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Analytical Study of the Seismic Performance of Steel-Braced Frames with Masonry Infill

Roohollah Ahmady Jazany¹; Iman Hajirasouliha²; Abdolreza S. Moghadam³; Hossein Kayhani⁴; and Hamidreza Farshchi⁵

Abstract: Special concentrically braced frames (CBFs) are widely used as efficient lateral-load resisting systems in seismic regions. In this study, experimentally validated finite-element (FE) models are used to investigate the effects of masonry infill and gusset-plate configuration on the seismic performance of CBFs. It is shown that the presence of masonry infill can increase the initial stiffness and ultimate strength of CBFs by up to 35 and 52%, respectively. However, the frame-infill interaction imposes high plastic strain demands at the horizontal re-entrant corner of gusset plate connections, which may lead to premature failure of fillet welds under strong earthquakes. Whereas using tapered gusset plates can significantly increase the fracture potential at fillet welds, gusset plates with elliptical clearance of eight times the plate thickness can lead to up to 54% lower equivalent plastic strain demands at both gusset plate connections and brace elements. Although the effects of masonry infill are usually ignored in the seismic design process, the results highlight the importance of considering those effects in the seismic design of CBF elements and gusset plate connections. DOI: 10.1061/(ASCE)ST.1943-541X.0001548. © 2016 American Society of Civil Engineers.

Author keywords: Concentrally braced frames; Masonry infill; Gusset plate; Fracture; Equivalent plastic strain; Metal and composite structures.

Introduction

Special concentrically braced frames (CBFs) are widely used as primary lateral-load resisting systems in seismic regions due to their ability in providing high lateral strength, stiffness, and ductility (ANSI/AISC 341 [ANSI/AISC 2010]). Whereas brace elements have a major role in controlling the inelastic lateral deformations of CBFs, gusset plate connections are designed to sustain large inelastic deformations after buckling of the braces (Thornton 1991; Uriz and Mahin 2004). Fracture of brace elements due to excessive postbuckling deformations can result in poor seismic performance of CBFs (Yoo et al. 2007). Premature failure of gusset plate connections and fracture of fillet weld lines are also considered as unfavorable fracture modes in CBFs (Yoo et al. 2008). Lehman et al. (2008) showed that the maximum strain demands of the beam and column elements of CBFs under earthquake loads may change significantly by changing the over strength characteristics of gusset plate connections. Hsiao et al. (2012) proposed a more accurate modelling approach to predict the seismic behavior of CBFs by including the local behavior of gusset plate connections as well as nonlinear geometric effects to simulate the buckling of brace elements. Roeder et al. (2006, 2012) presented a new design method to balance the gusset plate yield mechanisms with the tensile yielding or buckling of the braces. In a follow-up study, Lumpkin et al. (2012) conducted experimental tests on two three-story CBFs with concrete slabs, and concluded that the balanced-design procedure results in thinner and more compact gusset plate connections and also a higher overall ductility in the CBFs.

CBFs with masonry infill represent a typical construction practice adopted in many developing countries such as Iran (Fig. 1). The beam-column connections of these structures are usually simply supported or semirigid and, therefore, the seismic resistance is mainly provided by the frames with brace elements (Hashemi and Hassanzadeh 2008). It is shown in Fig. 1 that the damages to CFBs with masonry infill during the 2003 Bam earthquake were mainly due to the buckling of the braces, fracture of horizontal re-entrant corner of gusset plate connections and spalling of masonry infill. Several experimental programs have been conducted to investigate the actual seismic performance of steel frames with masonry infill under strong earthquake excitations (e.g., Mander et al. 1998; Moghaddam et al. 2006). These studies concluded that the presence of masonry infill improves the stability of the frame and the energy dissipation capacity of the system. However, observations from major devastating earthquakes (especially the Bam earthquake in Iran) highlighted that ignoring the contribution of masonry infill in the seismic design process can lead to the premature fracture of the connections in CBFs.

