PERFORMANCE-BASED PLASTIC DESIGN OF STEEL CONCENTRIC BRACED FRAMES FOR ENHANCED CONFIDENCE LEVEL

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ABSTRACT :

Concentrically braced steel frames (CBFs) are very efficient and commonly used steel structures for resisting seismic loads because they provide complete truss action. Based on research performed during the last twenty years or so, current U.S. seismic codes now include provisions to design ductile concentric braced frames called Special Concentrically Braced Frames (SCBFs). When designed by conventional elastic design methods, these structures can undergo excessive story drifts after buckling and yielding of bracing members. That can lead to early fractures of the bracing members, especially in those made of popular rectangular tube sections. Recent analytical studies have indicated that the confidence level to achieve collapse prevention performance objectives for typical SCBF can be extremely and unacceptably low when compared with special moment frames (SMFs). This paper presents results of a study in which a newly developed performance-based based plastic design (PBPD) methodology was applied to CBF with buckling type braces. Originally the method was developed and successfully applied to moment frames and more recently extended to other steel framing systems as well. 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 balance 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 velocity spectra. Plastic design is then performed to detail the frame members and connections in order to achieve the intended yield mechanism and behavior. Results of extensive inelastic dynamic analyses carried out on example frames designed by the PBPD method showed that the frames met all the desired performance objectives, including the intended yield mechanisms and the story drifts. Reliability-based evaluation in accordance with FEMA 351 indicated that the PBPD frames have confidence levels against global collapse much higher than those of corresponding SCBFs designed by current practice.

KEYWORDS: Concentrically Braced Frames, Performance-Based Design, PBPD, Plastic Design, Target Drift, Yield Mechanism

1. INTRODUCTION

Concentrically 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). In addition, the confidence level (FEMA, 2000) to achieve collapse prevention performance objectives for typical SCBFs can be extremely and unacceptably low when compared with special moment frames (Uriz and Mahin, 2004). This paper shows that the confidence level for collapse prevention of CBFs can be greatly enhanced when designed by a newly developed seismic design methodology, called Performance-Based Plastic Design (PBPD).

2. DESIGN OF STUDY FRAMES

2.1. CBFs Designed by Conventional Elastic Design Method (3V-NEHRP and 6V-NEHRP)

The three- and six-story Chevron type CBFs originally designed (Sabelli, 2000) as SCBF according to 1997 NEHRP design spectra (FEMA, 1997) and 1997 AISC Seismic Provisions (AISC, 1997) were used in this study. In those frames, the beams were designed based on the difference of nominal yield strength (P_y) and post-buckling strength of braces ($0.3\phi_c P_{cr}$, assuming out-of-plane buckling). The material overstrength factor, R_y (the ratio of expected yield strength to specified minimum yield strength), was not specified for design of beams in the 1997 Provisions, which led to considerable yielding in the beams at the location of brace intersection under major earthquakes (Sabelli, 2000). In this study, those frames were re-designed using the 1997 NEHRP spectra and the 2005 AISC Seismic Provisions (AISC, 2005), where the beams are required to be designed based on the difference of expected yield strength (R_yP_y) and nominal post-buckling ($0.3P_{cr}$) of braces. The member sizes of 3V-NEHRP and 6V-NEHRP frames are shown in Figures 1(a) and 2(a), respectively.

2.2. CBFs Designed by PBPD Method (3V-PBPD and 6V-PBPD)

Drift control is essential to achieving acceptable performance of CBFs. This can be accomplished by using the 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, 2006a; Chao and Goel, 2008). This design concept uses pre-selected target drifts (θ_p in Figure 3) and yield mechanism as performance limit states. The design base shear is 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 (Figure 4). The resulting design base shear obtained from energy balance can be expressed as (Chao and Goel, 2006b):

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

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

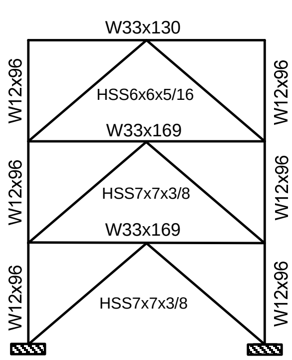
$$\gamma = (2\mu_s - 1)/R_\mu^2 \tag{2}$$

