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Performance-Based Seismic Design Method for Steel Concentric Braced Frames

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Abstract: This study presents a Performance-Based Plastic Design (PBPD) methodology for the design of steel Concentric Braced Frames (CBF). The design base shear is obtained based on energy-work balance using pre-selected target drift and yield mechanism. The 3 low-to-medium rise CBF 3, 6 and 9-story were designed by the proposed methodology and current seismic codes. Results of inelastic dynamic analyses showed that the PBPD frames met all the intended performance objectives in terms of yield mechanisms and target drift levels but the code compliant frames showed very poor response due to premature brace fractures leading to unacceptably large drifts and instability.

Key words: Performance based plastic design, steel concentric brace frame, inelastic dynamic analyses, nonlinear dynamic analyses and energy-work balance equation, Iran

INTRODUCTION

Steel Concentrically Braced Frames (CBFs) are very efficient steel structures that are commonly used to resist forces due to wind or earthquakes because they provide complete truss action. Based on research performed during the last 20 years or so, current seismic codes now include provisions to design ductile concentrically braced frames called Special Concentrically Braced Frames (SCBFs) (AISC, 2005). Since, the seismic forces are assumed to be entirely resisted by means of truss action, the columns are designed based on axial load demand only and simple shear connections are used to join the beams and columns (AISC, 2005; MacRae *et al*., 2004; Richards, 2009). It has been estimated that CBFs comprise about 40% of the newly built commercial construction in California during the last decade (Broderick *et al*., 2008). This change in the newly designed steel structures can be attributed to the simpler design of CBFs and also their high efficiency in resisting lateral load with reduced deflections compared to other systems such as SMRFs, especially after the 1994 Northridge earthquake. However, CBFs are generally considered less ductile seismic-resistant structures than other systems due to the buckling or fracture of the bracing members under large cyclic deformations. These structures can undergo excessive story drifts after the buckling of bracing members when designed by conventional elastic design methods (Annan *et al*., 2009). This can lead to early fractures of the bracing members, especially in those that popular rectangular tube sections (HSSs). Nevertheless, structural and nonstructural damage observed in code compliant SCBF buildings due to undesired failure modes have shown the need to develop alternative methodologies to better ensure the desired performance. Since, SCBF system has been widely used as part of seismic force-resisting systems, design methodologies and systematic procedures are needed which require no or little iteration after initial design in order to meet the targeted design objectives. Performance-Based Plastic Design (PBPD) method has been recently developed to achieve enhanced performance of earthquake resistant steel structures (Leelataviwat *et al*., 1999; Goel and Chao, 2009; Sahoo and Chao, 2010; Liao and Goel, 2012). In this method, pre-selected target drift and yield mechanisms were used 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. Therefore in the PBPD method, control of drift and yielding is built into the design process from the very start, eliminating or minimizing the need for lengthy iterations to arrive at the final design. This study presents first time application of the PBPD approach to seismic resistant SCBF system. SCBF structures present special challenge because of their complex and degrading (pinched) hysteretic behavior. In order to account for the degrading hysteretic behaviour the FEMA 440; C_2 factor concept was used in developing the process of

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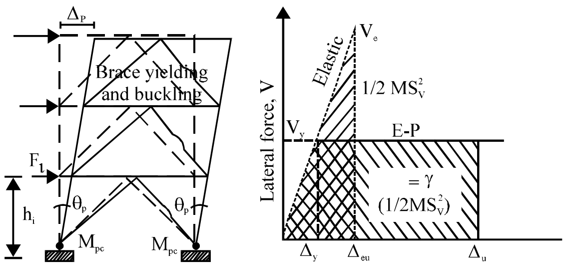

Fig. 1: PBPD concept

determining the design base shear for targeted drift and yield mechanism in the PBPD methodology (FEMA, 2006). The 3 baselines SCBF (3, 6 and 9-story) designed by conventional elastic design were selected for this study. Those frames were redesigned by the PBPD approach and also a fracture life criterion is employed for the HSS braces to prevent premature fracture. The baseline code designed frames and the PBPD frames were subjected to extensive time-history analyses. Results from analyses carried out on example frames designed by the PBPD method showed that the frames met all the desired performance objectives while the baseline frames showed very poor response due to premature brace fractures leading to unacceptably large drifts and instability.

Performance Based Plastic Design (PBPD): PBPD method uses pre-selected yield mechanism and target drift, as design criteria. These 2 design parameters are directly related to the distribution and degree of structural damage, respectively. In this method, the design base shear is determined using an energy-work balance concept where the energy needed to push an equivalent Elastic-Plastic Single-Degree of Freedom (E-P SDOF) system up to the target drift level is calculated, as a fraction of elastic input energy taken from the selected elastic design spectra (Fig. 1). Moreover in this study, a better representative distribution of lateral design forces is applied that is based on relative distribution of maximum story shears consistent with inelastic dynamic response results (Chao *et al*., 2007; Kharmale and Ghosh, 2012).

It also accounts for inelastic behavior and higher mode effects better than the distribution considered by the current codes. In order to achieve the intended yield mechanism and behavior, plastic design is then performed to detail the frame members and connections. Thus, 3 main components of the PBPD method are such as design base shear, lateral force distribution and plastic design. It should be mentioned that in this design method, the designer selects the target drifts according to acceptable damage and ductility and also selects a yield mechanism for desirable response and ease of post earthquake damage inspection and reparability and determines the design forces and frame member sizes for a given earthquake hazard (spectrum). Therefore, there is no need for factors, such as R, I, Cd, etc., as needed in the current design codes because those factors are based on a number of considerations including engineering judgment and plenty of debates already exist.

