A Seismic Design Method for Steel Concentric Braced Frames for Enhanced Performance

Shih-Ho Chao\textsuperscript{1} and Subhash C. Goel\textsuperscript{2}

ABSTRACT

Concentric braced frames (CBF) are very efficient and commonly used steel structures to resist forces due to wind or earthquakes. Based on research performed during the last twenty years or so, current seismic codes now include provisions to design ductile concentric braced frames called Special Concentric Braced Frames (SCBF). When designed by conventional elastic design methods, these structures can undergo excessive story drifts after buckling of bracing members. That can lead to early fractures of the bracing members, especially in those made of popular rectangular tube sections. This paper presents some results of a brief study in which a recently developed energy based plastic design methodology was applied to CBF with buckling type braces which exhibit somewhat “pinched” hysteretic loops. Originally the method was developed and successfully applied to moment frames and recently also extended to EBFs and STMFs. The design concept uses pre-selected target drifts and yield mechanisms as performance limit states. The design lateral forces are derived by using an energy equation where the energy needed to push the structure up to the target drift is calculated as a fraction of elastic input energy which is obtained from the selected elastic design spectra. Plastic design is then performed to detail the frame members in order to achieve the intended yield mechanism and behavior. Results of inelastic dynamic analyses carried out on example frames designed by the proposed method showed that the frames met all the desired performance objectives, including the intended yield mechanisms and the story drifts while preventing brace fractures under varied hazard levels. On the other hand, when designed by current code procedures as SCBF the same structures showed very poor response due to premature brace fractures leading to unacceptably large drifts and instability.

Keywords: Chevron, CBF, PBPD

INTRODUCTION

Concentric braced frames (CBFs) are generally considered less ductile seismic resistant structures than other systems due to the brace buckling or fracture when subjected to large cyclic displacements. Nevertheless, it has been estimated that CBFs comprise about 40\% of the newly built commercial constructions in the last decade in California (Uriz, 2005). This is attributed to simpler design and high efficiency of CBFs compared to other systems such as moment frames, especially after the 1994 Northridge Earthquake. However, recent analytical studies have shown that CBFs designed by conventional elastic design method suffered severe damage or even collapse under design level ground motions (Sabelli, 2000). This paper investigates the responses of a typical CBF designed according to current practice through nonlinear dynamic analyses using SAC ground motions (Somerville et al.,

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The reasons responsible for inferior seismic responses of CBFs are discussed and a performance-based design methodology is proposed for improving the behavior of CBFs under seismic excitation.

SEISMIC PERFORMANCE OF STUDY CBFs

CBF Designed by Conventional Elastic Design Method (3V-NEHRP)

A three-story Chevron type CBF used in a previous study (Sabelli, 2000) was redesigned by the current U.S. practice (AISC, 2005) using the NEHRP design spectra (FEMA, 1997). The building layout and final design sections are shown in Fig. 1, and Table 1 gives the corresponding design parameters. It should be noted that the braces were designed based on initial buckling strength \(2\phi P_r \cos \alpha\). Beams were designed based on the difference of expected yield strength and post-buckling strength \(0.3P_r\), assuming out-of-plane buckling) of the braces. The column design forces were based on gravity loading, post-post-buckling strength of braces, and vertical unbalanced load on the beams from the braces. The brace fracture life (Table 1), \(N_f\), was estimated by the following empirical equation, which was derived from test results of HSS braces under cycling loading (Tang and Goel, 1987):

\[
N_f = \begin{cases} 
262 \frac{(b/d)(KL/r)}{((b-2t)/t)^2} & \text{for } KL/r > 60 \\
262 \frac{(b/d)60}{((b-2t)/t)^2} & \text{for } KL/r \leq 60 
\end{cases}
\]  

(1)

where \(d\) is the gross depth of the section; \(b\) is the gross width of the section \((b \geq d)\); \(t\) is the wall thickness; \((b-2t)/t\) is the width-thickness ratio of compression flanges; \(KL/r\) is the brace slenderness ratio. Note that conventional elastic method does not consider the brace fracture life.

Nonlinear analyses were carried out by using the SNAP-2DX program, which has the ability to model brace behavior under large displacement reversals, as well as to predict the fracture of a brace having tube section (Rai et al., 1996). Gravity columns were included in the modeling by using continuous leaning columns, which were linked to the study brace frame through rigid beams. All beams and columns were modeled as beam-column elements. It is noted that, due to the presence of gusset plates, the beam-to-column connections at first and second levels of the frame were modeled by assuming fixed end condition.

A typical response of the 3V-NEHRP frame is shown in Fig. 2a under LA 02 ground motion (10% in 50yrs). It is seen that the first story drift exceeded 8% after 20 sec. due to early brace fracture (i.e., less fracture life). Significant yielding was also observed in columns due to rigid connections as shown in Fig 2b. This agrees with previous testing results that severe damage can occur in the vicinity of connection region if CBF is designed by current practice (Uriz, 2005; see Fig. 2c).

