Performance-Based Plastic Design (PBPD) of High-Rise Buckling-Restrained Braced Frames

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SUMMARY:
Current seismic design methods are primarily based on elastic analysis approach and use indirect ways to account for the inelastic behavior of structures, even though structures are expected to experience significant inelastic deformation under severe earthquakes. Therefore there is a need to develop a systematic design approach that not only requires minimum iterative procedure but also results in predictable and targeted seismic behavior of structures. Buckling-restrained braced frames (BRBFs) are primarily used as seismic-force resisting systems for buildings in seismically-active regions. This paper presents the seismic performance of high-rise BRBFs designed as per performance-based plastic design (PBPD) methodology in which target interstory drift and yield mechanism are included in the design process as the design parameters and performance objectives. Design of BRBs is carried out for lateral load demand computed based on energy-work balance concept and the frame members are designed using capacity design philosophy. An 18-story BRBF in which BRBs are arranged in two-story X-configuration is considered as study frame. The seismic behavior of the study frame designed by the proposed methodology is evaluated by nonlinear time-history analyses for both design basis earthquake (i.e., 10%/50yrs) and maximum considered earthquake (i.e., 2%/50 yrs) hazard levels. A total of forty recorded SAC ground motions are selected for the evaluation of seismic performance of the study frame. The main parameters evaluated in this study are interstory drift response, residual drift response, brace ductility demand, yield mechanism, and higher mode effect. The study frame reached its intended performance objectives in terms of yield mechanisms and target drift levels under both levels of seismic hazard.

Keywords: PBPD, Steel structures, Braced frame, Buckling-restrained brace, Seismic design, Seismic analysis

1. INTRODUCTION

Buckling-restrained braced frames (BRBFs) are primarily used as lateral-load resisting systems for buildings in high-seismic areas because of their enhanced energy dissipation potential, excellent ductility, and nearly symmetrical hysteretic response of buckling-restrained braces (BRBs) under both tension and compression. Different types of BRBs have been developed and tested in the United States and elsewhere (López and Sabelli, 2004). A typical BRB consists of (i) a yielding steel core encased in mortar-filled steel hollow section to restrain buckling, (ii) non-yielding transition segments, and (iii) non-yielding end zones. The buckling-restrained (core) segments of BRB, which are about 60-70% of the total length between work points (Richard, 2009), are laterally braced continuously by the surrounding mortar and steel encasement to avoid their buckling under compressive loads. A more comprehensive background on BRBs can be found elsewhere (Uang and Nakashima, 2000). Past (e.g., Sabelli, 2000; Fahnestock et al., 2007) have shown that BRBFs can be used to overcome several potential problems associated with the conventional steel concentrically braced frames (CBFs), such sudden degradation in strength and stiffness, reduced energy dissipation capacity, limited ductility, etc.

In general, BRBFs are designed to experience large inelastic deformations when subjected to major earthquake ground motions. However, most current seismic design methods are still based on elastic analysis approach and use indirect ways to account for the inelastic behavior. As such, the current performance-based design methodology heavily relies on an iterative “Assess Performance”, “Revision Design, and “Assess Performance” process to reach a design capable of achieving the intended performance (FEMA-445, 2006). Very limited guidelines are available for performance-based design of high-rise building structures. A high-rise building is herein defined as one exceeding 48.8 m (160 ft) in height as per Los Angeles Tall Building Structural Design Council (LATBSDC, 2011) specifications. Hence, there is a need of developing a systematic design approach that not only requires minimum iterative procedure but also results predictable and targeted seismic performance of high-rise structures under stated levels of seismic hazards.

2. OBJECTIVES AND SCOPE

This study presents a recently-developed performance-based plastic design (PBPD) methodology (Goel and Chao, 2008) for the design of high-rise buildings with BRBs. PBPD methodology considers the inelastic characteristics of structural components directly in the design to achieve the desired performance objectives of structures. This design methodology has already been successfully applied to various steel structural framing systems (Lee and Goel, 2001; Chao and Goel, 2006; Chao and Goel, 2008) and to low-to-medium rise BRBFs with both chevron and two-story X-braced configurations (Sahoo and Chao, 2010, 2010a). The robustness of proposed design methodology was verified through a series of nonlinear time-history analyses using computer package PERFORM-3D (CSI, 2007) for a typical 18-story BRBF under both design basis earthquake (DBE, i.e., 10% in 50 years) and maximum considered earthquake (MCE, i.e., 2% in 50 years) hazard level ground motions.

