Inelastic Torsional Response of Steel Concentrically Braced Frames

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SUMMARY:
Concentrically braced frames (CBFs) have limited energy dissipation capacity and low redundancy due to the likelihood of premature brace fracture under cyclic loading in addition to the brittle failure of brace connections. Variation in strength of braces can cause inelastic torsional response of a CBF. This study investigates the accidental torsional response in low-rise steel CBFs due to the variation in strength of braces. For this study, it was assumed that the variation in strength of braces would come from the expected yield stress rather than minimum specified yield stress of brace member. For this purpose, inelastic torsional response of a three-story building having perimeter CBFs subjected to strong earthquake ground motions is investigated in detail. The results are presented in the form of axial force-strain in brace members, base shear vs. roof displacement (pushover curve) and drift ratio through nonlinear dynamic response analyses.

Keywords: Torsional response, concentrically braced frames, strength variation

1. INTRODUCTION

Concentrically braced frames (CBFs) are considered to be one of the most cost-effective seismic load resisting systems against lateral loads in steel buildings. The main advantages of these systems are their efficiency in meeting lateral stiffness and strength requirements with minimum steel weight, and simplicity in design calculations. Overall gravity and lateral load analyses and design of a CBF can be easily performed by hand calculation. However, CBFs have limited energy dissipation capacity and low redundancy due to the likelihood of premature brace fracture under cyclic loading in addition to the brittle failure of brace connections (Akbas *et al.*, 2012). It is well-known that energy dissipation capacity of a brace member decreases as its slenderness ratio increases. Braces are the main lateral load carrying elements in CBFs and their axial force-deformation relation is substantially different from moment-rotation behavior in moment resisting frames. They exhibit non-symmetrical hysteretic behavior with significant strength degradation in compression.

Recent studies on CBFs have focused on addressing the questions on their inelastic behavior (Tremblay *et al.*, 2003; Tremblay and Poncet, 2005; Erduran and Ryan, 2011; Akbas *et al.*, 2012). Tremblay *et al.* (2003) performed an experimental study to determine the inelastic response of CBFs made with cold-formed rectangular tube sections. They also proposed simplified equations to predict the out-of-plane deformations of the braces. Tremblay and Poncet (2005) examined the seismic response of an eight-story CBF with mass irregularity. They concluded that mass irregularity did not have a significant impact on the elastic response for immediate occupancy level. Erduran and Ryan (2011) investigated the inelastic torsional response of a three story building with peripheral CBF for four different seismic hazard levels. They also evaluated the elastic response spectrum and pushover analyses methods to be used in estimating the torsional response of CBF subjected to biaxial ground excitation. Akbas *et al.* (2012) studied the collapse probability of ductile and non-ductile CBFs through nonlinear dynamic response analysis using the evaluation approach proposed by FEMA P695 (FEMA, 2009).

Torsional irregularities in a building whose structural system consists of CBFs are not only due to the difference between the centers of rigidity and mass, but also due to the variation in strength of braces. The main objective of this study is to investigate the accidental torsional response in low-rise steel CBFs due to the variation in strength of braces. For this study, it was assumed that the variation in strength of braces would come from the expected yield stress, R_yF_y, rather than specified minimum yield stress, F_y, of brace member, where R_y is the ratio of the expected yield stress to the specified minimum yield stress, F_y. For this purpose, inelastic torsional response of a three-story building having perimeter CBFs subjected to strong earthquake ground motions is investigated in detail. The results are presented in the form of axial stress-strain in brace members, base shear vs. roof displacement (pushover curve) and drift ratio through nonlinear dynamic response analyses.

2. STRUCTURAL MODEL AND EARTHQUAKE GROUND MOTIONS

2.1. Description and Design of the Three-Story Building

A three-story building with perimeter CBFs is selected for this study (Fig. 2.1). The plan of the building is symmetrical. The building has plan dimensions of 45.75m x 45.75m (150ft x 150ft) with a story height of 3.96m (13ft) for the second and third stories and 5.49m (18ft) for the first story. The building consists of five-bay frames in two orthogonal directions spaced at 9.15m (30ft). Perimeter braced frames are used in both orthogonal directions to resist lateral loads. Interior frames are assumed to be simply connected, i.e. they do not have any contribution for carrying seismic loads. The columns are assumed to be pinned to the ground. All the connections of frame to column, brace to girder, and girder to column are assumed to be pin connections as well. All the elements of building were designed based on the seismic design requirements in ASCE 7 (ASCE 7-10, 2010) and AISC 341 (AISC 341-10, 2010). The perimeter frames were designed as ductile CBFs with a response reduction factor of R=6. The member sizes are given in Table 2.1. Only one braced bay with inverted-V bracing in each perimeter CBF is used to resist seismic loads. The brace members are selected from HSS with a specified minimum yield stress of 317 MPa (46 ksi). The yield stress of all wide flange column and beams in the building is specified with a minimum specified yield stress of 345 MPa (50 ksi). The floor system of the building is assumed to provide diaphragm action and to be rigid in the horizontal plane. Thus, the inertial effects of each story level of the building are carried by each perimeter CBF resisting one half of the seismic mass of the building. The perimeter CBFs are numbered as Frame 1, 2, 3 and 4 as shown in Fig. 2.1.