MATERIALS AND METHODS

Determination of design base shear in PBPD method: In current seismic codes, design base shears are shown by decreasing the elastic strength demands to the inelastic strength demands using the response modification factors (R-factor). Based on the importance of specific structures the inelastic strength demands are more increased using occupancy importance factor. In general, the design base shear is specified from the code prescribed design acceleration spectrum that is represented as follow:

$$V = C_e\left(\frac{I}{R}\right)W \quad (1)$$

Where:

- C_e = The normalized design pseudo-acceleration
- I = The occupancy importance factor
- R = The response modification factor
- W = The total seismic weight

After finding the member sizes for needed strengths (that is basically performed by elastic analysis) the calculated drift using elastic analysis is multiplied by deflection amplification factor, such as C_d given in the codes and kept within specified drift limits (in the order of 2%). It is mentioned that the R-factor, shown in design codes for various structural systems are specified basically according to engineering judgment. Furthermore as pointed out earlier, the conventional design procedures in the codes are according to the elastic analysis approaches rather than displacement-based approaches, therefore the inelastic response beyond the elastic limit for a structure cannot be predicted precisely (Abdollahzadeh *et al*., 2013). A more rational design approach to overcome the shortcomings in the conventional approach was represented by Leelataviwat and modified by Lee and Goel (2001) that uses energy balance equation as the design basis with the structure pushed monotonically up to a target drift after the formation of a selected yield mechanism, as it is shown in Fig. 1. The required amount of external work to do that is calculated as a factor γ times the elastic input energy $E = 1/2MS_v^2$. The modification factor γ is based on the natural period of the structure that has significant influence on the earthquake input energy, as found out by many researchers (Uang and Bertero, 1988; Lee and Goel, 2001). Thus, the modified energy balances equation as follow:

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

Where:

- E_e and E_p = The elastic and plastic components of the energy needed, as the structure is pushed up to the target drift
- M = The total mass of the system
- S_v = The design pseudo-velocity

Using the geometric relationship between the 2 areas representing work and energy in Fig. 1, Eq. 2 can be written as:

$$\frac{1}{2}.V_y.\left(2\Delta_u - \Delta_y\right) = \gamma.\left(\frac{1}{2}.V_e.\Delta_{eu}\right) \tag{3}$$

Equation 3 can be further reduced into the following form:

$$\gamma\frac{\Delta_{eu}}{\Delta_y} = \frac{\left(2\Delta_u - \Delta_y\right)}{\Delta_{eu}} \tag{4}$$

The energy modification factor γ, depends on the structural ductility factor (μ_s) and the ductility reduction factor (R_μ) which is related to the period of the structure and can be as follow:

$$\gamma = \left(\frac{2\mu_s - 1}{R_\mu^2}\right) \tag{5}$$

Based on Eq. 5, γ is a function of the ductility reduction factor and the structural ductility factor. By using different methods, many researchers have studied the relationship between ductility reduction factor (R_μ) and structural ductility factor (μ_s) (Newmark and Hall, 1982; Miranda and Bertero, 1994). In this study, the method by Newmark and Hall because of its simplicity is used herein to relate the ductility reduction factor and the structural ductility factor for EP-SDOF, as shown in Table 1. Also, it should be noted that the other inelastic spectra proposed by some researcher, such as Miranda and bertero can be used.

The work-energy equation can be re-written in the following form:

Table 1: Ductility reduction factor and corresponding structural period range

Period range	Ductility reduction factor
$0 \le T < \frac{T_1}{10}$	$R_\mu = 1$
$\frac{T_1}{10} \le T < \frac{T_1}{4}$	$R_\mu = \sqrt{2\mu_s - 1}.\left(\frac{T_1}{4T}\right)^{2.513\,\mathrm{Log}\left(\frac{1}{\sqrt{2\mu_s - 1}}\right)}$
$\frac{T_1}{4} \le T < T_1'$	$R_\mu = \sqrt{2\mu_s - 1}$
$T_1' \le T < T_1$	$R_\mu = \frac{T\mu_s}{T_1}$
$T_1 \le T$	$R_\mu = \mu_s$

$T_1 = 0.57$ sec; $T_1' = T_1.\ (\sqrt{}2\mu\text{-}1/\mu_s)$ sec

$$\frac{1}{2}\frac{W}{g}.\left(\frac{T}{2\pi}\frac{V_y}{W}g\right)^2 + V_y\left(\sum_{i=1}^{n}\lambda_i h_i\right)$$
$$\theta_p = \frac{1}{2}\gamma\left(\frac{W}{g}\right).\left(\frac{T}{2\pi}S_a g\right) \tag{6}$$

Or:

$$\left(\frac{W}{V}\right)^2 + \frac{V_y}{W}\left(h^*.\frac{\theta_p 8\pi^2}{T^2 g}\right)\theta_p - \gamma S_a^2 = 0 \tag{7}$$

Where, V_y, λ_i and θ_p present the yielding base shear (can be also used as the design base shear), shear distribution factor for each floor i and the global inelastic drift ratio of the structure, respectively. The admissible solution of Eq. 7 gives the required design base shear coefficient, V_y/W:

$$\frac{V}{W} = \frac{-\alpha + \sqrt{\left(\alpha^2 + 4\gamma S_a^2\right)}}{2} \tag{8}$$

Where, α is a dimensionless parameter given by:

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

And:

$$h^* = \Sigma_{i=1}^{N}\lambda_i h_i \tag{9}$$

Special considerations for CBF in PBPD method: It is expected that the response of a degrading Single Degree of Freedom (SDOF) will be different from that of an equivalent Elastic-Plastic (EP) system under the same earthquake ground motion. The degrading behavior, although normally caused by the behavior of components can be expected in the system behavior as well. The degrading behavior can be Strength Degradation (STRD), Stiffness Degradation (SD) or pinched hysteretic behavior. Concentric brace frame structures present special challenge due to their complex and degrading (pinched) hysteretic behavior. Since, the original PBPD

