Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 1131

FURTHER DEVELOPMENT OF PERFORMANCE-BASED PLASTIC DESIGN METHOD FOR CONCENTERICALLY BRACED FRAMES

M. R. Bayat[1], S-H. Chao[2], S. C. Goel[3]

ABSTRACT

As part of previous studies, Performance-Based Plastic Design (PBPD) was applied to 3-story and 6-story Concentrically Braced Frames (CBF) with buckling type braces which exhibit "pinched" hysteretic loops under cyclic displacements. In this paper, some refinements in the design method are presented, which include: An alternative method to account for pinched hysteretic behavior in calculation of design base shear, a procedure to include effect of axial column deformation on the yield and target drifts which are needed at the initial stages of design. This estimation of yield and target drifts is of particular importance in design base shear calculation for tall CBF. The above modifications are then applied to the design of a mid-rise 9 story example frame. The results of inelastic dynamic analyses showed excellent performance under DBE as well as MCE hazard level ground motions.

Introduction

Concentrically braced frames (CBF) are efficient and economical seismic lateral force resisting systems. Based on research performed during the last twenty years or so, current seismic codes (AISC 2005) now include provisions to design ductile concentrically braced frames, called Special Concentrically Braced Frames (SCBF). However, when designed by conventional elastic methods, these structures can undergo excessive story drifts after buckling of bracing members. This can lead to early fractures of the bracing members, especially in those made of rectangular tube sections (Sabelli 2000, Uriz 2005).

In earlier study by the authors the PBPD method was applied to low-rise CBF with buckling type braces which exhibit "pinched" hysteretic loops under cyclic displacements (Goel and Chao 2008; Chao, Bayat and Goel 2008). The design concept, which was originally developed and successfully applied to moment frames, uses pre-selected target drifts and yield mechanisms as performance objectives. The design lateral forces are derived by using a modified energy equation to account for pinched hysteretic behavior of the braces.

1- Graduate Research Assistant, Dept. of Civil Engineering, Univ. of Texas at Arlington, email: mrezabayat@gmail.com
2- Assistant Professor, Dept. of Civil Engineering, Univ. of Texas at Arlington, email: shchao@uta.edu
3- Emeritus Professor, Dept. of Civil and Env. Engineering, Univ. of Michigan, Ann Arbor, email: subhash@umich.edu

Some refinements in the design method are presented in this paper, which mainly include: An alternative method to account for pinched hysteretic behavior in calculation of design base shear, a procedure to include effect of axial column deformation on the yield and target drifts which are needed at the initial stages of design. More realistic estimate of yield and target drifts is of particular importance in design base shear calculation for taller CBF. The above modifications are then applied to the design of a mid-rise 9-story example frame. In addition, the performances of two 9-story CBFs designed by using different lateral distributions have been studied. The results of inelastic dynamic analyses show excellent performance under DBE as well as MCE hazard level ground motions.

C_2 Factor Method for Design Base Shear

Determination of the design base shear for given hazard level is a key element in the PBPD method. It is calculated by equating the work needed to push the structure monotonically up to the target drift to that required by an equivalent elastic-plastic single-degree-of-freedom (EP-SDOF) system to achieve the same state. Assuming an idealized E-P force-deformation behavior of the system, the work-energy equation can be written as:

$$(E_e + E_p) = \gamma\left(\frac{1}{2}MS_v^{\,2}\right) = \frac{1}{2}\gamma M\left(\frac{T}{2\pi}S_a g\right)^2 \tag{1}$$

where E_e and E_p are, respectively, the elastic and plastic components of the energy (work) needed to push the structure up to the target drift. S_v is the design pseudo-spectral velocity; S_a is the pseudo spectral acceleration; T is the natural period; and M is the total mass of the system. The energy modification factor, γ, depends on the structural ductility factor (μ_s) and the ductility reduction factor (R_μ), and can be obtained by the following relationship:

$$\gamma = \frac{2\mu_s - 1}{R_\mu^{\,2}} \tag{2}$$

The solution of Eq. 1 gives the required design base shear coefficient, V_y/W:

$$\frac{V_y}{W} = \frac{-\alpha + \sqrt{\alpha^2 + 4\gamma S_a^2}}{2} \tag{3}$$

where α is a dimensionless parameter given by:

$$\alpha = \left(h^* \cdot \frac{\theta_p 8\pi^2}{T^2 g}\right) \tag{4}$$

The term θ_p represents plastic component of the target drift ratio, and $h^* = \sum_{i=1}^{N}(\lambda_i h_i)$.