3. PERFORMANCE-BASED PLASTIC DESIGN PHILOSOPHY

For the design of BRBFs, it is assumed that only the yielding of BRBs and the formation of plastic hinges at the column bases are permitted at the yield mechanism (Fig. 3.1). Hence, plastic hinges are not allowed to form in the beams and columns at the upper stories. The design base shear is computed using an energy-work balance concept where the energy needed to push an equivalent elastic-plastic single degree-of-freedom system up to the target drift level is calculated as a fraction of elastic input energy obtained from the selected elastic design spectra (Fig. 3.1). The design base shear for a BRBF can be expressed by (Goel and Chao, 2008):

$$V/W = \left(-\alpha + \sqrt{\alpha^2 + 4(\gamma/\eta)S_a^2}\right)/2 \tag{3.1}$$

where, V is the design base shear (note that the target drift, Δ_u, has been included in Eqn. 3.1); W is the total seismic weight of the structure; S_a is the spectral response acceleration obtained from code design spectrum; α is a dimensionless parameter depends on fundamental period (T), modal properties, and pre-selected plastic drift ratio (θ_p) and can be given by (Chao et al., 2007):

$$\alpha = \left(\sum_{i=1}^{n}(\beta_i - \beta_{i+1})h_i\right)\left(\frac{8\theta_p \pi^2}{gT^2}\right)\left(\frac{w_n h_n}{\sum_{j=1}^{n} w_j h_j}\right)^{0.75T^{-0.2}} \quad ; \quad \beta_i = \left(\frac{\sum_{j=i}^{n} w_j h_j}{w_n h_n}\right)^{0.75T^{-0.2}} \tag{3.2}$$

where β_i is the shear distribution factor at level i; w_i is seismic weight at level j; h_j is the height of level from the ground; w_n is seismic weight of the structure at the top level; h_n is the height of roof level ground; T is the fundamental natural period; g is the acceleration due to gravity. Eqn. 3.2 essentially represents a more realistic lateral force-distribution over the height of the structure when the inelastic behavior of the structure is considered (Chao et al, 2007). The factor of 0.75 in the exponent controls profile of lateral force distribution. In this study, two exponent values (i.e., 0.75 and 0.5) are as discussed later. The energy modification factor, γ, as given in Eqn. 3.1 can be related to structural ductility factor (μ_s = Δ_u/Δ_y; see Fig. 3.1) and ductility reduction factor, R_μ, by the following

$$\gamma = \left(2\mu_s - 1\right) / R_\mu^2 \tag{3.3}$$

The value of R_μ for a structural system can be determined by using the R_μ-μ_s-T relationship, such as an inelastic spectrum proposed by Newmark and Hall (1982), as shown in Fig. 3.1. Several structural systems exhibit significant reduction in strength and stiffness resulting "pinched" hysteresis response at the higher inelastic deformation levels. This reduction in energy dissipation capacity can also be accounted in the design by using an energy reduction factor, η. Since BRBFs exhibit full and stable hysteretic response, the value of η can be assumed as unity in Eqn. 3.1. A step-by-step PBPD procedure for a BRBF can be found elsewhere (Sahoo and Chao, 2010). Based on the calculated design base shear, the lateral forces at various story levels can be obtained by following a lateral distribution procedure proposed by Chao et al. (2007). Since columns generally share very limited story shear in braced frames, it is assumed that the story shear is entirely carried by BRBs only. Thus, the required size of BRBs at any story level is determined by resolving the computed story shear in the direction of braces and using their nominal yield strength values. Beams and columns (termed as non-yielding members) of BRBFs are designed based on capacity design philosophy for the maximum axial force and bending moment demand expected at the ultimate states of BRBs. The selected structural sections need to satisfy the compactness and lateral bracing requirements as per the ANSI/AISC 360-10 (2010) Provisions.