CBF Designed by Plastic Method (3V-PD)

It was seen from the responses of the 3V-NEHRP that CBF designed based on conventional method did not meet the intended performance due to uncontrollable damage in terms of location and degree. In view of this, a new design method based on plastic design concepts was used to enhance the performance of CBFs. As shown in Fig. 3a, a target yield mechanism was first selected which limits the yielding/buckling to the braces and column bases only. Then the braces were designed based on their ultimate state, i.e., tension yielding and post-buckling \(((\phi P_y + 0.5\phi P_r) \cos \alpha\), see Table 2). The
post-buckling strength was taken as $0.5P_{cr}$ instead of $0.3P_{cr}$ because the braces used for 3V-PD frame were made of double tube section (see Fig. 3b). Using built-up sections made of double tubes is an effective way to reduce width-thickness ratios without increasing the wall thickness of the sections (Lee and Goel, 1990). Moreover, this technique can utilize simple gusset plate connections with direct welding between the gusset plate and double tubes, without the inconvenience of making the necessary slots at both ends of a single tube member for welded gusset plate connections. Such built-up double tube members also buckle in-plane, which can eliminate possibility of damage of surrounding non-structural elements due to out-of-plane buckling of single tube section. The in-plane buckling of double tube section also simplifies the design of gusset plates because the plastic hinges will form in the brace instead of the gusset plates (three plastic hinges in brace). Tests carried out by Lee and Goel (1990) showed that double tube bracing members were able to dissipate more energy by sustaining more loading cycles when compared with single tube members. The post-buckling strength is nearly half the initial buckling strength due to the in-plane buckling (fixed end condition).

The beams and columns were designed as for the 3V-NEHRP frame. Further, in order to avoid the column hinging, beam shear splice was used in this study to prevent the beam moment transfer to column, as shown in Fig. 3b. Another advantage of using this scheme is that the connection can be shop-fabricated thereby enhancing the quality and reducing the field labor cost. In addition, based on preliminary study (Dasgupta and Goel, 2006), a minimum design fracture life of 200 (according to Eq. 1) for braces was specified to prevent early fracture of the braces. The final design sections for 3V-PD frame are shown in Fig. 3b and corresponding design parameters are given in Table 2. Comparison between Tables 1 and 2 shows that the use of double tube bracing members leads to smaller difference between the tensile and post-buckling strengths, thereby reducing the beam sizes.

Nonlinear dynamic analysis results showed that the 3V-PD frame behaved as intended, as shown in Fig. 4. No brace fracture was observed in all 10%/50yrs ground motions, which resulted in stable responses as indicated in Fig. 4a. Column hinging was also eliminated (except at the column base). This suggests that the intended yield mechanism (Fig. 3a) can be achieved by using the proposed design approach. It is seen that the seismic performance can be significantly enhanced by using the proposed design method without increasing the material weight (see Table 4). However, the drift still could not be controlled effectively. This was more evident when 3V-PD frame was subjected to 2%/50yrs ground motions (see Fig. 4b)

**CBF Designed by Proposed Performance-Based Plastic Design Method (3V-PBPD)**

Drift control is essential to achieve acceptable performance of CBFs. This can be accomplished by using a newly developed performance-based plastic design (PBPD) methodology, which has been successfully applied to moment frames, eccentrically braced frames, and special truss moment frames (Lee and Goel, 2001; Chao and Goel, 2005; Chao and Goel, 2006). This design concept uses pre-selected target drifts ($\theta_p$ in Fig. 3a) and yield mechanism as performance limit states. The design lateral forces are derived by using an energy (work) equation where the energy needed to push the structure up to the target drift is calculated as a fraction of elastic input energy which is obtained from the selected elastic design spectra (see Fig. 5a). Plastic design is then performed to detail the frame members in order to achieve the intended yield mechanism and behavior, as have been done in 3V-PD frame. The resulting design base shear obtained from energy balance can be expressed as:

$$V/W = \left( -\alpha + \sqrt{\alpha^2 + 4\gamma C_s^2} \right) / 2$$

where $V$ is the design base shear; $W$ is the total seismic weight of the structure; $\alpha$ is a dimensionless parameter, which depends on the period of the structure, the modal properties, and the intended drift level; $C_s$ is the design pseudo-acceleration coefficient based on code design spectrum. The energy
modification factor, $\gamma$, depends on the structural ductility factor ($\mu_s = \Delta_{\text{max}}/\Delta_s$) and the ductility reduction factor ($R_\mu = C_{eu}/C_\gamma$), which is related to the period of the structure and can be determined as:

$$\gamma = \left(2\mu_s - 1\right)/R_\mu^2$$  \hspace{1cm} (3)

The inelastic spectra by Newmark and Hall (1982) as shown in Figure 5b were used to obtain the above equation.