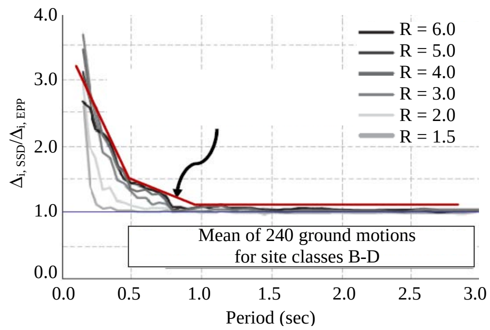

Fig. 2: Mean displacement ratio of SSD to EPP models computed with ground motions recorded on site classes B-D (FEMA, 2006). Mean plus standard deviation value is considered as C_2 factor in this study

approach was developed for Moment Frames (MFs) and the stiffness and strength of the braces in CBFs decrease under cyclic compression, using the same design base shear as MFs would not be appropriate. Therefore in this study, the design base shear for CBFs is determined by two modifications to account for P-delta effect and pinched hysteretic behavior. Since, development of the PBPD method for CBF structures is currently in progress, results from the study so far have been most promising. The design base shear was determined for 2 level performance criteria: A maximum story drift ratio ($\theta_u = 2\%$) for a ground motion hazard with a 2% probability of exceedance in 50 years (Maximum Credible Earthquake (MCE)) and a maximum story drift ratio ($\theta_u = 1.5\%$) with a 10% probability of exceedance in 50 years (10/50 and 2/3 MCE).

Pinched hysteretic behaviour: Many researchers have investigated the effect of degrading hysteretic behavior of Single Degree of Freedom Systems (SDOF) on resulting peak displacements. The results show that the peak displacements of systems with degrading hysteretic behavior are larger than those of non-degrading hysteretic behavior in the short period range but are about equal for longer periods. Therefore to explain this effect, approximate expressions have been suggested for modification for example, C_2 factor in FEMA 440, as seen in Fig. 2 (FEMA, 2006). This factor represents the effects of strength deterioration, stiffness deterioration and pinched hysteresis shape on maximum displacement response. Thus for a given structural system with degrading hysteretic behavior, the target design drift can be divided by the C_2 factor that would give design target drift for an equivalent non-degrading system. The design base shear can then be calculated by using this modified target drift. Also, it should be noted that in this study the mean-plus standard deviation is considered, as C_2 factor to obtain more significant results.

Table 2: Values of C_2 factor as function of R and T

R = 2~6	C_2 factors
T<0.2	3.15
0.2≤T<0.5	3.15-5.5(T-0.2)
0.5≤T<1.0	1.5-0.8 (T-0.5)
1.0≤T	1.1

The equations of simplified linear regression trend line of C_2 for different force reduction factor, R are summarized in Table 2.

After determining the value of C_2, the modified target design drift θ_u, ductility μ_S, ductility reduction factor R_μ and energy modification factor γ can be calculated as follows:

$$\theta_u^* = \frac{\theta_T}{C_2} \tag{10}$$

$$\mu_s^* = \frac{\theta_u^*}{\theta_y} = \frac{\theta_u}{\theta_y C_2} = \frac{\mu_s}{C_2} \Rightarrow \text{get } R_\mu^* \tag{11}$$

$$\gamma^* = \frac{2\mu_s^* - 1}{R_\mu^{*2}} \tag{12}$$

The design base shear can then be calculated by using this modified energy modification factor γ^* and Eq. 6 and 7.

Considering P-Δ effect in the lateral forces: Due to pinching hysteresis behavior of bracing system, it is important that P-Δ forces considered in the design of yielding members (braces). It was accomplished by adding P-Δ lateral force, $F_{i\text{-}PD}$ to the basic design force, F_i as seen in Fig. 3. The force $F_{i\text{-}PD}$ is determined equal to $P_i\theta_u$ where P_i represents the gravity load at story level i and θ_u presents the target design drift ratio that is considered constant for design purpose.

Overall design procedure in PBPD method: Plastic method is used to design the structural members that are expected to dissipate the earthquake energy inelastically (Designated Yielding Members (DYM)). For Concentrically Brace Frames (CBF) as seen Fig. 1, the plastic hinges may be confined to form only at the brace and column bases while keeping the vertical distribution of lateral strength of the structure close to the distribution of design story shear distribution. Previous studies have shown that it is desirable to have the distribution of structural strength along the building height follow the distribution of design story shears that was obtained and

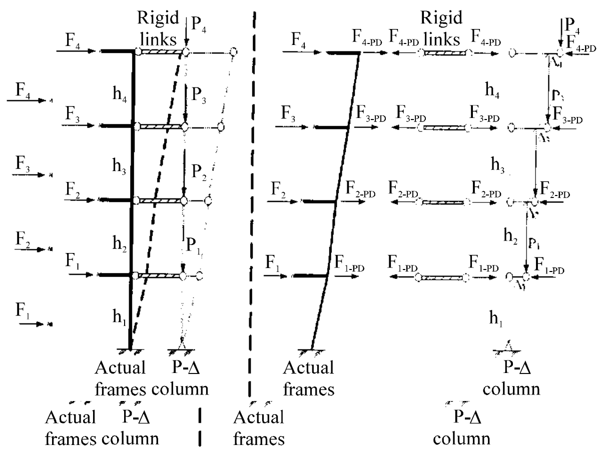

Fig. 3: Additional lateral forces due to P-Δ effect

calibrated by nonlinear dynamic time-history analysis results. This helps to distribute the yielding more evenly along the height, thereby preventing yielding from concentrating at a few levels. In order to plastic design, only basic knowledge is needed for design of members. Also to design members that should be remained elastic (non-DYM), such as columns, a capacity design approach by accounting for the strain-hardening and material overstrength of the DYM, as well as including the frame deformation (P-delta) effects as appropriate are required.

Design of bracing members: The 3 criteria are used in PBPD approach for design of bracing members, as described in the following. These are strength, fracture life and compactness criteria.