The effect of brick infill walls on the seismic performance of eccentrically braced frames was studied by Daryan et al. (2009). By using an explicit finite-element method, they demonstrated that the presence of masonry infill can increase the elastic range of the frame and may improve the energy dissipation capacity of the system.
force-displacement behavior, while the plastic deformation capacity of the frame will be deteriorated due to fragility of the masonry wall. However, the results of Daryan et al. (2009) study were based on the superposition of two distinct experimental tests on a CBF and a masonry infill wall and therefore could not capture the actual frame-infill interaction. In a more recent study, Ahmady Jazany et al. (2013) conducted a series of experimental tests to investigate the seismic performance of half-scale CBFs with masonry infill. They showed that the masonry infill can increase the lateral load-bearing capacity and the lateral stiffness of the frames by more than 40%. The results of their study also indicated that the presence of masonry infill may lead to premature fracture at the connection weld lines, which can significantly reduce the seismic performance and ductility of the CBFs.

To prevent or delay the premature failure modes in the connections of CBFs, the current study analytically investigates the seismic performance of CBFs with different gusset plate and weld configurations in the presence and the absence of masonry infill. Experimentally validated finite-element (FE) models are used to obtain more efficient design solutions for practical applications in seismic regions.

### Reference Experimental Program

The cyclic lateral load tests on CBFs conducted by Ahmady Jazany et al. (2013) are used to validate detailed FE models in this study. The test specimens consisted of a concentrically braced steel frame without masonry infill (CBF) and a concentrically braced steel frame with masonry infill (CBFI), which were built at half-scale. Fig. 2 shows the test specimens CBF and CBFI and the experimental test setup. The distance between the centerline of the columns and the height of the frame was 250 and 150 cm, respectively. The half-scale beam and column elements were fabricated using single IPE-270 and IPB-120 sections according to DIN-1025 (1995). Brace elements were UNP 60 with slenderness ratios \( \lambda_x = 56 \) and \( \lambda_y = 34 \). Infill panels consisted of half-scale solid clay bricks with the average size of 219 × 110 × 66 mm, which were placed in running bond with 22 courses.

The frame elements and gusset plate connections were designed based on ANSI/AISC 341-10 (ANSI/AISC 2010). The corner and midheight gusset plates were 280 × 280 × 12 mm, respectively. The beam-to-column connections were double seat angles. The gusset plate connections were designed with elliptical offset of 8 mm (tp is the gusset plate thickness) to provide a balance between the yielding of the gusset plates and braces as suggested by Yoo et al. (2007, 2008). The gusset plates were then welded to both brace and frame elements with 8 mm interface welds.

The CBFI specimen was designed to comply with the requirements of INBC-PART8 (INBC 2005). To evaluate the compressive strength of the masonry infill, 15 three-course block masonry prisms (couplet specimens) were tested based on ASTM C1314-14 (ASTM 2014). The average prism compressive strength was 7.53 MPa that is less than the average compressive strength of the brick and mortar. Based on common construction practice in Bam, a mortar with the similar characteristics to those applied for masonry infill was used to fill the gaps between the steel members.
and the masonry wall. The average thickness of the mortar was 8
and 5 mm next to the beam and column elements, respectively.
To monitor cracking patterns, a very fine layer of low strength plas-
ter was used on the masonry wall panel (less than 1 mm thickness).
The effect of this low strength layer on the seismic performance
of the CBFI was considered to be negligible. The experimental tests
were displacement control under ATC 24 (ATC 1992) cyclic loads
(Fig. 3). More information about the reference tests can be found in
Ahmady Jazany et al. (2013).

**Failure Modes of the Reference Tests**

Based on Ahmady Jazany et al. (2013) experimental results, the
first yielding in the CBF test specimen occurred in the bracing
elements at story drift angle of 0.008 rad. The yielding started be-
tween the corner and midheight gusset plates as shown in Fig 4(b),
which was followed by diagonal yield lines on the gusset plates.
Out-of-plane buckling of the braces started at story drift angle
of 0.012 rad and led to about 13.5 cm out-of-plane deformation
at story drift angle of 0.025 rad [Fig 4(a)]. The experimental test
was stopped at this stage to prevent damage to laboratory equip-
ment. As shown in Figs. 4(a and b), no fracture was observed at
brace and gusset plate connections. Concurrence of flaking of the
whitewashed areas on the brace elements [Fig. 4(a)] and the gusset
plates [Fig. 4(c)] implies that the concept of balanced design was
achieved in this test specimen. As shown in Figs. 4(a) and 5(a) there
was no sign of fracture on the gusset plate weld line up to the
failure point.