The building was designed with S_S=150%g, S_1=80%g and with estimated dead load of 3.84 kN/m^2 (80psf) and live load of 2.40 kN/m^2 (50psf). Beams and columns were modeled as beam-column elements, whereas inelastic steel bar element is used to model the axial behavior of braces (Fig. 2.2). Nominal compressive strength, P_{cr}, and nominal tensile strength, P_y, were computed based on AISC 360 (AISC 360-10, 2010) (Fig. 2.2). Residual compressive strength, $P_{residual}$ and the axial deformation at which it is reached were assumed to be $0.3P_{cr}$ based on AISC 341 (AISC 341-10, 2010) and $8\delta_y$, respectively (Fig. 2.2). δ_y is defined as the yield displacement corresponding to P_{cr}. A small yield plateau was assumed having constant length equal to δ_y after buckling occurred. Tension stretch effect due to the increase in buckling deformation in a cycle was also taken into account with a stretch factor of 0.05 (PERFORM-3D, 2011). Expected yield stress, R_yF_y, was used in defining the inelastic behavior of the braces, where R_y is 1.4 for HSS (AISC 341-10, 2010). Inelastic effects were also assigned to plastic hinges at beam and column ends in braced bays, i.e. material nonlinearity was considered in the analyses by defining bilinear moment-rotation relationship to beams and columns. Strain hardening was taken to be 5% in beam and column members. P-M (axial load-moment) interaction relation, suggested by AISC 360 (AISC 360-10, 2010), was used as the yielding surface of column elements. The first fundamental period of the building was found to be 0.66 sec. Damping ratio was taken as 5% and Rayleigh damping with the first and second natural frequencies were used in the analyses. Analyses were carried out using PERFORM-3D (2011) computer program.

2.2. Earthquake Ground Motions

Seven spectrum-compatible earthquake ground motions (LA21, LA22, LA25, LA26, LA36, LA37 and LA38) were selected for the nonlinear dynamic response analyses. These ground motions were used in a FEMA-sponsored research project on steel moment frames damaged in the 1994 Northridge earthquake and identified as having a 2% probability of exceedance in 50 years by SAC. Their mean response spectrum matches the 1997 NEHRP design spectrum, modified from soil type of S_B-S_C to soil type S_D and having a hazard specified by the 1997 USGS maps (Sommerville *et al.*, 1997). Normalized response spectra of the selected earthquake ground motions are given in Figure 2.3.

For the nonlinear dynamic response analyses, the strength of the braces was assumed to be varying for each perimeter CBF between R_yF_y and $1.2R_yF_y$. Eight cases were generated in order to study the inelastic torsional response of the perimeter CBFs (Table 2.2). Columns 2, 3 and 4 in Table 2 show the designated strength of the brace, applied frame in plan (see Fig. 2.1) and applied story, respectively. For example, Case 1 refers to the case where the strength of the braces in frame 1 at all stories is equal to $1.2R_yF_y$, i.e., the strength of the braces in other frames is equal to R_yF_y. For Cases 5, 6, 7 and 8, the strength of the braces is assumed to be equal to $1.2R_yF_y$ at only the first story braces of a given frame. A reference building with perimeter CBFs having no strength variation in the braces was also included in the analyses, i.e. expected yield stress, R_yF_y, was used in defining the inelastic behavior of these braces. The response of the perimeter CBFs for the Cases in Table 2.2 under the selected earthquake ground motions were believed to vary from moderate to severe and the inelastic torsional response of the CBFs would be evaluated rationally.