Strength criterion: In order to minimize the possibility of concentration of inelastic deformation in one or few stories, it is desirable to have the distribution of bracing member strength along the building height closely follow the distribution of design story shear. To resist the total design story shear, the braces are designed based on their ultimate state (plastic design), i.e., post-buckling and tension yielding, thus:

$$\left(V_{\text{story shear}}\right)_i \leq \left(\phi_t P_y + 0.5\phi_c P_{cr}\right)_i \cos\alpha_i \qquad (13)$$

Where:

$V_{\text{story shear}}$ = The story shear at level i for an equivalent one-bay frame

P_y and P_{cr} = The nominal axial tensile and compressive strength of bracing members

ϕ_t = ϕ_c = 0.9 (AISC, 2005)

α = The angle of bracing members with the horizontal (Fig. 4)

The design is carried out by assuming that both bracing members reach their ultimate inelastic strength. Note that for braces buckling in-plane, the post-buckling strength is taken as 0.5 P_{cr} but for braces buckling out-of-plane, 0.3 P_{cr} is used. In order to ensure in-plane buckling, braces are selected such that $K_x L/r_x > K_y L/r_y$.

Fracture criterion: According to previous studies when CBF is subjected to strong earthquake ground motions, early brace fractures may lead to excessively large story drifts and ductility demand on beams and columns. A fracture criterion for HSS braces is used in PBPD method for CBFs to prevent premature fracture. The brace fracture life, Nf is estimated by the following empirical equation which was derived from test results of HSS braces under cycling loading (Tang and Goel, 1989):

$$N_f = \left\{ \begin{array}{ll} 262\dfrac{\left(\dfrac{b}{d}\right)\left(\dfrac{kl}{r}\right)}{\left\{\left[\dfrac{(b-2t)}{t}\right]\right\}^2} & \text{for } \dfrac{kl}{r} > 60 \\ 262\dfrac{\left(\dfrac{b}{d}\right)60}{\left\{\dfrac{b-2t}{t}\right\}^2} & \text{for } \dfrac{kl}{r} \leq 60 \end{array} \right\} \qquad (14)$$

Where:

N_f = The fracture life representing the number of standard cycles, beyond which an HSS brace will fracture

d = The gross depth of the section

b = The gross width of the section (b>d)

t = The wall thickness

b-2t/t = The clear width-thickness ratio of compression flanges (Shaback and Brown, 2003; Uriz, 2005)

KL/r = The slenderness ratio

A minimum N_f = 100 for HSS braces is suggested. It should be noted that current design practice does not consider the brace fracture life in an explicit manner.

Compactness criterion: For compactness criterion, the required compactness ratio for the braces is also checked, as it is specified by AISC Seismic Provisions (AISC, 2005). However, the compactness requirement is generally satisfied for HSS braces with a minimum N_f = 100.

Design of non-yielding members: The design of non-yielding members including beams and columns is performed based on the capacity design approach, i.e., non-yielding members should have a design strength to resist the combination of factored gravity loads and the forces due to braces in their ultimate state.

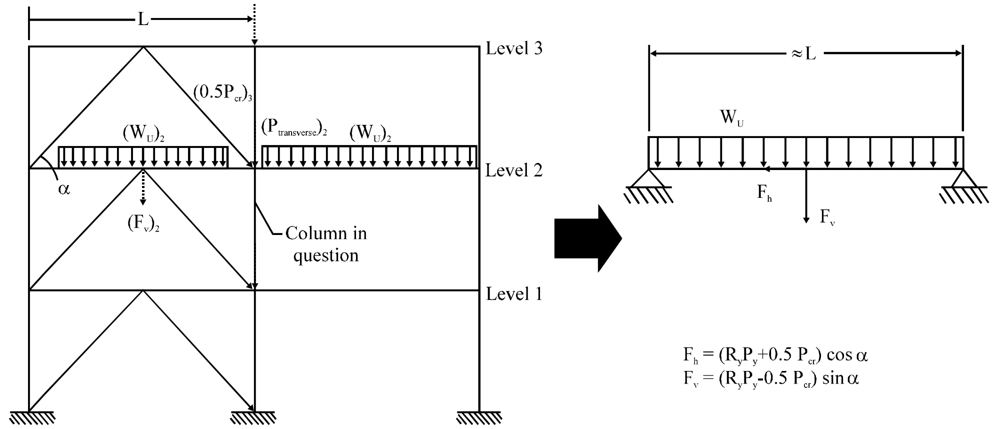

Fig. 4: Beam design for a chevron-type CBF

Design of beams: Design of beams should follow the criteria given in AISC Seismic Provisions (AISC, 2005). It should be noted that the post buckling strength of a brace is taken, as 0.5 P_{cr} for in-plane buckling. In addition, beams intersected by the braces should be designed assuming that no gravity loads are supported by the braces. A pin-supported beam model is used because shear splices are used at the ends. Due to the presence of high axial forces the design of beams should follow the beam-column design requirements. The unbalanced loads resulting from the braces are (Fig. 4):

$$F_h = \left(R_y P_y + 0.5P_{cr}\right)\cos\alpha \tag{15}$$

$$F_v = \left(R_y P_y + 0.5P_{cr}\right)\sin\alpha \tag{16}$$

Where:

F_h = The horizontal unbalanced force
R_y = The 1.3 is the ratio of the expected yield strength to the specified minimum yield strength and specified
P_y = F_yA_g is the nominal yield strength
P_{cr} = $F_{cr}A_g$ is the nominal compressive strength
F_{cr} = The axial critical stress

Design of columns: Due to the presence of beam shear splices, little or no moment is transferred into the columns. Thus only axial loads are considered for column design, including the fixed base 1st-story columns. Axial forces result primarily from the gravity loads and vertical component of brace forces. The 2 limit states are considered for the design of columns.