Ahmady Jazany et al. (2013) observations showed that the infill-
frame interaction in the CBFI specimen considerably increased the
strain demands of column elements and gusset plate connections.
The first yielding was observed in the steel columns at story drift
angle of 0.008 rad. The yielding of the gusset plate connections and
brace elements started almost simultaneously at story drift angle of

![Fig. 2. Schematic and side view of the reference test specimens in the Ahmady Jazany et al. (2013) study: (a and b) CBF specimen; (c and d) CBFI specimen](image-url)

![Fig. 3. Applied loading pattern (ATC 24)](image-url)
At this stage, a significant part of the whitewashed masonry wall flaked off around the midheight gusset plate connection. Subsequently, the brace elements exhibited out-of-plane buckling at drift angle of 0.012 rad, which was followed by the separation between the masonry infill and the braces as shown in Figs. 6(a and b). Stair-stepped cracks were then developed in the masonry infill followed by sliding cracks along the bed joints. This was concurrent with the fracture of the fillet weld at horizontal re-entrant corner of the gusset plate connection as shown in Fig. 5(b), and the test was terminated at this stage. The damages observed in the CBFI test specimen (i.e., out-of-plane buckling of brace elements, cracking of masonry infill and the fracture of fillet weld at horizontal re-entrant corner of gusset plate connections) compare very well with the observations of the 2003 Bam earthquake in Iran (Fig. 1).

The nonlinear hysteretic behavior of the CBF and CBFI test specimens are compared in Fig. 7(a and b). While the peak and the ultimate load capacity of the CBF specimen was 282 kN and 258 kN, respectively, the corresponding values for the test specimen CBFI reached 398 and 405 kN. Based on the results shown in Fig. 7, the strength degradation of the CBF and CBFI specimens was 9 and 22% at failure, which occurred at drift angles of 0.025 and 0.015 rad, respectively. This implies that the presence of masonry infill not only reduced the deformation capacity of the special concentrically braced frame from 0.025 to 0.015 rad, but also considerably increased the strength degradation of the system.

In general, the similarity between the experimental results and the damage to typical CBFs with masonry infill in the 2003 Bam earthquake in Iran (Fig. 1) can demonstrate the adequacy of the selected half-scale models in this study.

**Performance Parameters**

Previous studies on fracture of metals under cyclic loading have shown that the von Mises stress (or equivalent plastic stress) can be efficiently used to predict the yielding of steel material based on the results of simple uniaxial tensile tests (e.g., Yoo et al. 2008)

\[
\sigma_{eqv} = \frac{1}{2} \left[ (\sigma_x - \sigma_y)^2 + (\sigma_y - \sigma_z)^2 + (\sigma_z - \sigma_y)^2 + 6(\sigma_{xy}^2 + \sigma_{xz}^2 + \sigma_{yz}^2) \right]^{1/2}
\]

where \(\sigma_x, \sigma_y, \sigma_z, \sigma_{xy}, \sigma_{xz}, \text{ and } \sigma_{yz}\) are components of the stress tensor.

Also the equivalent plastic strain \(\varepsilon_{pl}^{eqv}\) has been widely used to evaluate the strain demands and fracture potential of steel connections (El-Tawil et al. 1998; Kanvinde and Deierlein 2005; Yoo et al. 2007, 2008). These studies suggested that the crack initiation can be adequately predicted by defining a threshold for

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**Fig. 4.** Comparison between experimental observations and FE results for CBF: (a) out-of-plane buckling of braces; (b) whitewashed area on brace element; (c) whitewashed area on gusset plate connection; (d through f) von Mises stress distributions in the corresponding FE model

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**Fig. 5.** (a) Yield lines on the gusset plate connection of CBF; (b) fracture of fillet weld at horizontal re-entrant corner of the gusset plate connection of CBFI
For a given stress-strain state, \( \varepsilon_{\text{pl}}^{\text{eqv}} \) is calculated using the following equation:

\[
\varepsilon_{\text{eqv}}^{\text{pl}} = \frac{1}{\sqrt{2(1 + \nu^2)}} \left[ (\varepsilon_x^{\text{pl}} - \varepsilon_y^{\text{pl}})^2 + (\varepsilon_y^{\text{pl}} - \varepsilon_z^{\text{pl}})^2 + (\varepsilon_z^{\text{pl}} - \varepsilon_x^{\text{pl}})^2 \right. \\
\left. + \frac{2}{3} (\gamma_{xy}^{\text{pl}} + \gamma_{yz}^{\text{pl}} + \gamma_{zx}^{\text{pl}}) \right]^{1/2}
\]