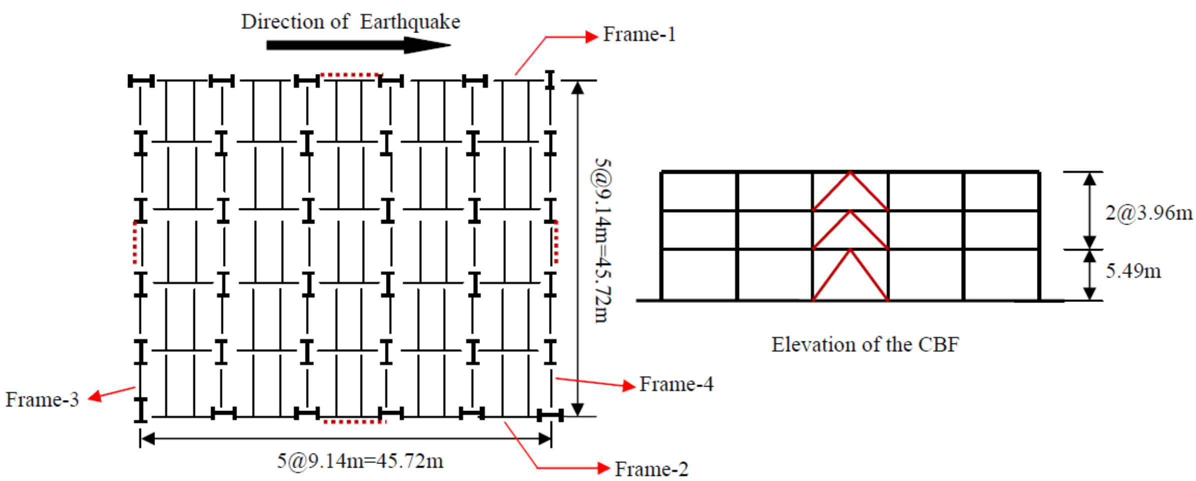


Figure 2.1. Plan and elevation of the 3-story frame

Table 2.1. Member sizes of the building

Story	Brace Members	Braced Bay Girders	Unbraced Bay Girders	Braced Bay Columns	Unbraced Bay Columns	Exterior Columns	Columns not on the perimeter	Girders not on perimeter
Roof	HSS7×7×1/2	W24×306	W21×44	W12×40	W12×40	W12×40	W12×50	W21×55
2nd Floor	HSS8×8×1/2	W24×335	W21×44	W12×40	W12×40	W12×40	W12×50	W21×55
1st Floor	HSS10×10×5/8	W36×395	W21×44	W12×65	W12×65	W12×40	W12×72	W21×55

