-story archetype designs for the 10% and 2%/50-year hazard levels

<table>
<thead>
<tr>
<th>Story</th>
<th>Proposed Chevron (%)</th>
<th>2% in 50 years</th>
<th>AISC Chevron (%)</th>
<th>2% in 50 years</th>
<th>X-Braced (%)</th>
<th>2% in 50 years</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Median</td>
<td>84th</td>
<td>Median</td>
<td>84th</td>
<td>Median</td>
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TABLE 8. Median and 84th percentile maximum story drifts for nine-story archetype designs for the 10% and 2%/50-year hazard levels

<table>
<thead>
<tr>
<th>Story</th>
<th>Proposed Chevron (%)</th>
<th>2% in 50 years</th>
<th>AISC Chevron (%)</th>
<th>2% in 50 years</th>
<th>X-Braced (%)</th>
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<tbody>
<tr>
<td></td>
<td>Median</td>
<td>84th</td>
<td>Median</td>
<td>84th</td>
<td>Median</td>
<td>84th</td>
</tr>
<tr>
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<td>0.64</td>
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<td>5</td>
<td>0.43</td>
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<td>6</td>
<td>0.28</td>
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<td>0.70</td>
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<td>0.60</td>
<td>0.81</td>
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<td>0.33</td>
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The nine-story frames had smaller drift levels than three-story frames for all designs, as shown in prior research.\textsuperscript{26} Again, the distribution of drift for nine-story frames was more uniform in the Proposed Chevron than the AISC Chevron, and the median maximum story drift for the Proposed Chevron was slightly smaller than that for AISC Chevron and X-Braced designs. However, unlike the three-story frames, the difference in the 84th percentile responses for the three designs was small. This is to be expected, because the drifts of nine-story frames are more influenced by global overturning than the three-story frames and the column sizes between the three designs were the same.

It should also be noted that the median maximum story drift for all designs for both building heights were less than 2\% in the 2%/50-year hazard level. This finding is true for the 84th percentile values at the 10%/50-year hazard level. This indicates that the Proposed Chevron frames meet or exceed the seismic performance expectations for braced frames.

5.3 Computed story drifts
Figures 15 and 16 are used to compare the maximum story drift demand obtained for the buildings subjected to the two ground motion suites for the three- and nine-story buildings, respectively. These figures show the probability (y axis) that the frame will not exceed the story drift on the x axis for a given hazard level (or one minus the probability that the drift will be exceeded). The figures were created using the maximum story drift, from all stories of a frame, for each ground motion time history in a hazard level suite. For example, Figure 15A shows that there is 50\% probability that a story drift of 0.75\% is not exceeded in the 10%/50-year hazard level ground motions for all of the three-story buildings, that is, 30 of 60 ground motions in that suite did not cause maximum drifts greater than 0.75\%.

Figure 15A,B shows that the drift was similar across all the three-story and nine-story frames for the 10%/50-year hazard. Similarly, the drift demands for the nine-story designs were similar for the 2\% in 50-year hazard level. However, for the three-story frames, Figure 15B shows that the Proposed Chevron had a significantly smaller probability of exceeding drifts between 2\% and 4\% than the AISC Chevron and X-Braced designs. For example, the Proposed Chevron have only a 10\% probability of exceeding 3\% story drift, whereas for the X-Braced and the AISC Chevron, there was a 20\% and 25\% probability, respectively. This is attributed to the larger number of occurrences of brace fracture for the X-Braced and AISC Chevron frames and demonstrates the benefits of increasing frame ductility by allowing limited chevron beam yielding. In all cases, the drift demands for the Proposed Chevron braced frames are similar or less than (see Figure 15B) the drift for the AISC Chevron and X-Braced frames.
5.4 Comparison of design and numerically simulated chevron beam demands for ground motion suites

Figure 17 shows median and 84th percentile values for the maximum vertical and horizontal force demand from the braces on the chevron beam supporting the first-story braces (i.e., the beam above the ground floor) in the 2%/50-year hazard level. The design vertical force demand on the beam computed by either Equations 3a or 5a, the current and proposed demands, respectively, and design horizontal demand on the beam computed by either Equations 2b or 4b, the current and proposed demands, respectively, are also shown in the figure with dashed lines. For the Proposed Chevron, the numerical results more consistently align with both design vertical and horizontal force demands on the beam (Equations 3 and 5), because the beam yielding controls the brace resistance as mentioned above. Note that the numerical results for beam force demand slightly exceed the demand from the design expressions. This is expected because the beams used actually have slightly larger strength than minimum required for both the Proposed Chevron and the AISC Chevron designs due to the application of resistance factors and limited beam sizes in design. For the AISC Chevron, it is apparent that whereas the numerical results agree well with the design vertical force demand, the horizontal demand from the analyses is 30% smaller than that used in design and given by Equation 2b. This is because the two opposing braces in a given story of a frame do not simultaneously reach their buckling and tension yield strengths, and instead, the maximum axial force in the beam corresponds better to Equation 4b, which is based on the braces simultaneously reaching their buckling capacity.