where \( \varepsilon_x^{\text{pl}}, \varepsilon_y^{\text{pl}}, \) and \( \varepsilon_z^{\text{pl}} \) = plastic strains; \( \gamma_{xy}^{\text{pl}}, \gamma_{yz}^{\text{pl}}, \) and \( \gamma_{zx}^{\text{pl}} \) = plastic shear strains; and \( \nu^2 \) = effective Poisson’s ratio. It should be noted that \( \varepsilon_{\text{eqv}}^{\text{pl}} \) may depend on the FE mesh size, crack location and stress conditions. Wang et al. (2011) study showed that using a threshold for \( \varepsilon_{\text{eqv}}^{\text{pl}} \) that is calibrated based on experimental results can be a reliable method to predict the crack initiation and propagation in welded connections.

**Experimentally Validated FE Models**

In this section, the nonlinear cyclic behavior of CBFs with and without masonry infill is studied using detailed FE models validated with the reference experimental tests discussed in previous sections. The ANSYS finite-element program is used to perform inelastic dynamic analyses. Fig. 8 shows the schematic view of the CBF and CBFI FE models and the critical points on the gusset plate connections. The steel elements and fillet welds were modelled using the 8-node three-dimensional (3D) solid element SOLID45. The material properties were obtained from the
Table 1. Parameters Used in the Modelling of Masonry Material

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c$</td>
<td>0.88 kg/cm²</td>
</tr>
<tr>
<td>$\eta$</td>
<td>15°</td>
</tr>
<tr>
<td>$\varphi$</td>
<td>38°</td>
</tr>
<tr>
<td>$f_c$</td>
<td>40 kg/cm²</td>
</tr>
<tr>
<td>$f_t$</td>
<td>1 kg/cm²</td>
</tr>
<tr>
<td>$\beta_1$</td>
<td>0.75</td>
</tr>
<tr>
<td>$\beta_2$</td>
<td>0.15</td>
</tr>
</tbody>
</table>

measured stress-strain relationships reported by Ahmady Jazany et al. (2013). Contact elements (CONTA174) were used to simulate the contact and sliding between adjacent surfaces in the seat angle connections. Nonlinear buckling and large displacement analysis were conducted to model the buckling behavior of the braces. The initial imperfections were taken into account by applying 0.000001 of the measured buckling displacements based on the dominant buckling mode shape of the brace elements as observed in the reference experimental tests.

The smeared crack element SOLID65 was implemented to model the mortar and masonry unites. Based on the experimental results reported by Ahmady Jazany et al. (2013), the Young modulus (E) and the Poisson’s ratio ($\nu$) of the masonry unites were considered to be 2,500 MPa and 0.25, respectively. The interaction between steel elements and masonry bricks was modelled using the contact pair elements CONTA174-TARGE170 with Coulomb friction coefficient ($\mu$) of 0.45 as suggested by Shaikh (1978). The fracture of the masonry material was modelled using the Drucker-Prager yield criterion with no strengthening hardening. Pourazin and Eshghi (2009) showed that this pressure-dependent yield model is suitable for the modelling of masonry infill as it is capable of considering different tensile and compressive yield strength values. The William and Warnkle (1975) constitutive model was used to simulate cracking and crushing of masonry infill. The parameters used for the modelling of masonry infill are summarized in Table 1.

The cyclic loading protocol shown in Fig. 3 was applied to the CBF and CBFI FE models. Fig. 7 compares the hysteretic response obtained from the FE models of the test specimens CBF and CBFI and their corresponding experimental tests. Based on this figure, it is evident that the FE models could accurately predict the nonlinear load-displacement behavior of the specimens with less than 10% error.

The von Mises stress distribution results for the CBF and CBFI specimens are displayed in Figs. 4 and 6, respectively. The results show a good agreement between the maximum von Mises stress at the critical points of the gusset plate connections and the fracture of fillet welds at horizontal re-entrant corner of the gusset plates in the reference experimental tests (Fig. 5). Based on Figs. 4(c and f), the von Mises stress distribution in the gusset plate connections is also match with the flaking of the whitewashed area on the gusset plates.

It is shown in Figs. 4(a and d) that the developed FE model could simulate the out-of-plane buckling behavior of the brace elements of the CBF specimen similar to the test observations. It is also shown that the von Mises stress distributions obtained from the FE model, in general, compare well with the yielding locations of the braces, recognized by the flaking of the whitewashed areas.