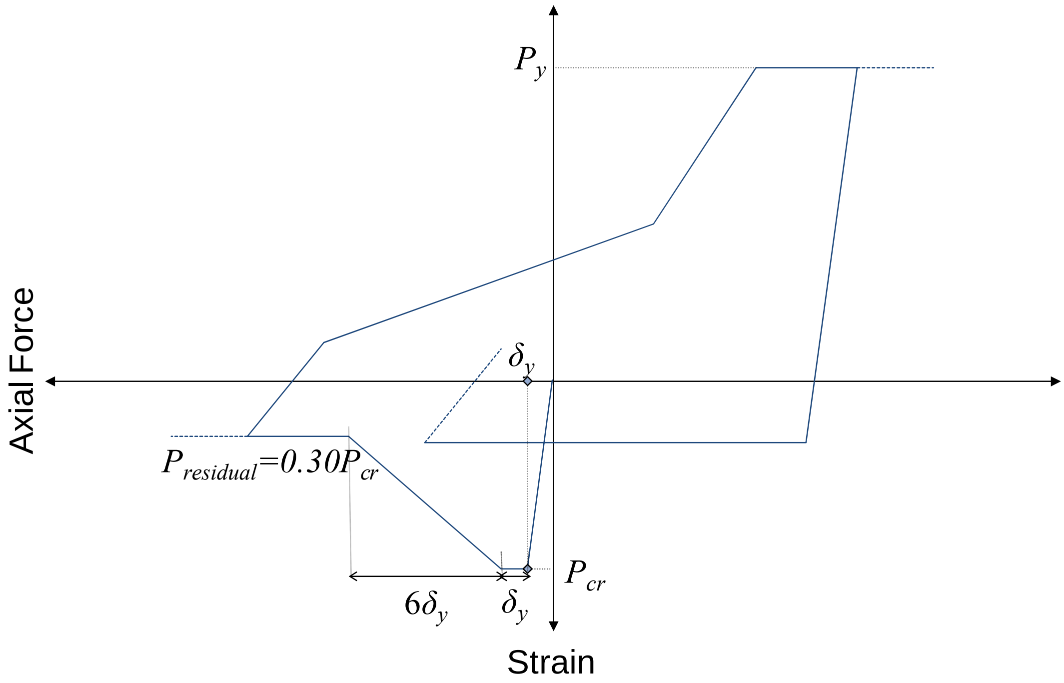


Figure 2.2. Hysteresis loop for axial behavior of braces

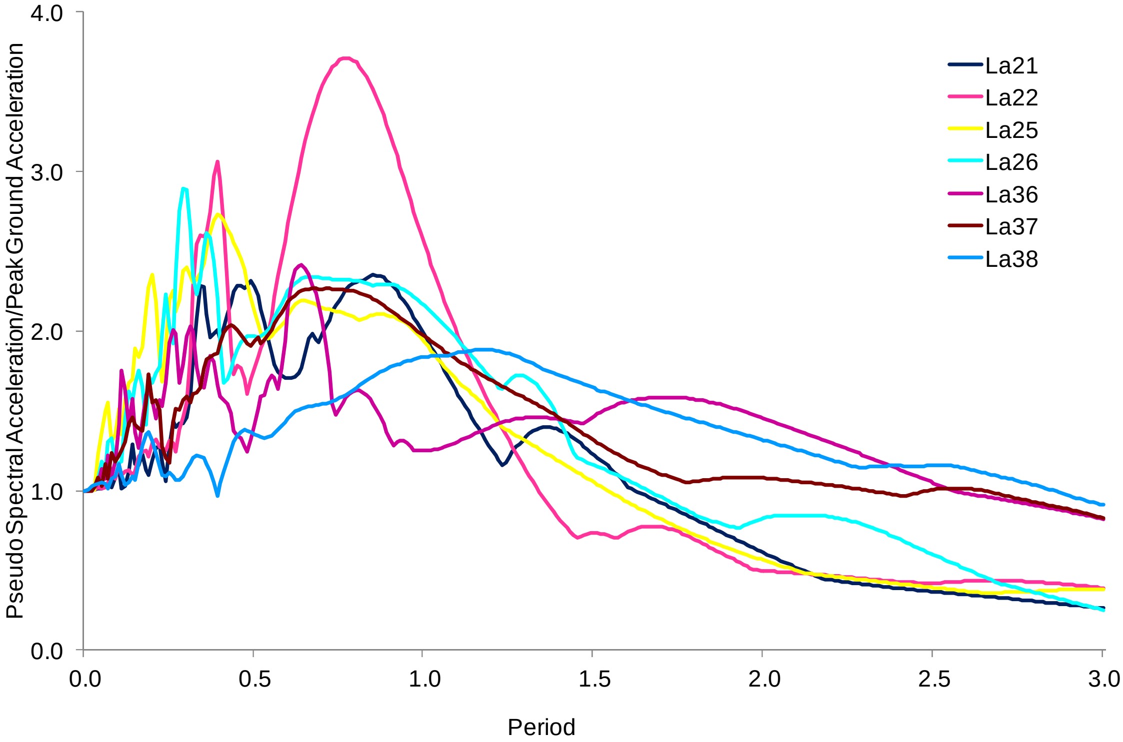
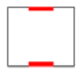
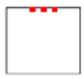


Figure 2.3. Normalized response spectra of the selected earthquake ground motions (ξ=5%)

Table 2.2. Strength variation in the perimeter CBFs

Case No	Strength of the Brace	Applied Frame		Applied Story
Case-1	$1.2R_yF_y$	Frame-1		Story-1,2,3
Case-2	$1.2R_yF_y$	Frame-1,2		Story-1,2,3
Case-3	$1.2R_yF_y$	Frame-1,2,3		Story-1,2,3
Case-4	$1.2R_yF_y$	Frame-1,2,3,4		Story-1,2,3
Case-5	$1.2R_yF_y$	Frame-1		Story-1
Case-6	$1.2R_yF_y$	Frame-1,2		Story-1
Case-7	$1.2R_yF_y$	Frame-1,2,3		Story-1
Case-8	$1.2R_yF_y$	Frame-1,2,3,4		Story-1

3. ANALYSIS RESULTS

The results from nonlinear dynamic response analyses are presented in the form of base shear vs. roof displacement (pushover curve), axial stress-strain in brace members, drift ratio and maximum axial compressive forces in the braces. Though eight different cases were defined as given in Table 2.2 covering a wide range of possible configurations, the results for the Cases 2, 3, and 4 and 6, 7, and 8 did not change at all. This is due to the fact that when the braces in both perimeter CBFs in the direction of earthquake have the same strength equal to F_y, R_yF_y or $1.2R_yF_y$, the strength variation in the braces perpendicular to the direction of earthquake does not affect the response. The response would differ and be significant for these Cases if there were some kind of torsional irregularity due to mass eccentricity, which is the next step of this on-going study. Torsional irregularity due to the variation in strength only exists in Cases 1 and 5. Thus, the results are presented together for Cases 2, 3 and 4 and 6, 7 and 8, because the response of the building corresponding to these torsionally regular cases was observed to be the same.

3.1. Pushover Curves

Fig. 3.1 shows the pushover curves in the form of base shear vs. roof displacement for all cases and the reference building. Lateral strength of the reference building was found to be 7,500kN (Fig. 3.1a). For Cases 1, 5, 6, 7 and 8, lateral strength of the building slightly increased (Figs. 3.1b, d, e) and remained around 7,500kN. It should be noted that in Case 1, only the strength of the braces in Frame 1 at all stories was taken as $1.2R_yF_y$ in the direction of earthquake, whereas the braces in Frame 2 was assumed to have expected yield stress, R_yF_y. For Cases 5, 6, 7 and 8, where only the braces at the first story of the CBF is equal to $1.2R_yF_y$, lateral strength of the building was not affected. However, for Cases 2, 3 and 4, lateral strength of the building increased about 8% (8,050kN) compared to that of the reference building (Fig. 3.1c). The strength of the braces in Frames 1 and 2 in the direction of earthquake was assumed to have $1.2R_yF_y$ for Case 2.