The calculation of the PBPD design base shear is also based on a lateral force distribution proposed by Chao, Goel, and Lee (2007), which can be expressed as $F_i = C'_{vi}V$, where:

$$C'_{vi} = (\beta_i - \beta_{i+1})\left(w_n h_n \Big/ \sum_{j=1}^{n} w_j h_j \right)^{kT^{-0.2}} \tag{5}$$

and $$\beta_i = V_i / V_n = \left(\sum_{j=1}^{n} w_j h_j \Big/ w_n h_n \right)^{kT^{-0.2}} \tag{6}$$

In the above equations, β_i represents the shear distribution factor at level i; V_i and Vn, respectively, are the story shear forces at level i and at the top (nth) level; w_j is the seismic weight at level j; h_j is the height of level j from the base; F_i is the lateral force at level i; and V is the total design base shear. The value of factor k in the exponent term was taken equal to 0.75.

As mentioned earlier, Eq. 3 for V_y was derived by assuming ideal elastic-plastic (E-P) force-deformation behavior and "full" hysteretic loops for the system. That is characteristic of a number of ductile steel framing systems, such as MF, EBF, STMF, and BRBF. For systems that do not posses such hysteretic property, such as steel braced frames with buckling type braces or RC frames, some modification is warranted. Two approaches have been tried which show good promise.

1) The energy capacity term, represented by the left hand side of Eq. 1, can be modified by a factor η to account for the reduced area of typical hysteretic loops as a fraction of the corresponding "full" loops (Chao and Goel 2006). Thus, Eq. 1 and 3 can be modified as Eq. 7 and 8, respectively:

$$\eta\left(E_e + E_p\right) = \gamma\left(\frac{1}{2} M S_v^{\,2}\right) = \frac{1}{2}\gamma M \cdot \left(\frac{T}{2\pi} S_a g\right)^2 \tag{7}$$

$$\frac{V_y}{W} = \frac{-\alpha + \sqrt{\alpha^2 + 4(\gamma/\eta) S_a^2}}{2} \tag{8}$$

2) The second approach, as used in this study, is based on consideration of the effect of degrading hysteretic behavior on peak (target) displacement. Investigators have studied the effect of degrading hysteretic behavior of single-degree-of-freedom systems on resulting peak displacements. The results show that the peak displacements are larger than those of systems with non-degrading hysteretic behavior in the short period range, but are about equal for longer periods. Approximate expressions have been proposed for modification factors to account for this effect, e.g., factor C_2 in FEMA 356 (FEMA 2000), as shown in Figure 1. Thus, the effective target design drift for a given structural system with degrading hysteretic behavior can be divided by the C_2 factor which would give design target drift for an equivalent non-degrading system. The C_2 factor used herein is based on the studies by Ruiz-Garcia and Miranda (2005) and is slightly different from the values given in FEMA 440. The approximate C_2 factor used in this study is shown in Figure 1. The design base shear can then be calculated by using this modified target drift and Eq. 3.

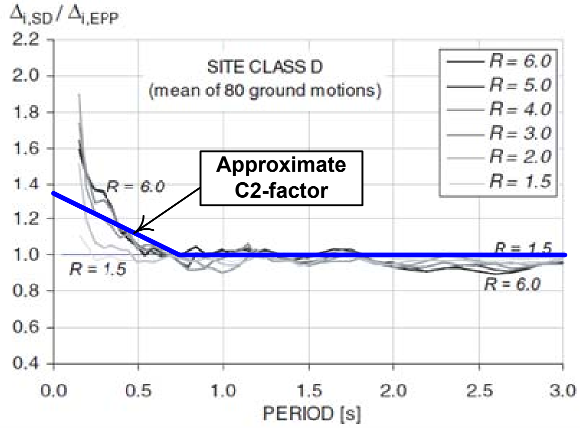


Figure 1. Mean displacement ratio of SD (Stiffness Degrading) to EPP models with ground motions recorded on site class D (from Ruiz-Garcia and Miranda 2005) and the approximate C_2-factor used in the current study.