The comparison between the von Mises stress distribution and the experimental results in Fig. 6 shows that the FE model of the infill panel could identify the crushing zones in the CBFI specimen. The out-of-plane buckling of the brace elements predicted by the FE models in Fig. 6(d) is in good agreement with the experimental observations in Fig. 6(b). The brace elements of the CBFI specimen exhibited about 5.2 cm out-of-plane buckling at the end of the experimental test, which is in good agreement with 4.7 cm out-of-plane displacement predicted from the FE model.

The comparisons mentioned herein indicate that the detailed FE models could simulate the buckling mode of the braces, fracture of the gusset plate connections, and the crushing of the masonry infill panel with an acceptable accuracy as was observed in the reference experimental tests and actual damages to CBFs in the 2003 Bam earthquake.

Effects of Gusset Plate Configuration on the Performance of CBFs

As discussed in the previous sections, the dominant failure mode of the test specimen with masonry infill (CBFI) was the fracture of fillet welds in the re-entrant corners of gusset plate connections. In this study, the experimentally validated FE models introduced in the previous section are used to examine the effects of different gusset plate configurations on the seismic performance of CBFs.

Companion Analytical Models

Four groups of CBFs were designed according to AISC (2010) by using straight and tapered gusset plates with linear and elliptical clearance (Table 2). Similar to the common practice, the frames were designed by ignoring the effects of infill-frame interactions. Fig. 9 demonstrates the configuration of the gusset plate connections of the designed CBFs with and without masonry infill.

The first configuration of the gusset plate was based on elliptical clearance of $8f_p$ ($f_p$ is gusset plate thickness), which was used in the reference experimental tests (Ahmady Jazany et al. 2013). As mentioned before, this gusset plate configuration is recommended by Yoo et al. (2007, 2008) to have a balanced failure in brace elements and gusset plate connections. The second gusset plate configuration had a $2f_p$ linear clearance based on AISC seismic provisions (2010). The third and the fourth configurations were tapered gusset plates with inclination angle of 15 and 25°, respectively. These two gusset plate configurations are used in engineering practice and their performance has been investigated by Yoo et al. (2007, 2008).
Using the previously mentioned gusset plate configurations, a total of eight FE models were developed to simulate the nonlinear seismic behavior of CBFs with the presence and the absence of masonry infill. A summary of the FE models including gusset plates’ geometry, frame elements and the angle and the length of the brace elements welded on the gusset plates are presented in Table 2.

### Table 2. Member Sizes and Gusset Plate Configurations of the FE Models

<table>
<thead>
<tr>
<th>Model</th>
<th>Category</th>
<th>Without infill</th>
<th>With infill</th>
<th>Assigned clearance</th>
<th>Gusset plate thickness (t_p) (mm)</th>
<th>Frame elements (\alpha)</th>
<th>Brace angle and brace length on gusset plate (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T2:2</td>
<td>Elliptical clearance</td>
<td>CBF-a</td>
<td>CBFI-a</td>
<td>8 (t_p)</td>
<td>280 × 280</td>
<td>IPE270</td>
<td>37°, 216</td>
</tr>
<tr>
<td>T2:3</td>
<td>Linear clearance</td>
<td>CBF-b</td>
<td>CBFI-b</td>
<td>Linear 2(t_p)</td>
<td>320 × 320</td>
<td>IPB120</td>
<td>37°, 210</td>
</tr>
<tr>
<td>T2:5</td>
<td>Tapered-T15</td>
<td>CBF-c</td>
<td>CBFI-c</td>
<td>8 (t_p)</td>
<td>Taper of 15°, 350 × 350</td>
<td>UNP60</td>
<td>37°, 286</td>
</tr>
<tr>
<td>T2:6</td>
<td>Tapered-T25</td>
<td>CBF-d</td>
<td>CBFI-d</td>
<td>8 (t_p)</td>
<td>Taper of 25°, 410 × 410</td>
<td>—</td>
<td>37°, 330</td>
</tr>
</tbody>
</table>

\(t_p\) and \(\alpha\) are used for beam, column, and brace elements, respectively.

Using the previously mentioned gusset plate configurations, a total of eight FE models were developed to simulate the nonlinear seismic behavior of CBFs with the presence and the absence of masonry infill. A summary of the FE models including gusset plates’ geometry, frame elements and the angle and the length of the brace elements welded on the gusset plates are presented in Table 2.