The response of the building subjected to the selected earthquake ground motions for each case was affected by the strength variation in the braces in perimeter CBFs significantly (discrete points in Fig. 3.1). For example, when subjected to LA38, the reference frame experienced about 7.45cm lateral displacement in the direction of earthquake (Fig. 3.1a). However, the lateral displacement of the building increased up to 11.12cm, 24.94cm, 11.11cm and 8.67cm lateral displacement for Cases 1; 2,3,4; 5; and 6,7,8, respectively. The maximum lateral displacement of 24.94cm among all cases is for the building where there is no torsional irregularity, but only variation in strength of the braces in both perimeter CBFs (Cases 2, 3 and 4). The average maximum lateral displacement for these cases increased about 60% compared to that of the reference building. The average maximum lateral displacement for Case 1, where the most significant torsional irregularity exists among all cases, increased about 54%.

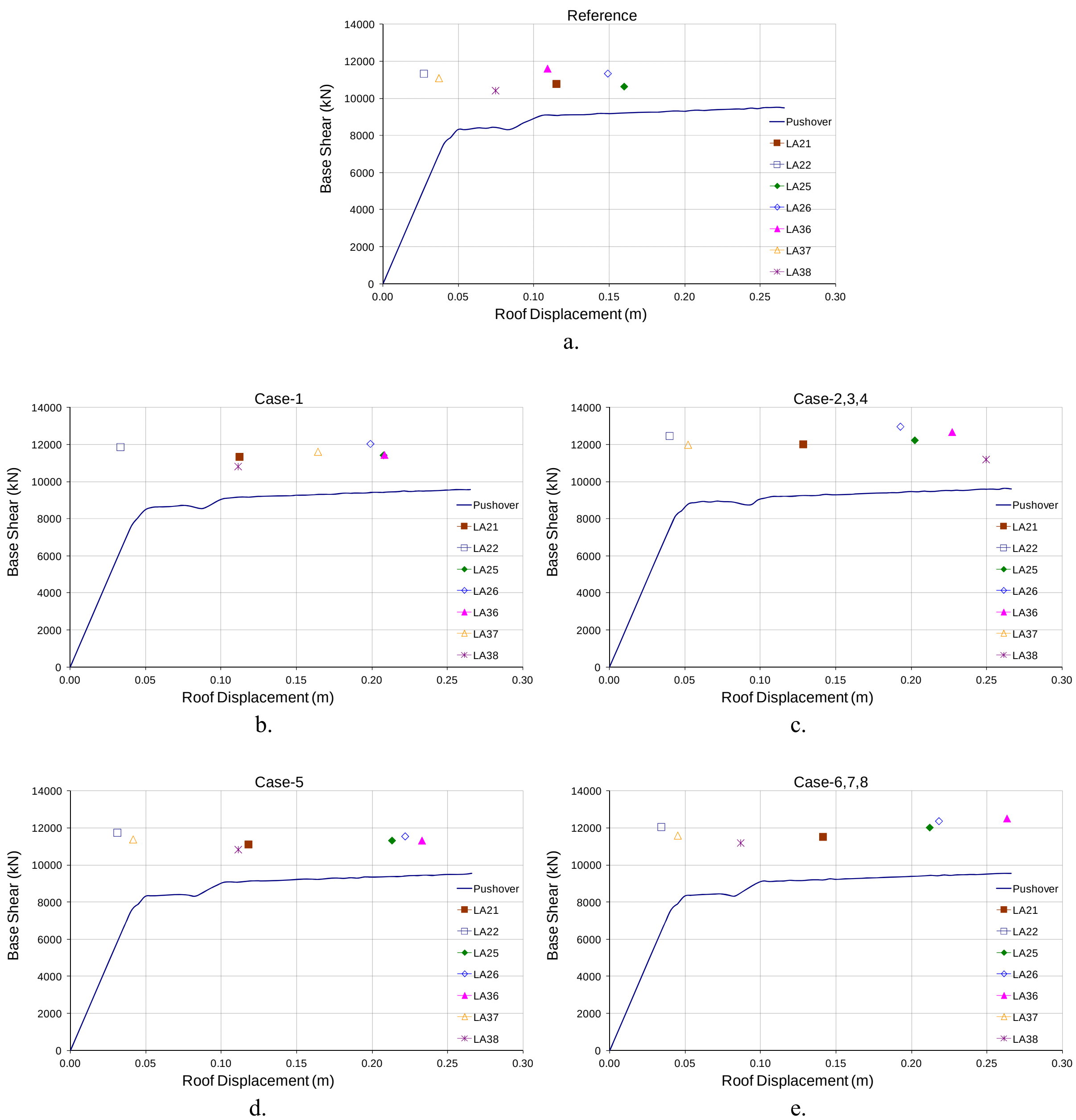


Figure 3.1. Base shear vs. roof displacements (pushover curves)