The design base shear shown in Eq. 2 was originally derived by assuming elastic-plastic hysteretic response for the structural elements in MF, EBF, or STMF. However, buckling of braces in CBFs leads to a “pinched” hysteretic response. Therefore using the same design base shear for a CBF would lead to major under-design. Parametric study based on a simple one-story one bay braced frame with pin-connected rigid beams and columns showed that the dissipated energy by CBF is approximately 35% of the energy dissipated by a structure with full elastic-plastic hysteretic loops, with both frames having equal strengths ($\eta = A_1/A_2 = 0.35$ in Fig. 6). Considering that other structural members such as gravity frames will also resist earthquake forces, a slightly higher $\eta = 0.5$ was used herein for design purposes. By using energy balance relation, the modified design base shear for CBFs can be determined by:

$$V/W = \left(-\alpha + \sqrt{\alpha^2 + 4(\gamma/\eta)C_e^2}\right)/2$$  \hspace{1cm} (4)

The three-story study braced frame was redesigned by using Eq. 4, along with the plastic design approach and a minimum design fracture life of 200 as used for 3V-PD frame. The pre-selected target drift was 1.25% for 10%/50yrs hazard level. Note that the design base shear obtained by Eq. 4 would be smaller if a larger target drift was selected. The resulting design base shear for 3V-PBPD is 1.7 times of that of 3V-NEHRP and 3V-PD. This is because of two reasons: 1) The drift control is built into the PBPD design base shear expression; 2) The base shear in the proposed method corresponds to the global yield mechanism ($C_yW$ in Fig. 5a) while that from NEHRP is intended for use at elastic design levels. The final design section and corresponding design parameters are shown in Fig. 7 and Table 3, respectively.

Typical responses of 3V-PBPD frame are shown in Figs 8a and 8b, under 10%/50yrs and 2%/50yrs ground motions, respectively. It is seen that the behavior is quite stable and drift was considerably reduced as compared with 3V-NEHRP and 3V-PD frames. Fig. 9 shows the maximum interstory drifts for both 3V-NEHRP (current U.S. practice) and 3V-PBPD (proposed method) due to twelve 10%/50yrs SAC ground motions. As can be seen, the 3V-NEHRP frame experienced large concentrated drift in the first story, accompanied by brace fracture and column hinging. On the other hand, CBF designed by the proposed PBPD approach resulted in more uniformly distributed story drift along the height of the frame (and generally within pre-selected target drift), while eliminating brace fracture and column yielding.

**CONCLUSIONS**

This paper presents performance of a concentric braced frame (CBF) designed by current U.S. practice under seismic excitation. The nonlinear dynamic analysis results showed that CBFs designed by conventional elastic method can suffer early brace fractures and damage in the vicinity of the connection region, which in turn leads to excessive story drift and possible collapse due to P-delta effect. Providing a means to relieve brace-beam-column connections from beam moment, such as by a beam shear splice, is essential to prevent undue damage in that region. On the other hand, behavior of CBFs when designed by the proposed performance-based plastic design (PBPD) methodology can be much better in terms of developing intended yield mechanism, preventing or delaying brace fracture,
and controlling the drift within target limits. This indicates that the confidence level of satisfactory CBF performance can be raised by using appropriate performance-base design methodologies, such as the one proposed herein. Further research work for improving the performance of CBFs by using the proposed PBPD approach is currently in progress.

ACKNOWLEDGMENTS

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### Table 1. Selected design parameters of 3V-NEHRP frame

<table>
<thead>
<tr>
<th>FL</th>
<th>Required Story Shear, $V_i$ (kips)</th>
<th>Brace Size, $(A_y, \text{in}^2)$</th>
<th>Design Strength, $2\phi P_c \cos \alpha$ (kips)</th>
<th>Difference between Tensile and Post-buckling Strengths, $P_y - 0.3P_c$ (kips)</th>
<th>Fracture Life, $N_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>3rd</td>
<td>203 HSS6×6×5/16 (6.43)</td>
<td>241</td>
<td>243</td>
<td>78</td>
<td></td>
</tr>
<tr>
<td>2nd</td>
<td>320 HSS7×7×3/8 (8.97)</td>
<td>386</td>
<td>328</td>
<td>71</td>
<td></td>
</tr>
<tr>
<td>1st</td>
<td>377 HSS7×7×3/8 (8.97)</td>
<td>386</td>
<td>328</td>
<td>71</td>
<td></td>
</tr>
</tbody>
</table>

*Note: the braces of 3V-NEHRP frame were chosen based on strength demand, compactness requirement, and weight (lightest among available sections).