### Rupture Potential of Different Gusset Plate Connections

The premature fracture of gusset plate weld lines in CBFs is an unfavorable failure mode (Farshchi and Moghadam 2004; Yoo et al. 2008; Ahmady Jazany et al. 2013), which can considerably affect the seismic performance of the whole structural system under strong earthquakes. To evaluate the rupture potential of gusset plate connections, equivalent plastic strains \(\varepsilon_{pl}^{eqv}\) are calculated at the critical points of the gusset plate connections. The critical points (Points a–d in Fig. 8) are identified based on the experimental test observations and also high stress demand regions in FE models. As discussed before, higher equivalent plastic strain \(\varepsilon_{pl}^{eqv}\) values are usually associated with an increased risk of crack initiation and fracture.

Fig. 10 compares the equivalent plastic strain \(\varepsilon_{pl}^{eqv}\) distribution of gusset plate connections with different configurations in the CBFs with and without masonry infill at story drift angle of 0.025 rad. The results indicate that the maximum \(\varepsilon_{pl}^{eqv}\) values on the horizontal re-entrant corner of the gusset plates in the frames with masonry infill (i.e., CBFI-a, CBFI-b and CBFI-c) are approximately three times higher than the corresponding values in the similar bare frames (i.e., CBF-a, CBF-b, and CBF-c). The results also show that the distribution of \(\varepsilon_{pl}^{eqv}\) in the re-entrant corners of the gusset plates is more uniform in the models without masonry infill. Based on Fig. 10, using an elliptical clearance gusset plate configuration (i.e., CBFI-a and CBF-a models) leads to considerably lower maximum equivalent plastic strains (up to 54% less) compared to other gusset plate configurations. It is also shown that, both in the presence and the absence of masonry infill, the gusset plates with tapered configuration exhibited maximum equivalent plastic strains. This implies that using tapered gusset plates, in general, will increase the potential of premature failure in the connections.

### Optimum Gusset Plate Configurations

To investigate the effects of using different gusset plate configurations on the plastic strain demands of the CBFs connections, Fig. 11 compares the equivalent plastic strain \(\varepsilon_{pl}^{eqv}\) at the critical points of the gusset plates (Points a–d in Fig. 8) at different story drift angles. As it was expected, for the same story drift angle, the equivalent plastic strains in the gusset plate connections of the CBFs with masonry infill were significantly higher (up to three times more) compared to those of the similar bare frames without masonry infill.
compared to those without masonry infill. The presence of masonry infill also changed the most critical points of the gusset plate connections. It is shown in Fig. 11 that the points c and d on the gusset plates (Fig. 8) exhibited maximum fracture positional in CBFI (with masonry infill) and CBF (without masonry infill) models, respectively. This observation is consistent with the equivalent plastic strain distributions shown in Fig. 10.

Based on the results of the reference experimental tests, the threshold for $\varepsilon_{\text{pl,eqv}}$ to initiate cracking in gusset plate connections was calculated to be in the range of 0.028 (lower bond) to 0.032 (upper bond). It is shown in Fig. 11(a) that the gusset-plate connections of the CBFI-a model (CBFI test specimen in Fig. 2) reached the crack initiation threshold at story drift angle of around 0.015 rad, which compares very well with the experimental results presented in Figs. 7(b) and 5(b). The maximum $\varepsilon_{\text{pl,eqv}}$ at the critical points of the gusset plate connections in CBF-a model (CBF test specimen in Fig. 2) was less than the cracking threshold up to the story drift angle of 0.025 rad, where the test was stopped. This implies that no cracking at the horizontal re-entrant corner of the gusset plates was expected at the end of the cyclic tests, which is confirmed by the experimental test observations [Fig. 5(a)].

According to AISC 341 (ANSI/AISC 2010) seismic provision, gusset plate connections in CBFs should be able to sustain an interstory drift angle of at least 0.025 rad, if they are connected to...
It is shown in Fig. 11 that the maximum equivalent plastic strains ($\varepsilon_{\text{pl}}^{\text{eqv}}$) at the critical points of all CBFs with masonry infill clearly exceeded the crack initiation threshold at story drift angle of 0.025 rad, due to the extra demand induced by the frame-infill interaction. For example, it is shown in Fig. 11(b) that the maximum $\varepsilon_{\text{pl}}^{\text{eqv}}$ in the gusset plate connections of the CBF model, designed based on AISC 341 (ANSI/AISC 2010) seismic provision, was less than the crack initiation threshold up to the story drift angle of 0.025 rad. However, in the presence of masonry infill, the maximum equivalent plastic strains in the gusset plate connections exceeded the calculated crack initiation threshold at story drift angle of around 0.015 rad. This means that this CBF cannot sustain the AISC required interstory drift angle, and hence is not qualified for seismic applications. These results can explain the extensive damage to CBFs with masonry infill in the 2003 Bam earthquake in Iran (Farshchi and Moghadam 2004).