The inelastic spectra proposed by Newmark and Hall (1982) as shown in Figure 5 (R_μ -μ_s-T relationship) were used to calculate γ from Eqn. (1). To account for the fact that buckling of braces in CBFs leads to a “pinched” hysteretic response, a factor ($\eta = A_1/A_2$ in Figure 6) is used in Eqn. (1) to express the energy dissipation ratio between a typical CBF and a frame with full elastic-plastic hysteretic loops (such as BRBF). Preliminary study suggests that $\eta = 0.5$ is reasonable for design purposes (Chao and Goel, 2006b). The two frames were re-designed by using Eqn. (1), along with the plastic design approach to detail the frame members to achieve the intended yield mechanism and behavior. The pre-selected target drift was 1.25% for 10%/50yrs hazard level. A minimum required design fracture life (N_f) of 100 was specified to prevent premature facture of HSS braces. The expected value of N_f, for HSS braces was estimated by the following empirical equation, which was derived from test results under cycling loading (Tang and Goel, 1987):

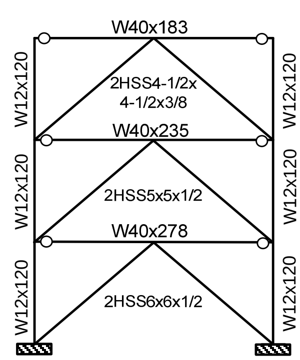
$$N_f = \begin{cases} 262 \cdot \{(b/d)(KL/r)\} / \{(b-2t)/t\}^2 & \text{for } KL/r > 60 \\ 262 \cdot \{(b/d) \cdot 60\} / \{(b-2t)/t\}^2 & \text{for } KL/r \le 60 \end{cases} \quad (3)$$

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; $(b\text{-}2t)/t$ is the width-thickness ratio of compression flanges; KL/r is the brace slenderness ratio. Note that current practice does not explicitly consider the brace fracture life in this form.

Built-up brace sections made of double HSS were used for the PBPD frames, as shown in Figure 7. This is an effective way to reduce width-thickness ratios without increasing the wall thickness of the sections (Lee and Goel, 1990). This technique utilizes 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 generally also buckle in-plane, which can eliminate the possibility of damage to surrounding non-structural elements due to out-of-plane buckling of single tube section. The in-plane buckling 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). Further, in order to avoid column plastic hinging due to presence of gusset plates (Figure 8), beam shear splice was used to prevent the beam moment transfer to column, as shown in Figure 7. Another advantage of using this scheme is that the connection can be shop-fabricated thereby enhancing the quality and reducing the field labor cost. The final design sections for 3V-PBPD and 6V-PBPD are shown in Figures 1(b) and 2(b), respectively. More complete design details can be found elsewhere (*i.e.*, Goel and Chao, 2008).

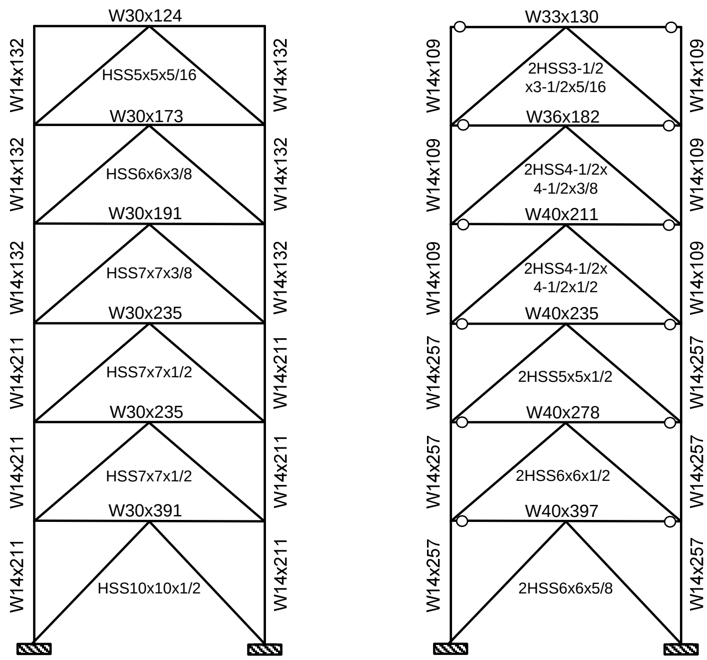


(a) 3V-NEHRP



(b) 3V-PBPD

Figure 1 Member sections for the 3-story CBF designed by (a) current design practice; and (b) PBPD