Effect of Column Axial Deformation on Yield and Target Drift

Yield and target drift are main parameters used in the PBPD method for calculation of design base shear. In case of CBF, and especially for tall frames, the drift can be significantly increased due to column axial deformations over that produced by brace elongation and shortening, as shown in Figure 2.

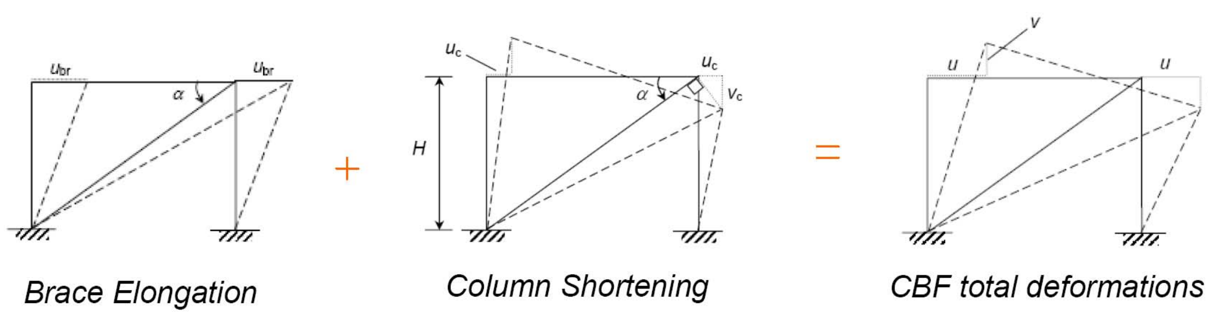


Figure 2. Different components of lateral drift in a braced frame, (a) Shear mode of deformation, (b) Flexural mode of deformation, and (c) Total deformations (from Calvi et al. 2006).

Thus, the total story drift in CBF (and other braced frames) can be obtained by adding shear component (due to brace deformation) and flexural component (due to column axial deformation), (Englekirk 1994). The shear component of the yield drift (YD) can be obtained as:

$$YD_{shear} = {2\varepsilon_y}/{\sin 2\alpha} \qquad (9)$$

where ε_y represents the yield strain of the brace material and α the angle of inclination of the brace with the horizontal. The following approximate expression can be derived for the additional drift due to column axial deformation:

$$YD_{flex} = \frac{\sigma_{avg}}{E} \times \frac{H}{L} = \frac{0.42\sigma_y}{E} \times \frac{H}{L} = \frac{0.42 \times 50}{29000} \times \frac{H}{L} = 0.000724 \frac{H}{L} \quad (10)$$

where σ_{avg} is the approximate average axial strain in columns and H is the total height of the frame.

Total yield drifts were calculated for four example frames, as shown in Figure 3, and presented in Table 1. The height, H_r, which is basically the total height minus the height of top story, is used in yield drift calculations since the axial force in the columns of the top story would be rather small in the case of an inverted-V brace configuration. The configurations of these frames are based on the SAC building models (Gupta and Krawinkler 1999). A typical floor plan for the 9-story building is shown in Figure 5. The lateral load resisting system in the original SAC buildings were MF, but Sabelli (2000) and more recently Richard (2008) have used similar building configurations for CBF structures. It can be noted that the estimated yield drifts are quite close to those obtained from the pushover analyses (e.g. Richards 2008). Since the columns in PBPD method are designed to remain elastic, the flexural yield drift (due to elastic column axial deformations) can be added to the basic target drift to obtain the modified target drift.

Table 1. Yield Drift for Example CBFs.

CBF Frame	H_r **(ft)**	L **(ft)**	α *(deg)*	YD_{flex}	YD_{shear}	YD_{total}	$YD_{pushover}$
3-Story	25	30	40.9	0.055%	0.317%	0.37%	0.35-0.4%
6-Story	70	30	40.9	0.155%	0.317%	0.47%	0.48%
9-Story	109	30	40.9	0.242%	0.317%	0.56%	0.5-0.6%
18-Story	226	20	52.4	0.753%	0.328%	1.08%	1.0-1.2%

The above procedure was used to obtain the required PBPD design base shears for four example CBFs (Tables 2 and 3). Two different hazard levels were considered to determine which one would govern the design.