Figs 11(c and d) show that, in general, using tapered gusset plate configurations increased the equivalent plastic strains ($\varepsilon_{\text{pl}}^{\text{eqv}}$) at the critical points of the gusset plate connections. For example, it is shown that the tapered gusset plate with taper angle of 25° (configuration Type d) exhibited up to 30% higher $\varepsilon_{\text{pl}}^{\text{eqv}}$ compared to the gusset plate with elliptical clearance (configuration Type a). Even in the frames without masonry infill, the maximum $\varepsilon_{\text{pl}}^{\text{eqv}}$ at the gusset plate connections with tapered configuration was around 50% higher than the cracking initiation threshold at story drift angle of 0.025 rad. Therefore, this type of gusset plate connection is not considered to be qualified based on AISC seismic provisions (2010).

For a better comparison, Table 3 presents the maximum $\varepsilon_{\text{pl}}^{\text{eqv}}$ at the critical points of the gusset plate connections and brace elements of the eight CBFs shown in Fig. 9. The ultimate story drift angle was considered to be 0.015 and 0.025 rad for the models with and without masonry infill, respectively, based on the ultimate story drift angles observed in the reference experimental tests as discussed in the previous sections.

| Table 3. Maximum Equivalent Plastic Strains of $\varepsilon_{\text{pl}}^{\text{eqv}}$ at Gusset Plates and Brace Elements in the Concentrically Braced Frames with and without Masonry Infill |
|------------------|------------------|------------------|------------------|------------------|
| Concentrically braced frames | Maximum $\varepsilon_{\text{pl}}^{\text{eqv}}$ at story drift angle 0.025 rad | Concentrically braced frames with masonry infill | Maximum $\varepsilon_{\text{pl}}^{\text{eqv}}$ at story drift angle 0.015 rad |
|                   | Gusset plates    | Braces           | Gusset plates    | Braces           |
| CBF-a             | 0.015            | 0.88             | CBFI-a           | 0.032            |
| CBF-b             | 0.029            | 0.95             | CBFI-b           | 0.043            |
| CBF-c             | 0.044            | 0.99             | CBFI-c           | 0.052            |
| CBF-d             | 0.049            | 1.12             | CBFI-d           | 0.065            |

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It is shown in Table 3 that in the CBFs without masonry infill, the brace elements exhibited significantly higher equivalent plastic strains ($\varepsilon_{pl}$) compared to the gusset plate connections. Therefore, the failure of the CBFs without masonry infill was always due to the buckling of the brace elements, as it was also observed in the reference experimental tests. However, in the presence of masonry infill, the most critical locations with maximum $\varepsilon_{pl}$ were on the gusset plates. This implies that gusset plate connections played a more dominant role in the failure of CBFs with masonry infill. Therefore, ignoring the effects of masonry infill in the seismic design of CBFs may reduce the deformability of the structural system and lead to a premature failure of the connections under strong earthquakes.

The results in Table 3 indicate that using the balance design concept (i.e., gusset plate configuration Type a) will lead to lower equivalent plastic strain demands at both gusset plate connections and brace elements. While tapered gusset plate configurations are widely used in engineering practice (especially in developing countries), it is shown in Table 3 that using tapered gusset plates (i.e., c and d configurations) will considerably increase the maximum $\varepsilon_{pl}$ at both gusset plates and brace elements. This is more evident for CBFs with masonry infill, in which tapered plates can result in two times higher plastic strains at the gusset plate connections compared to other configurations.