Pre-buckling limit state: Prior to brace buckling, no unbalanced force occurs in the beam and the design axial force in a typical exterior column is:

$$P_u = \left(P_{transverse}\right)_i + \Sigma\left(P_{beam}\right)_i + \left(P_{cr}\,\text{Sin}\,\alpha\right)_{i+1} \tag{17}$$

Where:

$(P_{transverse})_i$ = The tributary factored gravity load (1.2 DL+0.5 LL) on columns from the transverse direction at level i
$(P_{beam})_i$ = The tributary factored gravity load from the beam at level i (= 1/ 2 $(w_u)_i$ L)
$(P_{cr})_{i+1}$ = The buckling force of brace at i+1 level

Post-buckling limit state: An unbalanced force is created in the beam when a chevron type CBF reaches its ultimate state (Fig. 4) and for a typical exterior column, the axial force demand can be determined as follow:

$$\begin{aligned} P_u = {} & \left(P_{transverse}\right)_i + \Sigma\left(P_{beam}\right)_i + \\ & \left(0.5P_{cr}\,\text{Sin}\,\alpha\right)_{i+1} + \frac{1}{2}F_v \end{aligned} \tag{18}$$

Where:

F_v = The vertical unbalanced force
0.5 $(P_{cr})_{i+1}$ = The post-buckling force of brace at level (i+1)

Design of columns in medium high-rise buildings: The design axial force demand is then determined by the governing pre-buckling or post-buckling limit state. It is noted that the above approach assumes that all braces reach their limit states simultaneously. This is somewhat conservative for design of lower level columns, especially for high-rise buildings. Therefore, in this research for structures with periods larger than T≥0.7 sec, the probable maximum axial forces of columns are determined by Square Root of the Sum of Squares method (SRSS) (Redwood and Channagiri, 1991).

RESULTS AND DISCUSSION

Verification by nonlinear analysis

Frame designed by elastic method: The 3-6 and 9-story chevron-type CBFs were originally designed as SCBFs

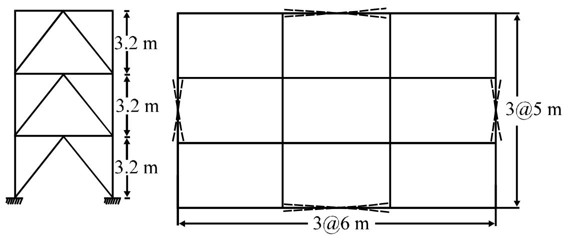

Fig. 5: Plan view of buildings

(baseline frame) according to 2005, BHRC design spectra and 2005, AISC seismic provisions (Redwood and Channagiri, 1991). The mentioned frames have similar stories in 3.2 m height and 6 m bay length. The braces were designed based on initial buckling strength ($2\varphi_c P_{cr} \cos\alpha$). The beams were designed based on the difference of nominal yield strength (R_y P_y) and post-buckling strength (0.3 P_{cr}, assuming out of plane buckling). Since, A36 steel was used for brace section, an over-strength factor of R_y = 1.3 was considered (AISC, 2005). The column design forces were based on gravity loading, the post-buckling strength of braces and the vertical unbalanced load on the beams from the braces. The baseline frames were then redesigned by the modified PBPD method by using the FEMA 440, C_2 factor approach and considering P-delta effect as discussed earlier. Typical floor plan is shown in Fig. 5 and important design parameters are given in Table 3. For response evaluation purposes, the PBPD frames and the baseline code compliant frames were subjected to nonlinear time-history analyses. To do nonlinear dynamic analyses, eleven earthquake records were selected, as shown in Table 4. Also, these records were modified such that their mean response spectrum matches the 2005 BHRC design spectrum (BHRC, 2005).

Table 3: Design parameters for PBPD frames

Design parameters	3-story	6-story	9-story
Sa (g)	0.960	0.81	0.670
T (sec)	0.310	0.55	0.750
C_2	2.545	1.45	1.300
Yield drift (%)	0.300	0.30	0.300
Target drift (%)	1.500	1.50	1.500
Modified target drift (%)	0.600	1.03	1.150
μ	2.000	3.70	4.300
R_μ	1.750	3.40	4.300
γ	1.000	0.55	0.425
α	2.850	2.88	2.950
V/W	0.250	0.14	0.085
V w/o PD (ton)	60.000	57.50	65.000
$\Sigma F_{i\text{-}PD}$ (ton)	4.000	8.00	12.000
Design base shear w/P-D	64.000	65.50	77.000

Table 4: Earthquake ground motion data

Earthquake	Years	PGA (g)	Moment magnitude	Duration (sec)
Kojour	2004	0.051	6.3	31.985
Tabas	1980	0.048	7.7	23.990
Kocaeli	1999	0.105	7.4	133.105
Landers	1992	0.115	7.3	49.940
Manjil	1990	0.184	7.4	60.400
Loma Prieta	1989	0.260	6.9	39.195
Imperial vally	1979	0.115	6.5	28.515
Zarand	2005	0.043	6.4	49.960
Cape	1983	0.153	6.4	43.940
Gulf	1994	0.095	6.7	59.980
Borrego	1996	0.130	6.9	39.970

Modeling the structure in software opensees: Nonlinear time-history analysis and modeling the structures were carried out by using the opensees software. This software is finite element software which has been specifically designed in performance systems of soil and structure under earthquake. In order to model the members in nonlinear range of deformation, following assumptions were assumed. All frame members, i.e., beams, columns and braces are considered as pin-ended. For the modeling of braces, nonlinear beam and columns element with the materials behaviour of $steel_{01}$ was used. Also to simulate the buckling behaviour of a brace under compression, an additional node at mid span of a brace and a small initial imperfection (L_{brace}/1000) is considered. Gravity columns were included in the modeling by using a lumped continues leaning column which was connected to the braced frame through rigid pin-ended links. The P-delta effect due to the gravity loads was also accounted for in the analysis.