As can be seen from the design base shear values in these tables, the MCE hazard level base shear governs for all cases, and should therefore be used if a dual hazard level performance objective is expected. A comparison between the obtained PBPD design base shears and the ASCE 7-05 code (SEI 2005) values as a function of T is shown in Figure 4. As can be seen, the DBE base shears are larger than the code values for short periods, but are almost the same for longer periods. The MCE base shears are much larger than the code values for shorter periods and slightly larger for longer periods.

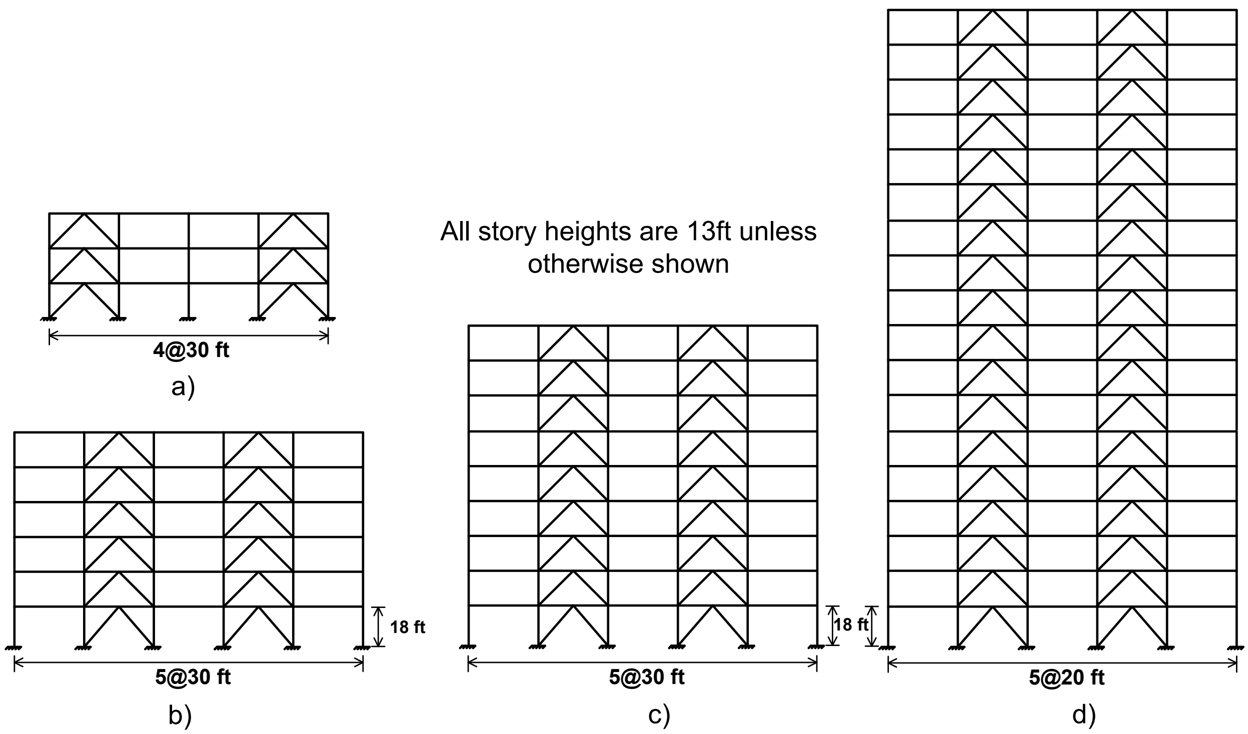


Figure 3. Elevation views of four example CBFs: a) 3-story; b) 6-story; c) 9-Story; d) 18-story.

Table 2. PBPD design base shear under DBE (2/3MCE) hazard level.

CBF	***H (ft)***	***L (ft)***	***α (deg)***	$Y.D._{total}$	TD_{basic}	TD_{mod}	**C2**	**PBPD V/W**
3-Story	39	30	40.9	0.37%	1.25%	1.31%	1.2	**0.336**
6-Story	83	30	40.9	0.47%	1.25%	1.41%	1.1	**0.281**
9-Story	122	30	40.9	0.56%	1.25%	1.49%	1.0	**0.166**
18-Story	239	20	52.4	1.08%	1.25%	2.00%	1.0	**0.111**

Table 3. PBPD design base shear under MCE hazard level.