3.2. Story Drift Ratios

Fig. 3.2 shows the story drift ratios at the story levels for all cases and the reference building when subjected to the selected earthquake ground motions. The maximum average drift ratio for the reference building was found to be 5.3% (Fig. 3.2a). For Cases 1, 2, 3, and 4, the maximum average drift ratio almost remained the same and slightly changed for Cases 5, 6, 7, and 8. LA36 caused a maximum drift ratio of around 8.0% for all cases. These limited results indicate that torsional irregularity due to the higher strength of braces in a CBF or torsional regularity due to the higher strength of braces in all CBFs in the direction of earthquake does not have a great impact on the drift ratio on low-rise CBFs. This observation was supported by the roof displacement vs. time graph as given in Fig. 3.3.

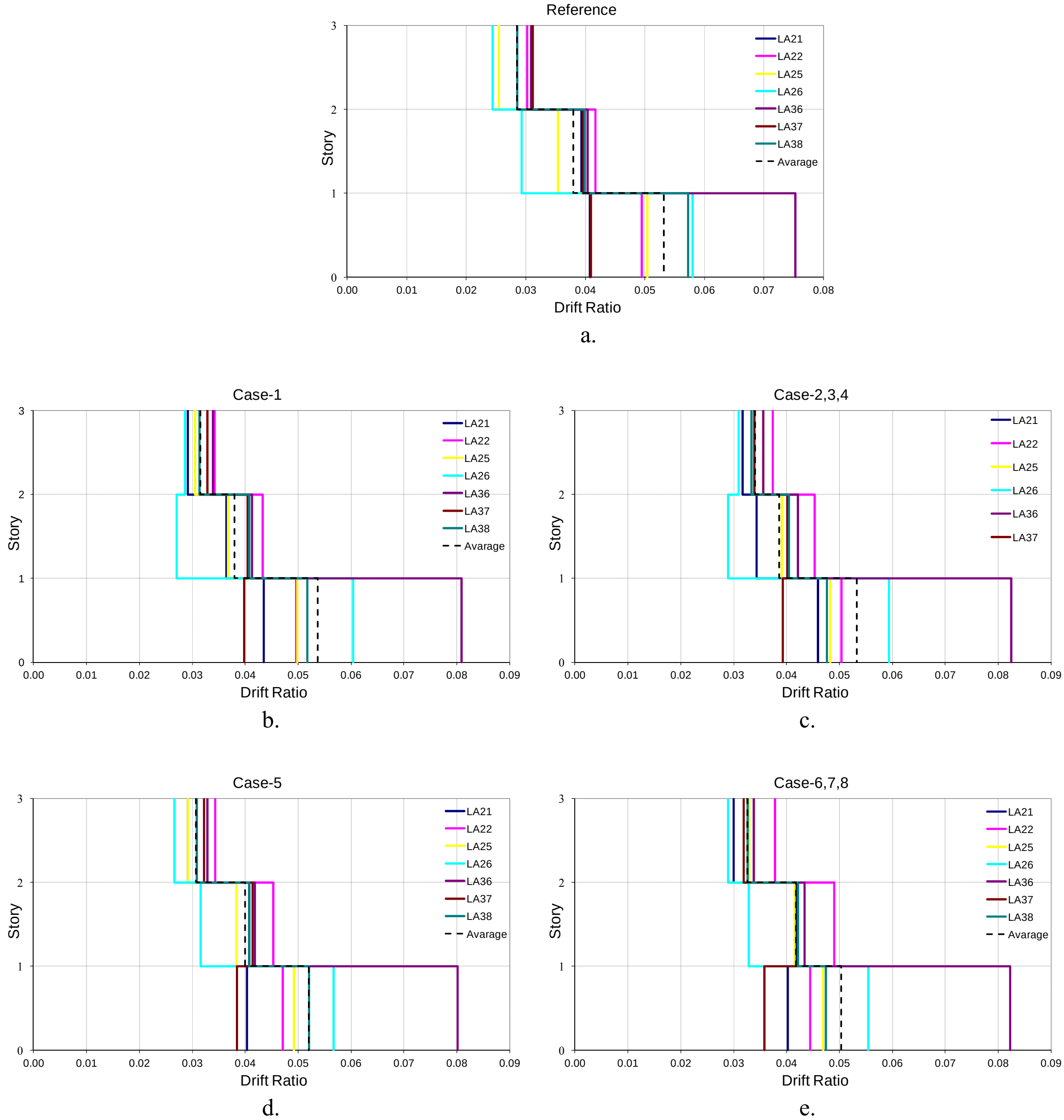


Figure 3.2. Story drift ratios at the story levels

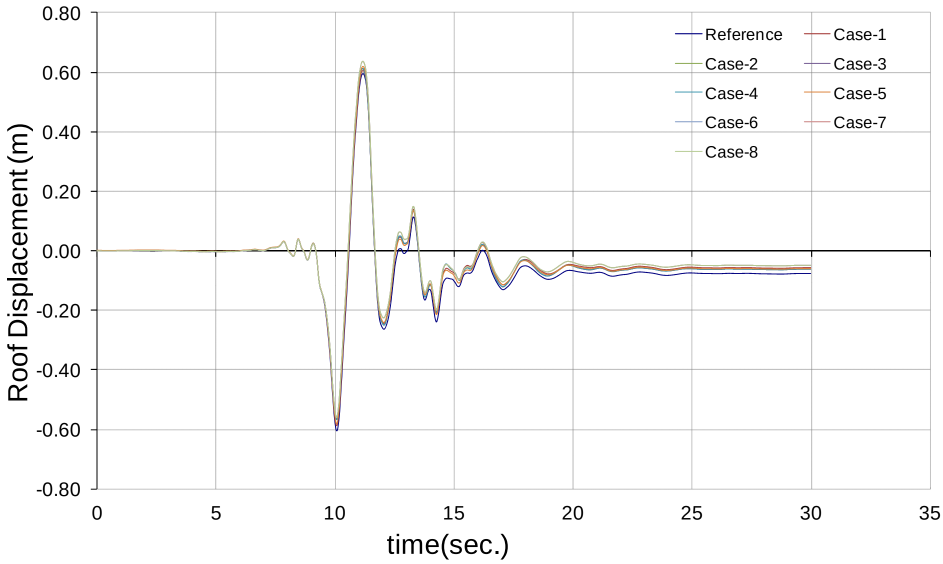


Figure 3.3. Roof displacement vs. time in the direction of earthquake

3.3. Axial Force vs. Axial Strain

Fig. 3.4 shows the representative axial force vs. strain at the first story braces in the CBF for all cases and the reference building when subjected to LA21 earthquake ground motions in the direction of earthquake. The first story braces reached their tensile strength (6,015kN) corresponding to R_yF_y for the reference building (Fig. 3.4a), whereas for all the other cases tensile strength demand remained below the tensile strength of the brace (7,215kN) corresponding to $1.2R_yF_y$ (Fig. 3.4b, c, d, e, f). The braces buckled in all cases. Maximum axial strain at the first story braces exceeded 0.04 for Cases 1, 2, 3, and 4 and remained below 0.4 for all other cases including the reference building.

3.4. Average Maximum Axial Compressive Force

Average maximum axial compressive forces at the first story braces in Frames 4 are given in Fig. 3.5 to investigate the effect of higher strength in braces in the direction of earthquake on the braces perpendicular to the direction of earthquake. Demand to capacity ratios (D/C) are also given in Fig. 3.5. The D/C ratio for the first story braces in Frame 4 in the reference building was about 3.5%, which was due to the gravity loading on the frame. However, the D/C ratio jumped to 14% for Case 1, which is the case, where the most significant torsional irregularity exists among all