Nonlinear analysis results and discussion: Nonlinear time-history analyses were carried out to evaluate the seismic performance of baseline and PBPD frames in terms of interstory drift ratio, residual drift ratio and yield mechanism. Interstory (or residual) drift ratio was defined as the ratio of the interstory (or residual) displacement to the corresponding story height (Fig. 6).

For clarity and brevity only the mean values of maximum interstory drifts and residual drift are shown. Sample time histories of story drift response (6-story frame) is shown in Fig. 6 for the Loma record. Each plot shows a response comparison between the baseline frame and PBPD frame. Figure 7 summarizes the maximum drift ratio for each of eleven ground motions. Both the baseline frame and PBPD frame are shown for a quick visual comparison. The maximum value of interstory drift ratio for 3-story baseline frame was 4.4% observed for Loma ground motion and for PBPD frame was 1.8% observed for Imperial record. The mean values of interstory drift ratios for the 3-story baseline frame and PBPD frame varied from 2.3-0.9 and 1.35-0.55%, respectively. Similarly, 6-story baseline frame and PBPD frame exhibited a maximum interstory drift ratio of 5.1% observed for Borrego ground motion and 2.12% for Imperial record, respectively. The

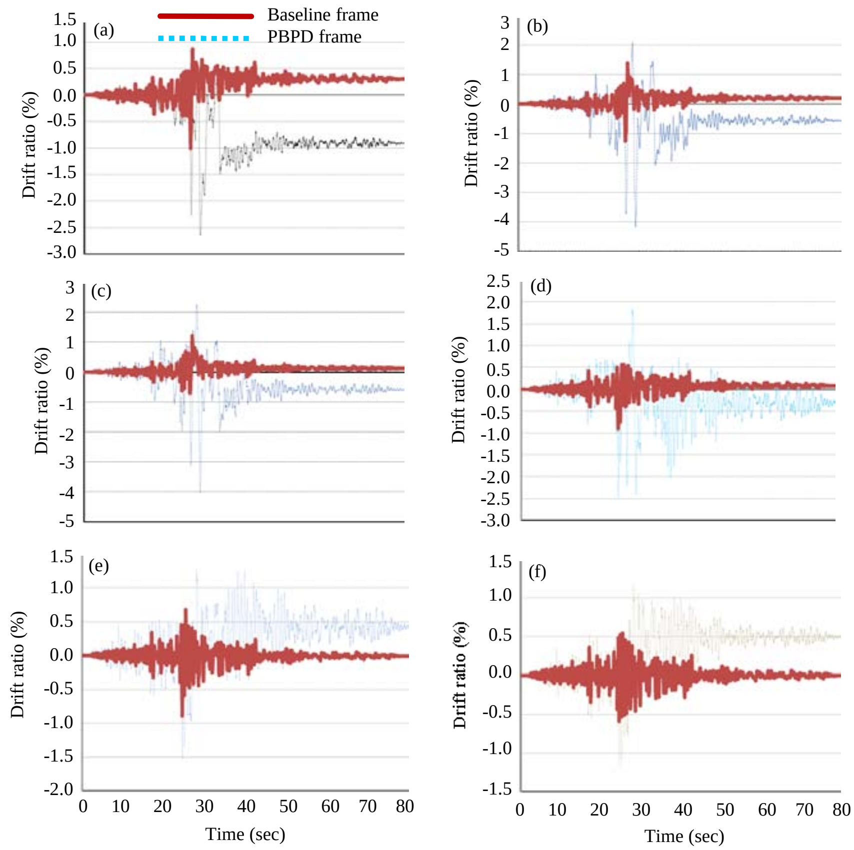

Fig. 6: Story drift time histories for both 6-story PBPD frame and baseline frame (the Loma record): a) 1st-story; b) 2nd-story; c) 3rd-story; d) 4th-story; e) 5th-story; f) 6th-story