CBF	***H (ft)***	***L (ft)***	***α (deg)***	$Y.D._{total}$	TD_{basic}	TD_{mod}	**C2**	**PBPD V/W**
3-Story	39	30	40.9	0.37%	1.75%	1.81%	1.2	**0.48**
6-Story	83	30	40.9	0.47%	1.75%	1.91%	1.1	**0.322**
9-Story	122	30	40.9	0.56%	1.75%	1.99%	1.0	**0.195**
18-Story	239	20	52.4	1.08%	1.75%	2.50%	1.0	**0.141**

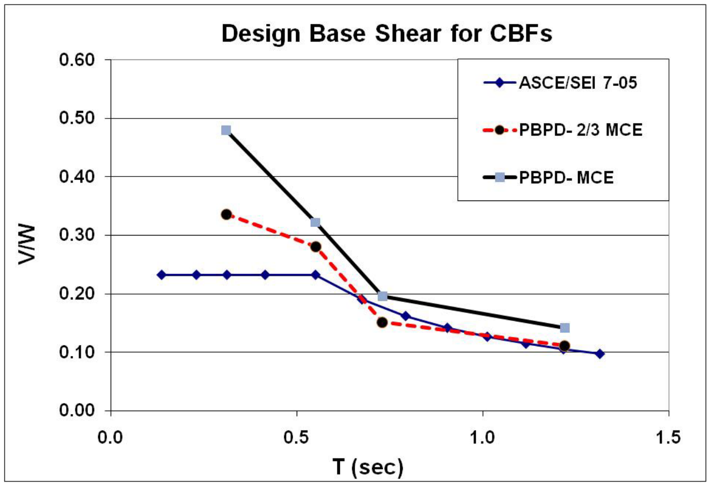


Figure 4. Comparison of PBPD design base shears with current code values.

Example 9-Story CBF

The 9-story SAC model building was designed as a CBF structure by using the above outlined modifications in the PBPD method. The plan view is shown in Figure 5, other details can be found in the reference reports (Gupta and Krawikler 1999, and Sabelli 2000). The calculated design base shear for the two hazard levels is shown in Tables 2 and 3. SAC ground motions of 2/3 MCE and MCE hazard levels for Los Angeles site (Somerville et al., 1997) were used for dynamic analyses. The results of these non-linear dynamic analyses, using SNAP-2DX program (Rai et al. 1996), are shown in Figures 6, 7, and 8. It should be mentioned that the frame was designed by using two different lateral force distributions corresponding to the k value of 0.75 and 0.50 in Eq. 5 and 6. It can be seen that by using $k = 0.50$ as the lateral distribution parameter, the upper story drifts become quite a bit smaller as compared to the case of $k = 0.75$. In addition, the story drift profile matches much better with the target drift limit for the case of $k = 0.50$ and also tends to be more uniform along the height.

Summary and Conclusion

Some refinements in the PBPD method for CBF were presented, including: An alternative method to account for pinched hysteretic behavior in calculation of design base shear, a procedure to consider effect of column axial deformations on the yield and target drifts. This estimation of yield and target drifts is of particular importance in design base shear calculation for tall CBFs. The above modifications were then applied to the design of a mid-rise 9 story example frame. In addition, the effect of using a slightly different lateral force distribution in design of this braced frame was also studied. The results of inelastic dynamic analyses showed excellent performance under DBE as well as MCE hazard level ground motions.