cases. For Case 5, where only the strength of the first story braces is equal to $1.2R_yF_y$ in Frame 1, the D/C ratio was found to be 12%, close to the one for Case 5. For Cases 2, 3, 4, 6, 7 and 8, D/C ratio was the same as the reference building as 3.5% indicating that D/C ratio is not affected when there is no torsional irregularity in the direction of earthquake.

4. SUMMARY AND CONCLUSIONS

The study is investigated inelastic accidental torsional response in a low-rise steel building with perimeter CBFs. The strength of braces causing torsional irregularity in the building was assumed to be varying between R_y F_y and $1.2R_yF_y$. Seven representative earthquake ground motions having 2% probability of exceedance in 50 years were used in analyses. The response of the CBFs subjected to these ground motions were believed to be in the range of moderate to severe to evaluate the inelastic torsional response of the CBFs rationally. From what was observed in this study, the following general conclusions can be made:

a. Lateral strength of the CBF is not affected significantly when there is there is a torsional irregularity due to the higher strength of braces in a CBF. However, lateral strength of the frame may increase significant amount when there is no torsional irregularity but higher strength of braces in all CBFs in the direction of earthquake.

b. The average maximum drift ratio is not sensitive to the torsional irregularity due to the higher strength of braces in a CBF or torsional regularity due to the higher strength of braces in all CBFs in the direction of earthquake.

c. Maximum axial strain increases due to the higher strength of braces in a CBF or in all CBFs in the direction of earthquake.

d. The strength variation in the braces in CBF perpendicular to the direction of earthquake does not affect the overall response of the building when there is no mass eccentricity and the braces in both CBFs in the direction of earthquake have the same strength.

e. Lateral displacement of the CBFs increase significantly when there is torsional irregularity due to the higher strength of braces in a CBF or no torsional irregularity but higher strength of braces in all CBFs in the direction of earthquake.

f. Demand to capacity ratios increases in the braces perpendicular to the direction of earthquake when there is torsional irregularity due to the higher strength of braces in a CBF.