(a) 6V-NEHRP (b) 6V-PBPD

Figure 2 Member sections for the 6-story CBF designed by (a) current design practice; and (b) PBPD

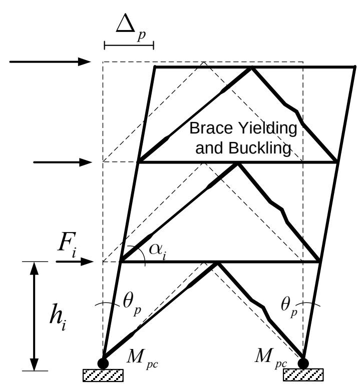


Figure 3 Yield mechanism of Chevron-type CBF

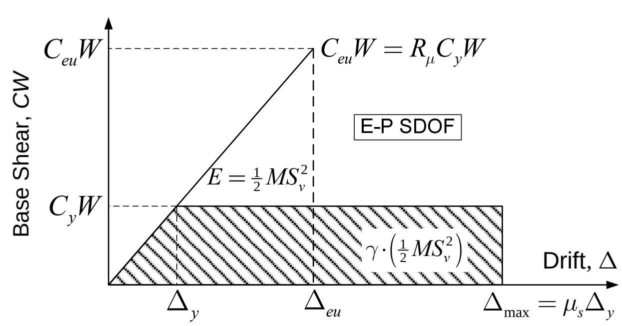


Figure 4 Energy balance concept

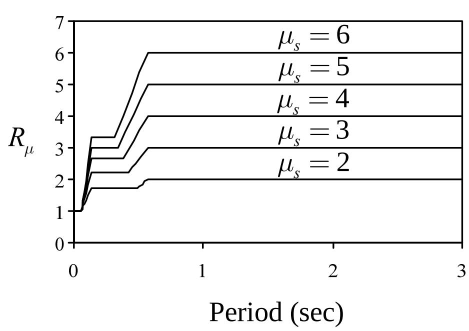


Figure 5 Inelastic response spectra by Newmark and Hall (1982)

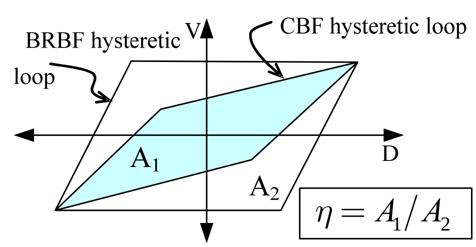


Figure 6 Energy reduction ratio, η

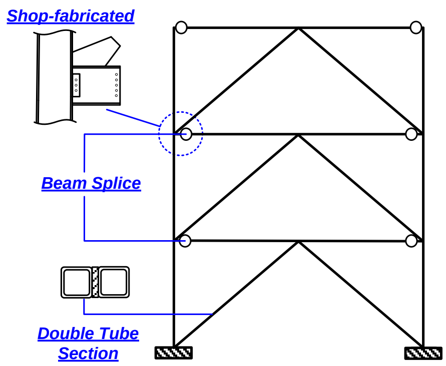


Figure 7 Connection details in PBPD CBFs

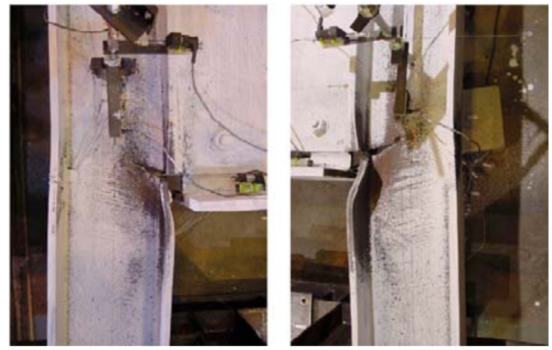


Figure 8 Fracture of beam-to-column connection (Uriz, 2005)

3. CONFIDENCE LEVEL EVALUATION

The main purpose here is to investigate the confidence level of study CBFs against global collapse under MCE (2%/50yrs) earthquakes. Performances of the NEHRP and PBPD frames were evaluated by following the procedure similar to that used in FEMA 351 for SMFs (FEMA, 2000). This probability-based quantitative approach involves evaluation of site-specific hazard, structural capacity, and structural demand, such that by having the hazard level and performance criteria the confidence level for the structure can be estimated. Although the method was originally introduced for evaluation of SMFs, the procedure has been modified so that it can be used for CBFs as well (Uriz and Mahin, 2004). It is noted that the parameters for CBFs used by Uriz and Mahin (2004) are approximate values, therefore, the numeric values of confidence levels obtained might not be as accurate as those for SMFs. Nevertheless, they are reasonable enough for comparison purposes since all parameters are the same for both NEHRP and PBPD frames.