To study the effects of gusset plate configurations on the nonlinear seismic behavior of CBFs with and without masonry infill, Fig. 12 compares the envelope of the cyclic response (i.e., backbone curve) of the eight CBF and CBFI analytical models with different gusset plate configurations. Table 4 summarizes the mechanical properties of the models including yielding force, ultimate strength and initial stiffness based on idealized bilinear backbone curves according to FEMA 356 (FEMA 2000). It is shown that, in the absence of masonry infill, the configuration of the gusset plates could affect the stiffness and the ultimate strength of the CBFs by up to 15 and 10%, respectively. However, as it was expected, the effect of gusset plate configuration on the strength and stiffness of CBFs with masonry infill was negligible (less than 4%). Table 4 also shows that, in general, using tapered gusset plates leads to a lower lateral stiffness and strength compared to other gusset plate configurations.

The results in Fig. 12 and Table 4 indicate that the presence of masonry infill, on average, increased the yielding force, ultimate strength and initial stiffness of the CBFs by 71, 44, and 27%, respectively. However, the CBFs with masonry infill show negative stiffness after the yield point (softening behavior) and also

| Table 4. Mechanical Properties of the CBFs with Different Gusset Plate Configurations |
|----------------------------------------|------------|------------|----------------|----------------|
| Mechanical properties of               | Elliptical clearance | Linear clearance | Tapered T15 | Tapered T25   |
| the analytical models                  | CBF-a      | CBFI-a     | CBF-b        | CBFI-b        | CBF-c        | CBFI-c      | CBF-d        | CBFI-d       |
| Yielding force (KN)                    | 195        | 315        | 178          | 320           | 190          | 323         | 185          | 319          |
| Ultimate strength (KN)                 | 270        | 376        | 282          | 385           | 256          | 388         | 260          | 382          |
| Stiffness (KN/m)                       | 26,270     | 30,208     | 25,320       | 31,230        | 23,810       | 31,524      | 22,950       | 31,051       |

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Summary and Conclusions

Detailed FE models, validated by an experimental joint study, were used to examine the effects of masonry infill and configuration of gusset plate connections on the seismic performance and failure modes of CBFs. Four groups of CBFs were designed by using straight gusset plates (with linear and elliptical clearance) and tapered gusset plates. It was shown that the developed FE models could predict the nonlinear cyclic behavior, damage development and failure patterns of CBFs with and without masonry infill similar to the reference experimental tests. Based on the results of this study, the following conclusions can be drawn:

- Although using masonry infill panels in CBFs can reduce the maximum strain demands at braces, the interaction between masonry infill and steel frame may significantly increase (by up to 300%) the maximum equivalent plastic strains at the horizontal re-entrant corner of gusset plate connections. Therefore, the failure of CBFs with masonry infill is usually due to the premature fracture of gusset plate weld lines as was observed in the reference experimental tests and the 2003 Bam earthquake.

- In the presence of masonry infill, the gusset plates designed by ignoring the effects of frame-infill interaction exceeded the crack initiation threshold at story drift angle of around 0.015 rad, which implies these connections do not meet the ANSI/AISC 341-10 (ANSI/AISC 2010) minimum requirements for seismic regions (i.e., story drift angle of 0.025 rad). This can explain the extensive damage to CBFs with masonry infill in the Bam earthquake.

- Gusset plate connections have a major role in controlling the inelastic behavior of CBFs. Using gusset plates with elliptical clearance of 8 tp (i.e., balance design) will lead to lower equivalent plastic strain demands (by up to 54%) at both gusset plate connections and brace elements. In contrast, using tapered gusset plate configurations can significantly increase the plastic strain demands and fracture potential at gusset plate connections, both in the absence and the presence of masonry infill.

- Whereas the configuration of gusset plates can change the initial stiffness and the ultimate strength of CBFs without masonry infill by up to 15%, the effect of gusset plate configuration on the mechanical properties of CBFs with masonry infill is negligible.

- The presence of masonry infill can increase the yielding force, ultimate strength, and initial stiffness of CBFs by up to 80, 52, and 35%, respectively. However, CBFs with masonry infill exhibit significantly lower ductility and postyield stiffness compared to similar frames without masonry infill.

- Using multiwythe masonry walls can considerably increase the fracture potential at the fillet welds of gusset plate connections under strong earthquakes. It was shown that CBFs with double-wythe and triple-wythe masonry walls exhibit up to 13 and 32% higher equivalent plastic strains at the gusset plate connections, respectively, compared to a similar frame with single-wythe wall.

- The outcomes of this study in general highlight the importance of considering the effects of infill-frame interactions in the design process of CBFs and gusset plate connections in seismic regions. The presented results can be directly used for more efficient design of CBFs using conventional displacement-based or force-based design methods.

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