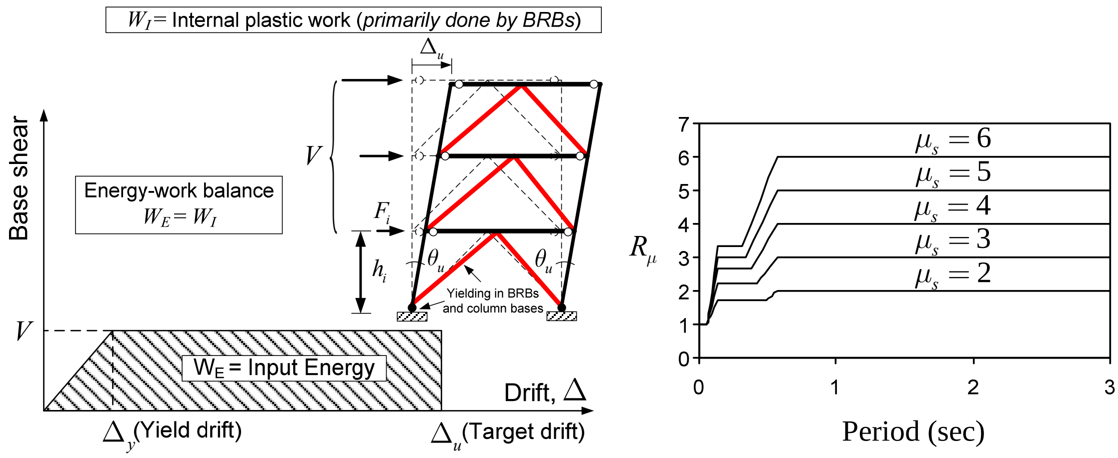
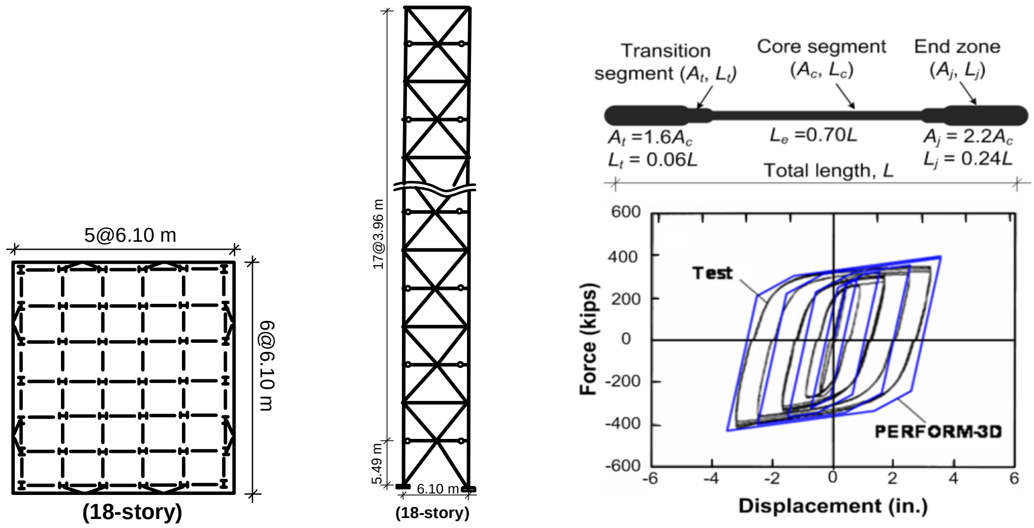


Figure 3.1. Energy-work balance concept used in PBPD methodology

4. BUILDING MODELS

As shown in Fig. 4.1(a), an 18-story building (Gupta and Krawinkler, 1999) was considered as study frame in which BRBs were arranged in two-story X-configuration. The building was assumed to be located on firm soil (site classification D) at a hypothetical site in Los Angeles. Typical bay width in each direction of the building was 6.10 m. There were a total of four braced bays in each direction of building. Except for the first story height of 5.49 m, typical story height of the building was 3.96 m. total height of 18-story BRBF was 72.9 m which exceeds the height limit of 48.8 m for tall buildings specified by LATBSDC (2011) document. However, the height of 18-story buildings was just below maximum height of 73.2 m allowed for the braced frames as per IBC (2009) provisions. Total seismic weight of the building was 107.5 MN. The spectral acceleration values at 0.2 sec. (S_{DS}) and 1 sec. (S_{D1}) were 1.11g and 0.61g, respectively. Pinned beam-to-column connections were used at beams where no gusset plates were present; otherwise the connections were assumed rigid due to the rotational restraint provided by the gusset plates.



(a) (b)

Figure 4.1. Details of (a) 18-story BRBF and (b) force-displacement response of BRB model (1 kips = 4.448 kN; 1 in. = 25.4 mm)

4.1. Design base shear for BRBF

A target drift ratio, θ_u, equal to 1.75% was selected in the present study for the DBE hazard level. Note that a smaller or greater values could be selected based on the engineer's judgment, which will lead to a greater and smaller PBPD design base shear, respectively. Table 4.1 summarizes design base shear values for the 18-story BRBF considering two different exponent values, as defined in Eqn. 3.2, of 0.75 and 0.5, respectively. It should be noted that this exponent value not only changes the lateral load distribution pattern but also will slightly change the magnitude of design base shear in PBPD method because α in Eqn. (3.1) changes. In general, use of a smaller exponent value will lead to smaller design lateral force and reduce drift response at the upper story levels. This can be helpful since the upper story in a high-rise structure can experience greater story drifts due to higher mode effect. The value of natural period of 1.70 sec. as per ASCE7-05 (2005) specifications was used for the design of the BRBFs. Note that, response reduction factor (R) and importance factor (I) are not explicitly required in the PBPD methodology. The value of yield drift was computed as 0.68% using a simplified equation proposed by Sahoo and Chao (2010a) based on the nonlinear analysis results of low-to-high BRBFs considering the realistic boundary conditions of BRBs. The values of design base shear for the 18-story BRBF were 0.047 and 0.044 for the respective exponent values 0.75 and 0.50 in Eqn. 3.2. For this particular building, the PBPD design base shear value was nearly the same as that predicted by current code-based design value of 0.05. Hence, the only difference was the pattern of lateral load distribution over the height of the BRBF. Separate design of BRBFs was not carried out for the MCE hazard level since the collapse prevention rather than drift control is the governing performance criteria under MCE hazard level. However the performance of the study frame designed for the DBE level was also checked under MCE ground motions by the nonlinear time-history analyses.