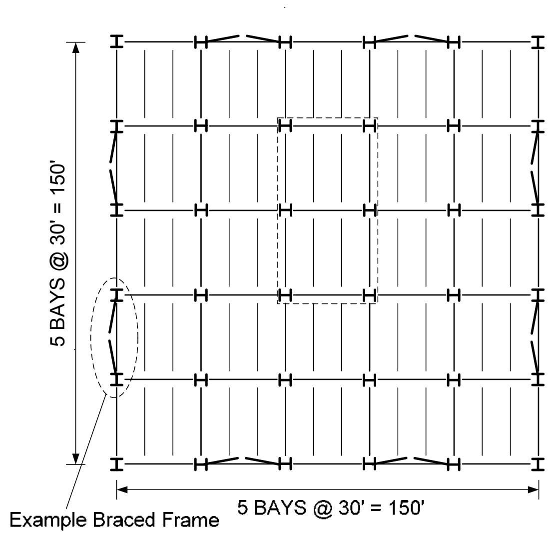
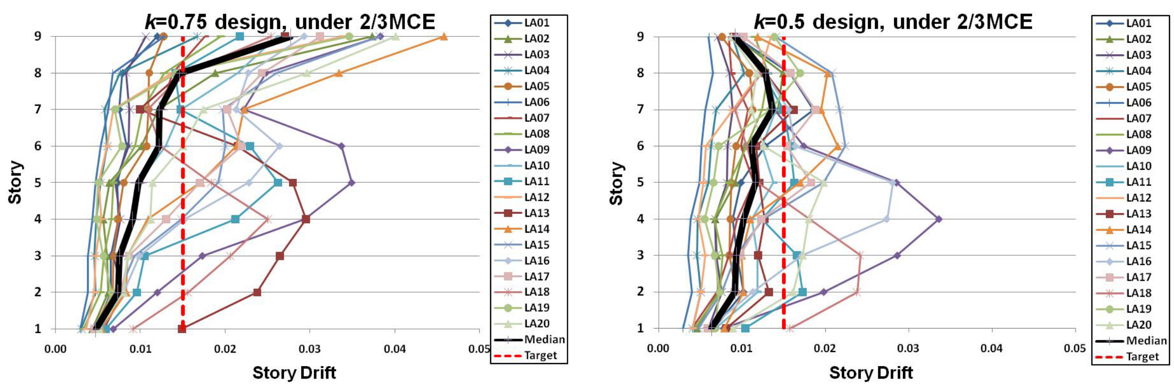
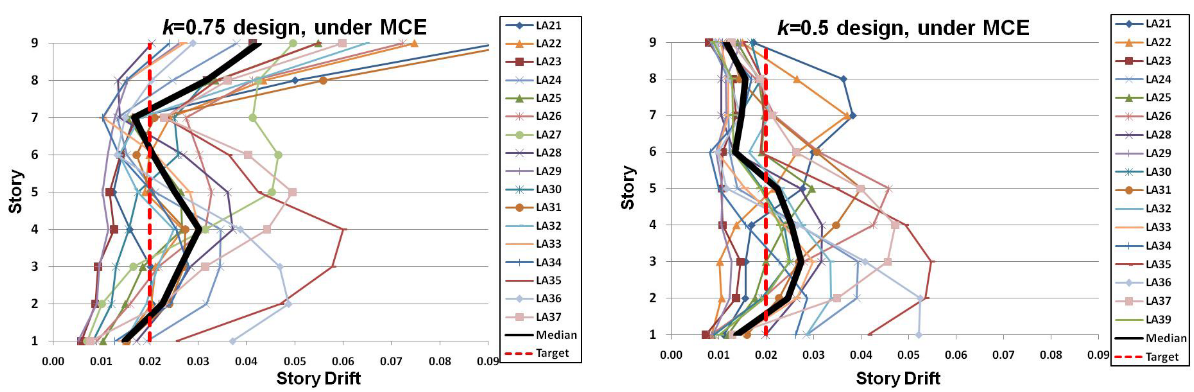


Figure 5. Plan view of 9-story SAC building.



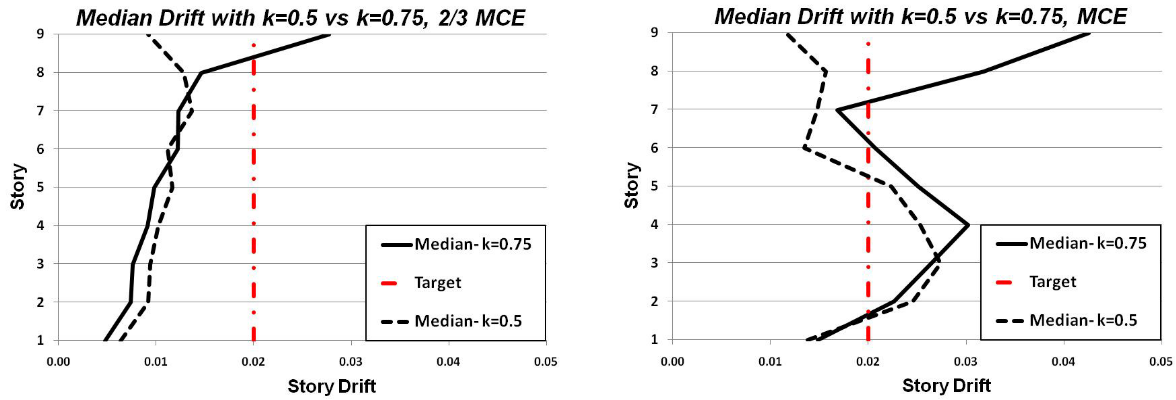
a) b)