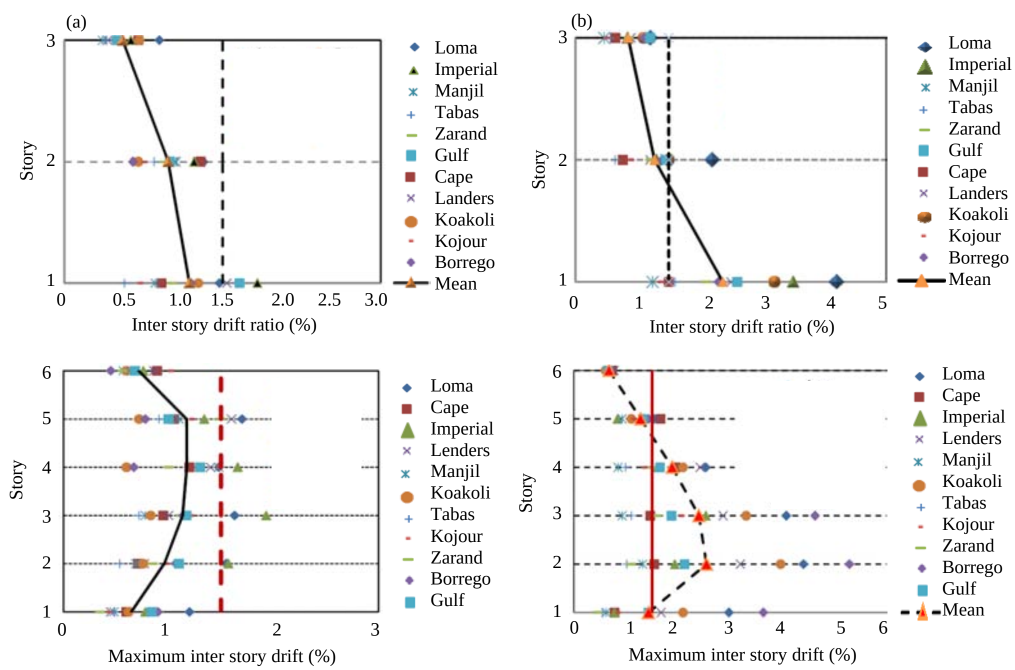

Fig. 7: Continue

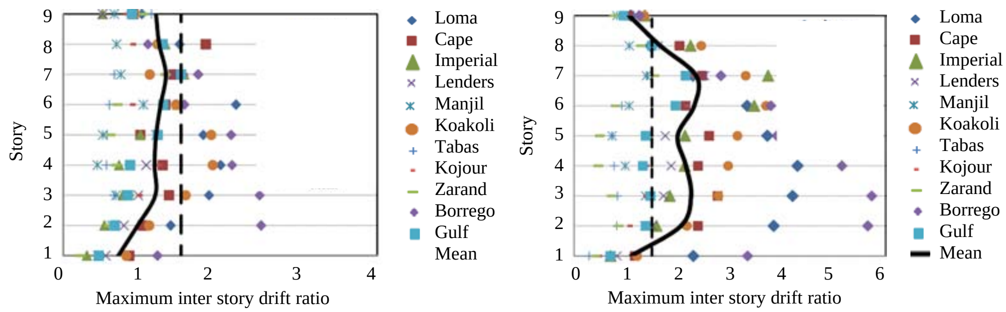

Fig. 7: Comparison of maximum inter story drifts for 3, 6 and 9-story concentric brace frames: PBPD frame; baseline frame

mean value of interstory drift ratios for the 6-story baseline and PBPD frames varied from 0.75-2.43 and 0.7-1.3%, respectively. The maximum interstory drift ratio for 9-story baseline frame was 5.62% for Borrego ground motion and for PBPD frame was observed 2.6% for Borrego record whereas the mean values of interstory drifts ratio were ranging from 1.13-2.12 and 0.85-1.35%, respectively. The results show that the mean maximum interstory drifts of the PBPD frames are well within the corresponding target values, i.e., 1.5% for 2/3 MCE. Moreover, the story drifts of the PBPD frames are more evenly distributed over the height as compared with those of the baseline frame where undesirable softness in the lower stories is evident which is caused mainly by plastic hinges in the columns. Formation of plastic hinges in the columns and story mechanism in the lower part of the baseline frames can be clearly noticed. In contrast, there are no unintended plastic hinges in the columns of the PBPD frame, resulting in more favourable deformed shape and yield pattern as intended in the design process. Since, drifts were well controlled by considering inelastic behaviour directly in the design of these PBPD frames, the P-delta effect had no appreciable influence on their overall behaviour but for baseline frame, especially for 9-story frame, the P-delta effect is very significant, so that the formation of plastic hinges in the columns and story mechanism in the lower part of the baseline frames can be clearly noticed. For example, Fig. 8 shows the formation of plastic hinges in 9-story baseline frame and PBPD frame under Loma record.

Also, the results obtained from 9-story CBF system designed by PBPD method as compared with baseline frame, confirm that the SRSS method proposed in column design for structures with period larger than 0.7 sec in PBPD method is excellent.

Figure 9 shows the residual drift ratio response of baseline frame and PBPD frame under eleven records. The mean values of residual drift ratio for 3-story baseline frame and PBPD frame varied from 0.25-0.87 and 0.1-0.25%, respectively. The 6-story baseline and PBPD

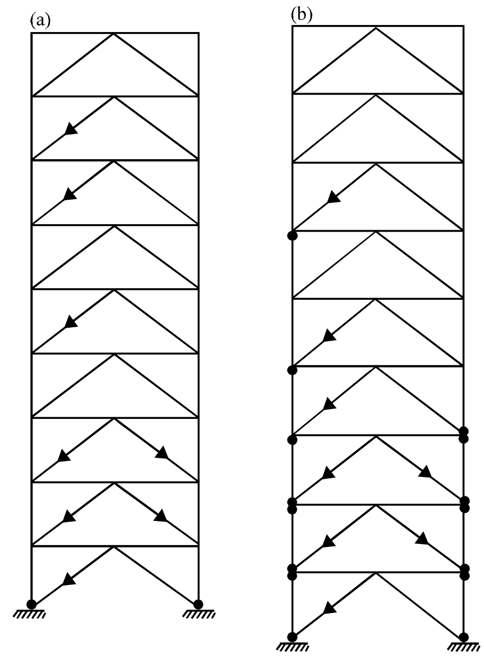

Fig. 8: Plastic hinge distributions for 9-story: a) PBPD frame; b) Baseline frames under Loma ground motion

frame exhibited mean residual drifts ranging from 0.3-0.5 and 0.1-0.2%, respectively. Similarly, the 9-story baseline and PBPD frame showed mean values of residual drift ranging from 0.27-0.67 and 0.1-0.22%, respectively. With notice to Fig. 9, the behaviour of PBPD frames were quite stable and residual drift was considerably less as compared with the baseline frames. For PBPD frames, the damage in terms of yielding and buckling was generally confined to the braces only and no brace fracture occurred, thus the intended yield mechanism and response was achieved while the baseline frame was subjected to sever damage and considerable residual drifts due to early brace fractures and plastic hinging in the columns as seen in Fig. 8.