4.2 Design of frame members

As stated earlier, the desired yield mechanism of BRBFs should involve the yielding of BRBs and the plastic hinges at the bases of first-story columns. All frame members were designed based on their nominal material strengths. BRB sizes were determined by assuming nominal material yield strength

248 MPa and strength-reduction factor of 0.9. The maximum force demands on beams and columns computed based on the maximum expected strengths of BRBs by applying material overstrength (R_y) of 1.1, compression strength adjustment factor (β) of 1.22 and strain hardening adjustment factor (ω) of 1.45 to the nominal yield strength values (Merritt et. al., 2003; Reaveley et al. 2004). Nominal material yield strength of 345 MPa with the value of R_y as 1.1 was used for the design of beams and columns. Table 4.2 summarizes the properties of BRBs and non-yielding members used in the 18-BRBF. Because BRBs arranged in a two-story X-configuration, the unbalanced force on beams is significantly reduced, leading to smaller beam sections.

Table 4.1. Computation of design base shear for 18-story PBPD BRBF

No.	Parameters	Case-I: *Exponent=0.75*	Case-II: *Exponent=0.5*
1	Target drift ratio, θ_u (%)	1.75	1.75
2	Total frame height (m)	72.9	72.9
3	Yield drift ratio, θ_y (%)	0.68	0.68
4	Fundamental period, T (sec.)	1.70	1.70
5	Inelastic drift ratio, $\theta_p = \theta_u - \theta_y$ (%)	1.07	1.07
6	Ductility reduction factor, R_μ	2.58	2.58
7	Structural ductility factor, μ_s	2.58	2.58
8	Energy modification factor, γ	0.63	0.63
9	Spectral acceleration, S_a	0.36	0.36
10	Exponent (alpha) in Equation (2)	0.75	0.50
11	Base shear coefficient, V/W	0.047	0.044
12	Design base shear, V (kN)	1275	1180

Table 4.2. Details of BRBs and structural sections of 18-story BRBF

Story	Tensile yield strengths of BRBs (kN)		Column sections		Beam (Both Cases)
	Case-I	Case-II	Case-I	Case-II	
1st & 2nd	1445 & 1144	1341 & 1064	W14x500	W14x500	W16x40 (All floors)
3rd & 4th	1134 & 1119	1058 & 1048	W14x398	W14x398	
5th & 6th	1100 & 1076	1036 & 1021	W14x311	W14x342	
7th & 8th	1047 & 1013	1003 & 981	W14x257	W14x283	
9th & 10th	974 & 930	956 & 926	W14x193	W14x211	
11th & 12th	879 & 822	893 & 853	W14x145	W14x159	
13th & 14th	758 & 685	808 & 756	W14x90	W14x109	
15th & 16th	603 & 508	694 & 619	W14x61	W14x74	
17th & 18th	396 & 258	525 & 394	W14x34	W14x43	

5. MODELING AND ANALYSIS

The seismic performance of BRBF was evaluated by nonlinear analysis using a computer program PERFORM-3D (CSI, 2007). Only two-dimensional analysis was conducted because the structural layouts of the building are nearly the same in both directions (Fig. 4.1(a)). Beams and columns were modeled as standard frame elements with plastic hinges lumped at the specified locations. Moment-Moment-rotation plastic hinges with axial-moment interaction were assigned to all beam and column elements since these members would carry axial force in addition to bending under earthquake excitations. All columns at the bases of the first level were assumed to be perfectly fixed to the P-Delta effect due to gravity loads resulted from the gravity frames in the building was modeled by an equivalent continuous column representing all gravity columns associated with the frame. The magnitude of axial load in these equivalent columns was computed from the total building weight (exclusive of tributary gravity load to the braced frames), divided by the number of braced frames a particular direction. Lateral stiffness and strength of these columns at each story level represent the sum of respective values of all gravity columns at that story level assuming their weak-axis bending. These columns were pinned at their bases and constrained to match the frame displacement at each level by using pin-ended rigid beams. Moment-rotation hinges with axial load-moment interaction added at ends of those columns so that the contribution of those gravity columns in resisting the lateral forces was also considered. Rayleigh damping (both mass- and stiffness proportional damping) of 2% were considered in all modes of structures in the time-history analyses.