Figure 18 shows median and 84th percentile values for the maximum base shear from 2%/50-year hazard level analysis normalized by the design base shear. Whereas X-Braced frames and AISC Chevron show the normalized maximum base shear of 2.0 that is close to the overstrength factor for SCBF in ASCE 7-16, Proposed Chevron frames show smaller values because the brace resistances are controlled by the beam strength. These results indicate that the smaller overstrength factor can be used for the capacity-based design of Proposed Chevron braced frames.
5.5 Comparison of seismic performance

The occurrence of several structural performance states associated with the serviceability, repair, and collapse prevention performance objectives was considered in evaluating the seismic performance of the braced frames. Note that for the 10%/50-year ground motions, there were no occurrences of any performance states mentioned above for all frames. Figure 16 shows the percentage of the ground motions for three- and nine-story buildings at 2% in 50-year hazard level that caused (i) vertical beam deflections, \( \delta \), in excess of \( L/100 \) in any story; (ii) vertical beam deflection, \( \delta \), in excess of \( L/50 \) in any story; (iii) occurrence of at least one brace fracture; and (iv) potential collapse indicated by exceeding 8% story drift at any story in the structure. Vertical beam deflection is associated with serviceability only. Brace fracture is associated with repair. Eight percent drift is associated with collapse. Note that the 8% drift limit for collapse is based on the nonsimulated failure mode of failure of the gravity frame beam-to-column connections as determined from test results from Liu and Astaneh-Asl.\(^{23}\)

As Figure 19 shows, the Proposed Chevron consistently had lower percentages of brace fracture and potential collapse. The differences were most pronounced in the three-story buildings. For the three-story frames, the AISC Chevron and X-Braced frames had braces fracture in over 40% of the ground motions, whereas the Proposed Chevron had brace fracture in fewer than 20% of those ground motions. Notably, all frames had a less than 10% probability of collapse in the 2%/50-year hazard level, which is an approximate target for new buildings designed per ASCE 7-16. Collapse probabilities were generally smaller for the Proposed Chevron, though there were no cases of collapse for either the nine-story Proposed Chevron or X-Braced frames at the 2%/50-year hazard level.

The trade-off for improved brace fracture and collapse performance in the Proposed Chevron frames is increased vertical deformation of the chevron beam. As expected, larger beam deflections were noted at the 2%/50-year hazard level, including in the AISC Chevron. The \( L/100 \) beam deflection performance state was exceeded in between 20% to 30% of the ground motions in the Proposed Chevron frames and in about 5% of the ground motions in the AISC Chevron frames. This performance state was also exceeded in a single ground motion in the nine-story X-Braced frame after brace fracture occurred.

6 SUMMARY AND CONCLUSIONS

A new design for chevron-configured SCBFs has been investigated and validated. The design encourages limited beam yielding following brace buckling, which has been shown to prolong the fracture life of the braces and improve seismic performance. Although well studied in experiments and numerical simulation studies, the impact of this new system on the response of buildings had not been investigated. To do so, a series of buildings was designed using conventional and the newly proposed designs. It is of note that the buildings using the proposed design method were approximately 20%–30% lighter than the conventionally designed chevron SCBFs. The buildings were analyzed using state-of-the-art numerical modeling methods using ground motions representing two seismic hazard levels. Numerical models of the frames were developed in the OpenSees finite element program. The models accurately simulate geometric and material nonlinearity including beam yielding,
brace buckling, brace fracture, and post-fracture frame behavior. The models were validated through comparison with the experimental results of a multistory chevron braced frame test specimen.

The nonlinear response history analyses showed that for the 10%/50-year hazard level, the response of the Proposed Chevron SCBFs were comparable with that of chevron SCBFs designed using current AISC Seismic Provisions and for X-Braced SCBFs. Moreover, even though the proposed design allows moderate beam yielding, the beam deflections for the 10%/50-year hazard level were not large, and no ground motion resulted in a beam deflection exceeding 1% of the beam length.

For the 2%/50-year hazard level, the nonlinear response history analyses demonstrated that the Proposed Chevron SCBFs had fewer instances of brace fracture and generally smaller story drifts relative to the AISC Chevron and X-Braced SCBFs designed to the current AISC Seismic Provisions. The beam yielding limited the tensile deformation of the brace and increased the drift at which brace fracture occurs, as observed in the previous experimental studies.

The results of this study indicate that chevron SCBFs with beams that undergo limited yielding are more economical and have equal or better seismic performance relative to chevron SCBFs designed to current requirements and multistory X-Braced SCBFs. Further, this performance was achieved using the Proposed Chevron beam design loads (i.e., the alternative load states) where the maximum tensile brace force required in the traditional plastic mechanism design is equal in magnitude to the expected buckling strength of the brace.

Further study is needed (and ongoing) to document seismic response for a wider range of parameters including number of stories, brace slenderness, and frame geometry. Additionally, the impact of column size on the performance of braced frames designed with the proposed and current AISC methods is also of interest.

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