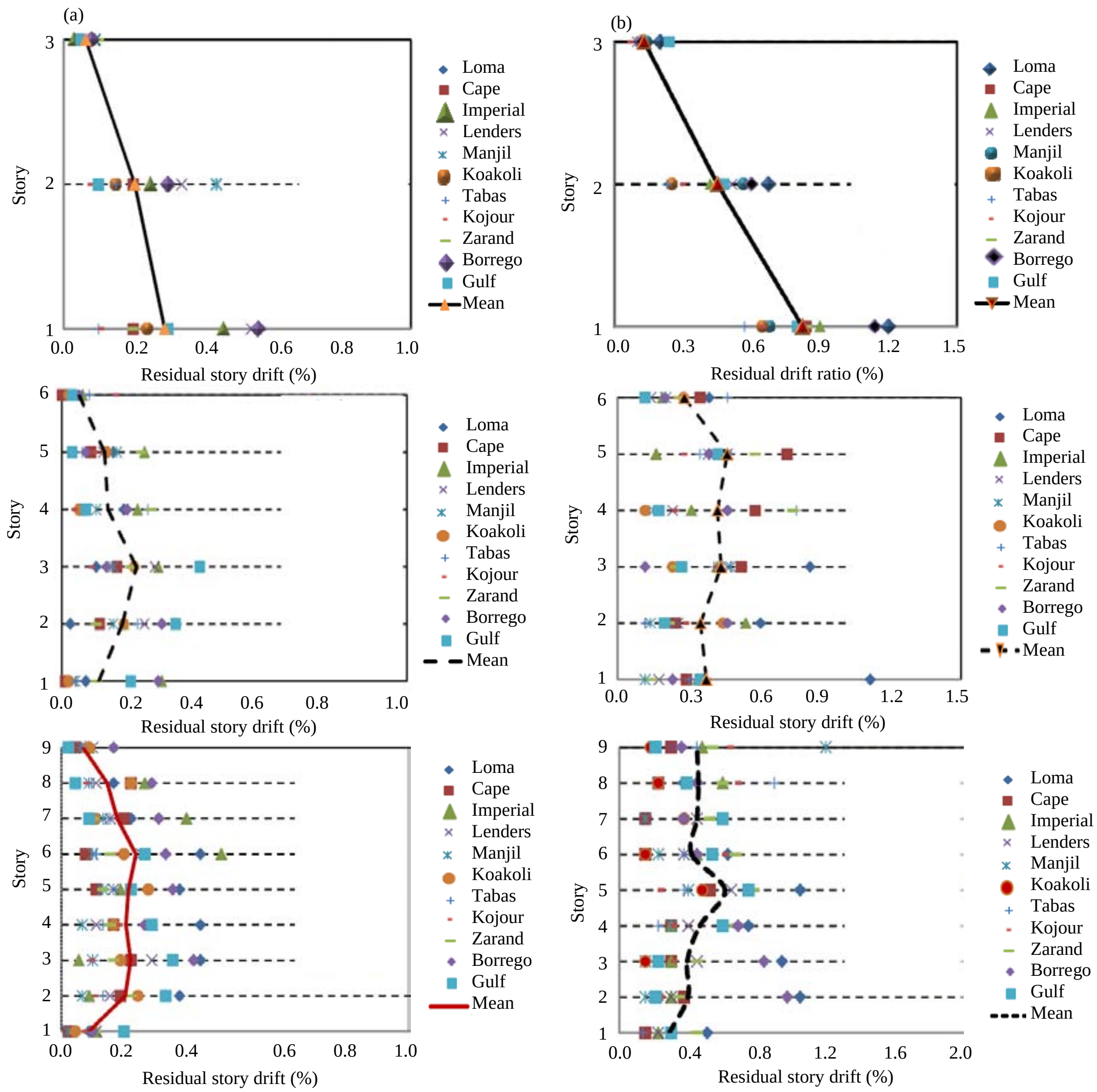

Fig. 9: Comparison of residual drifts ratio % for 3, 6 and 9-story concentric brace frames: a) PBPD frame; b) Baseline frame

CONCLUSION

The PBPD method is a direct design method which uses pre-selected target drift and yield mechanism, as key performance objectives which determine the degree and distribution of expected structural damage. The PBPD design procedure is not too different from what is done in current practice, yet it can be readily incorporated within the context of broader Performance-Based Earthquake Engineering (PBEE) framework. It does differ from the way PBEE is practiced currently which usually starts with an initial design according to conventional elastic design procedures using applicable design codes, followed by cumbersome and time-consuming iterative assessment process by using inelastic static or dynamic analyses till the desired performance objectives are achieved. The iterations are carried out in a purely trial-and-error manner. No guidance is provided to the designer, as to how to achieve the desired goals, such as controlling drifts, distribution and extent of inelastic deformation, etc. In contrast, the PBPD method is a direct design method which generally requires no evaluation after the initial design because the nonlinear behaviour and key performance criteria are built into the design process from the beginning. In this study by modifying, the determination of design base shear due to pinched hysteretic behaviour and P-Delta effect, the PBPD method was successfully applied to the design of CBF frames. The 3, 6 and 9-story baseline frames were redesigned by the modified PBPD method and the robustness and versatility of this method was evaluated by the seismic performance of those frames under 11 records representing the 2/3 MCE hazard level. Following conclusions are drawn from the present study:

- Since, stiffness degradation and strength deterioration are the major characteristics of typical SCBF system hysteretic behavior, C_2 factor is selected for modification of target design drift. C_2 factor method is based on consideration of the effect of degrading hysteretic behavior on peak (target) displacement. By converting target design drift by the C_2 factor to an equivalent non-degrading system, the design base shear for SCBF can be reasonably determined
- Due to strength degradation at braces of SCBF, it is necessary to include P-Delta effect in the determination of required axial capacity of braces, particularly for taller frames
- The results of nonlinear dynamic analyses showed that the PBPD frames responded, as intended in design with much improved performances over those of the corresponding baseline frames. So that the SCBF designed, as per PBPD methodology can successfully limit the maximum drifts within the pre-selected target drift level (1.5%), as well as achieve the intended yield mechanism under the DBE hazard level while the maximum drifts for baseline frames were larger than target drift level
- The maximum drifts for PBPD frames are generally uniformly distributed along the building height while the baseline frames experienced large concentrated drift in the lower story due to brace fractures and column hinging. Also, the PBPD frames exhibited smaller residual drifts compared to baseline frames
- The results of nonlinear dynamic analyses showed that considering SRSS method to design columns of high-rise building of SCBF designed by PBPD method is very significant

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