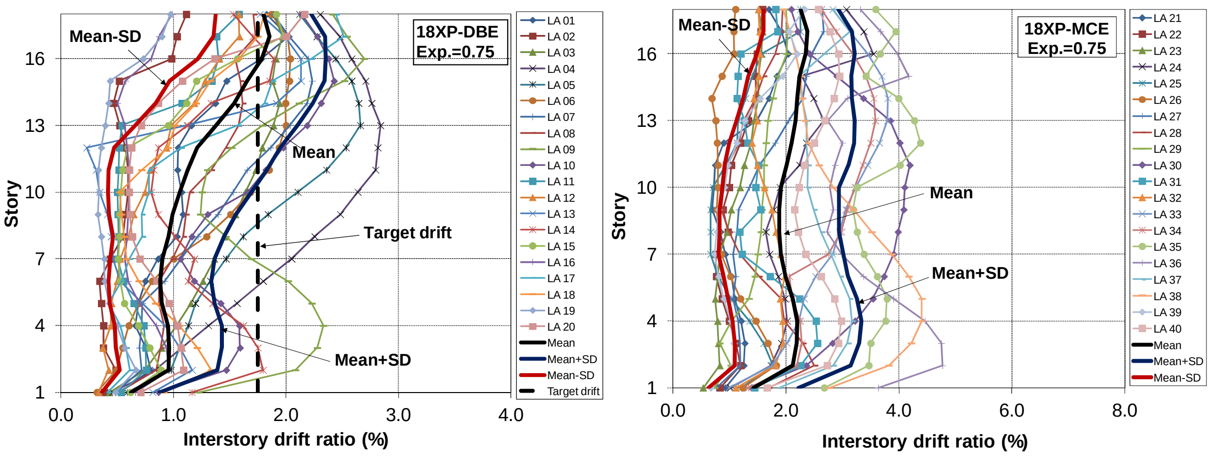
Standard BRB elements available in PERFORM-3D (CSI, 2007) were chosen to model all braces of BRBFs. In general, the area of transition (elastic) and end zones of BRBs are larger than that of the core (restrained yielding) segment to limit the yielding to the core segment only. As shown in Fig. 4.1(b), the area of transition and end segments of BRBs were assumed as 160% and 220% of the area of the core segment, respectively. Similarly, the length of transition and end segments were assumed as 6% and 24% of the total length of BRB (Huang and Tsai, 2002). Elastic modulus of steel was considered as 200 GPa to compute the axial stiffness of BRBs. The post-yield stiffness of core segments of BRBs in tension and compression was assumed as 3% of their initial stiffness. Both isotropic and kinematic hardening characteristics of BRBs were considered in the modeling of their force-deformation response which was obtained by comparing the hysteretic response of a typical BRB with the component test results (Merritt et. al., 2003) as shown in Fig. 4.1(b). To monitor the magnitude of plastic displacements of BRBs under various ground motions, the maximum allowable ductility of BRBs were specified as 15 and 25 times their yield displacements for DBE and MCE level analyses, respectively. Similarly, the respective values of allowable cumulative plastic displacement of BRBs were fixed as 200 and 400 times their yield displacements for DBE and MCE level ground motions, respectively (Fahnestock et al., 2007). Two suites of SAC ground motion records Somerville et al. (1997) representing DBE and MCE hazard levels were selected in this study for a hypothetical site in downtown Los Angeles with a probability of exceedance of 10% and 2% in 50 years, respectively. A total of forty records were obtained from twenty ground motions in both fault-parallel and fault-normal orientations. Ground motions LA 01-20 represent the DBE level earthquake hazard, whereas ground motions from LA 21-40 represent the MCE level hazard. These records were amplitude scaled so that the average of the two horizontal spectra matches the 1997 NEHRP spectrum. However, since the spectrum used in IBC (2009) is slightly different from that of 1997 NEHRP, the scale factors for those ground motions were changed accordingly, although the difference (which can either increase or decrease) was found to be marginal. The modified scaled factors to match the code spectrum were obtained from the PEER Ground Motion Database (PEER, 2011) by using weights of 0.1, 0.3, 0.3, and 0.3 for periods of 0.3, 1, 2, and 4 seconds, respectively. The scale factors were kept the same for two of the SAC ground motions that were not found in the PEER Ground Motion Database.