3.1. Modeling and Analysis

3.1.1 Drift Demands

Nonlinear dynamic 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 the fracture life of a tubular brace (Rai et al., 1996). Gravity columns were included in the model by using continuous leaning columns, which were linked to the braced frame through pin-ended rigid elements. Those gravity columns created significant P-Δ effect under large drifts. It is noted that, due to the presence of gusset plates, the beam ends at all levels (except for the top levels) of the NEHRP frames were modeled by assuming fixed-end condition. Twenty 2%/50yrs SAC LA site ground motions (Somerville et al., 1997) were used to obtain the maximum drift demands.

3.1.2 Drift Capacities

Incremental Dynamic Analysis (IDA) was employed, according to FEMA 351 (FEMA, 2000), to obtain the drift capacities of the frames. The maximum interstory drift was obtained through nonlinear dynamic analysis under varying intensities of twenty 2%/50yrs SAC ground motions (FEMA, 2000; Vamvatsikos and Cornell, 2002). Having the intensity (in terms of spectral acceleration, S_a) versus maximum story drift curve for each ground motion, the drift capacity for a particular ground motion can be estimated at the point where the slope of the curve falls below one-fifth of its initial slope. Additionally, as an upper bound, the drift capacities cannot be considered greater than 10%. Figures 9 and 10 give the IDA results for the three- and six-story frames, respectively. The drift capacities are shown in these figures by hollow circles on each curve.

3.2. Confidence Level

Randomness and uncertainty parameters as well as resulting confidence levels are shown in Tables 1 and 2, respectively. As can be seen, confidence level of the 3V-NEHRP frame against global collapse (<<1%) was dramatically improved when it was re-designed by the PBPD method (*i.e.*, 3V-PBPD frame). It is worth mentioning that the enhanced confidence level (>99.9%) is comparable to those of SMFs designed according to 1997 NEHRP provisions (Yun et al., 2002). It can be seen that although the median drift capacities for the two three-story frames are somewhat close, the drift demand of the 3V-PBPD frame is only about 22% of the 3V-NEHRP frame. Table 2 also shows that, the confidence level for 6V-NEHRP frame (23.3%) is higher compared to the 3V-NEHRP frame, but is still much below the 90% satisfactory level suggested by FEMA 351 for SMFs. The confidence level of 86.2%, for the 6V-PBPD frame is also quite close to the 90% level.

Table 1 Randomness and uncertainty parameters

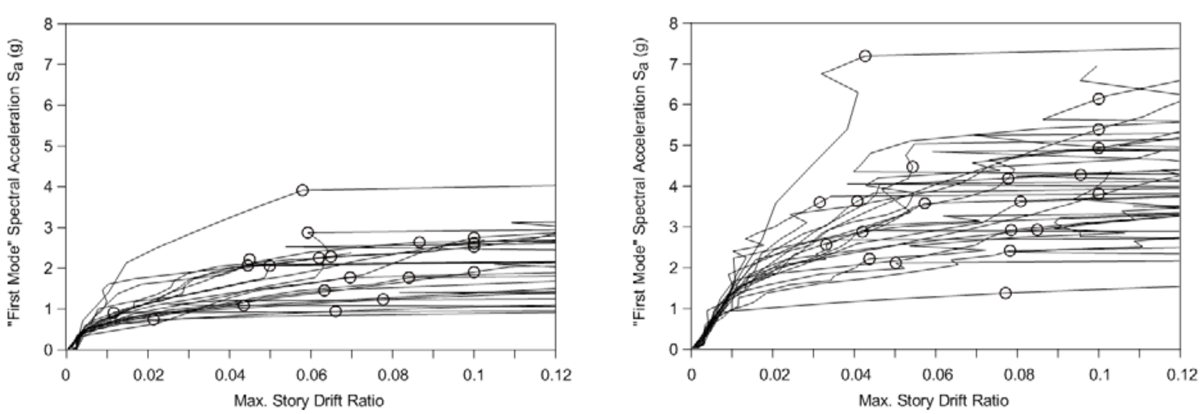
Frame	β_{RC}	ϕ_{RC}	β_{UC}	ϕ_{UC}	β_{RD}	β_{UT}
3V-NEHRP	0.537	0.649	0.15	0.967	0.890	0.30
3V-PBPD	0.394	0.793	0.15	0.967	0.545	0.30
6V-NEHRP	0.435	0.753	0.20	0.942	0.663	0.35
6V-PBPD	0.412	0.775	0.20	0.942	0.708	0.35