### Table 2. Selected design parameters of 3V-PD frame

<table>
<thead>
<tr>
<th>FL</th>
<th>Required Story Shear, $V_i$ (kips)</th>
<th>Brace Size, $(A_y, \text{in}^2)$</th>
<th>Design Strength, $(\phi P_y + 0.5\phi P_c) \cos \alpha$ (kips)</th>
<th>Difference between Tensile and Post-buckling Strengths, $P_y - 0.5P_c$ (kips)</th>
<th>Fracture Life, $N_f$</th>
</tr>
</thead>
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<tr>
<td>3rd</td>
<td>203 2HSS3×3×5/16 (5.88)</td>
<td>226</td>
<td>211</td>
<td>499</td>
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</tr>
<tr>
<td>2nd</td>
<td>320 2HSS 3-1/2×3-1/2×3/8 (8.18)</td>
<td>328</td>
<td>273</td>
<td>461</td>
<td></td>
</tr>
<tr>
<td>1st</td>
<td>377 2HSS4×4×3/8 (9.56)</td>
<td>397</td>
<td>299</td>
<td>283</td>
<td></td>
</tr>
</tbody>
</table>

*Note: the braces of 3V-PD frame were chosen based on strength demand, compactness requirement, weight, and fracture life.

### Table 3. Selected design parameters of 3V-PBPD frame

<table>
<thead>
<tr>
<th>FL</th>
<th>Required Story Shear, $V_i$ (kips)</th>
<th>Brace Size, $(A_y, \text{in}^2)$</th>
<th>Design Strength, $(\phi P_y + 0.5\phi P_c) \cos \alpha$ (kips)</th>
<th>Difference between Tensile and Post-buckling Strengths, $P_y - 0.5P_c$ (kips)</th>
<th>Fracture Life, $N_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>3rd</td>
<td>343 2HSS4×4×3/8 (9.56)</td>
<td>397</td>
<td>299</td>
<td>283</td>
<td></td>
</tr>
<tr>
<td>2nd</td>
<td>540 2HSS 4-1/2×4-1/2×1/2 (13.9)</td>
<td>589</td>
<td>418</td>
<td>395</td>
<td></td>
</tr>
<tr>
<td>1st</td>
<td>637 2HSS5×5×1/2 (15.76)</td>
<td>654</td>
<td>454</td>
<td>268</td>
<td></td>
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</table>

*Note: the braces of 3V-PBPD frame were chosen based on strength demand, compactness requirement, weight, and fracture life.

### Table 4. Material weight for one braced frame

<table>
<thead>
<tr>
<th>Weight Calculation</th>
<th>3V-NEHRP (lbs)</th>
<th>3V-PD (lbs)</th>
<th>3V-PBPD (lbs)</th>
<th>3V-PD/3V-NEHRP</th>
<th>3V-PBPD/3V-NEHRP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Braces</td>
<td>3505</td>
<td>3359</td>
<td>5590</td>
<td>0.96</td>
<td>1.59</td>
</tr>
<tr>
<td>Beams</td>
<td>14040</td>
<td>13530</td>
<td>19500</td>
<td>0.96</td>
<td>1.39</td>
</tr>
<tr>
<td>Column</td>
<td>7488</td>
<td>7488</td>
<td>7488</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Total</td>
<td>25033</td>
<td>24377</td>
<td>32578</td>
<td>0.97</td>
<td>1.30</td>
</tr>
</tbody>
</table>
Figure 1. (a) Layout of the study 3-story building; (b) Design sections for 3V-NEHRP.

Figure 2. (a) Drift response of 3V-NEHRP under LA02 (10%/50 years) ground motion; (b) Column plastic hinge rotation and brace fractures; (c) Fracture of beam-column connection (Uriz, 2005).
Figure 3. (a) Pre-selected yield mechanism for CBF; (b) Design sections for 3V-PD.

Figure 4. (a) Drift response of 3V-PD under LA02 (10%/50 years) ground motion; (b) Drift response of 3V-PD under LA27 (2%/50 years) ground motion; (c) Column plastic hinge rotation (LA 02); (d) Column plastic hinge rotation (LA 27).
Figure 5. (a) Structural idealized response and energy balance concept; (b) Inelastic response spectra by Newmark and Hall (1982).

Figure 6. Typical hysteretic responses for CBF and BRBF.

Figure 7. Design sections for 3V-PBPD.
Figure 8. (a) Drift response of 3V-PBPD under LA02 (10%/50 years) ground motion; (b) Drift response of 3V-PBPD under LA27 (2%/50 years) ground motion.

Figure 9. (a) Drift response of 3V-NEHRP under 10%/50 years ground motion; (b) Drift response of 3V-PBPD under 2%/50 years ground motion.