6. ANALYSIS RESULTS

Modal analyses were carried out to achieve a reasonable level of confidence in the models and designs. The natural period of 18-story BRBF was found to be 3.43 sec., which was greater than the upper bound value specified by ASCE7-05 (2005). Nonlinear time-history analyses were carried out to evaluate the seismic performance of BRBFs, in terms of interstory drift ratio, residual drift ratio, yield mechanism, and displacement ductility. Interstory (or residual) drift ratio was defined as the ratio of the interstory (or residual) displacement to the corresponding story height.

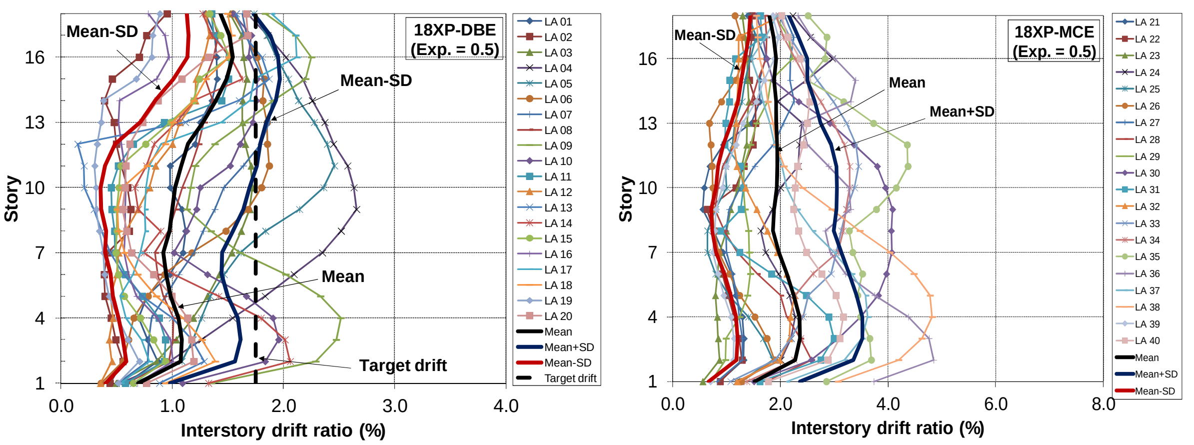
6.1. Interstory drift response

Fig. 6.1(a) shows the interstory drift response of 18-story BRBF designed considering the exponent value of 0.75 under DBE and MCE level ground motions. The maximum values of mean (μ) and mean (μ) +standard deviation (σ) of interstory drift ratios for the BRBF under DBE level ground motions 1.85% and 2.35%, respectively. Except at the top three stories, the interstory drift response at other levels was smaller than the target drift level of 1.75%. It should be noted that no iteration was carried to achieve the targeted performance of the BRBF. Similarly, as shown in Fig. 6.1(b), the maximum μ and (μ+σ) values of interstory drift ratio for the BRBF designed for exponent value of 0.75 were 2.38% and 3.34% under the MCE level ground motions. Since the target drift level was exceeded at three story levels of the BRBF, one variant of the same BRBF designed using exponent value of 0.75 was further studied in which same BRBs (i.e., Tensile yield strength of BRBs = 508 kN) were used in top three (16th-18th) story levels. All other parameters, such as, beams and columns sections, connections, etc. were kept exactly same as previous. Nonlinear time-history analysis results under conditions showed that maximum mean value of interstory drift at the 17th story reduced from 1.85% 1.64%. Similarly, the interstory drift response at the 18th floor reduced from 1.78% to 1.48%, that the mean interstory drift response of the BRBF did not exceed the target drift level of 1.75% at story level. Fig. 6.2(a) shows the interstory drift response of the BRBF designed for exponent value of 0.5 under DBE level ground motions. The maximum values of μ and (μ+σ) were found to be 1.54% 1.95%, respectively. The design base shear value for this case was 0.044 which was little smaller than the Case-I (0.047). As expected, the interstory drift response was higher at the lower story levels but smaller at the upper story levels. In contrast to the Case-I, the mean interstory drift response at all levels was smaller than the target drift level of 1.75%. As shown in Fig. 6.2(b), the maximum values μ and μ+σ interstory drift response of the BRBF (Case-II) under MCE level ground motions were to be 2.35% and 3.52%, respectively. It is noted that past experimental studied shows that BRBFs with the pinned beam-column-brace configuration exhibited excellent seismic performance and sustained only minor yielding up to story drift ratio of 4.8% (Fahnestock et al., 2007). Generally, in both study cases, the overall drift of the BRBF under the MCE level did not exceed a mean value of 3% and absolute maximum values of interstory drift ratio at any story level of 4.5% according to both the LATBSDC (2011) and PEER Tall Building Design (PEER, 2010) guidelines. This study indicates while using the exponent value of 0.75 in Eqn. 3.2 has been shown leading to uniform story drift ratios throughout the building height for low- to mid-rise structures (Chao et al., 2007), the use of exponent value of 0.5 results in interstory drift response within the target drift level at all story levels of the as well as uniform story drift response along the height for the high-rise buildings.