Figure 6. Story drifts for 9-story CBF under *2/3 MCE* level SAC ground motions, designed with (a) $k = 0.75$, and (b) $k = 0.5$ for the lateral force distribution.



a) b)

Figure 7. Story drifts for 9-story CBF under *MCE* level SAC ground motions, designed with (a) $k = 0.75$, and (b) $k = 0.5$ for the lateral force distribution.



a) b)

Figure 8. Comparison of median Story drifts for 9-story CBF under SAC ground motions; (a) under *2/3 MCE*, and (b) under *MCE* hazard level.

Acknowledgement

The research work reported in this paper was primarily supported by the young faculty start-up fund at the University of Texas at Arlington and in part by grants from the American Institute of Steel Construction and Nucor Research & Development. Any opinions, findings and conclusions are solely those of the authors.

References

AISC. 2005. ANSI/AISC 341-05: Seismic Provisions for Structural Steel Buildings. American Institute of Steel Construction, Chicago, IL.

ASCE. 2005. SEI/ASCE 7-05: *Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Engineers, Reston, VA.

ATC. 2005. FEMA 440: *Improvement of nonlinear static seismic analysis procedures*. Applied Technology Council, Redwood City, California and Federal Emergency Management Agency, Washington, D. C.

Calvi, G. M., Priestley, M. J. N., Sullivan, T. J., and Happold, B. 2006. Development of Displacement-Based Design and Assessment Methods; Progress Review: Year 1. *Presentation, European Centre for Training and Research in Earthquake Engineering (EUCENTRE), Nov 2006. (url: http://www.reluis.it).*

Chao, S-H., and Goel, S. C., 2006. A seismic design method for steel concentric braced frames for enhanced performance, *4th International Conference on Earthquake Engineering, Taipei, Taiwan.*

Chao, S.-H., and Goel, S.C., Lee, S.-S., 2007. A Seismic design lateral force distribution based on inelastic state of structures. *Earthquake Spectra* 23 (3), 547-569.

Chao, S.-H., Bayat, M. R., and Goel, S.C., 2008. Performance-Based Plastic Design of steel concentric braced frames for enhanced confidence level, *14th World Conference on Earthquake Engineering, Beijing, China* .

Englekirk, R. 1994. *Steel Structures: Controlling Behavior Through Design.* John Wiley & Sons, Inc.

Federal Emergency Management Agency. 2000. FEMA 356: *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*. Federal Emergency Management Agency, Washington, D. C.

Goel, S.C. and Chao, S-H., 2008. *Performance-Based Plastic Design: Earthquake Resistant Steel Structures*, International Code Council.

Gupta, A. and Krawinkler, H., 1999. Seismic Demands for Performance Evaluation Steel Moment Resisting Frame Structures, *Report No. 132*, The John A. Blume Earthquake Engineering Center, Dept. of Civil and Env Eng., Stanford University, Stanford, CA.

Rai, D.C., Goel, S. C. and Firmansjah, J., 1996. SNAP-2DX: General Purpose Computer Program for Static and Dynamic Nonlinear Analysis of Two Dimensional Structures, *Report No. UMCEE 96-21*, Dept of Civil and Environmental Eng., University of Michigan, Ann Arbor, MI.

Richards, P. 2008. Seismic column demands in ductile frames. *Journal of Structural Engineering*, 135 (1), 33-41.

Ruiz-Garcia, J., Miranda, E., Performance-based assessment of existing structures accounting for residual displacements, *Report TR-153*, The John A. Blume Earthquake Engineering Center, Stanford University, Stanford, CA.

Sabelli, R., 2000. FEMA/EERI Report: Research on improving the design and analysis of earthquake resistant steel braced frames, FEMA/EERI, Washington D. C.

Somerville, P. G., Smith, M., Punyamurthula, S., and Sun, J., 1997. Development of ground motion time histories for phase 2 of the FEMA/SAC Steel Project, *Report No. SAC/BD-97/04*, SAC Joint Venture, Sacramento, CA.

Uriz, P. 2005. Towards earthquake resistant design of concentrically braced steel structures, *Ph.D. Thesis*, University of California, Berkeley.