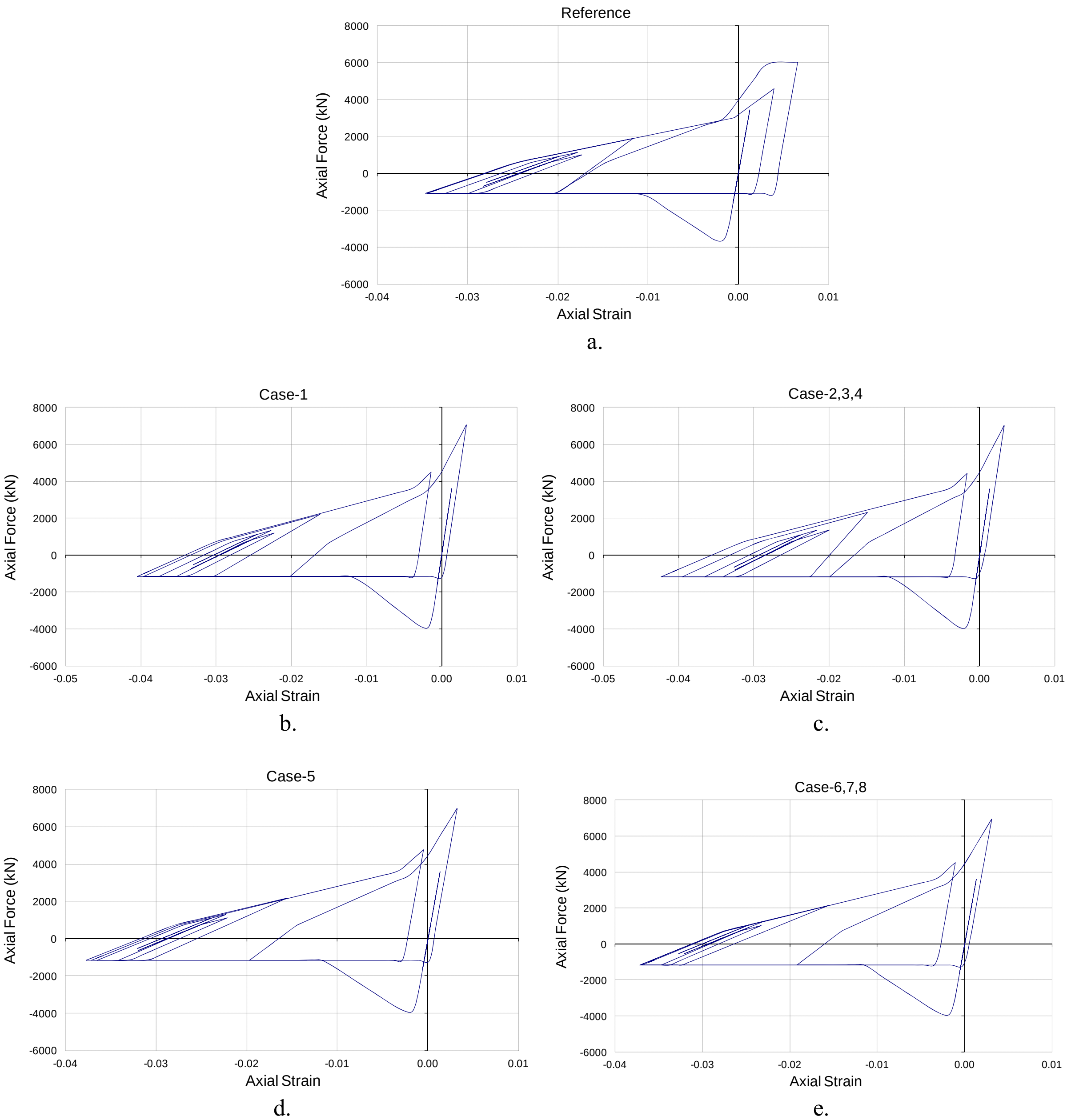


Figure 3.4. Axial force vs. axial strain at the first story braces in Frame 1

Figure 3.5. Average maximum axial compressive force at the first story braces in Frame 4 (See Fig. 2.1)

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