(a) (b)

Figure 6.1. Interstory drift response of BRBF (Case-I) under (a) DBE and (b) MCE level ground motions

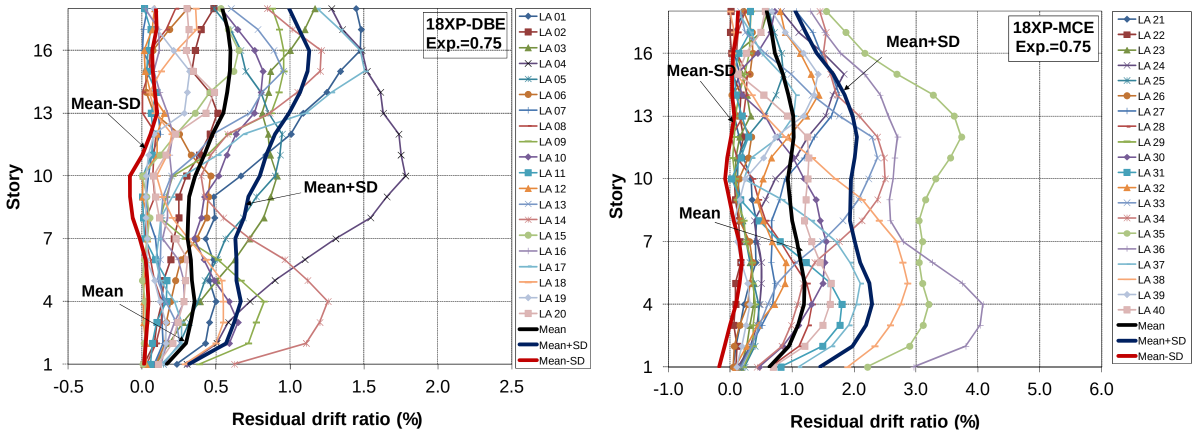


(a) (b)

Figure 6.2. Interstory drift response of BRBF (Case-II) under (a) DBE and (b) MCE level ground motions