*β_{RC}: standard deviation of natural logs of drift capacities due to randomness
*β_{UC}: standard deviation of natural logs of drift capacities due to uncertainty
*β_{RD}: standard deviation of natural logs of drift demands due to randomness
*β_{UT}: vector sum of logarithmic standard deviations for both demand and capacity considering all sources of uncertainty

Table 2 Summary of confidence level assessment for 3-story and 6-story CBFs

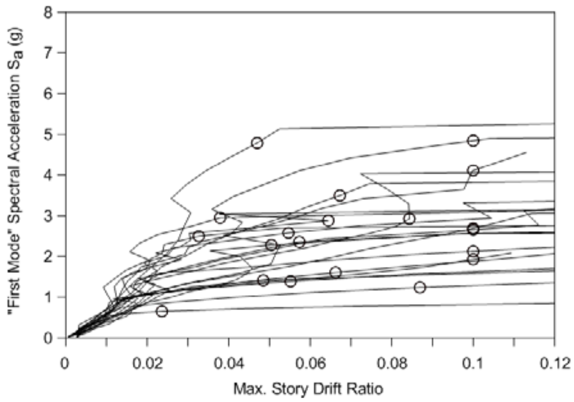
Frame	Median Drift Capacity (from IDA) $\boldsymbol{C}$	Capacity factor $\boldsymbol{\phi}$	Median Drift Demand $\boldsymbol{D}$	Demand factors		Confidence Parameter	Confidence Level (%)
				γ	γ_a	$\lambda = \frac{\gamma \cdot \gamma_a \cdot \boldsymbol{D}}{\boldsymbol{\phi} \cdot \boldsymbol{C}}$	
3V-NEHRP	0.064	0.628	0.068	3.37	1.06	6.04	**<< 1%**
3V-PBPD	0.078	0.766	0.015	1.56	1.06	0.41	**> 99.9%**
6V-NEHRP	0.065	0.709	0.035	1.93	1.06	1.55	**23.3%**
6V-PBPD	0.100	0.730	0.027	2.12	1.06	0.82	**86.2%**

*$\emptyset$: resistance factor that accounts for the randomness and uncertainty in estimation of structural capacity
*γ: demand uncertainty factor; * γ_a: analysis uncertainty factor

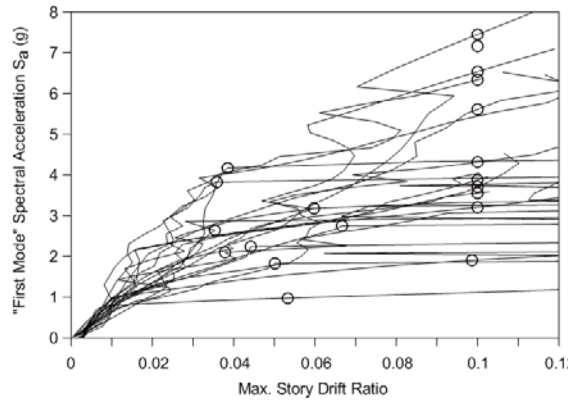


(a) (b)

Figure 9 IDA curves for (a) 3V-NEHRP and (b) 3V-PBPD frames under 2%/50yrs SAC ground motions



(a)



(b)

Figure 10 IDA curves for (a) 6V-NEHRP and (b) 6V-PBPD frames under 2%/50yrs SAC ground motions

4. CONCLUSIONS

Reliability-based evaluation by using the FEMA 351 procedure for SMFs, which accounts for randomness and uncertainty in the estimation of seismic demand and drift capacity, showed that steel concentrically braced frames (CBFs) designed by the performance-based plastic design (PBPD) method can have dramatically higher confidence levels against global collapse than those of CBFs designed in current practice. Also, those confidence levels can be similar to the target confidence levels for SMFs in current practice, *i.e.*, 90% or above.

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