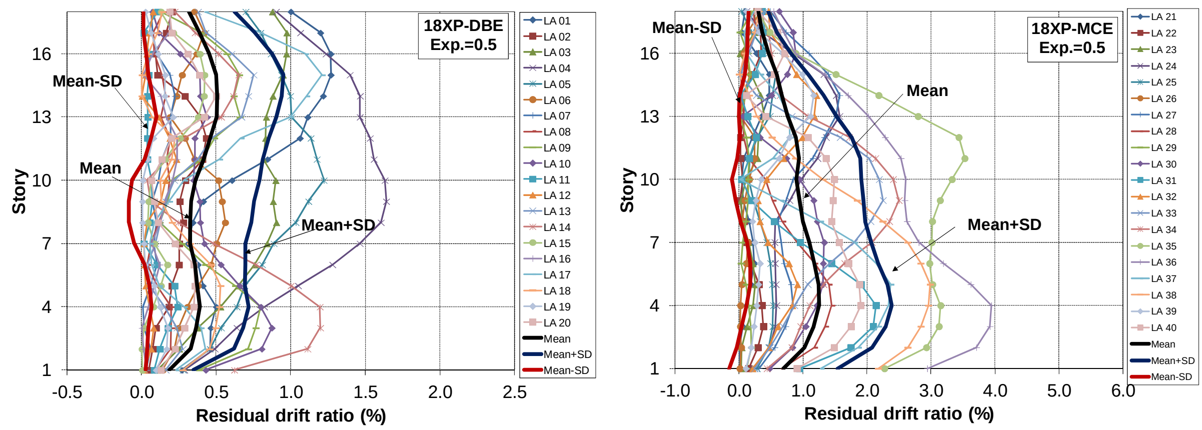
6.2. Residual drift response

The permanent displacement of BRBF was monitored from the drift levels at the end of earthquake ground motions. Fig. 6.3 shows the residual drift ratio response of the BRBF designed for exponent value of 0.75 (Case-I) under both DBE and MCE level ground motions. The BRBF showed maximum values of μ and (μ+σ) residual drift ratios of 0.67% and 1.13%, respectively under DBE level ground motion. The smaller residual drift response was observed at the lower story levels, whereas relatively higher residual drift values were noted at the upper story levels of the BRBF under the DBE level ground motions. The corresponding values under the MCE level ground motions were 1.18% and 2.26%. In contrast to the DBE level performance, the higher value of residual drift response was noted in the lower story levels. Fig. 6.4 shows the residual drift response of BRBF designed for exponent value of 0.5 (Case-II) under both DBE and MCE level ground motions. The maximum values of μ and (μ+σ) residual drift ratios under DBE level ground motion were 0.51% and 0.95%, respectively. Similarly, the maximum values of μ and (μ+σ) residual drift ratios under MCE level ground motion were 1.25% and 2.39 %, respectively. This shows that, using the exponent value of 0.5 in the design, the maximum value of residual drift of 18-story was reduced under DBE level ground motions, whereas the same is slightly increased under MCE level ground motions as compared to that using the exponent value of 0.75. For MCE level ground motions, it is required in both the LATBSDC (2011) and PEER Tall Building Design (PEER, 2010) guidelines that, "in each story, the mean of the absolute values of residual drift ratios shall not exceed 0.01; and in each story, the maximum residual story drift ratio in any analysis shall not exceed 0.015 unless proper justification is provided". While both cases generally meet the first requirement, it is seen that the second requirement can hardly be satisfied due to the characteristics of individual ground motions. A much smaller target drift for the design will be needed if these rules need to be strictly followed.



(a) (b)

Figure 6.3. Residual drift response of BRBF (Case-I) under (a) DBE and (b) MCE level ground motions



(a) (b)

Figure 6.4. Residual drift response of BRBF (Case-II) under (a) DBE and (b) MCE level ground motions

6.3. Brace ductility demand

The magnitude of inelastic deformation of BRBs is measured by brace ductility and cumulative plastic ductility. As stated earlier, the maximum values of brace deformation were set as 15 and 25 times their yield displacements under the DBE and MCE level ground motions in the nonlinear time-history analyses. Results showed that the18-story BRBF did not reach the maximum brace ductility demand of 15 under DBE hazard level in both cases. The average values of maximum ductility demand for the BRBF were 6.4 and 20.7 under the DBE and MCE level ground motions, respectively. This indicates that the ductility limit states of BRBs were not reached under both hazard levels. Prior experimental tests indicated that the behavior of BRBs can be largely controlled by cumulative plastic deformations. The values of maximum cumulative plastic displacements of BRBs were set as 200 and 400 times their yield displacements, respectively. The average value of maximum brace cumulative ductility for the BRBF was found to be 17.3 under the DBE hazard level. Similarly, the average value of maximum cumulative displacement ductility demands for the BRBF under the MCE level ground motions was 72.8. Thus, BRBs did not reach their cumulative plastic ductility limits under the DBE and MCE level ground motions. It should be mentioned that most prior isolated BRB tests were carried out under high cumulative ductility demand (approximately 300 to 1600), as opposed to the smaller cumulative ductility values observed in this study for the high-rise BRBFs.

6.4. Yield mechanisms

Under the DBE level ground motions, BRBF exhibited the yielding of BRBs in addition to the plastic hinges at the column bases. As desired, no plastic hinges were developed in beams and columns except at the column bases. Thus, the intended yield mechanism of BRBFs was achieved under the life safety hazard level. Under MCE level earthquakes, in addition to yielding of BRBs and plastic hinges at column bases, minor flexural yielding of beams and columns was observed at different story levels. However, these members did not reach their ultimate deformation and load-carrying capacity which prevented these BRBFs from partial or complete collapse under MCE ground motions. Thus, the target yield mechanism was also achieved in both design cases of 18-story BRBF using PBPD methodology.

7. CONCLUSIONS

Following conclusions are drawn from the present study:

1. BRBFs designed as per PBPD methodology can successfully limit the maximum drifts within the pre-selected target drift level, as well as achieve the intended yield mechanism under the life safety (DBE) hazard level. No iterative procedure is required to achieve the target drift level and yield mechanism in this design methodology.
2. The interstory drift and residual drift responses of the BRBF designed as per PBPD methodology were found to be generally within the acceptable limits as prescribed by LATBSDC and PEER Tall Building Design guidelines.
3. Using the exponent value of 0.50 instead of 0.75 in the PBPD lateral force distribution equation can reduce the maximum drift values especially at the upper story levels. This gives a more uniformly distributed story drift throughout the building height. The larger drifts at upper levels can also be significantly minimized by using same BRB sections throughout a few upper stories.
4. The maximum cumulative displacement ductility demands of the BRBs for the study high-rise BRBF were generally much smaller than their capacity, even under the MCE level ground motions.

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